Notice of Retraction

2016 International Conference on Smart City and Systems Engineering (ICSCSE)
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2016 International Conference on Smart City and Systems Engineering
10.1109/ICSCSE.2018.00045

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2018 3rd International Conference on Smart City and Systems Engineering (ICSCSE)

Experimental Assessment on Seismic Failure Modes of Bridges in Liquefiable Ground with or without Overburden Crust

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Abstract—This paper focuses on the impact of overburden clay crust on seismic failure modes of bridges in liquefiable ground by one-g shake-table tests. Two identical pile-group-supported bridge specimens are embedded separately into saturated sand deposits with or without overburden clay crust and both are subjected to a scaled ground motion obtained from the 1999 Chi-chi Earthquake. Excess pore pressure, structural acceleration and curvature demands are interpreted to reveal the seismic failure modes. Test results show that overburden crust can significantly affect the failure modes by the way of transferring damage from pile heads to piles at crust-sand interface and pier bottoms.

Keywords- Bridge; Liquefaction; Overburden Crust; Seismic Failure Mechanism

I. INTRODUCTION

Pile foundations for bridges may be embedded into soil deposits involving clay crust overlying saturated sands, which may liquefy under earthquakes, triggering variations of soil strength and stiffness. These variations can affect vertical and lateral soil resistances, which, together with the complex soil-pile interactions in the clay crust, may cause damage to pile foundations and associated collapses of bridges. Despite great efforts have been made in past few decades to reveal the seismic behavior of such bridges, recent earthquake disasters still show the lack of knowledge on this issue [1], especially the unclearness on seismic failure modes. Previous studies mainly focused on the seismic behavior of piles or pile-supported structures in liquefied and laterally spreading ground [2]–[6], where the lateral spreading occurs due to a sloping crust overlying liquefiable sand layers. To simplify this problem as well as provide novel insights into the seismic failure mechanism of bridges in liquefiable soils with overburden crust, it is reasonable to isolate the impact of overburden crust from the whole soil-bridge systems. In addition, opportunities exist that bridges in liquefiable soil with overburden clay crust may become those without the clay crust due to scour hazard. Therefore, it is an emerging topic to evaluate the impact of overburden crust on the seismic failure modes of bridges in liquefiable ground.

This study designs a pair of shake-table tests to isolate the impact of overburden clay crust on the seismic failure modes of bridges. To this end, two identical 2×2 reinforced concrete (RC) pile-group-supported bridge specimens are constructed and embedded separately into liquefiable soil deposits with or without overburden clay crust and both are subjected to a scaled ground motion originally obtained from the 1999 Chi-chi Earthquake. Representative test results, including excess pore water pressure, acceleration and curvature demands, are interpreted to reveal the seismic failure modes of the test specimens. It is worth noting that this study concentrates on aclinic ground scenarios. Sloping ground scenarios that may trigger lateral spreading will be assessed in a future paper.

II. SHAKE TABLE TEST PROGRAM

A. Overview of test setup

In order to isolate the impact of overburden clay crust, two shake-table tests are designed, where two identical bridge specimens are embedded separately into liquefiable soil deposits; one with and the other without the overburden clay crust.

Figure 1 physically shows the shake-table tests prepared for testing. Dimensions of the laminar box are 2.0 m (length) × 1.5 m (width) × 2.0 m (height). Boundary effects of the laminar box are negligible at distances of 40 cm from the container wall [7]. This laminar box has been used for shake-table tests on assessing soil-structure interactions [8]–[10].

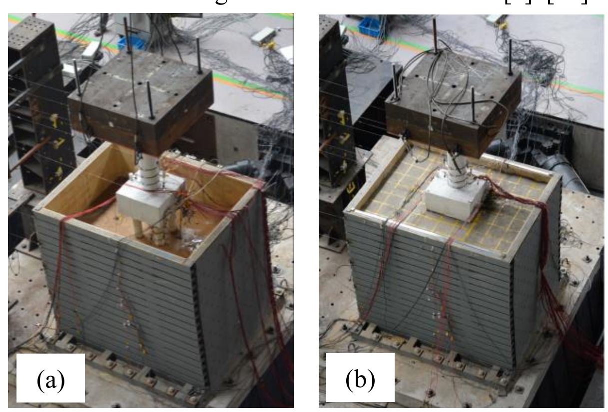

Figure 1. Overview of the shake-table test: (a) without overburden clay crust; and (b) with overburden clay crust

