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Research on Lateral Dynamic Response of Pile Foundation in Liquefiable Soil based on Non-liner Pseudo Static Analysis Method

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In this paper, the author discussed the lateral dynamic response of pile foundation in liquefiable soil based on non-liner pseudo static analysis. Modified model of dynamic p-y curves of pile foundation in liquefiable soil layer are proposed and its reliability are also verified. The basic form of main trunk circuit of pile p-y curve is gained and the difference and connection with the p-y curve in API Code is got through the study on the influencing factors, modification method and mode of the new model construction of the p-y curve in liquefiable soil strata. Using double-parameters modified method construct a new computational model. The new formula overcome shortage of rapid rise of soil-pile interaction force within the deformation is small. The reliability of the new formula is verified through shaking table test in this paper.

1. Introduction

The pile foundation disasters occur often, especially in liquefiable soil layer through all previous earthquakes. Study on lateral capacity of pile in liquefiable soil layer has generally become important subject in aseismic design of soil and foundation engineering in the world. There are obvious deficiencies in aseismic design method of pile in liquefiable site for methods used at present. One of the main causes is that the existing theory and experience for aseismic design method of pile foundations depend on static condition. Except of that, the shortage or misunderstanding to mechanism of liquefied-related pile-soil interaction is the other troubles.

State-art-of researches on the subject are conducted and, moreover, several important problems are pointed out based on shaking table tests as well as post-earthquake investigation and numerical simulation. The design of pile-soil interaction shaking table test is completed to reach the purpose for comparison of multi-cases tests. A lot of data and acknowledges are obtained. In test design, four destinies saturated sand, non-liquefiable sand and silt; two kinds of pile and pile-head models static test under sinusoidal cyclic loading and seismic wave input are included; The time dependent processes of accumulating pore water pressure and the strain of pile at different soil layer depth as well as the interaction force of soil and pile and displacement, acceleration of the pile head.

Relationship among acceleration, pore water pressure, deformation and modulus of soil layer, the inputting acceleration, bending moment of pile and interaction force of soil and pile, response of pile-head is analyzed through many tests. Some new phenomena and acknowledgement are obtained. The results in the paper show the movement of soil layer surface and dynamics response of pile foundation in the liquefied soil has specialties compared with non-liquefiable soil (Khamis et al., 2017). The relative displacement between soil and pile in liquefied soil is much greater than non-liquefied soil. The raising rate of relative displacement is greater than decreasing rate of soil modulus. The soil layer acceleration and pile-head acceleration, bending moment of pile and input acceleration wave in the non-liquefied layer coincide with each other in the form and bending moment of pile near bottom is less than in soil surface.

Surface acceleration in liquefied soil is analogous with pile-head initially. Then the responses raise and reach to maximum as pore water ratio reaches to 0.8. Then, the response decreases obviously under the sinusoidal wave and for seismic input, high frequent decreases in and low frequent amplify. However, they are both agreeable with the base input in the form. The bending moment of pile in liquefied soil layer is greater than

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that near soil surface as the pore water pressure reaches to 0.8. After that, the bending moment does not decrease but is amplified.

The mechanism of liquefaction-induced dynamics pile-soil interaction is revealed by post-earthquake analysis, two scales shaking table tests and numerical simulation. The results in the paper indicate inertia force of pile-head dominates the pile destruction in non-liquefied soil and the soil is pushed by the pile. In this case, soil and pile assume high frequent responses, which are all, be controlled by soil layer acceleration. In liquefied soil layer, however, the relationship between soil and pile is soil-pushing pile since the pile is pushed by the soil and the rate of displacement rising is greater than decreasing of soil modulus. There are obvious correspondence relations in the pore water pressure, soil deformation and soil-pile interaction force. The destruction of pile foundation is controlled by liquefaction-related soil deformation not by the inertia of pile-head. The relative displacement and force of soil-pile and bending moment of pile behave at low-frequency response since they are all be controlled by liquefaction-related soil displacements.

2. Overview

Earthquake is amongst one of the most important serious and fearful natural hazards threatening human lives and property in china and over the world. Its features include that it happens within an interval less than a second or longer, and leave behind great pain and massive losses in properties, life, buildings, and bridges and also leads to another kind of disasters such as explosion, flooding, landslides, and so on. These incidents are not only by means of strong shake but also due to ground deformations happen as a result of liquefaction. Liquefaction is a phenomenon that takes place when a saturated soil subject to shear stress cycles, combined with a rapid decrease in volumetric strain. If there is no time for drain, the water exists in the soil skeleton, is will be some generations for a high excess pore pressure (EPP) which can reduce the frictions between soil particles due to its presence, and reduce the mean effective stress and finally, the stiffness of the soil as well as shown in Figure 1.

