Centrifuge Investigation of Seismic Behavior of Pile Foundations in Soft Clays

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ABSTRACT

The seismic behavior of pile foundations in soft clays is a very complex problem with interacting piles, soils and superstructure. Using ground improvement, for example, Cement Deep Soil Mixing (CDSM), to restrict lateral displacements of piles during earthquakes is a viable, but not fully explored technique. This paper presents experimental methods and procedures for centrifuge tests on single piles with and without ground improvement under both static and dynamic loadings. The data collected from the centrifuge test were used to analyze the differences between piles in unimproved and CDSM-improved soft clay. The test results confirmed that the CDSM can improve the seismic behavior of pile foundations and demonstrated that the pile response is directly related to the size of the ground improvement zone.

INTRODUCTION

Pile foundations are integral part of many civil engineering structures. The seismic behavior of pile foundations is a very complex problem with interactions between soils, piles, and superstructure. This complexity is further exacerbated when weak soils such as soft clays and liquefiable loose sands surround the pile foundation. The behavior of pile foundations in liquefiable sands has been studied extensively; however, similar investigations of soft clays or seismic response of piles in improved soils are scarce. The current seismic design practice calls for avoiding inelastic behavior of pile foundations by restricting their lateral displacements because it is difficult to detect damage to foundations following an earthquake. Limiting the lateral displacement of a pile foundation is relatively easy to achieve in competent soils. In case of weak soils, the current practice is to use an increased number of more ductile, larger diameter piles that are difficult to design and expensive to construct (see Zelinski et al. 1995). An innovative and more cost-efficient solution to this problem is to improve the soil surrounding the pile foundation. For structures undergoing seismic retrofit with existing pile foundations in weak soils, improving the soils may be the most cost effective option to improve the seismic behavior of the foundation. Although abundant research has been carried out on the seismic behavior of piles in loose liquefiable sands (e.g., Ohtomo 1996; Boulanger and Tokimatsu 2006), very few studies have addressed the seismic behavior of piles in soft clays or effects of ground improvement around the piles in soft clays. This technique is not widely used in seismic regions due to lack of fundamental understanding of the behavior of improved and unimproved soils and the interactions between them as well as with the piles during earthquakes. Soft clays are quite prevalent in earthquake prone areas of U.S., but have received little attention from the research community. Meymand (1998) studied soil-structure interaction in an artificial soft clay using shake table tests, but without ground improvement. Lok (1999) and Mayoral et al. (2005) measured p-y curves for the artificial soft clay used by Meymand (1998) using pot testing (a model pile is cyclically loaded in a container of soil in the laboratory). Rollins et al. (2010) used field experiments to investigate the lateral resistance of piles in soft clays with soil replacement, soil mixing and jet grouting. Their studies confirmed that the soil mixing and jet grouting provide effective ways to increase the lateral resistance of existing pile group foundations. In the present study, a centrifuge test was designed to investigate both static and dynamic behavior of single piles in unimproved and CDSM-improved soft clays. The test results confirmed that the CDSM method can improve the seismic behavior of pile foundations and demonstrated that the pile response is related to the size of the improvement zone.

CENTRIFUGE TESTING

Material properties

According to Peck et al. (1974), soft clays have undrained shear strength lower than 25 kPa. To satisfy the requirements of the testing soil, i.e., low strength and acceptable permeability, commercially available Kaolin (No. 1 Glaze clay from the Old Hickory Clay Company in Kentucky) and a white fine sand from the George Townsend & Co. Inc. in Oklahoma were selected for mixing. The mixture consisted of 50/50 Kaolin/sand by weight with an initial water content of 64% (or 2 LL). The coefficient of consolidation of the mixture was 4.5 x 10^{-8} m^2/s.

The recipe for the CDSM was selected as follows: Kaolin:sand ratio 1:1 and the mixing water content for the Kaolin/sand mixture of 34%, the cement-dry soil mix ratio of 1:10 by weight and the water-cement weight ratio of 1:1. After mixing the Kaolin/sand and the cement with water separately, the cement slurry was added to the soil-water mixture to get a uniform soil. The unconfined strength of the cement/soil mix was about 380 kPa after 7-day curing, 470 kPa after 14-day curing and 650 kPa after 28-day curing, respectively.

