GEOTECHNICAL ENGINEERING ANALYSIS AND DESIGN FOR PILE RESPONSE TO LIQUEFACTION-INDUCED GROUND DEFORMATION

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SUMMARY

This paper describes an analytical model and design procedure for foundation piles subjected to large lateral ground deformation triggered by liquefaction. The model involves the use of p-y curves, but avoids the empiricism associated with the selection of degradation coefficients or reduction factors. To obtain a proper p-y characterization of the reaction between laterally deformed liquefied soil and an embedded pile, triaxial extension is recognized as the most appropriate analogue for the loading conditions. A suite of undrained triaxial extension tests was carried out using Nevada sand to establish the relevant strength and deformation parameters. Using the material parameters obtained from these tests, 2-D FLAC analyses were performed to develop strain-softening p-y curves. Application of these p-y curves to the analyses of centrifuge experiments involving lateral spread effects on piles yields good agreement between the computed and measured responses. The strain-softening model provides excellent predictions of the measured peak and residual moments. Furthermore, the computed soil pressure diagrams agree well with the recommendations made by the Japan Road Association, which were calibrated using case histories from the 1995 Kobe earthquake.

INTRODUCTION

Liquefaction-induced permanent ground deformation has been the cause of extensive damage to pile foundations of buildings, bridges, and waterfront structures. Presently, there are several methods for analyzing the response of a single pile subjected to such lateral loadings. One of the most commonly used method involves p-y curves. Application of the p-y approach for analyzing lateral spread effects requires that appropriate p-y relationships be developed for liquefied soil conditions. Currently, p-y characterization of liquefied soil relies heavily on empirical rules involving the application of degradation coefficients or reduction factors to the strength and stiffness of the unliquefied soil [Meyersohn, 1994; Chaudhuri, 1998; Ishihara and Cubrinovski, 1998].

To account for the fundamental behavior of liquefied soil in a more rigorous manner, p-y relationships need to be assessed relative to the undrained behavior of sand and its constitutive relationships. This paper provides a description of an analytical model that was developed to evaluate soil-pile interaction for conditions of liquefaction-induced lateral spread. The analytical model was developed at Cornell University in conjunction with centrifuge experiments performed at Renssalaer Polytechnic Institute (RPI). The centrifuge experiments [Abdoun, 1997] provide a means of calibrating and checking the model under controlled conditions of inertial similitude in which key soil layering, soil properties, and pile characteristics are varied to assess the model's predictive capabilities under diverse conditions of practical interest for design. This paper focuses on the stress transfer from liquefying soil to vertical piles during lateral spread and the corresponding application of this characterization in the numerical simulation of soil-pile interaction.

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MODEL ASSUMPTIONS AND IDEALIZATIONS

Figure 1a shows a condition of lateral spread in which the soil moves horizontally against the pile foundation. The resulting stress state involves complex three-dimensional stress relations and shear transfer. To reduce the complexity of the problem, the simplified interaction model proposed in this paper invokes the key assumption that stress transfer from soil to pile during lateral spread is controlled principally by static undrained loading in response to relative soil-pile movement rather than dynamic load components governed by cyclic ground shaking. Large lateral movement induced by liquefaction is accompanied by large monotonically increasing shear strains in the soil surrounding the pile. These strains are likely to exceed their cyclic counterparts, with the static loading component becoming dominant as the magnitude of lateral spread increases. By uncoupling the static and dynamic components of undrained shear deformation, the problem is simplified significantly. The assumption that pile response is controlled by static loading conditions is fully testable; static loading can be evaluated by laboratory soil testing, and the results incorporated in an analytical model that is compared against centrifuge measurements and field observations.