Analysis and estimation of pile response to lateral spreading address vital issues for safe design of pile foundations against liquefaction-induce lateral spread. The “lateral spreading” is termed as the down slope movement of the saturated, loosely- deposited, gently sloping ground during earthquake shaking. A well reachable archived document example of the consequences of lateral spreading was from earthquakes in Peru, Guatemala, Tangshan earthquake and Japan's quake 2011 are shown in Figures 2. Soil liquefaction in sloping ground may result in the downslope mass shifting of liquefiable sands and represents as a one of the greatest contributors to damage of piles and pile-supported structures embedded in liquefiable sloping ground during earthquakes.

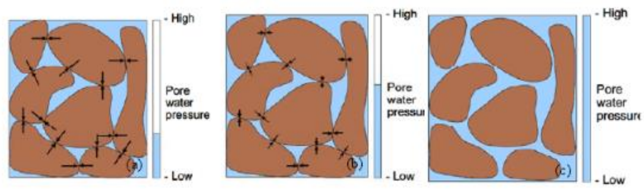

Figure 1. Illustration of soil deposits: (a) before liquefaction, (b) when EPP is increasing and (c) during liquefaction (blue bars and arrows represent EPP and contact forces between grains, respectively)

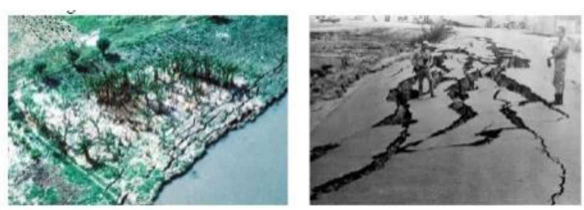

Figure 2: laterals spreading in: (a) the 1970 Peru Earthquake and (b) the 1976 Guatemala Earthquake

The significance of soil liquefaction-related damage to pile foundations in liquefiable sloping ground have been clearly demonstrated during earthquakes such as the 1964 Niigata, 1989 Loma-Prieta and 1995 Hyogoken-Nambu et al., The onset of liquefaction and any lateral permanent displacements will be small in the absence of driving stresses acting on the horizontal ground (Sarkar et al., 2014). The sloping ground is subjected to driving static shear stresses due to the weight of the sloping soil before the occurrence of seismic loading. Hence, with the increasing excess pore pressure, the permanent soil deformations in sloping ground could be very large due to these initial driving shear stresses downslope but the effective stress may not reach near zero at the onset of liquefaction (Finn, 2014). These stresses will act as driving forces and may cause very large ground deformations, even before the onset of soil liquefaction, which in turn potentially cause severe damage to foundations. However, there still remain uncertainties to be understood about the exact nature of dynamic soil- pile interaction problem in liquefiable sloping soil.

There are a number of methods mainly including experimental and numerical, which could be utilized to predict such behaviors. Pile behavior in liquefiable sloping ground under earthquake is a complex problem. Prediction of dynamic behaviors of piles in liquefiable sloping soils is very difficult but achievable (Tang and Ling, 2014). Research efforts worldwide continue to address this challenge with simplified empirical procedures as well as realistic and accurate numerical approaches. Three-dimensional (3D) nonlinear finite element method (FEM) is becoming increasingly feasible for analysis of dynamic soil-pile interaction in liquefying ground. A number of shake-table experimental studies have been conducted to investigate dynamic behaviors of soil-pile interaction in liquefiable horizontal soils. This research attempts to simulate representative shake-table experiments to gain more insights into dynamic behaviors of soil-pile-structure systems in liquefiable horizontal soils by three-dimensional (3D) nonlinear effective stress finite element (FE) analysis.