The piles were aluminum tubes (type: 6061-T4) with an O.D. of 15.875 mm and a wall thickness 0.889 mm. The length of the pile was 609.6 mm. The modulus of elasticity of the pile was 66.1 GPa and the yield strength was 167.5 MPa. On top of the piles, a steel seismic mass (0.55 kg) was attached for dynamic loading test.

Model construction and instrumentation

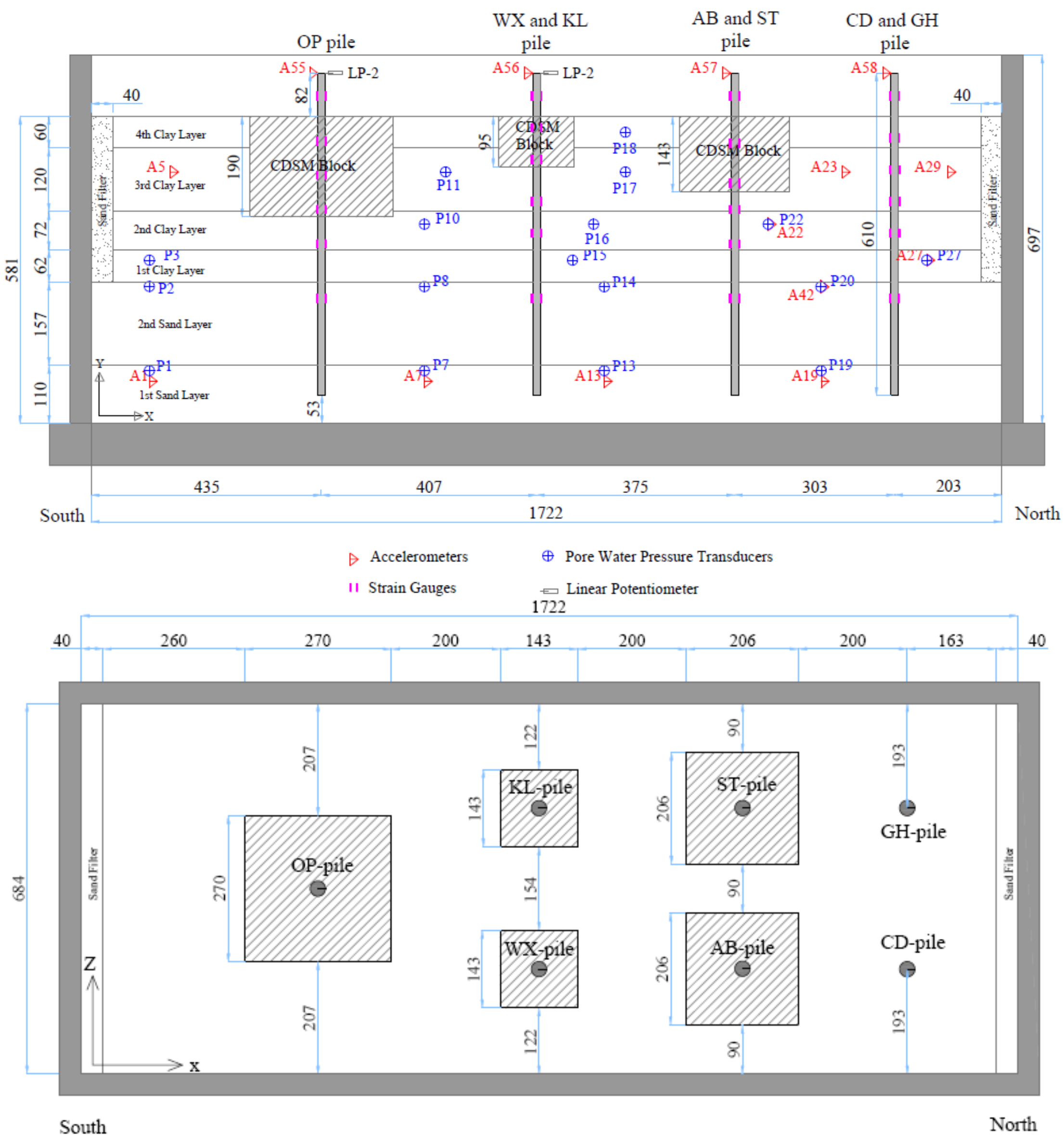


Figure 1. Side view and plan view of the centrifuge model (unit: mm, selected instruments are shown for clarity)