In Figure 1b, the loading conditions on the upstream and downstream sides of the pile are shown as being analogous to undrained triaxial extension (TE) loading and compression (TC) unloading, respectively. Because load transfer occurs principally on the upstream side, it appears that triaxial extension, as opposed to compression, is a more appropriate analogue to characterize soil-pile interaction under lateral spread involving large horizontal displacements of the soil mass. This distinction is important, as it has been widely reported by Vaid et al. [1990] and Yoshimine and Ishihara [1998], among others, that soil specimens tested in triaxial extension exhibit lower strengths and more contractive behavior then identical specimens tested in compression. By incorporating relevant soil parameters that pertain to the triaxial extension mode of loading, the proposed interaction model strives to provide a simple but realistic representation of field loading conditions.

LABORATORY TEST PROGRAM AND RESULTS

The development of the proposed interaction model begins with the strength characterization of liquefiable soil in undrained triaxial extension. An experimental program was undertaken to determine the undrained strengths and moduli of Nevada test sand prepared at several relative densities and tested at different initial confining pressures. The properties of this sand are well documented by Abdoun [1997], who prepared the RPI centrifuge models using the same granular material. Furthermore, both the triaxial specimens and centrifuge models were prepared using the method of air-pluviation, thus ensuring a consistent soil fabric in both sets of tests.

Selected results from a suite of triaxial tests are presented in Figure 2. In this study, the Dr = 40% specimens are of special interest because they correspond to the initial state of the sand prepared in the centrifuge models. The stress-strain measurements shown in Figure 2a strongly suggest that, under undrained extension, two distinct shear strengths are mobilized during the course of straining. The peak strength is mobilized at very small strains, corresponding to the maximum deviatoric stress, while the minimum strength is associated with what Ishihara [1993] has identified as the quasi-steady state condition. In the literature, this condition has also been called phase transformation, critical state, or simply, steady state. In this study, the term ‘minimum strength’ is used as a simple expedient for identifying the lowest strength condition without adopting a terminology that implies an underlying cause or mechanism. The mobilization of two distinct undrained strengths, and the transition from one to the other, are important features of the soil-pile interaction model proposed in this paper.

Undrained triaxial compression tests were carried out by Yamamuro and Lade [1997] on Nevada sand with similar relative density (Dr=42% compared with Dr=40% in this study) and grain size characteristics (% fines = 6% compared with zero fines content in this study). In contrast to the contractive behavior illustrated in Figure 2 for triaxial extension, their results showed fully dilatant behavior even at a very low confining pressure of 25 kPa. Such observations are consistent with the generally recognized fact that sand exhibits contractive behavior at much higher relative densities under triaxial extension than compression [Vaid et al., 1990; Yoshimine and Ishihara, 1998]. This aspect of soil behavior emphasizes the importance of applying the appropriate mode of loading in strength characterization tests.

Figure 2b plots the measurements in the stress space, while Figure 2c presents the data in the form of the isotropic consolidation line (ICL) and quasi-steady state line (QSSL). The same figures also show the stress and state paths for two specimens consolidated to p'_i = 100 kPa and 400 kPa. Based on the measured stress and state paths shown, Goh and O’Rourke [1999] showed that, analytically, both the peak and minimum deviatoric stresses can be linearly related to the initial effective confining pressures. Experimentally, this is supported by the correlations shown in Figures 3a and 3b. Although there is some nonlinearity of the data in Figure 3a at higher confining pressures, the overall regression is not significantly different from the results obtained using a low pressure subset, as shown in the inset on the same figure. For depths up to about 10m, such as in the centrifuge tests, the low pressure subset is a good representation of the stress conditions in the soil. The peak and minimum deviatoric stresses shown in Figure 3 can, in turn, be related to their respective undrained shear strengths to obtain the following strength-pressure relationships [Goh and O'Rourke, 1999] :

minimum strength : $s_{u\text{-}min} = 0.034\ p'_i$ (1)

peak strength : $s_{u\text{-}pk} = 0.17\ p'_i$ (2)

In the next section, these relationships are used in the numerical derivation of p-y curves.

