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Centrifuge Modeling of Cyclic Softening in Low Plasticity Clays Partially Induced by Seismic Soil-Structure-Interaction

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ABSTRACT

Seismic soil-structure-interaction (SSI) has significant influence in seismically-induced ground failure; however, despite recent advances, the contribution remains poorly understood and is routinely neglected in geotechnical engineering practice. Observations from the 1999 Kocaeli, Turkey and 1999 Chi-Chi, Taiwan Earthquakes indicates SSI effects may be pronounced in the cyclic softening-induced ground failure of low-plasticity, fine-grained foundation soils. The authors recently completed a centrifuge modeling program that seeks to fill this knowledge gap by evaluating the SSI-based seismic demand imparted by shallow foundations on fine-grained soils and the corresponding contribution to cyclic softening-induced ground failure. This paper presents preliminary results from the modeling program, including observed peak ground acceleration (PGA), peak structural acceleration, and structure settlement. The influence of SSI effects is demonstrated by comparing observed structural settlement to structure factor of safety evaluated using dynamic bearing capacity calculations.

INTRODUCTION

Earthquake-induced ground failures are a source of significant economic loss and life safety risk during earthquakes. Ground failure potential is typically evaluated based on demands arising solely from vertically propagating shear waves in the free-field (i.e. site response). These demands are compared with the cyclic soil strength to evaluate failure potential. However, for many infrastructure systems, seismic demands arise from both site response and soil-structure interaction (SSI). SSI effects can both increase and decrease demands relative to those from site response alone (e.g., Dashti et al. 2010a, b; Hayden et al., 2015), although these SSI effects are generally not considered in engineering practice.

Ground failures related to soil strength loss can broadly be categorized as exhibiting “sand-like” (e.g. liquefaction) or “clay-like” (e.g. cyclic softening) behavior. Case histories of structural damage due to liquefaction of sand-like soil are many and typically involve surface evidence of liquefaction both in the vicinity of the structure and in the free field. By contrast, cyclic softening of clay-like soil has been observed to occur beneath structures of considerable mass (generally > 3 stories), but not in the free-field during the 1999 Chi-Chi, Taiwan Earthquake (e.g., Chu et al., 2008) and the 1999 Kocaeli, Turkey Earthquake (Bray et al., 2004). This suggests that SSI plays an important role in creating the conditions whereby cyclic softening develops below shallow foundations for fine-grained, plastic soils.

The authors recently completed a centrifuge modeling program to study the influence of SSI effects on cyclic softening. The program consists of two centrifuge tests, designated JZB01 and JZB02, performed at the Center for Geotechnical Modeling (CGM) at the University of California, Davis. The centrifuge tests incorporated three structures of various mass and stiffness properties, but with approximately the same contact pressure imparted by strip footing foundations that supported the structures. The design and performance of a structure with a single column, Structure 1, is discussed herein to demonstrate the goals and insights from the testing program. The two other structures included in each model were single-bay portal frames; however, results from these structures are omitted here for brevity.

CENTRIFUGE TESTING PROGRAM

Centrifuge tests JZB01 and JZB02 were performed on the 9-meter-radius centrifuge at the CGM. Test JZB01 was performed at an induced gravity due to centrifugal acceleration, $N = 40g$; test JZB02 was performed at $N = 57g$. Because the two models were tested at different N values, results are presented in model scale units when discussing model configuration, and in prototype scale units when discussing recorded data or bearing capacity analysis. Centrifuge scaling relationships used to compute prototype units are presented by Kutter (1995). Data reports available on the platform DesignSafe summarize testing details for tests JZB01 (Buenker et al., 2019a) and JZB02 (Buenker et al. 2019b).

JZB01 Model Configuration. The JZB01 model consisted of bentonite-based clay overlying dense sand. Structures were founded on a thin surficial sand layer overlying the clay. The model was constructed in the CGM’s hinged plate container (HPC). The HPC consists of a solid aluminum base with aluminum bulkheads along the long container axis. Aluminum annular ring assemblies mounted on runners in the bulkheads form the container short axis walls. The HPC rings exhibit low shear stiffness and may be deformed by hand when empty. Figure 1 presents the JZB01 model. Instruments in the figure with the designation “A-” are integrated circuit piezoelectric (ICP) accelerometers, instruments with the designation “P-” are pore pressure transducers, and instruments with the designation “LP-” are linear potentiometers.

