Title

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MONOTONIC AND CYCLIC BEAM ON NONLINEAR WINKLER FOUNDATION ANALYSES OF PILE FOUNDATIONS IN LATERALLY SPREADING GROUND

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ABSTRACT

Static beam on nonlinear Winkler foundation (BNWF) analyses are compared with data from a suite of centrifuge tests of pile groups in liquefied and laterally spreading ground. Analyses that utilize monotonic load paths are shown to reasonably predict bending moments, and pile cap displacements were reasonably predicted when limited to less than about 0.4 meters. However, the analyses under-predicted larger pile cap displacements, with the error being attributed primarily to the accumulation of displacement during repeated loading cycles due to cyclic ratcheting. The influence of cyclic ratcheting is explored using cyclic BNWF analyses, and predicted cap displacements are larger for cyclic load paths compared with linear load paths in cases where the pile tips fail in the axial mode (i.e. plunging or pullout).

Introduction

Extensive damage to pile foundations has been caused by liquefaction and lateral spreading of the surrounding soils [e.g., Japanese Geotechnical Society (JGS) 1996, 1998]. Lessons learned from case histories have provided important guidance for selecting input parameters and loading conditions to use in design computations, but the number of case histories is not sufficient to fully validate analytical methods. Furthermore, case histories typically do not provide the detailed data required to measure fundamental loading mechanisms, and it is therefore difficult to ascertain just how general observations from case histories are, and whether they would hold true for different conditions. Model studies can provide sufficient detailed data to identify fundamental loading mechanisms that occur during lateral spreading, thereby supplementing case histories to provide a rational basis for deriving robust analytical approaches.

Pile groups in lateral spreading soils are often analyzed using a static-seismic beam on nonlinear Winkler foundation (BNWF) analytical approach, in which the foundation components are modeled as beam-column elements, and soil-structure interaction is modeled using p-y materials for lateral subgrade reaction, t-z materials for pile shaft friction, and q-z materials for pile tip end bearing. As shown in Fig. 1, demands from laterally spreading layers can be represented by imposing free-field soil displacements

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on the free ends of the p-y materials (e.g., Boulanger et al. 2003). Inertia forces can be applied simultaneously with lateral spreading demands. Static methods inherently neglect many dynamic features that are known to affect pile foundation response (e.g., accumulation of displacement due to cyclic ratcheting, dilatant behavior of liquefied sand), and aim to use "statically-equivalent" loading conditions to envelope the field response. However, conditions in which "statically-equivalent" methods provide accurate solutions are not yet clear.

This paper compares results of monotonic BNWF analyses with data from a series of centrifuge tests described by Brandenberg et al. (2005). A brief explanation of the centrifuge test program is presented first, with focus on the observations that most significantly influence static BNWF analytical methods. Then the design guidelines adopted for this study are presented, along with recommended modifications to loading conditions and input parameters to account for liquefaction and lateral spreading. Numerical methods utilized in this study are then presented, followed by results of predicted versus measured peak bending moment and pile cap displacement. The influence of cyclic ratcheting on pile cap displacements is observed using the analytical BNWF method with cyclic ground displacement paths, which helps explain why measured pile cap displacements for some of the tests were under-predicted using monotonic ground displacement paths.

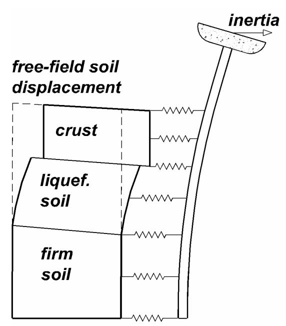

Figure 1: Beam on nonlinear Winkler foundation (BNWF) method with impose free-field soil displacements.

