Seismic Behaviour of Piles in Non-Liquefiable and Liquefiable Soil

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Seismic Behaviour of Piles in Non-Liquefiable and Liquefiable Soil

A. Fouad Hussein¹ · M. Hesham El Naggar²

Abstract

This paper investigates the nonlinear soil-pile-structure interaction (SPSI) employing three-dimensional (3D) nonlinear finite element (FE) models verified with the results of large-scale shaking table tests of model pile groups-superstructure systems. The responses of piles in both liquefiable and non-liquefiable soil sites to ground motion with varying intensities were evaluated considering both kinematic and inertial interaction. The calculated piles and soil responses agreed well with the responses measured during the shaking events. The numerical models correctly predicted the different pile deformation modes that were exhibited in the experiments. The FEA was then employed to perform a parametric study to evaluate the kinematic and inertial effects on the piles' response considering different ground motion levels and piles characteristics. It was found that the bending moment of piles in the liquefiable site increases significantly, compared to the non-liquefiable site, due to the loss of lateral support of the liquified soil, and the maximum bending moment occurs at the interface between the liquified and liquefied sand layers. The inertial interaction contributes the most to the bending moments at the pile top and the interface between the top clay and liquefied loose sand layers. For piles with a larger diameter, the bending moment due to kinematic interaction increases significantly and the bending moment distribution corresponds to short (rigid) pile behaviour. In addition, the piles at the saturated site displace laterally as a rigid body during strong ground motions because the pile base loses the lateral support due to the soil liquefaction. Finally, the kinematic interaction effect becomes more significant for piles with higher elastic modulus.

Keywords: Liquefaction · Soil-Pile-Structure Interaction · Shake Table · Seismic · Dynamic Finite Element · OpenSees

Declarations

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Competing interests

The authors declare that they have no competing interests or conflicts of interest

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All data are fully available

Code availability

software application "OpenSees" open source

Authors' contributions

A. Fouad Hussein: Conceptualization, Methodology, Investigation, Resources, Numerical Analysis, Validation, Data curation, Writing - original draft. **M. Hesham El Naggar:** Supervision, Methodology, Resources, Writing, review, editing

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1 Introduction

Analysis of soil-pile-structure interaction (SPSI) is an essential consideration in the design of major structures, such as high-rise buildings, bridges, and infrastructure that are supported by deep foundations installed in loose sand (Maheshwari and Sarkar 2011). In regions characterized by significant seismic hazard, the pile foundation could experience severe damage during large earthquakes, which could lead to catastrophic failure of the supporting structure (Cubrinovski et al. 2017; Cubrinovski et al. 2014; Dash et al. 2009; Finn and Fujita 2002; Palermo et al. 2011; Wotherspoon et al. 2012). Sites with saturated loose sand layers are susceptible to liquefaction due to the build-up of pore water pressure (PWP). The potential of liquefaction and consequent damage have attracted significant research efforts to investigate the complex SPSI in liquefiable soil (Holzer et al. 1989; Kramer 1996). Moreover, if the soil layers are sloped, the ground could move laterally causing lateral spreading, which adds additional lateral forces on piles (Abdoun and Dobry 2002; Cubrinovski and Ishihara 2006; Ebeido et al. 2019; Ishihara and Cubrinovski 1998; Su et al. 2016). Failures of pile foundations and the supporting structures during recent earthquakes (e.g., 1989 Loma Prieta earthquake, 1995 Kobe earthquake, and 1999 Chi-Chi earthquake) underscore the importance of better understanding and evaluation of seismic SPSI effects (Boulanger et al. 1995; Chu et al. 2006; Su et al. 2020; Sugimura et al. 2004). Bhattacharya et al. suggested that the failure of piles in liquefiable soil is due to pile buckling owing to the loss of support from the surrounding soil, eventually causing a plastic hinge in the pile. This failure mechanism is the same as pile buckling resulting from installation errors or weak support of soft clay (Bhattacharya et al. 2009; Bhattacharya and Bolton 2004a; Bhattacharya and Bolton 2004b; Bhattacharya et al. 2005; Bhattacharya and Madabhushi 2008). The difficulty of analyzing the SPSI in liquefiable soil is related to the involvement of PWP, the inertial force from the interaction between the superstructure mass with the piles-soil system, and the kinematic interaction between the pile and soil. The superstructure will generate an inertial force when it oscillates, which is transmitted eventually as lateral force and bending moment on the pile cap. This subjects the piles to additional lateral and axial loads, hence increasing the bending moment on the pile shaft. Excess pore water pressure (EPWP) is generated during the shaking, which reduces the pile axial capacity, and can lead to an excessive post-earthquake settlement. The loss of soil resistance along a section of the pile shaft due to liquefaction can cause buckling and axial instability of the pile, and deterioration in its bending stiffness which may initiate plastic yielding (Bhattacharya and Bolton 2004b; Boulanger et al. 1999; Veletsos and Meek 1974).

Model tests for evaluating the SPSI, such as centrifuge and 1g shaking table tests, provide useful insights into different aspects of seismic SPSI and generate a beneficial database that can be utilized for verifying advanced numerical models to further investigate the complex phenomena. In scaled modelling experiments, typical measurements include the responses of soil, piles, and superstructures (Abdoun et al. 2003; Kagawa et al. 2004). Yasuda et al. (2000) conducted large-scale shake table tests to investigate the behaviour of piles in liquefiable soil considering the potential of ground flow. Tokimatsu et al. (2005) studied the effect of inertial and kinematic forces on pile stresses based on the results of a large shaking table testing program for piles installed in both dry and saturated sand. Cubrinovski et al. (2006) performed large-scale shake table tests on two flexible piles and two stiff piles subjected to ground displacement. They reported that the key parameters, which controlled the pile response were the lateral ground displacement, ultimate pressure from the surface layer, and stiffness reduction in the liquefied soil. The behaviour of a low-cap pile group in liquefied dense sand was examined using shake table tests (Tang et al. 2016). Ebeido et al. (2019) performed large-

scale shake table experiments to study the response of singles pile and pile group to liquefaction-induced lateral spreading.

This experimental database provides valuable information for calibrating advanced numerical models for high-performance computational simulation tools. There are several approaches to model the SPSI. The beam on the nonlinear Winkler foundation (BNWF) model is the simplest and is most commonly used, which estimates the piles' response to lateral ground deformation due to liquefaction by simulating the piles as beam elements and the soil as longitudinal and transversal springs. These simplified models can capture the pile bending moments and the reduction of the soil lateral resistance due to liquefaction. They incorporate empirical p-y curves to describe the relationship between load and deformation (Ashour and Ardalan 2012; Cubrinovski and Ishihara 2004; Liyanapathirana and Poulos 2005; O'Rourke et al. 1994; Su et al. 2016), where the spring stiffness can allow non-proportional relations between the load and the displacement.