The centrifuge model was prepared at the Center for Geotechnical Modeling at UC Davis. The model configuration is presented in Figure 1 in the model scale. The inside dimensions of the model container FSB1 (Flexible Shear Beam 1) were 1722 mm long, 697 mm deep and 684 mm wide. All tests were performed at a centrifugal acceleration of 30 g. Therefore, prototype dimensions can be obtained by multiplying the model dimensions by 30.

Two dense Nevada sand layers were prepared via dry pluviation. The bottom sand layer had a thickness of 110 mm with a void ratio of 0.567, while the second sand layer was 157 mm thick with a void ratio of 0.531. The dense sand layers were for drainage during clay consolidation and to anchor the piles. After the sand layers were built, the sand layers were fully saturated with de-ionized water. Then the soft clay mixture for the first clay layer was poured into the container and was consolidated under vertical effective stress of 190 kPa. Similarly, three more clay layers were poured into the container and consolidated to target vertical effective stresses of 150 kPa, 55 kPa and 25 kPa, respectively. The final thicknesses of the four clay layers after consolidation were 67 mm, 72 mm, 120 mm and 60 mm in model scale. Some selected instruments are also shown in Figure 1. Preliminary numerical analyses carried out by Kirupakaran et al. (2010) were used to decide the sensor locations. On both ends of the container, 40 mm coarse sand was placed along the clay layers to expedite consolidation.

After consolidation and instrumentation, excavation was carried out and the precast CDSM blocks were placed in the pre-determined locations. The dimensions of the CDSM blocks were: 9D×9D×6D, 13D×13D×9D and 17D×17D×12D (Note: D is the O.D. of the pile and D = 15.875 mm). The piles were then driven through the holes, which have a slightly larger diameter than the O.D. of the instrumented piles at the center of the CDSM blocks. The gap between the pile and the CDSM block was filled with a grouting material (Plaster of Paris) to ensure a rapid and good contact between the piles and the CDSM blocks. During the whole test, the water table was kept at 10 mm above the soil surface.

Test sequence

For this centrifuge model, cyclic lateral loading tests on unimproved pile GH and CDSM-improved pile ST (see Figure 1) were performed. During the cyclic lateral loading tests, 10 small horizontal displacements were applied to the pile top before the maximum lateral displacement was reached. After the lateral loading tests were completed, the seismic masses were bolted on top of each pile and base shakings were applied. In total, three motions were applied: two Santa Cruz motions with peak base accelerations of 0.15g and 0.3g (prototype scale), respectively and one large Kobe motion with a peak base acceleration of 0.7g (prototype scale).

DATA ANALYSIS

The centrifuge test results are presented and discussed below. Please note all dimensions hereinafter are presented in prototype scale unless stated. A complete description of the scaling laws can be found in Kutter et al. (1992).

Static loading test results

The moment vs. depth curves for the unimproved pile GH under a series of lateral pushing and pulling tests are presented in Figure 2. Displacements from the south end to the north end of the container are considered positive. In general, the moments from a depth of 0 to 5.0 m vary linearly. The pile appears stiff with respect to the soft clay, especially the top two layers.