NUMERICAL DERIVATION OF P-Y CURVES

Soil-pile interaction under lateral load is a complex three-dimensional problem. At intermediate to large depths, however, the displacements in the vertical direction are small compared to their horizontal counterparts. Under this condition, the load transfer mechanism may be modeled as a two-dimensional plane strain problem. Chen and Poulos [1993], for example, have used plane strain conditions to analyze the soil-pile interaction associated with pile groups. In this study, a similar approach is adopted, in which 2-D simulations are performed using FLAC to obtain the p-y response for a single pile laterally loaded in a strain-softening elasto-plastic medium.

Details pertaining to the numerical model, including the specification of strength and moduli parameters based on triaxial tests results, are found in Goh and O'Rourke [1999]. In Figure 4, computed normalized p-y responses are shown for the limiting conditions of perfect and no adhesion between the soil and pile. These conditions simulate a rough and smooth soil-pile interface, respectively.

The results from FLAC analyses can be compared against the theoretical values reported by Randolph and Houlsby [1984], who used limit state theories to show that the ultimate normalized loads associated with a rigid circular pile moving through a perfectly plastic, cohesive material are 11.94 (rough) and 9.14 (smooth) respectively. The FLAC results show that, due to strain softening in the soil, the peak resistances are approximately 80~85% of their non-softening counterparts. More significantly, the residual resistances mobilized in the soil are drastically reduced to 20~25% of their peak values.

The strain-softening p-y curves are also characterized by the slopes of the loading branch k_L and unloading branch k_u . For the p-y curves shown in Figure 4, the steep slopes associated with the loading branch are obtained directly from FLAC analyses. However, the unloading branches have been modified to achieve 3-D elastic stability of the pile during rebound. As explained by Goh and O'Rourke [1999], the steep unloading slopes of the p-y curves obtained from 2-D FLAC analyses result in an instantaneous 'snapback' of the pile during soil softening. This is analogous to the well-known rupture problem encountered in rock mechanics when the load frame is insufficiently stiff relative to the unloading stiffness of the rock specimen being tested. In the field, however, pile response during rebound involves the interaction of stored energy dissipation and viscous soil response that impedes rapid changes in loading and pile geometry. Such behavior is supported by the RPI centrifuge experiments, in which the pile rebound was observed to be a stable and gradual process. As explained by Goh and O'Rourke [1999], the unloading branch shown in Figure 4 represents the maximum allowable slope that is required to obtain a stable pile response during rebound.

COMPARISON OF MODEL PREDICTIONS WITH CENTRIFUGE MEASUREMENTS

Strain-softening p-y curves similar to those in Figure 4 were used in BSTRUCT to analyze the RPI centrifuge experiments. BSTRUCT is a finite element (FE) code [Meyersohn, 1994] that can be used for the analyses of piles, frames, pipelines and other buried structures. The soil-pile configuration is modeled using a combination of beam elements, spring-slider elements and/or rotational springs. By accounting for both geometric and material non-linearities of the pile and soil, it is possible to study pile performance under large soil displacements that strain the pile material beyond its elastic limit.

Figure 5 shows the soil-pile arrangement and the FE model corresponding to RPI centrifuge model 3. Details pertaining to the centrifuge experiment, such as model preparation, instrumentation and test results, are found in Abdoun [1997]. The soil springs that model the liquefiable soil are spaced 0.0625m apart, and are governed by the normalized strain-softening p-y curve shown in Figure 4 for the smooth soil-pile interface. Through a calibration process explained in Goh and O'Rourke [1999], it was found that good compliance between the computed and measured responses is obtained when y_{min} /D is set to 0.75. At the base of the pile, the behavior of the non-liquefiable sand is modeled using the p-y curves back-figured by Abdoun [1997] from centrifuge lateral pile load tests carried out in this lightly cemented material. Figure 5c shows the free-field lateral displacement profile measured in the centrifuge at the end of lateral spread. The discretized profile is applied in the form of incremental displacements to the free-field nodes of the corresponding soil springs.