Figure 2 schematically shows the shake-table tests and associated instrumentations. The bridge specimen consists of a 2×2 RC pile-group with pile diameters of 0.1 m (d), center-to-center distances of 3d and lengths of 19d. The pile tips stand on the base of the laminar box by concreting with a pre-embedded steel plate with four collars. The pile heads are connected through an RC cap with dimensions of 0.6 m (length) × 0.6 m (width) × 0.3 m (height), which supports a single pier with a diameter of 0.214 m (D) and a height of 1

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DOI 10.1109/ICSCSE.2018.00045

m. A 4-ton iron block is fixed to the pier head to represent the bridge superstructure. For the case without overburden crust, piles are embedded into a homogenous saturated medium dense sand layer with a thickness of 15*d*, whereas for the other case, a 4*d*-thickness clay crust layer (undrained shear strength of $S_u \approx 60$ kPa) overlies an almost identical sand layer (both cases with a relative density of $D_r \approx 50\%$).

Before seismic excitations, white noise excitations are applied to obtain fundamental periods of the specimens; those are 0.51 sand 0.38 s for cases without and with the overburden crust, respectively. These periods generally fall into the common range of fundamental period for ordinary multi-span girder bridges (0.26 ~ 0.65 s) [11].

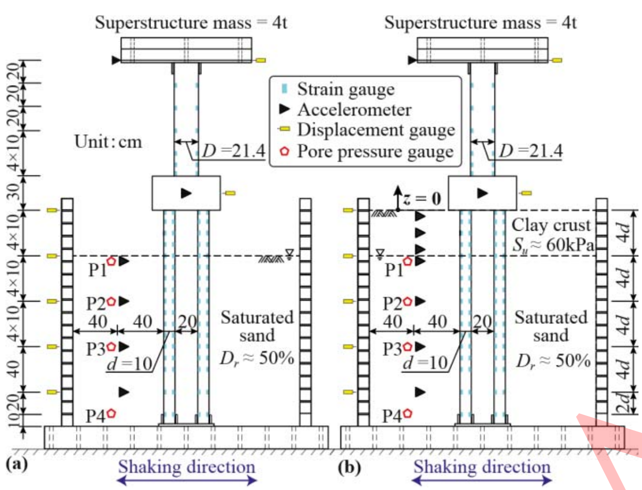

Figure 2. Schematic of the shake-table test: (a) without overburden clay crust; and (b) with overburden clay crust

B. Structural and soil properties

Based on a survey of column and pile sections of bridges in practice, a longitudinal reinforcement ratio of 2% and a transverse reinforcement ratio of 0.8% are assigned to the specimens (both piers and piles). More specifically, for the pier, longitudinal reinforcements are provided by ten ϕ10-mm rebars with a concrete cover of 2 cm. Spiral ϕ6-mm stirrups are arranged with an interval of 6 cm for confinements. With regard to the piles, six ϕ6-mm rebars are assembled as longitudinal reinforcements with a concrete cover of 1 cm. Confinements of the piles are supplied with spiral ϕ3.5-mm stirrups with an interval of 4 cm.

Shanghai sands are used to prepare a fully saturated soil profile. Tests on the particle size distribution of the sands indicate a poorly graded property with a mean grain size of 0.33 mm, a coefficient of uniformity of 2.06, maximum and minimum dry densities of 1.654 g/cm^3 and 1.429 g/cm^3, respectively.

To prepare a homogeneous saturated medium dense sand deposit with $D_r \approx 50\%$, the rain drop technique is used to prepare the liquefiable sand deposit layer by layer (10 cm-thickness for each layer). Details for the ground preparation procedure can be found in Wang et al. [10].

C. Instrumentation

Figure 2 illustrates the instrumentation of the shake-table tests. Amounts of different sensors, including accelerometers, displacement, strain and pore pressure gauges, are adopted to obtain excess pore pressure ratio, acceleration and curvature demands during the test.