Recent research efforts adopted complex 3D finite element models to investigate the soil nonlinear behaviour by simulating the whole SPSI system to properly account for the interaction between the different soil layers with different stiffnesses, piles and superstructure. However, this analysis requires complex mathematical and computational modelling efforts which require extensive specialized computational tools (Kampitsis et al. 2015). Meanwhile, recent developments in numerical simulation, including sophisticated and realistic constitutive models for soil behaviour, provide powerful tools for analyzing the seismic SPSI. Consequently, several researchers investigated the pile behaviour in liquefied soil employing numerical models (Chae et al. 2004; Elgamal and Lu 2009; Maheshwari et al. 2004; Yang and Jeremić 2003). Numerical studies overcome the disadvantages of experimental simulation such as scaling and hence can be used to validate experimental observations and to further study different aspects of SPSI considering a wide range of soil and pile parameters (Cheng and Jeremić 2009). Cubrinovski et al. (2008) studied the effect of non-liquefied crust layer on the bending moment of piles by performing three-dimension effective stress analysis. Assimaki and Shafieezadeh (2013) performed non-linear finite element models to investigate the SPSI in liquefied soil-induced lateral spreading and validated the numerical model with physical tests. He et al. (2017) calibrated a numerical model employing results of large-scale shaking table experiments to investigate the effect of soil permeability on triggering liquefaction and the liquefaction-induced lateral force. Su et al. (2020) simulated the response of the SPSI system behind a quay wall in liquefiable soil-induced lateral spreading and calibrated the model using the results obtained from large shake table tests.

The studies reviewed above explored the response of piles in either dry soil or saturated and liquefiable soil individually. None of these studies investigated the SPSI response considering the same configuration of piles in similar site conditions but with liquefiable versus non-liquefiable soil to allow for direct comparison of the obtained responses and delineating the performance aspects and failure mechanisms for each case. It is particularly important to separate the kinematic and inertial effects considering liquefaction conditions.

Xu et al. (2020) performed shaking table tests to investigate the seismic response of the SPSI in dry and saturated sands, employing a similar test configuration (soil profile and pile group) to enable direct comparison of the nonlinear response for non-liquefiable and liquefiable soil cases. They applied different shakings including, 0.05g sine wave beat motion to induce linear soil-pile responses and the Wenchuan earthquake scaled motion of 0.3g to induce nonlinear soil-pile responses. Due to the ever-existing limitations of experimental programs, these experiments were limited to one soil profile and a limited range of earthquake shaking.

To further extend the observations from this comparative study, the present paper develops rigorous 3D nonlinear finite element models to investigate the dynamic response of the SPSI considered in the shaking table testing program to evaluate the effects of different soil and pile parameters (Xu et al. 2020). The numerical models were first validated by the experimental results then were employed to explore the kinematic and inertial effects on the SPSI.

2 Description of shake table experiments

Xu et al. (2020) performed a series of shaking table tests to compare the seismic performance and failure mechanisms of SPSI in non-liquefiable and liquefiable loose sand sites. The tests were performed using the large-scale table facility of the State Key Laboratory of Building Safety and Environment at China Academy of Building Research. The shaking table dimensions were 6m x 6m with operation frequencies ranging from 0.1Hz to 50Hz. The soil was enclosed in a laminar shear box with dimensions of 3.2m x 2.4m x 2.2m, see Fig. 1. The soil model was 2.0m deep and consisted of three layers: 0.30m of a non-liquefiable clay crust and a plasticity index of 13.3%, 1.2m loose sand with 60% relative density, and 0.50m of dense sand with 90% relative density at the bottom. The top clay crust and the bottom dense sand would accelerate the build-up of the PWP in the loose, potentially liquefiable, layer. The intermediate loose sand was classified as poorly graded silica sand with relative density, $D_r = 60\%$, the effective diameter of 0.35mm, the mass density of 1570 kg/m³, friction angle, $\phi = 37.5^\circ$ and elastic modulus, $E_s = 3.82\text{MPa}$. The water table was located at the surface of the clay crust to ensure that the liquefiable sand layer would be fully saturated during the test. Xu et al. (2020) indicated a potential disturbance of the soil profile during installation, and all the soil parameters were therefore assumed as average values. The poorly graded sand and the soil disturbance explain the low measurement of the soil parameters, especially in the test with water included. These assumptions are examined in the numerical model calibration, and sensitivity analysis for the constitutive model was performed to explore the effects of different parameters on the response of the shaking table tests.

A 2x2 pile group was installed in the soil bed, and the pile heads were connected by a 0.25m thick square cap (0.8m x 0.8m), and the piles were spaced at 5 times the pile diameter. The concrete compressive strength of the pile cap was 27.6MPa and the concrete compressive strength of the piles was 17.1MPa. The pile cap was reinforced with bars 5 mm in diameter spaced at 80mm across the cap. The piles had diameter, $d = 0.1\text{m}$ and length, $L = 1.65\text{m}$, with the pile toe located 10cm above the base. Each pile was reinforced with 6 longitudinal steel bars with a diameter of 4 mm. The top 1.40m of the pile was reinforced by 1.2mm diameter spiral stirrups with a 10mm pitch, while the rest of the pile had stirrups with 20mm pitch. A model superstructure was bolted on top of the pile cap, which comprised two rigid steel blocks weighing 410Kg each, connected by a flexible steel column. Both the liquefiable and non-liquefiable soil experiments had the same configuration for soil layers, pile group, and superstructure, but the soil was saturated in the liquefiable site.

Fig. 1 Pile foundation-simplified superstructure-soil bed

3 Finite element model

The 3D soil-pile-structure interaction numerical model was established using the OpenSees Framework (Mazzoni et al. 2006), which is an open-source computational platform that aids in the development of applications to simulate geotechnical structures for static and seismic analysis (Yang and Elgamal 2002). The post-processing of these analyses was performed using GID (Website: <http://www.gidhome.com>).

3.1 Geometry and Meshing

Fig. 2 shows the soil model discretization, in which the thickness of soil elements along the depth was 0.30m. The maximum element size in the longitudinal direction ($L=3.60\text{m}$) was 0.30m, and the maximum element size was 0.4m in the transverse direction ($W = 2.4\text{m}$). The dominant frequency of the Wolong ground motion in the Wenchuan Earthquake was 4.69Hz (Xu et al. 2020), while the maximum frequency of a wave propagating efficiently through a soil element with shear wave velocity, $V_s = 100\text{m/sec}$, $F_{\max}=V_s/(4H) = 62.25\text{Hz}$ (considering $H = 0.4\text{m}$). Thus, the element size was adequate as the maximum frequency was well above the dominant frequency of the Wolong motion, and the soil discretization would ensure proper propagation of the ground motion through the soil elements. A finer mesh could cause redundant elements within the wavelength and possibly numerical problems (Zerwer et al. 2002). The annular zone around the pile was divided into 6 elements (each 0.05m) to simulate the pile diameter and the surrounding 10 cm soil interface element, as shown in Fig. 2 section B-B. Fig. 3. shows a 3D view for the soil section discretization. The model had 3933 nodes and 3168 elements.