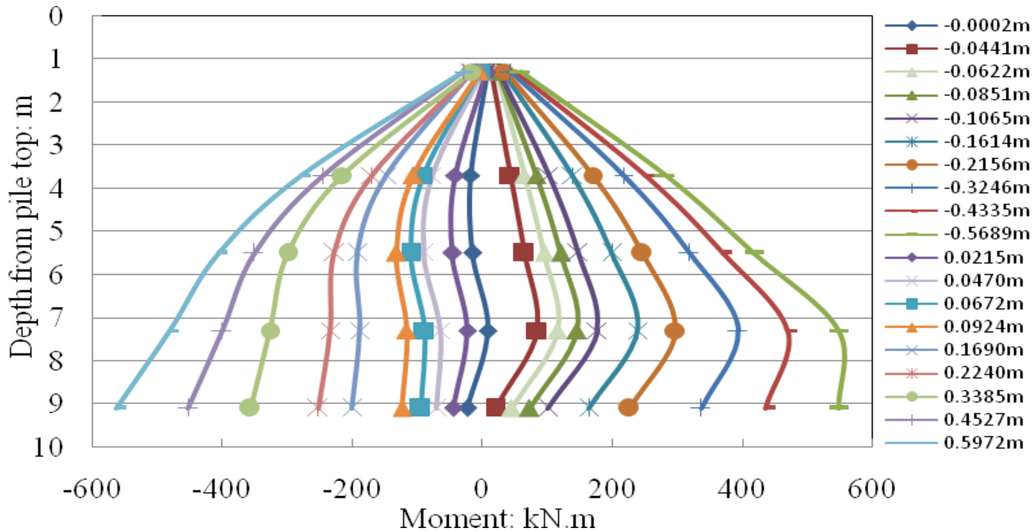


Figure 2. Moment vs. depth for unimproved pile GH

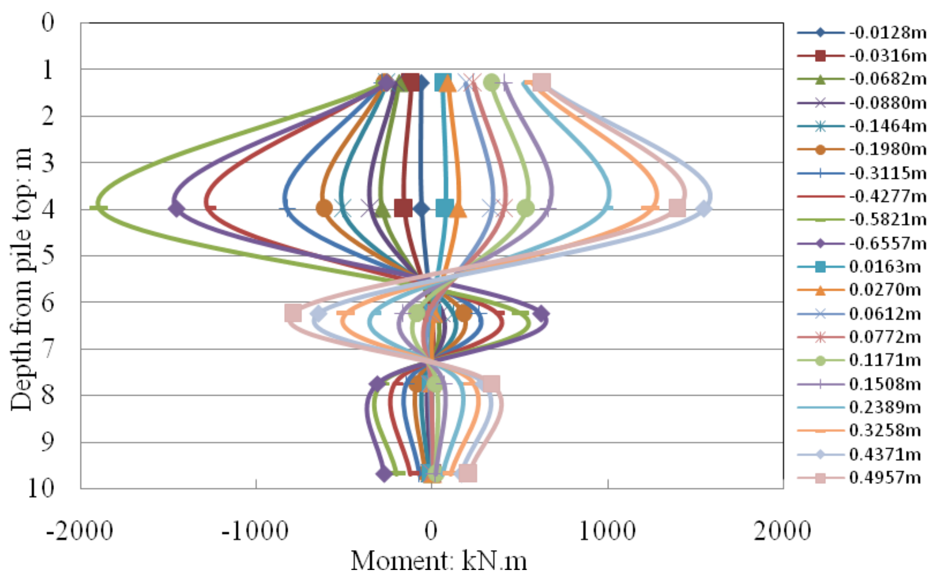


Figure 3. Moment vs. depth for CDSM-improved pile ST

The moment vs. depth curves for the pile ST with medium CDSM improvement are presented in Figure 3. The moment distribution along the CDSM-improved pile was very different from the unimproved pile. The significant difference came from the existence of the CDSM block. The maximum bending moment along the pile ST was about 2000 kN.m when the lateral displacement was about 0.6 m. However, for the same lateral displacement, the maximum bending moment along the unimproved pile GH was less than 600 kN.m, significantly lower than that for the CDSM-improved pile ST. Compared with the unimproved pile, it required a much larger lateral force to push or pull the CDSM-improved pile to the same lateral displacement.