The numerical simulation employed a solid-fluid fully coupled u-p formulation based on Biot's dynamic coupled theory (Li et al., 2016) and a recently developed soil constitutive model based on a multi-surface plasticity framework. Aspects of model response including excess pore pressure and acceleration of soil, pile acceleration and pile bending moment during liquefaction are examined to validate this numerical simulation methodology. Upon calibration, the finite element analysis has produced a model response that reasonably matched the good-quality data from the shake-table experiment shake-table experiment. Using the above calibrated FE modeling with the identical parameters, the computational modeling for simulating dynamic soil-pile interaction in liquefiable gentle sloping soils continues to be developed and parametric study is presented to investigate dynamic behaviors of bridge pile foundations.

3. Finite element modeling of dynamic soil-pile interaction in liquefiable ground model and algorithm

A shake-table experiment was conducted and shown in Figures 3 including the schematic view and the measurement instrumentation in Institute of Engineering Mechanics, China Earthquake Administration. A large-scale box is set on a shake-table in Figure 3. In the experiment, a single reinforced concrete pile with 0.2 m diameter is tested to simulate the response of the pile foundation subjected to soil liquefaction under nearly harmonic base excitations with dynamic sine wave. The pile tip was connected to the soil box bottom in fixed condition. On the pile heads, a rigid mass model (360 kg) was set in fixed condition as the superstructure. The Harbin-sand with a non-uniformity coefficient of 3.375, mean particle diameter of 0.32 mm, specific gravity of 2.66, maximum void ratio of 0.968, maximum dry density of 1.92 g/cm 3, and minimum dry density of 1.46 g/cm 3 was adopted. The soil profiles used in the test consisted of two horizontal soil layers. The lower layer (1.2 m in thickness) was modeled as saturated sand with the upper "model clay" layer (0.3 m thick) being weaker reconstituted salty clay.

Water table level was located at 0.3 m near the soil interface between clay layer and liquefied sand layer. The saturated sand layer was of relative density 58% and with permeability coefficient of about 0.0045 cm/s. The experimental results such as accelerations and EPP in the ground, accelerations of the pile and strains of the pile were measured and presented in the following sections in comparison with the simulation. To reduce computation time, the model was halved at the line of symmetry along the centerline of the pile as indicated in Figure 4. Nodes along this line were restrained from perpendicular movement across the line of symmetry. Length in the longitudinal direction is 3.8 m, while that in transversal direction (in this half-mesh configuration) is 1.5 m. The finite element meshes of brick elements in 3D finite element model are used to represent the soil deposits. Spacing of the soil elements varied throughout the model from large at the outside to small nearer the area of interest at the inside.

The clay soil domain is expressed by 20-node brick element, without considering EPP, and the saturated sand domain is represented by 20-8 node, effective-stress fully coupled (solid- fluid) brick elements with pressure dependent material. As such, 20 nodes describe the solid translational degrees of freedom, with the eight-corner nodes also describing the fluid pressure in Figure 5, where 20 represents the number of nodes for the solid phase and 8 represents the number of nodes for the fluid phase. The FE matrix equation is integrated with time using a single-step predictor multi-corrector scheme of the new mark type. For each single step, the solution is obtained using the modified Newton-Raphson approach with Krylov subspace acceleration. In order to ensure numerical stability in case of nearly undrained and incompressible pore fluid condition, the Babuska-Brezzi condition should be met (Hung et al., 2014). Consequently, the functional shape of the solid phase should be one degree higher than that of the fluid phase. Therefore, a 3×3 Gauss-Legendre integration rule is used in the evaluation of the matrices related to the solid matrix and a 2×2 rule for the matrices related to the fluid phase. In this idealized model, there is single circle reinforced concrete pile-column.

The pile is 0.2 m in diameter and 2.3 m in length, of which the portion of the pile extending 0.8 m above ground can support a point mass at the top of the pile to model the superstructure during the test. The pile and column are modeled as elastic beam-column elements available in the finite element model. These beam elements have 6 degrees of freedom at each end for translation and rotation in the x, y and z directions. The material properties of the model and the finalized soil parameters used in for the 3D nonlinear dynamic analyses are summarized in the paper directly from shake-table test. Contraction parameters c 1 and c 2 were performed during preliminary computer runs. As dilation was not observed in the experiment, soil dilation parameters d 1 and d 2 were deactivated in the numerical simulation for each motion Events due to its frequency. Quasi rigid beam elements normal to the longitudinal axis of the piles are used to model the actual size of the cross of the pile. These quasi rigid elements are elastic beam-column elements with rigidity 1000 times larger than the pile rigidity (Hung et al, 2014; Garbini, 2014).