Fig. 2 Longitudinal section for the geometry of soil-pile-superstructure interaction used in numerical modelling

Fig. 3 3D view for the soil mesh discretization using GID (Website: <http://www.gidhome.com>)

3.2 Defining Elements and Materials

The soil elements were modelled utilizing 8-node hexahedral linear isoperimetric elements (Brick u-p Element) (Yang et al. 2008). The element has four degrees of freedom: three for the solid displacement in the three perpendicular directions (u), while the fourth is for the fluid pressure (p). This element simulates the dynamic response of the solid-fluid fully coupled material, and its ability to capture the dynamic response of solids was verified for soil with permeability less than 10^{-3} in the complete frequency range (Zienkiewicz et al. 1980; Zienkiewicz et al. 1999). The constitutive material model PressureDependMultiYield02 (PDMY02) was used to simulate the nonlinear response of the sand layers. The material response is elastic-plastic, and it is sensitive to pressure change. The behaviour of the clay layer is simulated employing the PressureIndependMultiYield elastic-plastic material model, in which the plasticity appears only in the deviatoric stress-strain response; however, the volumetric stress-strain response in the linear-elastic response is independent of the deviatoric stress and it is insensitive to the confining pressure variation (Elgamal et al. 2002; Yang and Elgamal 2002; Yang et al. 2008).

The soil parameters for the constitutive models were selected based on the recommended values of the model (Yang et al. 2008) and correlations with the parameters measured from the shaking table test. The sand minimum and maximum voids ratios were estimated using the correlations with its particles' mean diameter and uniformity coefficient (Sarkar et al. 2019). The voids ratio was then used to calculate the low strain shear modulus at each depth through the correlation (Das and Ramana 2011):

$$G_r = A \frac{(B - e)^2}{1 + e} p r^C \quad (1)$$

Where e is the voids ratio, p is the effective confining pressure in kPa, $A=3230$, $B=2.97$, and $C=0.5$.

Table 1 provides the parameters of the constitutive models for the different soil layers, while Table 2 presents the parameters used to account for liquefaction in the constitutive model. A thin-layer element was introduced to simulate the soil-pile-structure interaction, with a thickness that ranges from 0.01m to 0.1m depending on the relative stiffness between the two adjacent elements (Desai et al. 1984). For efficient seismic analysis, the element is modified by incorporating the Rayleigh damping to simulate energy dissipation (Miao et al. 2016). A thin element 0.1m thick (to account for the high pile-soil relative stiffness) with Rayleigh damping was used surrounding the pile (as shown in Fig. 2) to simulate the pile-soil interface. The shear and normal stiffnesses, and shear parameter of the interface element were reduced by a factor of 0.70 as recommended by Ghalibafian (2006). The soil elements within the annular zone around the pile were discretized to 0.05m corresponding to the size of the pile elements and the interface thickness (Lu et al. 2011).

Table 1 Constitutive model parameters for different soil layers

Parameters	Crust Clay	Loose Sand	Dense Sand
Thickness, m	0.30	1.20	0.50
Relative Density D_r , %	-	60	90
Soil Density ρ , kg/cm ³	1.60	1.60	1.60
Reference Pressure p_r , KPa	101	101	101
Reference Shear Modulus G_r , MPa	12.5	16.55	19.12

Reference Bulk Modulus B_r , Mpa	40.0	43.03	49.71
Friction Angle at peak shear strength ϕ_f , °	0	35.0	37.0
Cohesion c , KPa	25.0	0.1	0.1
Peak Shear Strain γ_{max}	0.1	0.1	0.1
Pressure Dependent Coefficient d_p	0.0	0.5	0.5
Phase transformation angle ϕ_{pt} , °	-	26	26
Contraction coefficient c_1	-	0.02	0.04
Contraction coefficient c_2	-	5.0	5.0
Contraction coefficient c_3	-	0.0	0.15
Dilation coefficient d_1	-	0.0	0.06
Dilation coefficient d_2	-	3.0	3.0
Dilation coefficient d_3	-	0.0	0.15
Voids ratio e	-	0.90	0.75
Horizontal permeability coefficient, m/s	1E-9	6.6E-5	6.6E-5
Vertical permeability coefficient, m/s	1E-9	6.6E-5	6.6E-5

Table 2 Soil liquefaction parameters

Parameter	Value	Description
Liquefaction constant liq_1	1	Damage parameter to define accumulated permanent shear strain as a function of dilation history
Liquefaction constant liq_2	0	Damage parameter to define biased accumulation of permanent shear strain as a function of load reversal History
Cs1	0.9	Parameters defining a straight critical-state line e_c in $e-p'$
Cs2	0.02	space
Cs3	0.7	

The pile and the superstructure were simulated using 3D nonlinear displacement-based beam-column elements, while the cap and rigid links elements were modelled as elastic beam-column elements (Mazzoni et al. 2006). A fiber section was implemented to capture the elastoplastic behaviour of the pile cross-section, including cover, concrete core, longitudinal reinforcement, and the spiral stirrups as shown in Fig. 4 (a) and (b). The uniaxialMaterial Concrete02 constitutive material model was used to describe the concrete part (the core-confined and outer-cover-unconfined) (Kent and Park 1971; Scott et al. 1982), while the Giuffrè-Menegotto-Pinto material model (uniaxialMaterial Steel02) was used to simulate the behaviour of the steel part (Filippou et al. 1983; Giuffrè 1970).

Table 3 and Table 4 illustrate the parameters used to describe the concrete and steel models, respectively. The superstructure cross-section shown in Fig. 4 (c) was also described by a fiber section and its material behaviour was simulated utilizing the constitutive model axialMaterial Hardening.

Rigid beam-column link elements were used to fill the space of the pile within the soil, with elastic stiffness ten thousand times the elastic stiffness of the pile ($E_{link}=104 E_{pile}$) (Su et al. 2017); the rigid links were normal to the pile cross-section (Elgamal et al. 2008). The pile cap was modelled with high flexure stiffness and the total mass of the cap was assigned to the center of the cap at depth of 0.125m. The rigid link and the pile cap were simulated as elastic material. The pile head had the same degrees of freedom as the pile cap, i.e., the same displacement (Li and Motamed 2017). Table 5 illustrates the properties of the parameters used to describe the pile cap and rigid link elements. The two superstructure masses (410 kg each) were assigned at 2.0m and 3.0m, for the bottom and top masses respectively, above the ground level and top of the pile cap (Xu et al. 2020). Table 6 illustrates the properties of the parameters used to describe the superstructure constitutive model.

Fig. 4 Reinforced concrete pile and superstructure cross-sections: (a) pile cross-section; (b) fiber discretization of pile cross-section; (c) superstructure hot-rolled steel H-section (unit: mm).