D. Adopted ground motion

This paper presents the results of shake-table tests under the Chi-chi 0.3g motion, which is originally obtained from the 1999 Chi-chi Earthquake. Figure 3 shows its acceleration time history and corresponded acceleration spectrum. As can be seen that this ground motion contains amounts of high frequency contents, with a predominant period of 0.24 s.

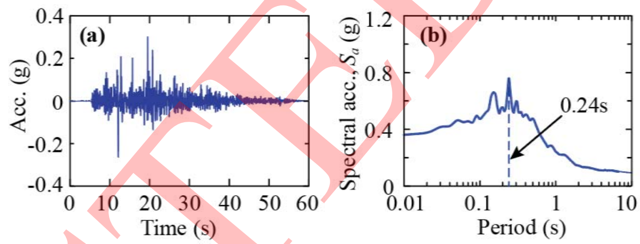

Figure 3. Adopted Chi-chi 0.3g ground motion: (a) acceleration time history; and (b) acceleration spectrum

III. Test Result Interpretation

A. Excess pore pressure ratio

Figure 4 compares excess pore pressure ratio (r_u) results at different locations between two cases (with and without overburden crust). r_u is defined in Equation (1).

$$r_u = \frac{\Delta u}{\sigma'_{v0}} \tag{3}$$

where Δu is the recorded excess pore pressure and σ'_{v0} is the initial vertical effective stress.

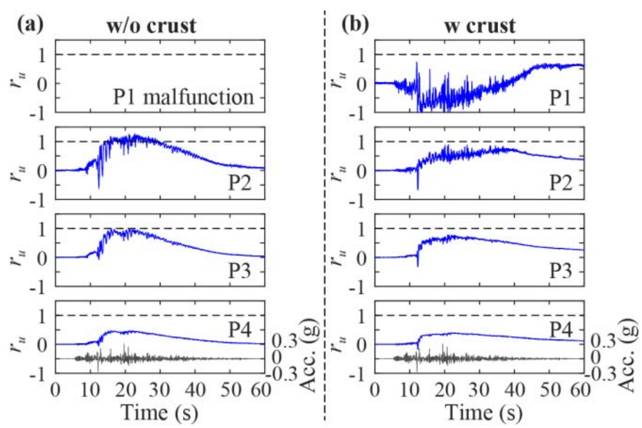

Figure 4. Excess pore pressure ratios: (a) without overburden clay crust; and (b) with overburden clay crust

It can be found from Figure 4 that the sand liquefied under the Chi-chi 0.3g ground motion for both cases (r_u mostly approaches 1.0). Moreover, r_u in the case without crust develops larger than that in the case with crust. This is attributed to the fact that the overburden crust significantly increases σ'_{v0}. In other words, the existence of overburden clay crust shows a tendency to impede the development of r_u. In addition, both cases show dilation tendency (immediate reduction of r_u) during the development of r_u. For example, r_u at P2 for the case without crust immediately drops at around 12 s, even to a negative value, and then recover soon. These fluctuation phenomenon (dilation tendency) occurs several times during the development of r_u.

B. Soil Acceleration

Figure 4 compares peak soil accelerations along depth between these two cases. It is clear that a generally amplified tendency is observed from the bottom of the laminar box to the soil surface for the case without crust, as expected. By contrast, an apparent maximum acceleration is observed at the crust-sand interface, which is due to large differences of strengths and stiffnesses between these two soil layers.

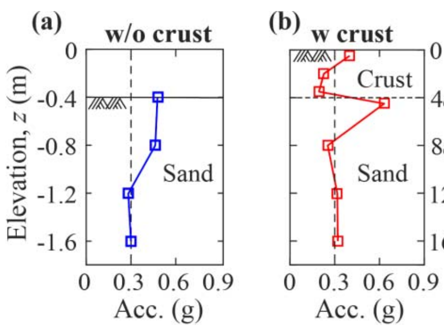

Figure 5. Peak soil accelerations along depth: (a) without overburden clay crust; and (b) with overburden clay crust

C. Structural Acceleration

Figure 6 shows superstructure acceleration time histories and corresponded spectral acceleration ratios for these two cases under Chi-chi 0.3g. From Figure 6(a), it is seen that peak accelerations (× in Figure 6) occur at moments where dilation tendency take place. Figure 6(b) indicates that the case with crust exhibits a smaller predominant period (i.e., period at the peak spectral acceleration ratio) as compared to the case without crust. Recalling the acceleration spectrum of the Chi-chi 0.3g ground motion, the case with smaller predominant period consequently displays a larger spectral acceleration ratio, as expected.