Figure 6 compares the model predictions with the centrifuge measurements. In Figure 6a, the computed and measured moment histories at a pile depth of 5.75m are presented. Two computed responses are shown, corresponding to the use of (i) the nonlinear p-y curve obtained from FLAC analyses (Figure 4), and (ii) its trilinear approximation. There is little noticeable difference between the two computed responses, which suggests that a trilinear p-y profile may be used as a simplification without much loss in accuracy. Due to its proximity to the boundary separating the liquefiable layer from the cemented sand, the moments measured at this location are the maximum moments induced in the pile. The effect of the sinusoidal base excitation is manifested in the cyclic component of the measured response, while the dominant effect of lateral spread is reflected in the mean trend of the measured moment history. This measured trend is characterized by a peak and a residual moment, both of which are well replicated in the computed responses. Figure 6b shows the comparison between the measured and computed bending moment distributions along the pile at two stages during lateral spread, corresponding to the instants at which the peak and residual moments are attained. In both cases, the model predictions agree favorably with the centrifuge measurements.

Figure 7 shows the soil-pile arrangement and the FE model corresponding to RPI centrifuge model 5a, which is identical to model 3 except for the presence of a pile-cap. For the most part, the modeling considerations are the same as those described for model 3. The restraining effect of the pile cap, however, must be taken into account. This is achieved by specifying the appropriate pile-cap properties for the affected beam elements, and re-tuning the p-y response of the soil springs adjacent to the pile cap to reflect the passive pressures that may be mobilized over this depth. An approximate estimation of the forces acting on the pile cap is presented in Goh and O'Rourke [1999].

Figure 8 compares the model predictions with the centrifuge measurements when the pile cap is present. As shown in Figure 8a, the computed bending moment history near the boundary between the liquefied soil and cemented sand agrees favorably with the measured response. The comparisons also turn out very favorably for the bending moment distributions plotted in Figure 8b. Although the trends are similar to those recorded in model 3, it is noted that both the measured peak and residual moments are significantly increased when the pile-cap is present. This is largely due to the contribution of the passive earth pressures acting on the pile-cap, the effects of which are well replicated in the numerical response when the p-y curves of the affected soil springs are specified accordingly.

COMPARISON OF MODEL PREDICTIONS WITH THE JAPANESE HIGHWAY CODE

Figure 9 shows the pressure diagrams for models 3 and 5a corresponding to the instant at which the maximum moments develop along the pile. The distributions are approximately triangular, which reflect the assumed linear variation of the undrained soil strengths with depth. The pressure reversal near the 6 m boundary indicates that there is pile displacement relative to the soil at this level. This implies that the slightly cemented sand does not act as a perfect restraint. Neglecting this slight pressure reversal, the magnitude of the computed pressure exerted near the base is approximately 41 kN/m^2.

Based on case histories calibrated after the 1995 Kobe earthquake, the Japan Road Association (JRA) [1996] recommends that the pressure acting on piles due to ground flow be taken as 30% of the overburden pressure. This inherently gives rise to a triangular pressure distribution acting on the pile during lateral spread. For a soil which has a total saturated unit weight of 20 kN/m^3, the overburden pressure at a depth of 6 m is 20 x 6 = 120 kN/m^2. Using the JRA recommendation, the liquefied soil pressure acting at this level is estimated as 0.3 x 120 kN/m^2 = 36 kN/m^2 . The resulting pressure diagrams, shown shaded in Figure 8, are in good agreement with the computed distributions.

CONCLUSIONS

Soil-pile interaction during lateral spread is a complex three-dimensional problem. In this paper, a simplified model is proposed in which triaxial extension is used as the most appropriate analogue for the development of undrained shear stress in soil during liquefaction and its transfer to piles during lateral spread. To establish the relevant soil strength and deformation parameters, a suite of triaxial extension tests was carried out using Nevada sand. By accounting for both peak and minimum strengths, strain-softening p-y curves were numerically generated using 2-D FLAC analyses. Application of these p-y curves to the analyses of centrifuge experiments involving lateral spread effects on piles show favorable agreement between the computed and measured responses. The strain-softening model provides excellent predictions of the measured peak and residual moments. Furthermore, the computed soil pressure diagrams agree well with the recommendations made by the Japan Road Association (1996), which were calibrated using case histories from the 1995 Kobe earthquake.