Table 3 Constitutive model for concrete used in the fiber section

Parameter	Confined core concrete	Unconfined cover concrete
Compressive strength f_c , Mpa	-18.81	-17.1
Strain at compressive strength ϵ_c	-0.005	-0.003
Crushing strength f_{cu} , Mpa	-4.32	-3.05
Strain at crushing strength ϵ_{cu}	-0.025	-0.01
Concrete modulus of elasticity E_c , GPA	15.1	15.1

Table 4 Constitutive model for Steel used in the fiber section

Parameter	Value
Yield Strength f_y , Mpa	240
Elastic Modulus E , GPA	21
Strain-Harding ratio b	0.01

	R0	18
Parameters to control the transition from elastic to plastic branches	cR1	0.925
	cR2	0.15

Table 5 Constitutive model for the Elastic Element

Parameter	Pile	Cap/Rigid Link
Area Cross-section A_s , m	0.00785	78.5
Elastic Modulus E_p , GPa	15.0	15.0 E4
Shear Modulus G_p , GPa	11.5	11.5
Moment of Inertia I , m^4	4.9E-06	4.9E-6

Table 6 superstructure constitutive model parameter (uniaxialMaterial Hardening)

Parameter	Cap/Rigid Link
Elastic Modulus E_s , GPa	300
Yield stress F_y , MPa	360.0
isotropic hardening Modulus H_{iso}	0.0
kinematic hardening Modulus H_{kin}	1.0E3

3.3 Input Motion

The shake table experiments comprised harmonic sine waves with an amplitude of 0.05g and scaled Wolong ground motion record (Li et al. 2008), as shown in Fig. 5 and Fig. 6. The Wolong peak acceleration was scaled to 0.3g and the 220-sec earthquake motion was scaled to 70sec. The peak acceleration of real motion was reduced because the ground motion dominant frequency was close to the natural frequency of the liquefied soil test, which posed risk to the safety and integrity of the experiment setup should higher intensity were to be used. The dominant frequency of the motion was 4.69Hz, which matched the motion used in the tests. A harmonic wave with a peak amplitude of 0.05g was also applied to study the linear response. It is worth noting that during the test, the shake table actuator could not produce the same small amplitude for both liquefied and non-liquefied tests, but in the numerical study, the same motion was used for the two different sites.

Fig. 5 Sine wave beat a) Acceleration time history, and (b) Fourier spectra

Fig. 6 Wolong ground motion: (a) Acceleration time history; and (b) Fourier spectra

3.4 Boundary Conditions and Analysis Stages

To simulate the physical model experiment, staged analysis was adopted to capture the initial conditions before the seismic loading was applied (Wang et al. 2016). Different boundary conditions were applied to the model at different stages as explained below.

- 1- In the first stage, only the soil block was implemented using the u-p brick elements. The soil base nodes were fixed against the movement in the gravity direction only, and the base outer points were fixed against the translation along x and y-directions. This stage represented an elastic stage, and the soil behaviour was assumed to be linear elastic.
- 2- In the second stage, the state of soil was updated to account for plasticity, and the initial free-field stresses were obtained from a plastic gravity analysis. The base boundary condition was fixed in all directions for all nodes. The 4th degree of freedom of the soil elements defines the conditions of the pore pressure. Therefore, boundary conditions were specified to constrain the nodes above the water level (dry nodes) to account for the built-up of the PWP.

- 3- The third stage comprised installing the structural elements (piles, rigid links, pile cap, and superstructure). The three degrees of freedom of the pile and rigid link nodes and the soil interface elements were tied together. Structure element masses were defined along with the mass of the superstructure, then plastic gravity analysis was performed to calculate the consolidation and settlement due to the pile group installation. This analysis would not capture the effect of the pile installation such as stress relaxation for drilled piles or soil densification for driving piles, but these are minimal in the current analysis (Wang et al. 2016). The response during gravity steps was evaluated using the Newmark integration method with the time integration parameters of $\gamma=1.50$ and $\beta=1.0$.
- 4- In the seismic analysis stage, a shear beam boundary was applied to the node at both sides of the model at each depth to ensure that the movements of the nodes in the horizontal direction were equal at the same height (Su et al. 2020). The permeability coefficients of the different soil layers were updated as per to account for the undrained behaviour during the seismic motion. The base motion was implemented to the fixed base of the model using the UniformExcitation command (Mazzoni et al. 2006) with a time step of 0.01 second, and the seismic analysis was performed.

For the dynamic steps, the solution was obtained using the modified Newton-Raphson approach (KrylovNewton) (Carlson and Miller 1998; Mazzoni et al. 2006). The initial tangent stiffness of the system was used for all steps to achieve an energy increment test less than 10^{-4} . Finally, the Rayleigh damping was defined by a damping ratio of 2% for the dry site and 3% for the saturated site, with an estimated first mode frequency of 1.99, adopted to simulate the energy dissipation and enhance the stability of the numerical analysis. A low stiffness-proportional coefficient of 0.0006 and 0.0008 was used for the dry and saturated sites, respectively.

4 Results and Discussion

4.1 Excess pore water pressure build-up

The excess pore water pressure ratio (r_u) is expressed as the ratio between the measured variation in pore water pressure, (Δu), to the initial vertical effective stress σ_{vo} ($r_u = \frac{\Delta u}{\sigma_{vo}}$) at a specific depth. When r_u reaches 1, the soil loses its shear strength and behaves like a liquid. During the shaking table tests, sensors were placed at different depths through the potentially liquefiable soil to monitor the generation and dissipation of the water pressure [45]. Fig. 7 (a, c) compares the calculated and measured PWP build-up for both the 0.05 g sine beat wave motion and the 0.3g Wulong ground motion. Fig. 7 (b, d) shows a rise in the PWP at the free field points and the points near the piles, which can be used to evaluate the soil-pile interaction effect on the propagation of PWP.

Fig. 7 (a) shows that the calculated maximum r_u was less than 0.5 at all depths, which was slightly higher than the measured PWP during the weak 0.05g sine wave motion. The calculated and measured r_u values indicate that the weak shaking did not trigger liquefaction in the loose sand.

Fig. 7 (c) shows that both calculated and measured r_u for the Wulong motion reached 1 (i.e., liquefaction occurred) at all points up to a depth of 1.2m and decreased to 0.8 at depth of 1.5. Maximum r_u was less than 0.6 at the base of the sand layer, which is attributed to the densification of the sand and the higher overburden pressure. Moreover, both the experimental and numerical results depicted the triggering of the liquefaction after 11 seconds of the Wulong motion.

However, the experimental results for r_u were slightly less than the calculated values at the lower depths. These observations indicate that the numerical model correctly captured the built-up PWP and liquefaction.

Fig. 8 shows a 3D visualization of the EPWP distribution for the Wolong motion at 15.21 seconds. Fig. 8 also shows the soil deformation (scaled up 70 times) after it reached the liquefaction state.

The calculated EPWP at the far-field point and near the pile are compared. For the points near the pile, the behaviour was oscillatory during the PWP build-up, which corresponded to the first 5 seconds with 0.05g acceleration, and the peak accelerations in the Wolong motion between seconds 6 and 25. This is attributed to the vibration of piles, which developed then dissipated the PWP at the pile-soil interface as a gap opened and closed causing significant increases and decreases in r_u , especially during Wolong motion. This response was also observed in the experiments especially for the top points but was not observed at depth because the gap was limited to the surficial layer. Similar behaviour was also reported by Dash and Bhattacharya (2015) and Wang et al. (2016).

Fig. 7 Calculated excess pore pressure ratio at saturated test: (a) free field points for 0.05g sine beat motion; (b) near pile points for 0.05g sine beat motion; (c) free field points for 0.3g Wolong ground motion; (d) near pile points for 0.3g Wolong ground motion

Fig. 8 Excess pore pressure build-up for 0.3g Wolong ground motion.