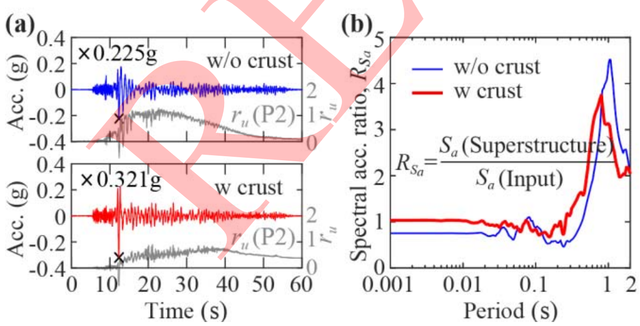

Figure 6. Superstructure accelerations for cases with and without overburden clay crust: (a) Time histories; and (b) spectral acceleration ratios

D. Curvature Distribution and Failure Mode

Figure 7 illustrates the curvature distributions along the pier and one of the piles, which are obtained from the pairs of strain gauges at the same elevation. Note that pile group effects are not apparently detected for the studied pile center-to-center distance of 3*d* in liquefiable soil. It is seen that the most vulnerable part for the case without crust occurs at the pile head above the soil surface. In addition, a potential underground damage area is detected at a depth around 6*d*. By contrast, in the case with crust, curvature demand at the bottom of pier significantly increases, since the crust improves the lateral stiffness of the bridge specimen, which leads to a smaller fundamental period that triggers a larger demand at the pier. As for the pile portion, the curvature demand at pile head significantly decreased while that at the crust-sand interface greatly increased. The former result is because that the rotation of pile head is restrained by the crust. The later phenomenon is attributed to the larger differences of lateral resistances and stiffnesses between the crust and the sand, which corresponds to the soil acceleration responses mentioned above (Figure 5). In general, it can be concluded that the crust can transfer the seismic damage from pile heads to piles at crust-sand interface and pier bottoms.

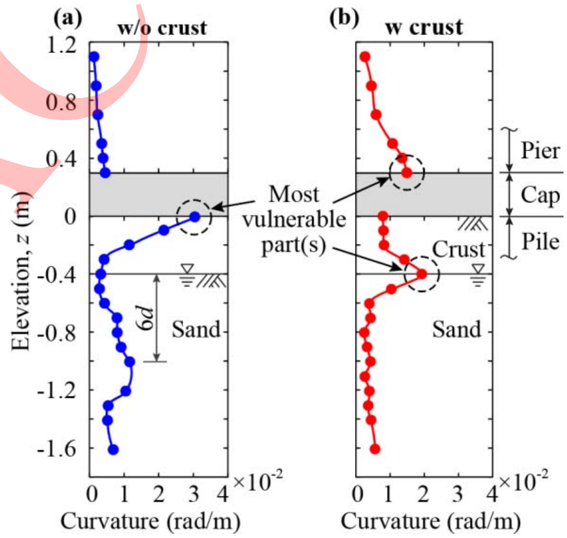

Figure 7. Curvature envelopes along depth: (a) without overburden clay crust; and (b) with overburden clay crust

IV. CONCLUSIONS

This paper presents the results of a pair of shake-table tests, where 2×2 pile-group-supported bridge specimens are embedded separately into liquefiable soil deposits with or without overburden clay crust. Representative test results, including excess pore water pressure, soil and structural acceleration and curvature demands, are interpreted to reveal the seismic failure modes of the test specimens. It is found that the overburden crust can significantly affect the failure modes by the way of transferring damage from pile heads to piles at crust-sand interface and pier bottoms.

Acknowledgements

This work is supported by NSFC (Grant No. 51778469), China Postdoctoral Science Foundation, and Fundamental Research Funds for the Central Universities.

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