Figure 1. JZB01 Model Configuration (model scale units).

JZB02 Model Configuration. The JZB02 model consists of layered kaolinite-based clay overlying dense sand. Dense sand lenses existed between the clay layers. Structures were founded on a thin surficial sand layer overlying the clay. The model was constructed in the CGM's flexible shear beam (FSB) container. The FSB consists of a solid aluminum base with semi-rigid side walls composed of stacked aluminum rings. The rings are separated by layers of Neoprene rubber. The FSB is flexible relative to soil but is less flexible than the HPC and cannot be readily deformed by hand. Figure 2 presents the JZB02 model.

Figure 2. JZB02 Model Configuration (model scale units).

Clay Properties. The JZB01 clay consisted of a synthetic blend composed of 95 percent Sil-co-sil ground silica and 5 percent bentonite, with a plasticity index (PI) of 14. The JZB02 clay consisted of a synthetic blend composed of 78 percent Sil-co-sil ground silica and 22 percent kaolinite, with a PI of 9. Each clay was consolidated from slurry on the CGM's hydraulic press prior to centrifuge testing. The maximum consolidation pressure was 60 kPa for JZB01, and 140, 180, and 200 kPa for the top, middle, and bottom layers, respectively for JZB02.

JZB01 and JZB02 clays were based on SBFW and SKFW synthetic blends, respectively, developed and tested by Eslami (2017). Based on direct simple shear test data, Eslami expressed clay undrained shear strength, s_u normalized by effective vertical consolidation stress, σ'_{vc} as a function of overconsolidation ratio, OCR :

$$\text{JZB01 Clay (SBFW): } \frac{s_u}{\sigma'_{vc}} = 0.213OCR^{0.718} \quad (1)$$

$$\text{JZB02 Clay (SKFW): } \frac{s_u}{\sigma'_{vc}} = 0.198OCR^{0.594} \quad (2)$$

Using the above relationships, the average undrained shear strength for the JZB01 clay layer as 15 kPa and for the JZB02 clay layer as 22 kPa. These values are the average undrained shear strengths for one footing width below the bottom of the Structure 1 footing, and account for free-field consolidation stresses. The structures exert additional increments of total and shear stress that are not considered in estimating the undrained strength.

Structure 1. Structure 1 consisted of a 24-inch-long (model scale) A36 steel plate mounted to the top flange of a 6061 – T6 Aluminum Association standard I-beam. The steel plate and aluminum I-beam were connected by high strength steel bolts. Total structure mass was 12.45 kilograms. Model-scale structural frequency was approximately 37 Hz; meaning, the prototype-scale structural frequency was 0.92 Hz and 0.65 Hz for tests JZB01 and JZB02, respectively. Structure response during testing was monitored using ICP accelerometers, strain gages, and linear potentiometers.

Ground Motions. Scaled versions of four different motions (Step, Sweep, TCU, and Kobe) were applied to the centrifuge models during spinning using the servo-hydraulic shakers about the long axis of the model container (i.e., in the left-right direction in Fig. 1). Figure 4 presents the peak ground acceleration (PGA) of each motion in the far field. The figure also presents peak acceleration experienced by Structure 1. Accelerations were measured by the ICP accelerometers indicated in the legend of each figure. ICP positions for base and far field PGAs were presented in Figures 1 and 2. ICP position A1 is at the top of Structure 1. As the base shaking intensity increases, the ratio of structural acceleration to PGA decreases. This is an

indication of nonlinearity in the system. Since the structures remained elastic during shaking, the nonlinearity arises from soil-foundation-structure interaction due to nonlinear soil response.

Figure 4. Peak Acceleration from Ground Motions (prototype scale).

BEARING CAPACITY EVALUATION

Structure 1 was designed based on preliminary calculations that utilize a limit equilibrium procedure to compute the factor of safety against bearing failure. The critical design considerations were (i) the factor of safety should be adequately high immediately after spin-up, before the clay had time to consolidate under its own self-weight and the structural load, and (ii) the factor of safety should be low enough that SSI effects would be anticipated to induce ground failure. Ground motions were modeled as inclined eccentric loads applied as seismically induced shear, V_e , bending moment, M_e , and eccentrically positioned axial loading, P demand, as shown in Figure 5. These loads were evaluated based on peak structural acceleration at the top of the structure (ICP position A1) and bending strain in the structure web (strain gage position SG1).