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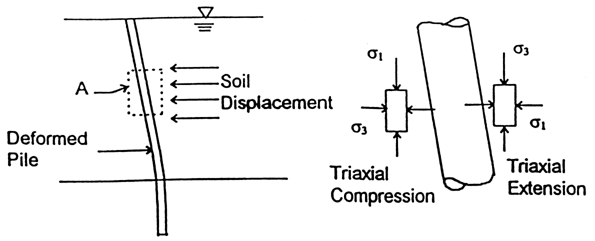
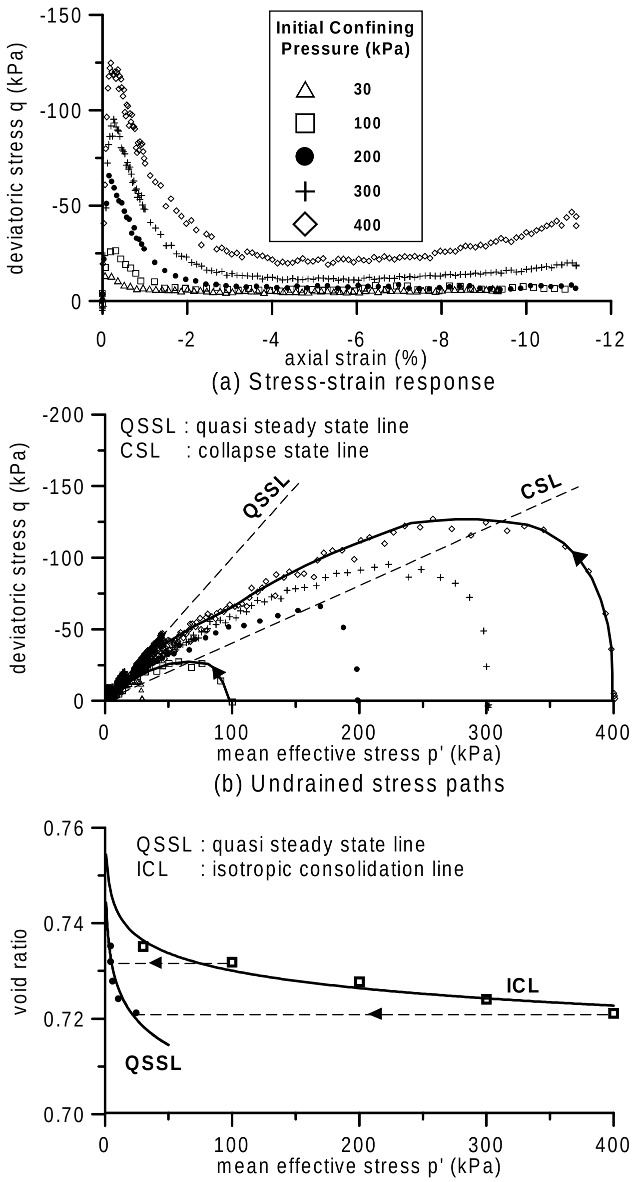


(a) Cross Section (b) Detail A

Figure 1 Schematic of Relative Soil-Pile Movement and Analogous Triaxial Loading Conditions