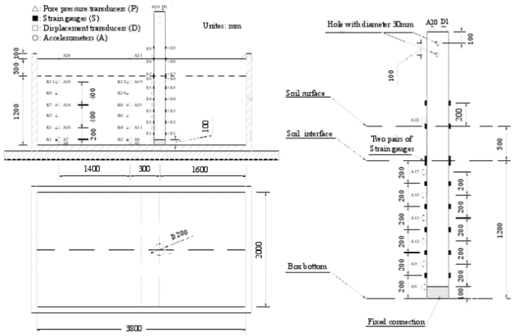

Figure 3: Setup of shake-table experiment

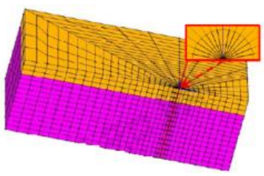

Figure 4: Finite element modelling

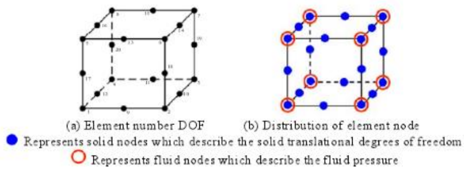

Figure 5: 3D solid-fluid coupled brick elements

4. Experiment and data analysis

Using the finalized numerical modeling and parameters, several studies are undertaken in light of this calibration process for dynamic loading and its predicted response was verified and evaluated with the experimental results. These accelerations used to excite the shake-table experimental model were applied to the base of the finite element model in the shaking direction which included: 1 Hz sinusoidal wave with peak acceleration of 0.1 g and cycle numbers of 20 times as Event A, and 2Hz sinusoidal wave base excitation with peak acceleration of 0.15 g and cycle numbers of 20 times as Event B. Experimental and computed excess pore pressure and acceleration time histories of sand in Events A and B is presented in Figures 6 and 7 respectively.

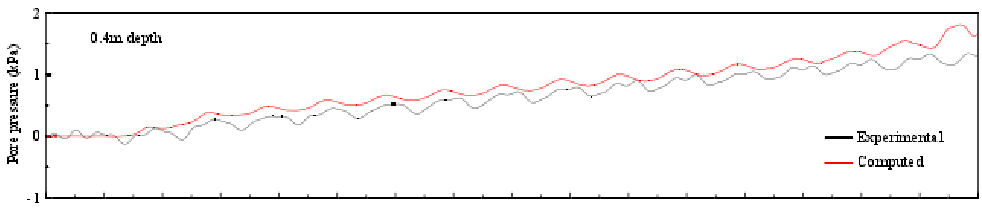

Figure 6: Experimental and computed excess pore pressure time histories of sand in Events A and B

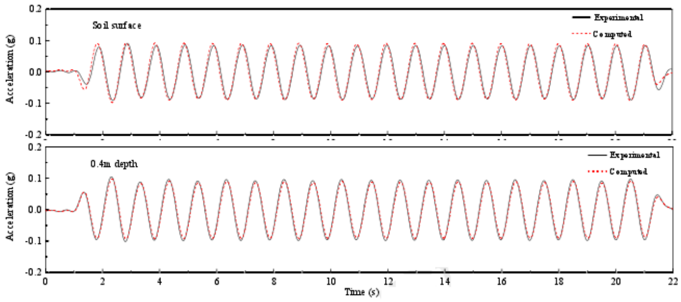

Figure 7: Experimental and computed acceleration time histories of sand

To depict the change of excess pore water ratios more clearly, the ratio was normalized and obtained by dividing the measured excess pore pressure by the initial vertical effective stress at any given depth. And a Ru of 100% indicates complete soil liquefaction. In general, good agreement was achieved between the computed and experimental excess pore pressure responses in Events A and B. The excess pore pressure gradually decreased bottom-up in the vertical direction and the slight contract behavior and dilative tendency were observed in Event A. Experiments and numerical analysis have shown that excess pore pressure ratio rapidly increased and equaled to 1.0 or slightly lower from bottom to top in the vertical direction and its at 0.4 m depth reached the liquefied state at about 10 seconds after shaking in Event B, as indicated by excess pore pressure histories. Once the soil liquefied, there is some more fluctuation of EPPs from numerical analysis in Event B. EPP shows most signs of strong contract behavior and dilative tendency as the sharp spikes in EPPs are observed.