Centrifuge Test Program

A series of dynamic centrifuge tests was performed on the 9-m radius centrifuge at the University of California, Davis. Details of the tests were presented by Brandenberg et al. (2005) and only an abbreviated summary is provided herein. Model configurations were variations of the example layout shown in Fig. 2. The models consisted of a nonliquefiable clay crust (liquid limit ~ 88, plasticity index ~ 48) over liquefiable loose sand (D_r ~ 35%) over dense sand (D_r ~ 75%). The layers sloped gently toward an open channel carved in one end of the crust. The crust was composed of reconstituted overconsolidated Bay mud with average undrained shear strengths ranging from 22 kPa to 44 kPa in the various models. Pile diameters were 0.73 m or 1.17 m, center-to-center pile spacing was four diameters, and the piles were fixed into a large pile cap embedded in the crust. Single-degree-of-freedom superstructures were attached to the pile caps for some tests. Models were tested in a flexible shear beam container at centrifugal accelerations of 38.1 or 57.2 g. Each model was shaken with realistic simulated earthquake motions conducted in sequence with sufficient time between shakes to allow pore pressures to dissipate. Water was used as the pore fluid. Results are presented in prototype units.

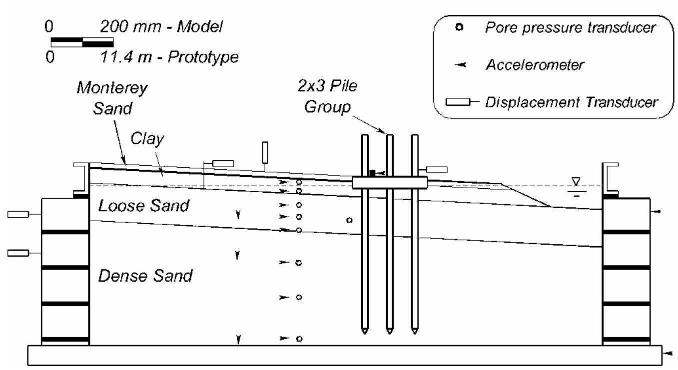

Figure 2: Example centrifuge model layout with most of the 100 instruments omitted for clarity.

Centrifuge Test Observations

Raw and processed data were used to characterize the loading mechanisms from the centrifuge tests, and data processing methods were presented by Brandenberg et al. (2005). Fig. 3 shows data quantities from a large Kobe motion for a pile foundation with 1.17-m diameter piles. The Kobe motion was preceded by a sequence of three previous motions. Peak bending moments and peak pile cap displacements were associated with peak downslope crust loads during strong shaking, and the crust loads subsequently decreased even though relative displacements between the pile cap and crust continued to increase. The transient lurching component of ground displacement was sufficient to mobilize large crust loads during strong shaking, and large crust loads acted simultaneously with large pile cap inertia forces, which is contrary to design guidelines that assume the two load components do not act simultaneously (e.g., TRB 2002). The liquefiable sand layer exhibited transient drops in excess pore pressure caused by the dilatancy behavior of the loose sand due to a combination of free-field strains imposed by shaking and local strains imposed by the piles. Large upslope subgrade reaction forces (p) were observed at the time that the peak bending moments and peak pile cap displacements occurred, which is contrary to the small downslope loads often imposed on the piles in static BNWF methods (e.g., JRA 2002).

BWNF Approach and Adopted Guidelines

Loading mechanisms that occur during lateral spreading depend on dynamic phenomena (e.g., cyclic ratcheting, shaking-induced dilatancy in the loose sand, transient lurching of the clay