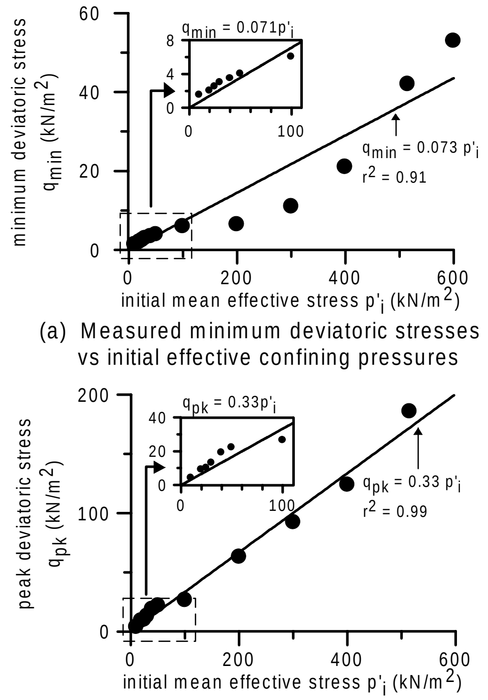


(a) Stress-strain response

(b) Undrained stress paths

(c) State paths in e-log p' space

Figure 2 Test Results from Triaxial Extension Tests Performed on Nevada Sand (Dr=40%)



(a) Measured minimum deviatoric stresses vs initial effective confining pressures

(b) Measured peak deviatoric stresses vs initial effective confining pressures

Figure 3 Minimum and Peak Deviatoric Stresses of Nevada Sand (Dr=40%) at Different Initial Confining Pressures

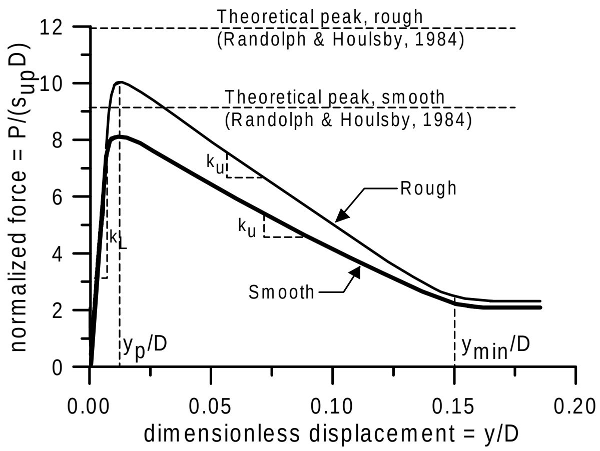
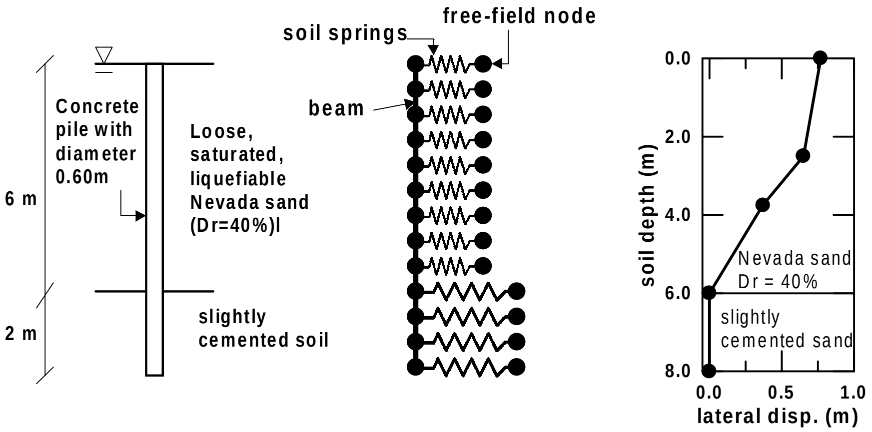
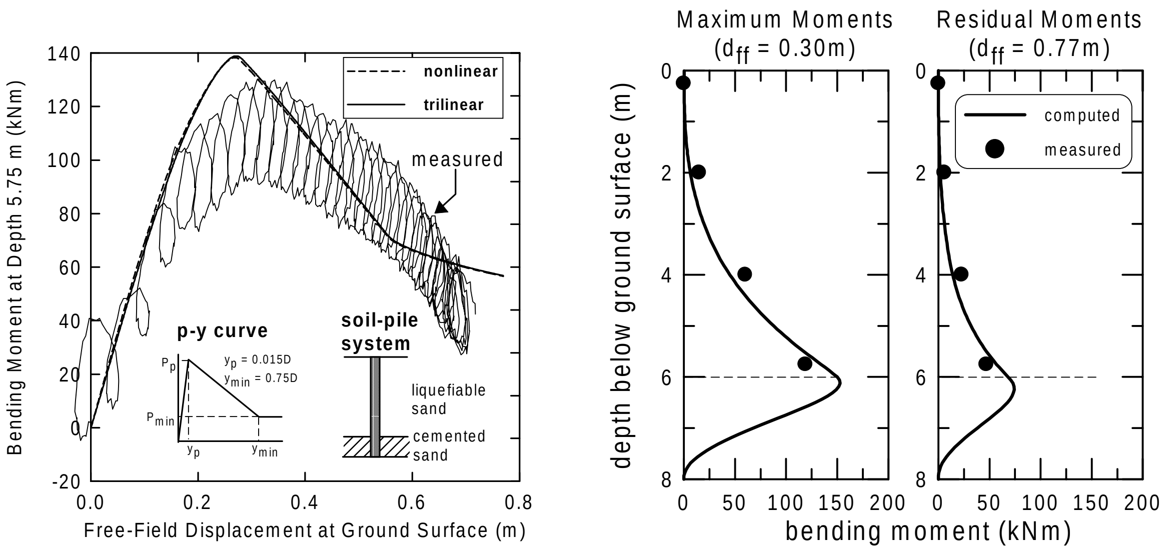