It was also observed that EPP accumulated slowly during small-amplitude shaking in Event A but the obvious accumulation of EPP was observed under the stronger shaking in Event B. Generally, close correlation is noted between the numerical and experimental EPPs. However, at any given depths, the experimental EPPs seem to be higher than those in the numerical results during the shaking. This may be due to pore water outflows from soil surface and box sides causing the lower EPP responses during the test; the sand is treated as an untrained material in numerical analysis. It also has been mentioned that the soil box in the experiment does not continuously move freely as a shear-beam especially in low-amplitude shaking. Similarly, compatibility is observed in the computed and recorded acceleration time histories of sand. At the two specific locations, acceleration responses of sand show strong acceleration spikes similar to those recorded and the computed spikes are in phases under low-amplitude shaking in Event A.

An inspection of acceleration time histories of soil surface from the simulation, were revealed to be a little higher than those from the experiment. Accelerations of soil surface from numerical analysis begin to lag slightly in the experimental measurements during shaking Event B. Although, from the simulation there is still lag the experimental recordings in the numerical results are slightly higher than the measurements in Event B. At 0.4 m depth, acceleration time histories of the sand gradually become larger due to liquefaction during shaking from simulation and experiment in Event B. It was also found that acceleration response of the sand and soil surface displays magnification effect on input base shaking before and after initial liquefaction, which is obviously different from the understanding that generally liquefied soil weakens acceleration response of the sand. This may be due to the different function from liquefied soil during soil liquefaction stage.

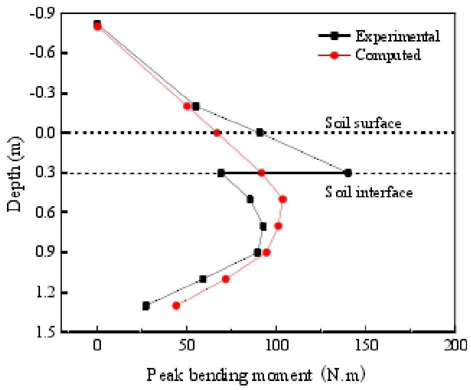

Figure 8: Experimental and computed pile bending moments distribution

Experimental and computed acceleration time histories of the pile and superstructure and bending moment distribution on the pile in Event A is displayed in Figures 8 showing acceleration response of superstructure, soil surface and the pile at 1.2 m depth. The experiment and the simulation agree closely in both amplitude and phase, especially under the lower EPP and shaking amplitude. Acceleration response of superstructure is larger than that of the pile near soil surface and at 1.2 m depth. Only the computed acceleration response of superstructure, soil surface and the pile at 1.2 m depth are a little smaller than the observed ones at 6 sec for the amplitude in Event A. In general, numerical results on pile bending moment profiles including the tendency and the amplitude are close to the experimental measurements in Event A. The peak bending moment occurs at the soil interface from the experimental recordings in Event A. However, the computed peak bending moments on the pile occur in the underlying sand near the soil interface in Event A. Obviously, the computed peak bending moment on the pile above the soil interface gives lower results than the experimental recordings. On the contrary, the pile bending moments predicted in the simulation are higher than the measurements from the experiment below the soil interface.

5. Conclusions

In this paper, the author discussed the lateral dynamic response of pile foundation in liquefiable soil based on non-liner pseudo static analysis. The reasonability of the main computational method of seismic lateral capacity of pile is discussed and the disadvantages of the pseudo-static and coefficient-discounted method are also pointed. The results in the paper indicate both the coefficient-discounted method and using lateral displacement of pile instead of soil-pile relative displacement are not agreeable with the real situation. Fundamental idea of pseudo-static method is not agreeable with mechanism in actual. The pile response cannot be obtained simply by discounting lateral stiffness of non-liquefiable soil layer. Calculated results by using pseudo-static method disagree with real situation especially in small displacement range. Calculating results by using pseudo-static method are quite conservative even cannot be accepted in engineering practice. Modified model of dynamic p-y curves of pile foundation in liquefiable soil layer are proposed and its reliability are also verified. The basic form of main trunk circuit of pile p-y curve is gained and the difference and connection with the p-y curve in API Code is got through the study on the influencing factors, modification method and mode of the new model construction of the p-y curve in liquefiable soil strata. Using double-parameters modified method construct a new computational model. The new formula overcome shortage of rapid rise of soil-pile interaction force within the deformation is small. The reliability of the new formula is verified through shaking table test in this paper.

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