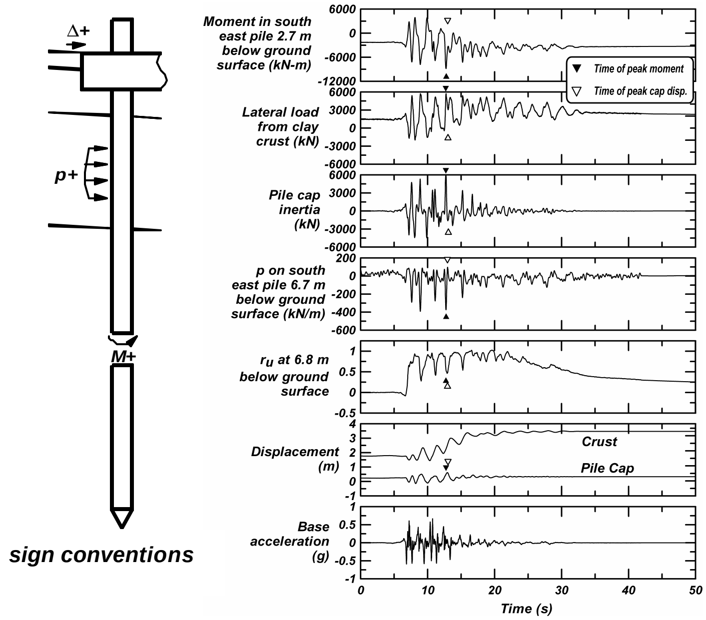

Figure 3: Time series from test SJB03 with 1.17-m diameter piles.

crust) that cannot reasonably be captured using static BNWF approaches with monotonic load paths. Rather than accurately tracking every nuance of the dynamic behavior, the BNWF approach seeks "statically-equivalent" loading conditions to envelope the potential field response. However, there are insufficient data to validate selection of any particular set of loading conditions and guidelines for estimating input parameters, and calibration with case histories and model studies is required to obtain more robust design guidelines. The approach adopted in this study was to select a single set of loading conditions and guidelines for input parameters, and subsequently observe the ability of the BNWF model predictions to envelope the peak bending moments and pile cap displacement measured across a suite of centrifuge tests.

Input parameters selected for the BNWF analyses were presented in detail by Brandenberg (2005), and only a brief overview is presented herein. Properties of the p-y elements in the clay crust layer were based on Matlock's (1970) relations for soft clay for static loading conditions. Matlock's cyclic loading corrections were not used because they were calibrated for a large number of loading cycles (e.g., wind or wave loading) in sensitive clay soils, and are inaccurate and unconservative to use in lateral spreading conditions. Properties of the p-y materials in the sand layers were based on the API (1993) relations, and subsequently softened to account for the effects of liquefaction. Shaft friction was represented using t-z elements distributed along the length of the piles, and end bearing was modeled using q-z elements at the tips of the piles. The pile foundations were therefore allowed to rock under the imposed lateral loads, as observed in the centrifuge tests. Vertical friction forces along the sides of the pile cap were modeled through t-z elements distributed along the length of the cap.

Lateral spreading displacements were estimated using Newmark's (1965) sliding block procedure, and the ground displacement profile was selected to match the characteristic profile observed in the centrifuge tests (e.g., Fig. 1). The displacement discontinuity between the clay crust and underlying liquefiable sand layer was caused by the low-permeability crust trapping water that was flowing upward through the liquefiable sand. A high void ratio zone formed in the sand immediately beneath the crust due to void redistribution (e.g., Kulasingam et al., 2004). Inertia forces from the pile caps and superstructures were imposed simultaneously with free-field lateral spreading displacements. The structural accelerations were estimated from equivalent linear site response analyses of the soil profiles.

Numerical Methods

The BNWF analyses were performed using the open system for earthquake engineering simulation (OpenSees) developed by the Pacific Earthquake Engineering Research (PEER) Center. Piles were modeled as elastic beam column elements with section properties that matched the model piles. The pile cap was modeled using stiff beam column elements connecting the pile heads. Soil springs were modeled as zeroLength elements with PySimple1, TzSimple1 and QzSimple1 materials (Boulanger et al. 2003). The analyses were two-dimensional, and the 2x3 pile group was modeled as a 1x3 pile group with the stiffness of the soil springs and pile elements doubled. Soil displacements were imposed using displacement patterns applied to the free-ends of the zeroLength p-y elements, and inertia forces were modeled using load patterns applied to the pile cap nodes.

