Back-Calculation of Winkler Foundation Parameters for Dynamic Analysis of Piles from Field Test Data

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ABSTRACT

Prediction of response of a pile-soil system during a large magnitude earthquake requires consideration of various aspects such as the nonlinear and elasto-plastic behavior of soil and pile-soil interface, pore water pressure development in soil, radiation of energy away from the pile, etc. While laboratory and in situ soil properties may be used in a rational analysis method to perform the response prediction, reliability of the prediction may be greatly improved by conducting a suitable site-specific field experiment and calibrating the analysis method against the field results. Using a Statnamic device in unison with a spring/mass system, a single pile was excited in the horizontal direction at the level of the pile head with a dynamic motion involving a few cycles. While the pile was fully instrumented, some of the strain gauges failed during the test. The time histories of the pile head displacement, pile head acceleration, pile displacement at the ground level and the applied load were successfully measured. Using these preliminary results, the potential for back-calculating stiffness and damping parameters for a Beam on Nonlinear Winkler Foundation (BNWF) analysis method is explored. It is shown that a well-designed field test can provide adequate information to calibrate the analysis method, and thereby, to increase the reliability of the analytical predictions. Also explored here is the potential for use of BNWF method for problems involving liquefaction. One such problem is analyzed using BNWF and a fully-coupled finite element computer program (HOPDYNE) and the results compared and discussed.

INTRODUCTION

Analysis of structures, supported on piles and subjected to a dynamic loading such as the earthquake loading and ship collision loading, requires an accurate evaluation of the soil-structure interaction effect. The dynamic pile-soil interaction is complicated by many factors. The behavior of soil is highly nonlinear and elasto-plastic. When the amplitude of pile motion is large, a gap could form at the pile-soil interface, causing the interface behavior to be highly nonlinear. In saturated soils, a cyclic loading could produce a significant positive excess pore water pressure, further reducing the stiffness and strength characteristics of the soil. Finally, the energy supplied to the pile is expended in the surrounding soils by plastic dissipation and radiation in a complex manner. While numerical techniques such as the finite element method have the potential for capturing most of these nonlinearities, it is desirable to have a simpler method for use in design analyses. One such method is the so-called Beam on Nonlinear Winkler Foundation
(BNWF) method, where the effect of the soil surrounding the pile is represented by a Winkler foundation involving springs and dashpots as shown in Fig. 1. (Also shown in Fig. 1 are above-ground spring and dashpot to be discussed later.) The discussion here is limited to highly nonlinear problems. Here, one choice for the soil springs is the so-called nonlinear p-y curves, where p refers to the lateral load and y refers to the lateral deflection. Reese et al. (1974) proposed an empirical relationship for sands, and this is used here to represent the soil. The reader is referred to this original reference for the details. The properties required for defining the p-y relation are the friction angle of the soil $\phi$, the effective density of the soil $\gamma'$ and the initial slope of the p-y curve $k_1$. The value of $\phi$ governs the maximum value of p. $k_1$ depends on the density of the soil.

![Fig. 1. Details of Beam of Nonlinear Winkler Foundation Model](image)

The relationship proposed by Reese et al. (1974) is an approximate one, and may or may not exactly represent a given field soil deposit. Also, the relationship was proposed and verified only for static and slow cyclic loading, and its validity has not yet been proven for dynamic loading. Due to its simplicity, it is adopted here, but using modified parameters as

$$
\phi^* = F_\phi \phi \quad \text{and} \quad k_1^* = F_k k_1
$$

The objective of the study presented here is to evaluate site-specific values of the modifiers $F_\phi$ and $F_k$. 