4.2 Free-field soil acceleration amplification and deformation

Shape acceleration arrays (SAA) were used to measure the deformations and accelerations of the piles and soil at specified points (Xu et al. 2020). The measured accelerations revealed amplification of the input motion as the seismic waves propagated towards the surface. Fig. 9 compares the measured and calculated profiles of the acceleration amplification in terms of the acceleration amplification factor (AAF) for the dry and saturated soil tests during the 0.05g sine beat wave and the 0.3g Wolong events. For the 0.05g sine beat wave event, the measured AAF at the surface was 2.2 and 7 for the saturated and dry sites, respectively. Meanwhile, the calculated AAF was 2.3 and 6.4 at the same locations as shown in Fig. 9 (a). The lower AAF value in the saturated site is due to soil softening because of the EPWP and increased damping, which dissipated more energy compared to the dry site.

The experimental and numerical results for the Wolong ground motion presented in Fig. 9 (b) show AAF values of 1.0 for the points in the dense sand, however, the topsoil layers exhibited different responses. For the dry site, the motion amplified through the top two layers and the measured and calculated AAF values were 1.32 and 1.41. On the other hand, the saturated site exhibited de-amplification of the acceleration with AAF of 0.64 and 0.73 for the experimental and numerical results, respectively. The de-amplification of the acceleration is attributed to the strength degradation of the liquefied soil. Although the AAF was less than 1 throughout the liquefied sand and the top crust clay, the AAF increased through the crust clay. There are some differences between the experimental and numerical results, which could be due to the possibility of soil disturbance during the performance of tests, and the inability of the actuators to produce the sine wave at exactly 0.05g. However, the experimental and numerical AAF results followed the same trends, and the numerical model could capture the amplification and de-amplification in different sites for different motions.

Fig. 9 Soil peak acceleration amplification factor at section A-A (a) 0.05g sine beat wave: (b) 0.3g Wolong ground motion

The calculated lateral soil displacements are compared with the measured values in Fig. 10. The measured and calculated lateral displacements increased from bottom to top. For the 0.05g sine beat wave, the measured lateral displacements increased from 2mm to 8mm for the saturated site, and up to 6mm for the dry site, while the maximum calculated displacement was 5.0mm for both sites. For the Wolong ground motion, the calculated maximum lateral displacement was 10mm and 12.3mm for the saturated and the dry sites, which is less than what was measured in the experiments. The calculated and measured lateral displacements were higher in the dry sand site compared to the saturated sand site, which is attributed to the increased energy dissipation of the liquefied soil.

Overall, the large deformations and acceleration amplification in the dry sand tests and the liquefaction in the saturated sand tests were predicted fairly accurately by the developed finite element model. The differences between numerical predictions and experimental observations can be attributed to the uncertainty of the measurements and interpretations of the soil parameters and the inability of the actuators to produce the sine wave at exactly 0.05g.

Fig. 10 Soil displacement at section A-A: (a) 0.05 g sine beat wave; (b) 0.3 g Wolong ground motion.

Fig. 11 shows the lateral displacement time histories of the dry and saturated sites during Wulong ground motion. The lateral displacement of the saturated site had a longer duration as shown in Fig. 11 (b), indicating a longer vibration period of the soil profile. Fig. 12 exhibits the 3D visualization of the lateral deformation (scaled up 70 times) for both the dry and saturated sites at the second 11.71 of the Wulong events. For the dry site, the soil block lateral deformation was distributed along with the soil profile almost uniformly, while the lateral deformation increased at a higher rate (almost concentrated) within the liquefied soil layer compared to the dense sand at the bottom and the clay crust at the top.

Fig. 11 Soil lateral displacement time history for Wenchuan earthquake: (a) dry test- (b) saturated test

Fig. 12 Soil deformed shape and lateral displacement from Wulong motion (Second 11.72): (a) dry test; (b) saturated test

4.3 Pile Acceleration Amplification

The SAA was used to measure the pile's accelerations and deformations during the experiments. Correspondingly, the pile accelerations and deformations were calculated at the same locations. Fig. 13 compares the numerical and experimental pile AAF values during the 0.05g sine wave and Wulong earthquake ground motions for the dry and the saturated sites. The trends and the AAF values obtained from the numerical model agree well with the experimental results. For the 0.05g sine wave, the calculated AAF increased from 1.0 at the base to 4.3 and 2.1 at the pile top for the dry and saturated sites, while the maximum experimental values were 3.3 and 2.52 for the dry and saturated sites. The difference between the calculated and measured values is attributed to the inability of the actuators to produce the

sine wave at exactly 0.05g (the actual peak acceleration was less than 0.05g). For the Wolong motion, the calculated and experimental AAF in the dry site increased from bottom to top with a maximum value of 1.99 and 1.83 at the pile top. Meanwhile, the AAF was 1 in the dense soil and decreased through the liquified soil to a minimum calculated and measured values of 0.64 and 0.7 at 1.0m below surface.

Comparing the soil and pile AAF values during the 0.05 g sine wave (Fig. 9 (a) and Fig. 13 (a)), it is observed that the pile AAF was less than that for the soil. On the other hand, the Wolong motion caused larger oscillation in the pile compared to the soil in the dry site, while for the saturated site the soil acceleration was de-amplified more than the pile's acceleration as noted from Fig. 9 (b) and Fig. 13 (b).

Fig. 13 Peak acceleration amplification factor for pile 2 (a) experiments with 0.05 g sine beat wave: (b) experiment with 0.3g Wolong ground motion

4.4 Pile Lateral Displacement and Vertical Settlement

The pile lateral deflection was significantly affected by the ground motion intensity and the site condition. For the weak motion, the pile maximum lateral displacement was less than 1.5mm for both dry and saturated sites. The pile lateral displacement increased during the large Wolong motion, i.e., 7.37mm and 5.47mm for the dry and saturated sites, respectively. The pile group in the saturated site, however, exhibited a larger rotational response. Also, the pile base lateral displacement was 0.0mm during the dry tests and was -0.65mm during the saturated test. This indicates the pile group rotated in an almost rigid body mode, as opposed to flexural deflection at the top during the dry test.

Fig. 15 displays the calculated vertical settlement time history of piles 1 and 2 in the dry and the saturated sites for both ground motions. The piles static settlement was 0.35mm and 0.5mm at the dry and saturated sites. During the shaking events, the pile group settled more: a gradual increase in settlement of piles was observed during the 0.05g sine wave ground motion to a maximum of 1.8mm and 1.6mm for the dry and saturated sites, respectively. For the Wolong ground motion, the settlement increased to 3.3mm for the dry site and 6.1mm for the saturated site. It is noted that a large difference in the settlement was observed after the 11.7 seconds when liquefaction occurred. Again, a longer duration of response cycles of the pile-soil system is observed after the soil liquefied. It is also noted from Fig. 15 (d) that the

piles experienced uneven settlement, which corresponded to the rigid body rotational response of the group in the saturated site. This rigid body rotation of the pile-cap-structure system was also observed in the experiments. These results clearly indicate pile groups exhibit different performance characteristics and failure mechanisms in non-liquefiable and liquefiable sites subjected to the same excitation. It is also noted that the settlement of the pile group during strong shaking events is attributed to the densification of loose sand, in addition to settlement associated with liquefaction.