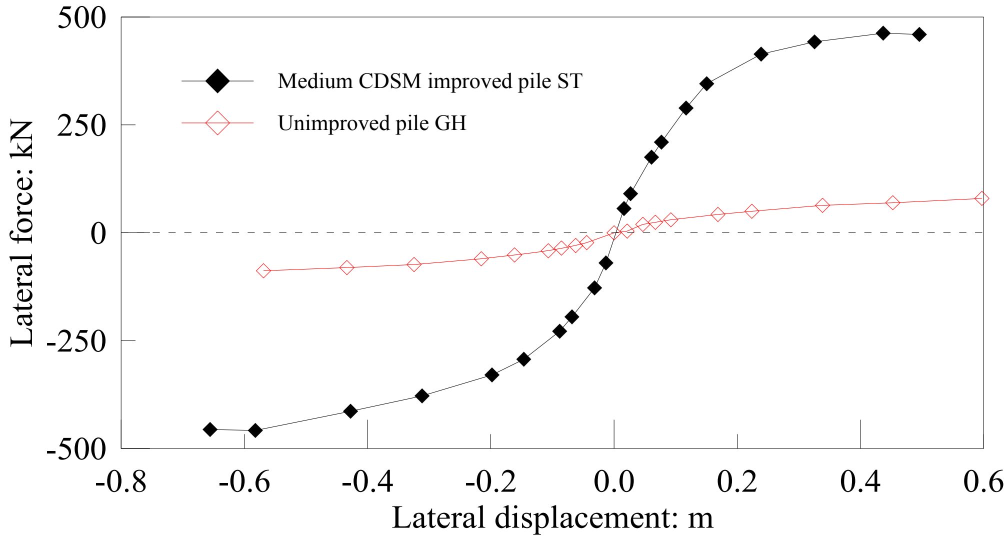
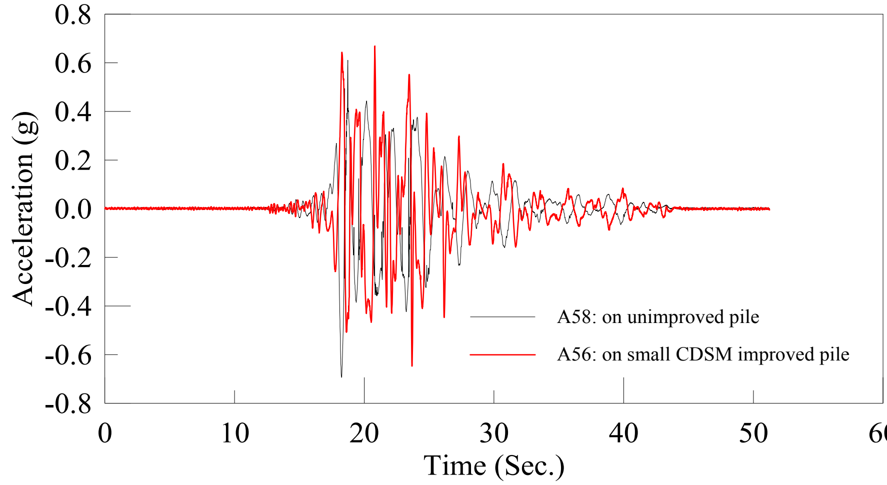


Figure 4. Deflection vs. force for piles GH and ST

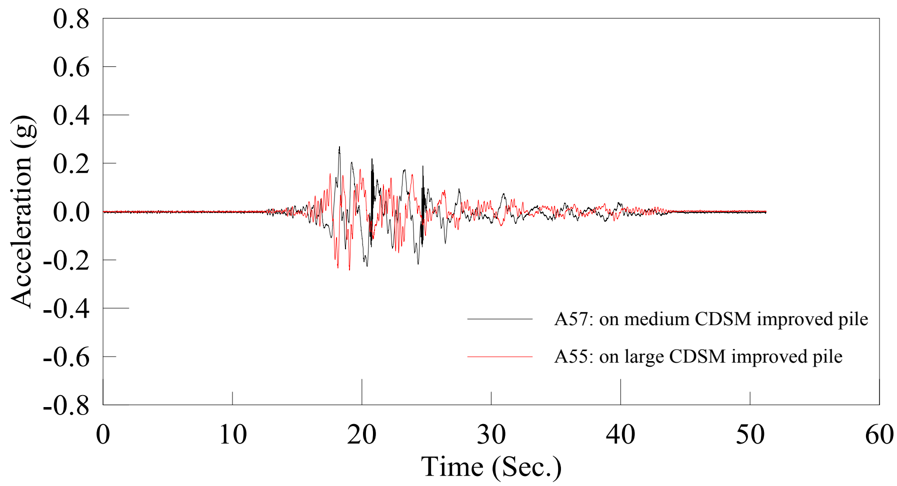
Figure 4 presents the deflection vs. force curves for the piles GH and ST. It was obvious that the CDSM block significantly increased the lateral stiffness of the pile. To reach the same lateral displacement, much higher lateral force had to be applied on top of the CDSM-improved pile than the unimproved pile. The static lateral loading tests confirmed that the CDSM method can improve the lateral stiffness of the pile significantly and the lateral displacement can be effectively reduced.