The analyses were conducted by first applying gravity loading using a vertical load pattern, and subsequently imposing the horizontal displacement and inertia load patterns simultaneously. Loads and displacements were imposed incrementally using a static load control integrator, with the number of increments required depending on the nonlinearity in the foundation response. Force convergence was obtained when the norm of the displacement residuals was smaller than a specified tolerance. Penalty constraints were used to enforce the prescribed displacement boundary conditions. Numbering of nodal degrees of freedom was performed using a reverse Cuthill-McKee algorithm, and the system of equations was solved using a Newton-Raphson algorithm.

BNWF Analysis Results

Comparison between the predicted and measured profiles of displacements, bending moments and subgrade reaction loads (p) are shown in Fig. 4. The peak bending moment magnitude (-8838 kN·m) was measured at 2.7 meters below the ground surface (at the depth of the strain gauge on the pile nearest to the pile cap), where the predicted bending moment had an 11% larger magnitude (-9812 kN·m). The peak measured pile cap displacement was 0.48 m, while the predicted cap displacement was 0.38 m (21% smaller). Predictions of peak bending moments and pile cap displacements were reasonable for this case, though the distributions of bending moment and pile displacement deviated more significantly from the observed response due to differences in the dynamic loading conditions during the centrifuge test compared with the static conditions in the analyses. For example, large upslope subgrade reaction forces were observed in the centrifuge tests, while small downslope subgrade reaction forces were applied in the analysis. This caused a large predicted positive peak bending moment at about 10 meters depth, while a much smaller positive peak occurred in the centrifuge test. Distributions of bending moment and pile displacement may be important in some cases (e.g., less rebar may be placed in drilled shafts at depths where bending moments are small). However, this paper focuses on the peak bending moments and peak pile cap displacements.

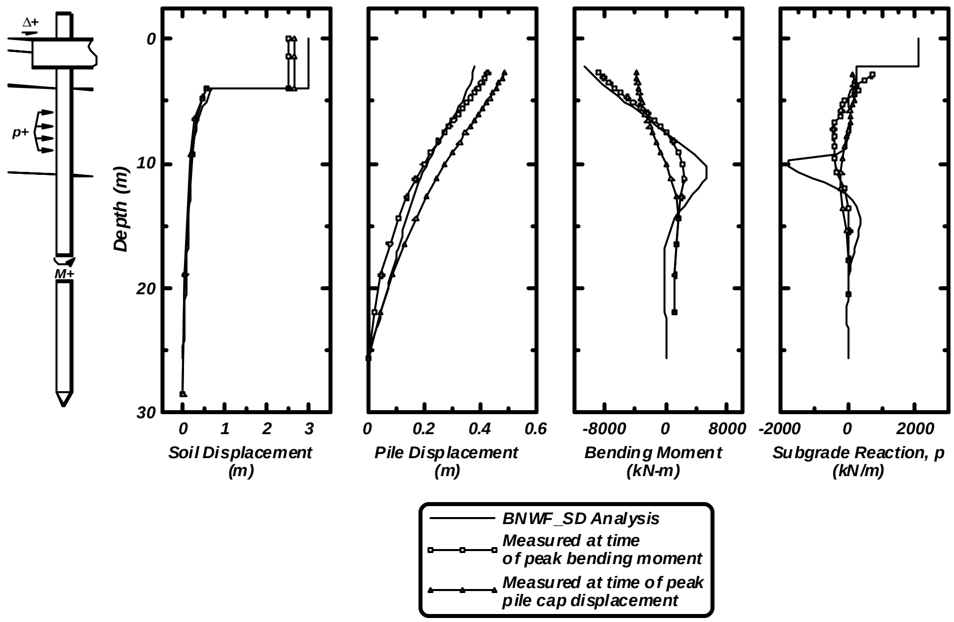

Figure 4: Results of BNWF analysis of pile foundation from test SJB03 for a large Kobe motion.