Fig. 14 Pile lateral displacement time history for Wenchuan earthquake: (a) dry test (b) saturated test

Fig. 15 Pile Vertical settlement time history curves: (a) dry test-0.05g motion; (b) saturated test-0.05g motion; (c) dry test-Wenchuan earthquake (d) saturated test-Wenchuan earthquake

4.5 Pile Bending Moment

Strain gauges were attached to the pile reinforcement to measure the pile's strains, which were then used to calculate the bending moment (Xu et al. 2020). Correspondingly, the time history of the pile's bending moments and the results are compared with the measured values. The comparison demonstrates that the calculated bending moments were slightly larger than the experimental results. For the 0.05 sine beat wave, the measured maximum bending moments were 40N.m at the saturated site and 12N.m at the dry site, while the calculated values were 52N.m at the saturated site and 18 N.m at the dry site. This may be attributed to the higher acceleration used in the numerical model compared to the actuator input motion during the experiments. The interface factor which controls the reduction of the soil shear strength at the interface elements could also affect the calculated bending moment. For the Wulong motion, the measured maximum bending moments were 380N.m at the saturated site and 28 N.m at the dry site, while the calculated values were 420N.m at the saturated site and 31 N.m at the dry site (i.e., the difference is about 5%). The maximum bending moment along the shaft varied for piles in the dry site, indicating an inflection point, and residual bending moment existed at the end o shaking as shown in Fig. 16 (a). The bending moment of the pile in the saturated site was much higher than that in the dry site due to the degradation of the pile's lateral support, but there was no residual moment after the shaking as shown in Fig. 16 (b).

Generally, the results indicated that the numerical model captured the pile bending moment characteristics correctly and depicted the different deformation modes for both the saturated and dry sites. Yet, the experimental results showed that piles did not have any macroscopic failure, because the bending moments were less than the yield moment; there were no signs of plastic hinges, which was also predicted by the numerical model.

Fig. 16 Pile bending moment time history for Wenchuan earthquake: (a) dry test- (b) saturated test

5 Inertial and Kinematic Effects on Piles response

Piles seismic performance is dominated by the coupling between the inertia forces, and the kinematic effects that arise from the ground movement interacting with the pile shaft. This topic was investigated through physical tests, cross-correlation analysis techniques and pseudo-static analysis (Abdoun and Dobry 2002; Tokimatsu et al. 2005; Wang et al. 2016) methods. However, full 3D finite element analysis can clarify the ambiguity of the complex conjugation between the kinematic and inertial forces, especially when the liquefaction phenomenon is involved. Plastic hinges are created through the pile shaft because of the excessive internal bending moment induced from the coupling between the kinematic and the inertial force. The location of these hinges depends on the intensity of ground motion (kinematic

effect), masses on the pile top (inertial effect), and pile configuration. A parametric study was conducted in this study to explore the variation of the kinematic and inertial effects on the pile response.

In this section, the pile along the initial movement direction is denoted as the Leading pile, while the pile in the opposite direction is referred to as the Trailing pile. The results of the bending moments shown in the following graphs are the maximum values through the time histories and they are normalized by the yield bending moment (PNBM) for the pile cross-section with $d=0.1\text{m}$. The calculated bending moment is compared to the results from the shaking table configuration.

5.1 Effect of Kinematic Interaction

To explore the kinematic effect of the ground motion on the pile response, the superstructure was removed, and the response was calculated for the 0.3g Wolong motion scaled to 0.4g, 0.5g and 0.6g. The analyses were performed for both the saturated and dry sites. Fig. 17 (a, b) shows the pile normalized bending moments (PNBM) of the saturated site considering the piles with no superstructure. The PNBM increased for all piles with the rise of the input motion; however, the bending moments remained below the yield moment (maximum PNBM = 0.89 at the peak acceleration of 0.6g). The PNBM more than doubled (2 and 2.5) for the leading and trailing piles, which occurred at depth of 0.6 m. The bending moment increase was less along the lower part of the liquefied soil due to the high lateral resistance of the dense sand layer.

Fig. 17 Kinematic interaction effect on piles bending moment $d=0.1\text{m}$ (saturated site no-superstructure): (a) leading pile; (b) trailing pile

The pile diameter affects pile stiffness and correspondingly, the kinematic pile-soil interaction. Thus, analyses were conducted to study the kinematic effect (without superstructure) of piles with $d= 0.2\text{m}$ and $d= 0.3\text{m}$, see Fig. 18. The piles were spaced at $4d$, and the pile longitudinal reinforcement was maintained constant at 0.96% of the concrete

section. For the 0.3g input motion used in the tests, the maximum bending moment of the pile with $d=0.1\text{m}$ was 420N.m , while the maximum bending moment for the pile with $d=0.2\text{m}$ and the pile with $d=0.3\text{m}$ was 3719N.m . presents the maximum bending moments for piles with $d=0.2\text{m}$ and 0.3m . also shows that the maximum bending moment increases from the base to a maximum value at the top (i.e., no inflection point), which indicates short (rigid) pile behaviour due to the low slenderness ratio, $(L/d) = 8.25$ and 5.5 for the piles with $d = 0.2\text{m}$ and 0.3m , respectively. The maximum bending moment occurred at the pile top and increased tremendously as the pile diameter increased due to the higher section stiffness. For example, the maximum bending moment during the 0.6g ground motion was 2552N.m for $d=0.2\text{m}$ and was 8372N.m for $d=0.3\text{m}$. Despite the increase of the bending moments, no plastic hinges occurred because of the increase in the pile bending capacity (PNBM was 54% and 50.7% for $d=0.2\text{m}$ and 0.3m , respectively) during the 0.6g ground motion.

Fig. 18 Kinematic interaction effect on piles bending moment (saturated site, no-superstructure): (a) $d = 0.2\text{m}$; and (b) $d = 0.3\text{m}$

Fig. 19 shows the lateral displacement of piles with $d=0.1\text{m}$, $d=0.2\text{m}$, and $d=0.3\text{m}$. As expected, the pile lateral displacement increased as the peak acceleration of ground motion increased. The lateral displacement of the pile was also influenced by its diameter. For pile with $d=0.1\text{m}$, the pile rotated at a point just above its tip, and the lateral displacement increased to a maximum at the pile head for the ground motions with peak accelerations of 0.3g , 0.4 , and 0.5g . However, the pile displaced laterally in almost rigid body movement for the case of peak acceleration of 0.6g because the dense sand layer liquefied and lost its shear strength. For piles with $d=0.2\text{m}$ and $d=0.3\text{m}$ in Fig. 19 (b, c), due to the increased pile rigidity, it displaced laterally almost as a rigid body with some rotation, resulting in maximum displacement at the pile head. This was particularly pronounced for the strong shaking as the dense soil layer liquefied.

Fig. 19 Lateral displacement of piles at the saturated site without superstructure: (a) pile diameter 0.1m; (b) pile diameter 0.2m; (c) pile diameter 0.3m.