Figure 4 Normalized Strain- Softening p- y Curve



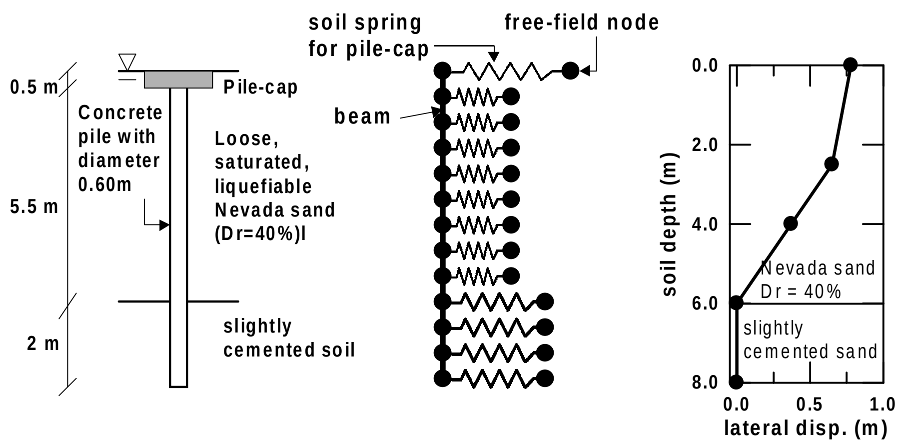
(a) Model 3 Soil-Pile System (b) Finite Element Model (c) Free-Field Lateral Displacements