Comparisons between the peak predicted bending moments and pile cap displacements are presented in Fig. 5 for 21 different ground motions from five centrifuge tests. Bending moments were predicted quite well, with a coefficient of variation of about ¼, and a slight over-prediction on average. Pile cap displacements were reasonably predicted when large-diameter piles limited the pile cap displacement to less than about 0.4 meters (i.e. for tests SJB03, DDC01 and DDC02, all with 1.17-m diameter piles), but predictions were considerably lower when small diameter piles permitted larger cap displacements (i.e. for tests SJB01 and PDS03, both with 0.73-m diameter piles). The primary reason for this error is believed to be accumulation of cap displacement during repeated loading cycles, called cyclic ratcheting.

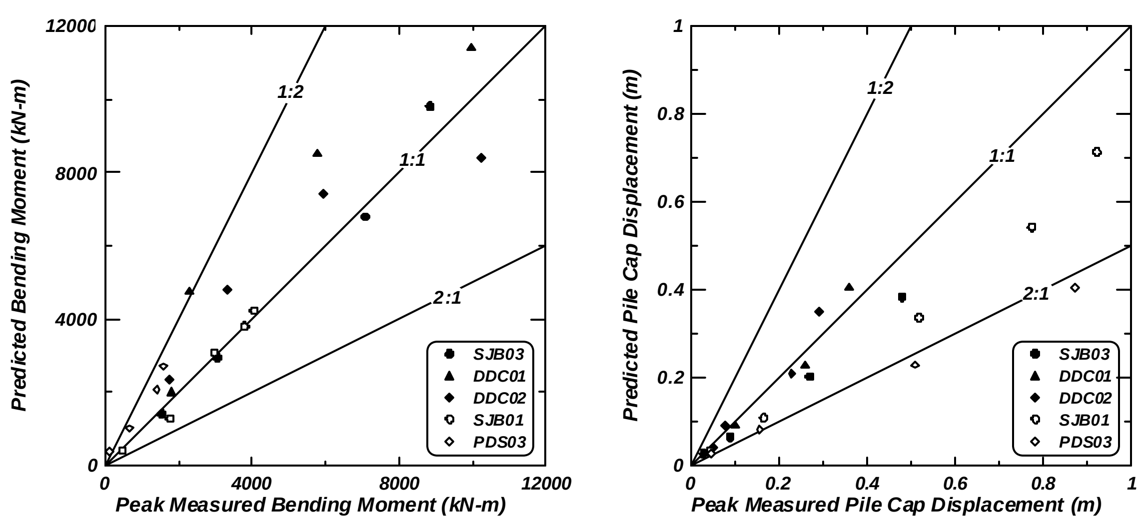

Figure 5: Measured versus predicted bending moments and pile cap displacements.

Influence of Cyclic Ratcheting on Predicted Pile Cap Displacement

Cyclic ratcheting can be caused by axial failure of the pile tips (i.e. plunging and/or pullout). The influence of cyclic ratcheting was observed using BNWF analyses with various cyclic load paths instead of the monotonic linear load paths used in previous sections. To isolate the effect of ratcheting caused by ground displacements alone, inertia forces were not applied. Gapping on the downslope side of the pile cap was modeled using bi-directional p-y behavior on the pile cap, where the downslope capacity was equal to the passive force plus friction forces, while the upslope capacity was equal only to the friction forces. This configuration assumes that the gap remains open during shaking, as occurred during the large motions in the centrifuge tests. A six-pile group with 0.73-m diameter piles was used in the analyses since cyclic ratcheting is suspected to contribute to the larger-than-predicted cap displacements for the 0.73-m diameter pile groups.