Fig. 20 (a) demonstrates that the kinematic effect on the bending moment of piles (without superstructure) in the dry site was relatively small. However, the bending moment increased almost proportionally through the piles as the ground motion intensity increase; and the maximum bending moments occurred at the pile top and near the middle of the loose sand with an inflection point at the intersection between the clay and the sand layer. Fig. 20 (b) shows that the pile lateral displacement also increased from zero at the pile base to the maximum value at the pile top, and as expected, was increased as the ground motion acceleration increased.

Fig. 21 presents the bending moment for piles with $d=0.1$ supporting the superstructure at the saturated site. Fig. 21 shows that several sections of the pile approached the yield state. The bending moment of pile cross-section at the interface of clay and sand layers exceeded the yield moment (i.e., plastic hinge developed) for both the leading and trailing piles, while the cross-section at the intersection between the loose and dense sand layers approached yield state with a maximum PNBM of 0.8 (cracks initiated). This increase is attributed to the inertia forces and the weak resistance of the clay layer. It is also noted that the bending moment increased consistently through the pile shafts because of the inertial effect. These results demonstrate that when the piles lost the lateral support of the liquified layer, the bending moment increased significantly, especially along the liquified part.

Fig. 20 Kinematic interaction effect on piles (dry site without superstructure): (a) pile bending moment; (b) pile displacement

Fig. 21 Kinematic interaction effect on piles bending moment (saturated site with the superstructure, $d = 0.1\text{m}$): (a) leading pile; (b) trailing pile

Two major parameters define the pile stiffness: the cross-section dimensions and the material elasticity modulus. Thus, the effect of pile material elastic modulus on kinematic interaction is investigated in this section by considering the response of steel pipe pile, with the same diameter as the test pile (i.e., 10cm external diameter) and the wall thickness of 8mm. The pile elastic modulus was assumed to be 2.1×10^8 MPa, and the steel unit weight was 78 kN/m^3 . The pipe pile section modulus is 49 cm^3 , and the bending moment capacity is 18 E3 N.m ; assuming yield strength of 355 MPa. A series of analyses were performed by varying the peak ground acceleration, and the results of the maximum bending moments through the time history are shown in Fig. 22. The maximum bending moment during the 0.3g Wolong earthquake was 1098 N.m at the leading pile and 1497 N.m at the trailing pile, while the maximum bending moment recorded during the shaking table test (as well as the calculated) for the 0.1 m reinforced concrete pile was 420 N.m , which represents an increase of 356% due to the pile rigidity. Fig. 22 (a) shows that the bending moment increases as the peak acceleration increases, and the bending moment for the 0.6 g case is more than twice that for the 0.3 g case. Fig. 22 (b) shows that the trailing piles bending moments increased with higher ground motion, but this increase was observed to be located through the loose sand, and it became prominent at the intersection of loose and dense sand layers. The minor reduction of the bending moment in the clay soil is attributed to the deterioration of the soil passive resistance with higher ground motion.

Fig. 22 Kinematic interaction effect on steel pipe piles bending moment (saturated site with the superstructure, $d = 0.1 \text{ m}$, $t = 8 \text{ mm}$): (a) leading pile; (b) trailing pile

The kinematic interaction for the steel pile at the dry site was evaluated. Fig. 23 illustrates that the increase in pile stiffness caused a higher bending moment; the maximum bending moment during the 0.3 g Wolong motion increased by almost 50% over the bending moment for the concrete pile (from 31 N.m to 45.4 N.m). It is also found that the bending moment increased by 300% from the 0.3 g motion to the 0.6 g motion. Fig. 23 shows that the trailing pile experienced a higher

bending moment than the leading pile, which is attributed to the high passive resistance of the soil against the lateral displacement of the piles.

Fig. 23 Kinematic interaction effect on steel pipe piles bending moment (dry site - the superstructure, $d = 0.1\text{m}$, $t=8\text{mm}$): (a) leading pile; (b) trailing pile

Fig. 24 (a, b) presents the steel pile lateral displacement at the saturated and the dry sites. As expected, the pile lateral displacement of the pile decreased at both the dry and the saturated sites, and the reduction of the lateral displacement is attributed to the increase of the pile stiffness.

Fig. 24 Kinematic interaction effect on steel pipe piles lateral displacement (with the superstructure, $d = 0.1\text{m}$, $t=8\text{mm}$): (a) saturated site; (b) dry site

5.2 Inertial Effects

Several failure mechanisms could be attributed to the inertial force, including the development of plastic hinges due to excessive bending moments, buckling failure and the loss of foundation capacity. Fig. 25 shows the response of piles in the saturated site considering different superstructure mass defined in terms of mass factor, $MF = \text{considered mass/test mass}$. Fig. 25 (a, b) shows that the leading and trailing piles experience increased PNBM as MF increases. The calculated maximum bending moments for $MF \geq 2$ exceeded the yield moment and caused plastic hinging at the clay-sand interface. The pile used 88% of its bending capacity at the $MF=1$, while it used 136% of its bending capacity at $MF=5$; that indicates a 53% difference from $MF=1$ to $MF=5$. The excessive increase of the bending moment at the pile top is due to the significant increase of the acceleration at the pile top and the mass at the top increases the fixation of the pile top.

Fig. 25 Inertial effect on bending moment (saturated site, $d = 0.1\text{m}$): (a) leading pile; (b) trailing pile

The inertial effect was investigated for the piles at the dry site subjected to the Wulong motion with $0.3g$ peak acceleration, and varying superstructure mass. Fig. 26 shows that the PNBM values were small, and the bending moment increased at the intersection of the layers. The rise of the superstructure mass increased the PNBM from 0.06 at $MF=1$ to a maximum value of 0.1 at $MF=5$. The minor effect on the bending moment is attributed to the higher soil lateral subgrade reaction at the dry site which degrades at the saturated site with the EPWPR and the pile becomes laterally unsupported.

Fig. 26 Inertial effect on the piles at the dry site for pile diameter of 0.1m

The effect of the inertial interaction on the pile's lateral displacements is displayed for the saturated and dry sites in Fig. 27. Unlike the kinematic effect, the difference between the pile lateral displacement at the saturated and dry sites was small.

Fig. 27 Lateral displacement of piles: (a) saturated site – d = 0.1m; (b) saturated site - d = 0.2m; (c) dry site – d = 0.1m

To investigate the effect of the pile diameter on the inertial effect at the saturated site, several models have been conducted using pile with d=0.20m. Fig. 28 shows the maximum bending moments during the time history at the

saturated site supporting different superstructure mass during the 0.3g Wolong motion. Similar to the kinematic effect, the pile bending moment increased with a higher pile section, also with a larger superstructure mass. There were no signs of plastic hinges since the maximum bending moment was 3021N.m at MF=5 which is less than the section bending capacity of 4750N.m. The distribution of the pile bending moment displayed in Fig. 28, increased from zero at the pile base to a maximum value at the pile top, indicates a rigid pile response. On the other hand, the bending moment displayed in Fig. 28 shows somewhat flexible pile behaviour, with maximum bending moments at the intersections between the layers. The pile with $d=0.2\text{m}$ had a maximum PNBM of 41% and 64% for MF=1 and 5, respectively.