Dynamic loading test results

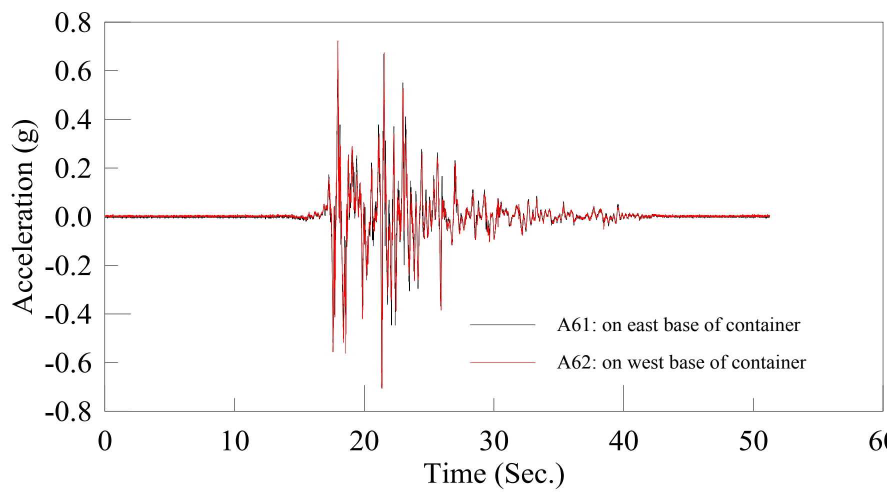
Selected test results from the Kobe earthquake are reported here. Figures 5(a) through 5(c) present the acceleration vs. time from the accelerometers at different locations. All the accelerometers were used to measure the horizontal accelerations. A58 was attached on the unimproved pile CD. A56 was attached on the pile WX with the small CDSM block, while A57 and A55 measured the accelerations on top of the piles AB and OP with the medium and large CDSM blocks, respectively. A61 and A62 represented the accelerations on the east base and the west base of the container. The peak base acceleration of the Kobe earthquake was 0.7g, which is shown in Figure 5(c).



(a) Accelerations vs. time from A56 and A58



(b) Accelerations vs. time from A55 and A57



(c) Accelerations vs. time from A61 and A62

Figure 5. Acceleration vs. time for the Kobe earthquake motion

It is of importance to note that the peak accelerations on top of the unimproved pile and the pile improved by a small CDSM were very close to the peak base acceleration, while the peak accelerations on top of the other two piles, i.e. the piles improved by the medium and the large CDSM blocks, were much smaller than the peak base acceleration. During the same earthquake, the acceleration on top of the pile AB which was improved by the medium CDSM block was about 0.3 g, while the acceleration on top of the pile OP with large CDSM block was only 0.17 g. The effect of the CDSM blocks on decreasing the maximum accelerations on top of the piles was very clear. The larger the improvement dimensions, the larger the acceleration reduction.

Figure 6 presents the excess pore water pressures vs. time within the clay. The static vertical effective stresses at the sensor locations are also provided here. It can be seen from Figure 6 that the pore pressure generation was not significant within the clay layers. During shakings, the deeper locations generally generated higher excess pore water pressures. The excess pore water pressures at different locations (e.g. P11, P16 and P17) were about 20% of the initial vertical effective stresses. The locations P16 and P11 showed substantially more cyclic pore pressures than the location P17. The negative pore pressures registered at various locations are related to the dilation of the slightly overconsolidated soil during cyclic loading. In general, pore pressures recorded within the sand layers were negligible.