Fig. 6a shows a comparison between the linear load path used in analyses in the previous section, and a cyclic load path consisting of 5 ¼ sinusoidal cycles of 0.3-m transient displacement amplitude, and a monotonic bias such that the final displacement for the cyclic load path was 3 meters (same as for the monotonic load path). Axial failure (i.e. plunging and pullout) occurred at the pile tips because the capacities of the q-z elements were smaller than the axial demands caused by lateral loading. Fig. 6b shows pile cap displacement versus ground

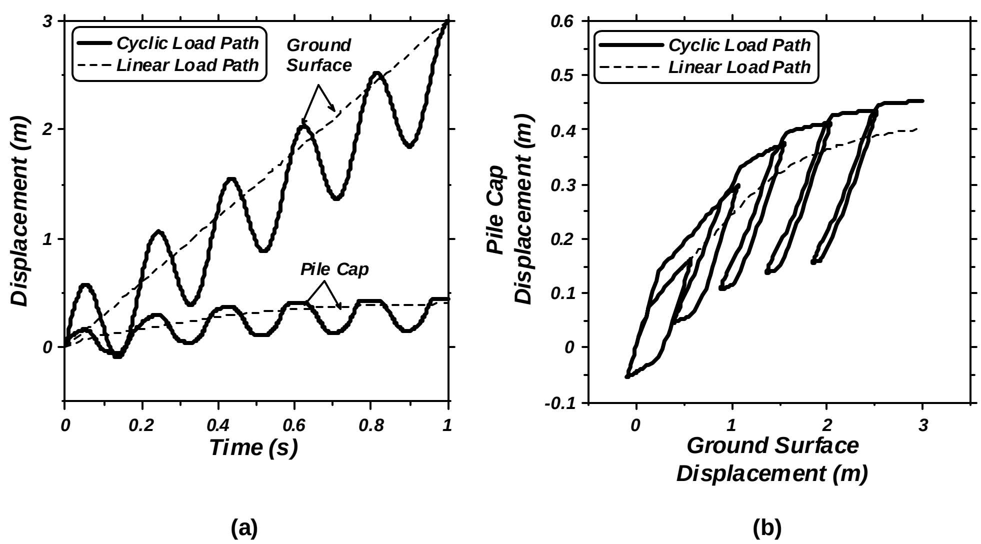

Figure 6: Comparison of cyclic load path and linear (monotonic) load path.

surface displacement. Cyclic ratcheting is manifested as the gradual increase in pile cap displacement accumulated during unloading and subsequent reloading. The pile cap displacement was about 0.40 meters at the end of the linear load path, and was 0.45 meters at the end of the cyclic load path.

The cyclic load path analyses were repeated for different numbers of cycles (1.25, 5.25, 10.25, 15.25 and 20.25 cycles), different values of transient displacement amplitude (0.1, 0.3, and 0.5 meters), and also for cases where the axial capacity of the pile tips was large enough to prevent axial failure. Final crust displacement was 3 meters for all cases. Fig. 7 shows the results of these analyses. For small pile tip capacity, predicted cap displacement generally increased as the number of loading cycles increased, and as the transient displacement amplitude increased. However, cap displacement sometimes reached a threshold where more cycles or larger transient displacement amplitude did not produce more pile cap displacement. For the case where large axial pile tip capacity prevented failure of the pile tips, the predicted cap displacements were about 0.23 meters and were independent of load path. Hence, cyclic ratcheting affected pile groups whose piles failed in the axial mode at their tips, but was not a factor when the tip capacity exceeded demand.