Fig. 28 Inertial effect on bending moment of the pile with $d = 0.2\text{m}$ at the saturated site

5.3 Piles Vertical Settlement Response to Inertial Forces

During the static analysis, the increase of the superstructure mass caused higher settlement, and the settlement values at time 0 in Fig. 29 (a) show that the settlement increased from 0.53mm at MF=1 to 2.28mm at MF=5 (430% increase). On the other hand, the variation of pile settlement with an increase in superstructure mass changing during the dynamic motion was not significant as it was mainly governed by the static and kinematic effects. The dense sand layer did not liquefy during the 0.3g ground acceleration (maximum EPWPR was 0.6), so it retained its strong resistance (stiffness) to support the higher masses. The pile group rotation in Fig. 29 (b) also indicates that the variation of mass has a minor effect. The lateral and vertical deformation response for the pile with $d=0.2\text{m}$ was similar to the pile with $d=0.1\text{m}$. Therefore, the serviceability conditions are significantly controlled by the kinematic force, and marginally by the inertial forces.

Fig. 29 Piles settlement for $d = 0.1\text{m}$: (a) pile settlement time history; (b) pile group rotation

6 Conclusion

Several 3D finite element analyses, using the OpenSees platform, were performed to address the dynamic response of the soil-pile group-superstructure interaction through non-liquefiable and liquefiable soil states. The numerical modelling was based on the results of four shaking table experiments at the China Academy of Building Research aiming to distinguish between the behaviour of the soil and piles in different site conditions during different intensity motions. Overall, a reasonable agreement between the tests and the numerical analysis was achieved concerning the dynamic response of the soil and the piles.

The numerical models were validated by the shaking table test results. The validated models were then employed to study the kinematic and inertial effects on the soil-pile interaction. A comprehensive parametric study was conducted by varying the pile diameter, pile material, intensity of ground motion, and the superstructure masses. The main findings from the study are summarized below.

- 1) As expected, the pile bending moments increase as the peak acceleration of ground motions. The increase at the saturated site is more significant relative to the dry side due to the loss of the lateral support of the liquified soil. The maximum bending moment occurs at the interface of the loose and dense sand layers.
- 2) The inertial interaction due to the superstructure mass contributes the most to the bending moments at the pile top and the interface of the top clay and loose sand layers. These bending moments increase as the supported mass increases.
- 3) Piles with larger diameters experience a significant increase in bending moment due to kinematic interaction, and the bending moment distribution corresponds to short (rigid) pile behaviour due to the low slenderness ratio.
- 4) As expected, the pile lateral displacement increases as the peak acceleration increases for both the dry and saturated sites. However, the piles displace laterally as a rigid body at the saturated site during strong ground motions as the pile base loses the lateral support due to the soil liquefaction, especially for larger diameter piles.
- 5) The increase of the pile material elastic modulus causes a dramatic increase in the bending moment due to the kinematic interaction, but the lateral displacement decreases owing to the increased pile stiffness.
- 6) The variation of dynamic pile settlement with the superstructure mass is marginal given that the dense soil layer at the base does not liquefy.

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Figures

Figure 1

Pile foundation-simplified superstructure-soil bed

Figure 2

Longitudinal section for the geometry of soil-pile-superstructure interaction used in numerical modelling

Figure 3

3D view for the soil mesh discretization using GID (Website: <http://www.gidhome.com>)

Figure 4

Reinforced concrete pile and superstructure cross-sections: (a) pile cross-section; (b) fiber discretization of pile cross-section; (c) superstructure hot-rolled steel H-section (unit: mm).

Figure 5

Sine wave beat a) Acceleration time history, and (b) Fourier spectra

Figure 6

Wolong ground motion: (a) Acceleration time history; and (b) Fourier spectra

Figure 7

Calculated excess pore pressure ratio at saturated test: (a) free field points for 0.05g sine beat motion; (b) near pile points for 0.05g sine beat motion; (c) free field points for 0.3g Wolong ground motion; (d) near pile points for 0.3g Wolong ground motion

Figure 8

Excess pore pressure build-up for 0.3g Wolong ground motion.

Figure 9

Soil peak acceleration amplification factor at section A-A (a) 0.05g sine beat wave: (b) 0.3g Wolong ground motion

Figure 10

Soil displacement at section A-A: (a) 0.05 g sine beat wave; (b) 0.3 g Wolong ground motion.

Figure 11

Soil lateral displacement time history for Wenchuan earthquake: (a) dry test- (b) saturated test

Figure 12

Soil deformed shape and lateral displacement from Wolong motion (Second 11.72): (a) dry test; (b) saturated test

Figure 13

Peak acceleration amplification factor for pile 2 (a) experiments with 0.05 g sine beat wave: (b) experiment with 0.3g Wolong ground motion

Figure 14

Pile lateral displacement time history for Wenchuan earthquake: (a) dry test (b) saturated test

Figure 15

Pile Vertical settlement time history curves: (a) dry test-0.05g motion; (b) saturated test-0.05g motion; (c) dry test-Wenchuan earthquake (d) saturated test-Wenchuan earthquake

Figure 16

Pile bending moment time history for Wenchuan earthquake: (a) dry test- (b) saturated test

Figure 17

Kinematic interaction effect on piles bending moment $d=0.1\text{m}$ (saturated site no-superstructure): (a) leading pile; (b) trailing pile

Figure 18

Kinematic interaction effect on piles bending moment (saturated site, no-superstructure): (a) $d = 0.2$ m; and (b) $d = 0.3$ m

Figure 19

Lateral displacement of piles at the saturated site without superstructure: (a) pile diameter 0.1m; (b) pile diameter 0.2m; (c) pile diameter 0.3m.

Figure 20

Kinematic interaction effect on piles (dry site without superstructure): (a) pile bending moment; (b) pile displacement

Figure 21

Kinematic interaction effect on piles bending moment (saturated site with the superstructure, $d = 0.1\text{m}$): (a) leading pile; (b) trailing pile

Figure 22

Kinematic interaction effect on steel pipe piles bending moment (saturated site with the superstructure, $d = 0.1\text{m}$, $t=8\text{mm}$): (a) leading pile; (b) trailing pile

Figure 23

Kinematic interaction effect on steel pipe piles bending moment (dry site - the superstructure, $d = 0.1\text{m}$, $t=8\text{mm}$): (a) leading pile; (b) trailing pile

Figure 24

Kinematic interaction effect on steel pipe piles lateral displacement (with the superstructure, $d = 0.1\text{m}$, $t=8\text{mm}$): (a) saturated site; (b) dry site

Figure 25

Inertial effect on bending moment (saturated site, $d = 0.1\text{m}$): (a) leading pile; (b) trailing pile

Figure 26

Inertial effect on the piles at the dry site for pile diameter of 0.1m

Figure 27

Lateral displacement of piles: (a) saturated site – $d = 0.1\text{m}$; (b) saturated site – $d = 0.2\text{m}$; (c) dry site – $d = 0.1\text{m}$

Figure 28

Inertial effect on bending moment of the pile with $d = 0.2\text{m}$ at the saturated site

Figure 29

Piles settlement for $d = 0.1\text{m}$: (a) pile settlement time history; (b) pile group rotation