Figure 5 Description and Finite Element Model of RPI Centrifuge Model 3



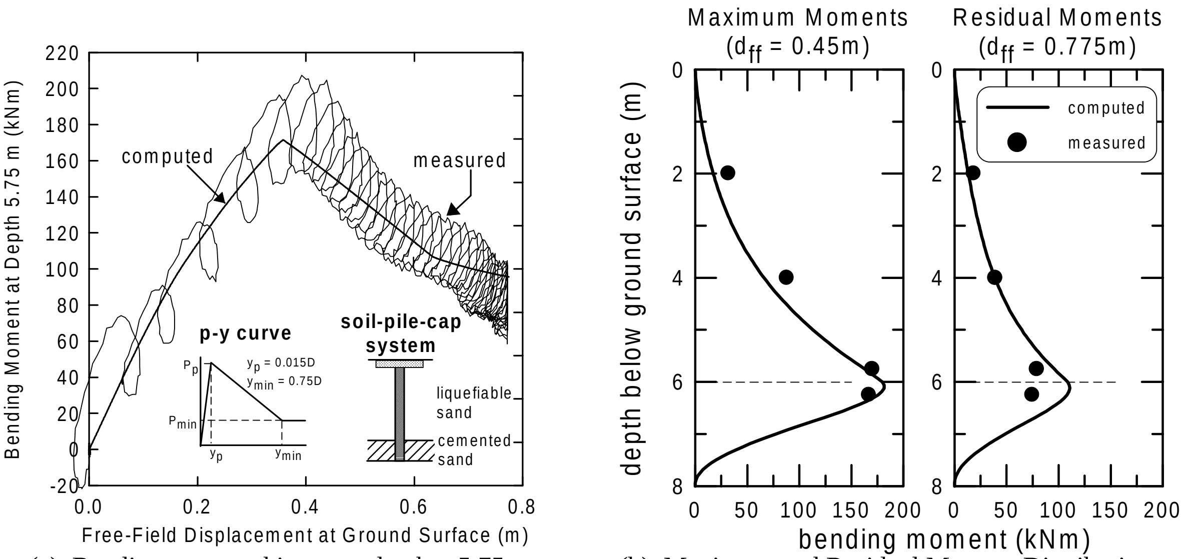
(a) Bending moment history at depth = 5.75m (b) Maximum and Residual Moment Distributions

Figure 6 BSTRUCT Predictions of RPI Centrifuge Model 3



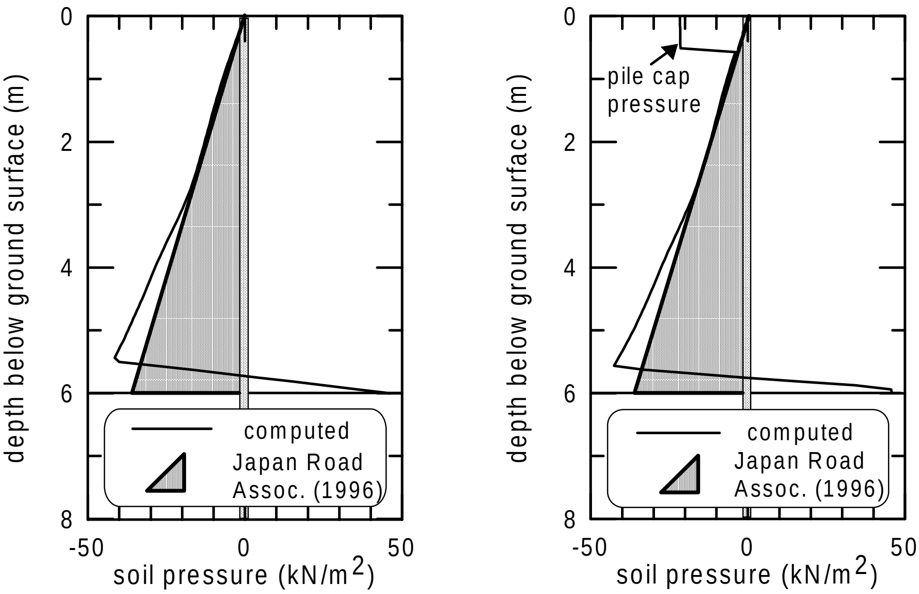
(a) Model 5a Soil-Pile-Cap System (b) Finite Element Model (c) Free-Field Lateral Displacements

Figure 7 Description and Finite Element Model of RPI Centrifuge Model 5a



(a) Bending moment history at depth = 5.75m (b) Maximum and Residual Moment Distributions

Figure 8 BSTRUCT Predictions of RPI Centrifuge Model 5a



(a) Model 3 (b) Model 5a

Figure 9 Comparison of BSTRUCT Pressure Diagrams with Japan Road Association's Recommendations