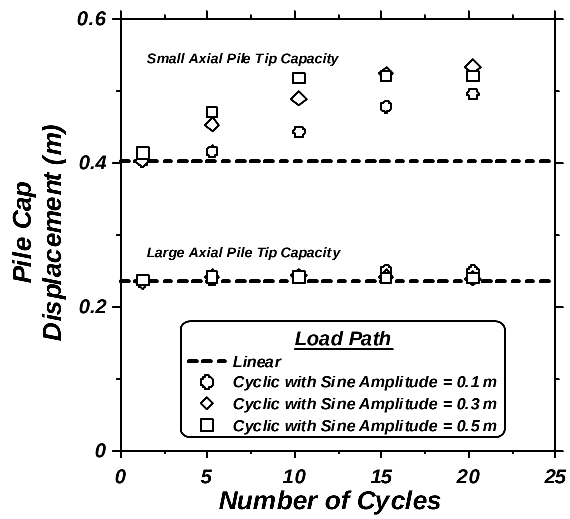

Figure 7: Influence of number of loading cycles, axial pile tip capacity, and amplitude of transient ground displacement on accumulation of pile cap displacements.

Discussion

Measured peak pile cap displacements were about 50% to more than 100% larger than displacements predicted using a linear displacement path in static BNWF analyses for tests with 0.73-m diameter piles (PDS03 and SJB01). Predicted cap displacements were up to about 30% larger when cyclic displacement paths were imposed than when linear displacement paths were imposed, hence additional factors must have contributed to the large measured cap displacements. Structural inertia forces occurred simultaneously with ground displacements in the centrifuge tests, but inertia was omitted from the BNWF analyses. Downslope inertia force cycles would increase cyclic cap displacements, thereby adding to the ratcheting effect. Cyclic mobility behavior of the saturated sand layers caused by dilatancy could also contribute to ratcheting behavior, but was not included in the BNWF analyses.

The sequence of base motions applied to the centrifuge models imposed a large number of loading cycles on the pile foundations compared with the number of cycles that occurred during any single base motion alone. Shaking sequence was accommodated in the BNWF analyses by imposing the total estimated ground displacements (including displacements from previous base motions). Hence, comparison between the BNWF analyses and centrifuge tests are compatible. An alternative approach would be to compare BNWF results with incremental displacements that occurred during each ground motion. The influence of cyclic ratcheting on incremental displacements might more reasonably capture the influence expected for a single motion instead of a sequence of motions. However, incremental displacements depend on stress history imposed on the foundations during previous earthquake motions, which was not captured in the BNWF analyses. Comparing with total displacements was deemed simpler and more straightforward than attempting to account for stress history in the BNWF method and compare incremental displacements.

Conclusions

Monotonic beam on nonlinear Winkler foundation (BNWF) analyses, that utilized a single set of "statically-equivalent" loading conditions and input guidelines, reasonably predicted peak bending moments measured during a series of dynamic centrifuge tests. Bending moment was predicted with a coefficient of variation of about ¼, and was slightly over-predicted on average. Pile cap displacements were predicted well when large-diameter piles were used to limit pile cap displacements to less than about 0.4 meters throughout the series of simulated earthquakes. However, under-predictions of about a factor of two were observed for tests where smaller-diameter piles (i.e. 0.73 m) permitted large pile cap displacements. Cyclic ratcheting contributed to the displacements in the centrifuge tests, but was not modeled in the linear load paths used in the BNWF analyses. Additional BNWF analyses with cyclic load paths demonstrated the mechanisms of cyclic ratcheting.

Under-prediction of cap displacement caused by cyclic ratcheting can be avoided by providing sufficient axial capacity at the pile tips to ensure that the piles do not fail under the combined effects of gravity loading, lateral loading, and any down-drag loads that might develop during liquefaction and lateral spreading. Boulanger and Brandenberg (2004) used a neutral plane solution to show how pile settlements caused by down-drag loads were small when axial failure was prevented. Brandenberg (2005) performed a monotonic BNWF sensitivity study, and found that variations in axial capacity did not significantly affect predicted bending moments and cap displacements when the capacities were large enough to prevent failure, but analysis results were sensitive to axial capacity when failure did occur at the pile tips. Preventing axial failure provides better pile group performance (smaller pile cap displacements) and also enables monotonic BNWF predictions to better envelope field conditions.

Acknowledgments

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