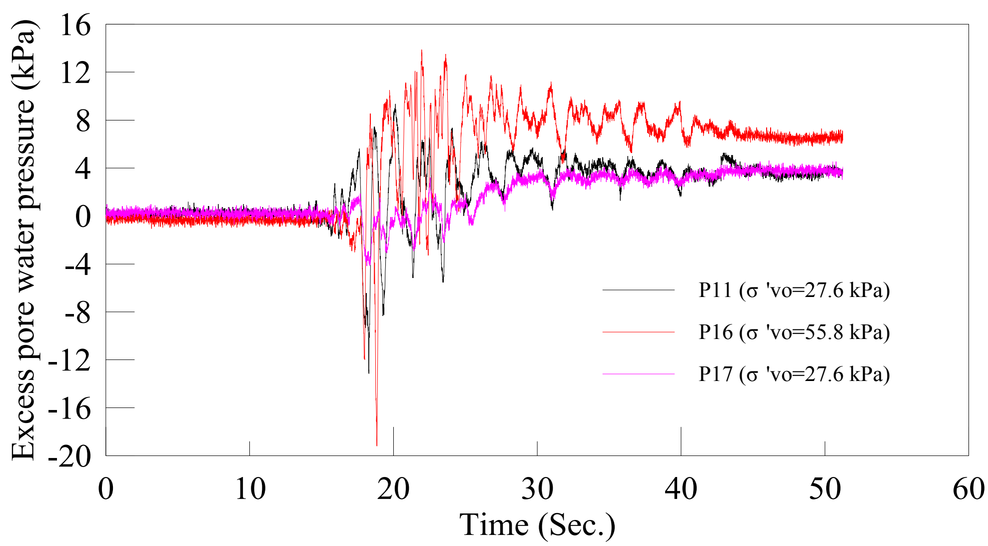


Figure 6. Excessive pore water pressure vs. time during the Kobe earthquake

Figure 7 shows the lateral displacements of the two piles improved by CDSM during the Kobe earthquake. It is quite clear that the lateral displacement was directly related to the dimensions of the CDSM block. The pile with the bigger CDSM block underwent lower lateral displacements during shaking. The maximum displacement of the pile improved by the small CDSM block was almost two times that of the pile surrounded by the large CDSM. The test results confirmed that the lateral displacement can be effectively controlled with CDSM under both static and dynamic loading events.

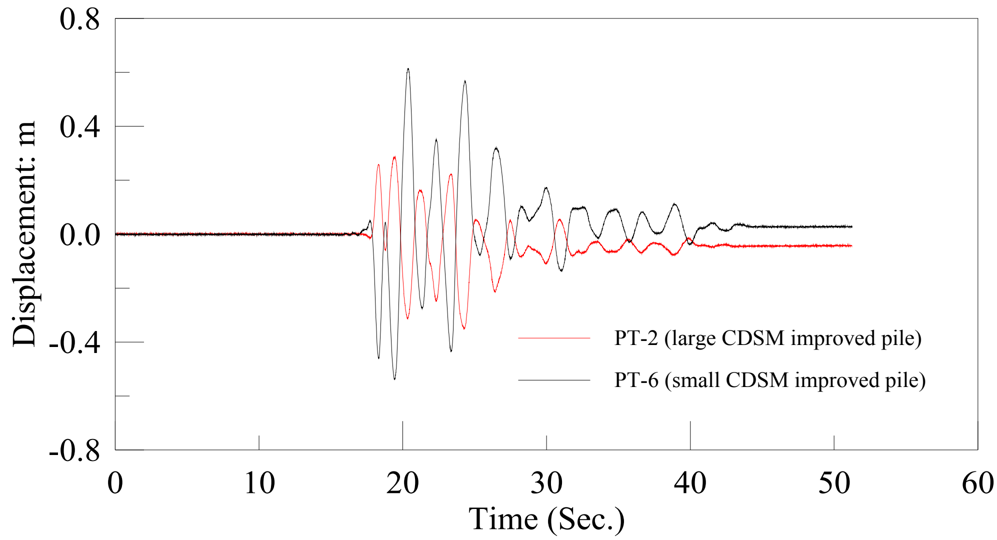


Figure 7. Displacement vs. time during the Kobe earthquake

CONCLUSIONS

The static and dynamic loading tests on single piles with different CDSM configurations confirm that CDSM is an effective way to increase the lateral stiffness of the pile foundation and the lateral displacement can be significantly reduced using CDSM. The magnitude of the lateral displacement is directly related to the dimensions of the CDSM block. The bigger the CDSM block, the smaller the dynamic displacements. The peak acceleration of the top of the pile during an earthquake can also be significantly reduced by the CDSM method.

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