The corresponding ultimate capacity provided by the soil for a strip footing founded on cohesive, frictionless clay is:

$$q_{ult} = 5.14s_u i_c + q \quad (3)$$

where i_c is the load inclination correction and q is the overburden surcharge. The footing is lighted embedded ($D_f < B_f$) with groundwater depth, D_w at footing depth ($D_w \sim D_f$); therefore,

depth correction factors accounting for the shearing resistance of overburden material and groundwater correction factors were not incorporated.

Figure 5. Structure 1 Configuration for Bearing Capacity Evaluation

With i_c by Vesic (1975), the equation expands to:

$$q_{ult} = \left(5.14s_u - \frac{2V_e}{B'_f} \right) + q \quad (4)$$

FS against bearing capacity failure is therefore:

$$FS = \frac{q_{ult}}{q_a} = \frac{q_{ult}B'_f}{P} \quad (5)$$

We used this framework to evaluate static FS and the peak structural acceleration needed to generate seismic FS equal to one for Structure 1. Table 1 summarizes our design results from this analysis procedure.

Table 1. Summary of Structure 1 Properties

Centrifuge Test	Prototype Scale B_f , (m)	Prototype Scale Static q_a , (kPa)	Prototype Scale Static q_{ult} (kPa)	Design Static FS	Design Peak Structure Acceleration Required to for Seismic FS = 1 (g)
JZB01	4.1	78	111	1.4	0.10
JZB02	5.8	111	159	1.5	0.23

STRUCTURE PERFORMANCE

Structure 1 experienced bearing capacity failure during tests JZB01 and JZB02. In each test, the failure occurred progressively, with small vertical residual settlement and tilt following low amplitude ground motions and gradually higher settlement and tilt with increasingly high amplitude motions. Figure 6 indicates the progressive settlement of Structure 1 over the suite of applied ground motions. Settlement is terminated after ground motion numbers 7 and 11 for JZB01 and JZB02, respectively, because the structure experienced full bearing capacity failure and toppled after these ground motions.

We used the limit equilibrium-type bearing capacity framework outlined in the previous section to evaluate structure performance. Figure 7 shows residual settlement versus factor of safety against bearing failure. Settlement tended to increase as FS decreased, and some settlement occurred even for cases with FS > 1.0. This is an indication that the system did not behave in a manner consistent with limit equilibrium concepts. Rather, the failure was strain-based and consisted of the progressive displacement of the foundation until failure occurred. In Figure 7, blue symbols denote cases where far-field ground failure was not predicted based on procedures by Boulanger and Idriss (2007), while orange symbols denote cases where cyclic failure is predicted. Note that the cyclic failure analysis assumes level ground conditions, and neglects static and dynamic shear stresses and total stresses exerted by the structure. Cases for which cyclic failure was predicted did have higher settlements than cases for which cyclic softening failure was not predicted.

Figure 6. Structure 1 Prototype Scale Settlement.

Figure 7. Structure 1 Normalized Settlement versus Seismic FS.

CONCLUSION

Structures shaken in a recent centrifuge model testing program were evaluated using conventional seismic bearing capacity analysis, as well as far-field cyclic softening analysis. The bearing capacity analysis indicated that settlement increased as FS decreased, but the trend was not an abrupt increase in settlement when FS crosses the threshold at 1.0. This indicates that the problem is more complicated than implied by limit equilibrium assumptions and challenges the attendant implication that foundations will be suitably stable at FS greater than one. Structural settlement was higher for cases where far-field cyclic softening failure was predicted. The limit equilibrium analyses performed herein account for stresses induced by SSI, whereas the far-field cyclic softening analysis accounts for stresses induced by vertically propagating shear waves. In reality, both mechanisms exert demands on the soil simultaneously. Therefore, an integrated approach is needed that considers demands from vertically propagation waves in addition to those from SSI. Subsequent research efforts by the authors resulting from centrifuge models JZB01 and JZB02 will be directed towards evaluating these SSI effects.

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