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\(\phi\) refers to the friction angle of the soil, \(\gamma'\) is the effective density of the soil, \(k_1\) is the initial slope of the p-y curve.
Considering a problem of wave propagation in a linear viscoelastic, infinite rod, it can be shown that the rod can be truncated at a finite distance from the source of excitation, and portion of the rod from the truncation location and infinity can be exactly represented by a dashpot of coefficient

\[ C_{\text{rad}} = a \rho V \]  \hspace{1cm} (2)

where \( a \) is the cross sectional area of the rod, \( \rho \) is the mass density of the rod, and \( V \) is velocity of wave propagation in the rod. The damping is referred to as the radiation damping. When the wave is a shear (s-) wave, \( V = V_s \) and when it is a p-wave, \( V = V_p \), where \( V_s \) and \( V_p \) are s- and p-wave velocities respectively. Berger et al. (1977) applied this model for accounting for the radiation damping in the pile-soil interaction problem. The simplified wave propagation model used by Berger et al. (1977) is shown in Fig. 1(c), where it is seen that the energy is assumed to be carried away in the form of p-waves in the direction of shaking and s-waves in the direction perpendicular to shaking. The net damping coefficient is given by

\[ C_{\text{rad}} = 2 a \rho (V_s + V_p) \]  \hspace{1cm} (3)

More accurate models would lead to frequency-dependent damping coefficient, which is not convenient for nonlinear time-domain analysis as the one of interest here.

The radiation damping model works well only for purely linearly problems. The energy dissipation due to plasticity dominates in highly nonlinear problems. While this may be accounted for by the use of material damping, there is no rational method of quantifying this. Here, it is assumed that the net damping can still be represented by the Berger’s model, but with a modified damping coefficient as

\[ C_s = F_c C_{\text{rad}} \]

The modifier \( F_c \) is to be evaluated from the field data.

**DETAILS OF EXPERIMENT**

The loading is applied in the horizontal direction at the level of the pile head. The Statnamic device produces an impulse loading, with a rise plus fall time of about 120 ms. The device has been extensively used by the last author in conducting lateral load tests on pile groups for evaluating the static group capacity. When the displacement amplitude of loading is large, the oscillation of the pile decays in less than a cycle in most cases. One of the objective of the study was to find ways to cause the pile to oscillate at least for a few cycles (say, 4 to 5), so that the effect of pile-soil interface gapping and pore water pressure build-up can be captured. In this preliminary study, however, the tests were conducted on a partially-saturated sand. It was analytically found that when the impulse loading is applied through a spring/mass oscillator as shown in Fig. 1(a), the pile experiences a cyclic motion, with the number of cycles depending on the spring constant \( k \), mass of the pile head \( m_1 \), the second mass \( m_2 \), amplitude and duration of the load.
pulse, and the stiffness and damping characteristics of the pile-soil system. When the load is applied directly on the pile head with no spring/mass system, the energy is injected directly into the soil and dissipated quickly. When the loading is applied through the spring/mass system, a portion of the energy produced by the Statnamic device is stored in the spring, and released gradually in producing a few cycles of motion.

![Fig. 2. Schematic of the Field Test Arrangement](image)

The photograph of the experimental setup is shown in Fig. 2, where the Statnamic reaction mass is placed on a steel I-beam. The second mass \( m_2 \) consists of mainly the mass of the Statnamic projectile containing the piston and load cell. The spring is attached to the pile head through a 10,000lb capacity load cell. A special rail system was designed and fabricated to allow the projectile to slide back and forth in the horizontal direction. Once the projectile is shot out of the reaction mass, the projectile travels at a certain velocity and latches onto the spring. A catching mechanism was designed and fabricated for this purpose. From this point on, the second mass stays attached to the first mass (i.e., the pile head mass) through the spring. (Another series of tests was conducted with the spring attached between the pile head and fixed support, and the load applied directly onto the pile head. The details will not be described here.)

The tests were conducted in a test pit, 20’x20’ (6.1m\( \times \)6.1m) in plan area and 18’ (5.5m) deep. The pit contains partially saturated sand (filled by the last author in the past), and the pile was driven into this soil and tests conducted. The sand existed at a medium dense state. The properties needed for the analysis here are estimated to be: \( \phi = 35^0 \) and \( \gamma' = 100 \) lb/ft\(^3\) (15.7 kN/m\(^3\)). From Reese et al. (1977), for a medium dense sand, \( k_1 = 103,680 \) lb/ft\(^3\) (16,535 kN/m\(^3\)).

The pile is a thin-walled steel pipe with an inside diameter of 3.87 inches (9.83cm) and wall thickness of 0.065 inches (0.1651 cm). Towards the end where the pile head is, the wall thickness is 0.544 inches (1.38 cm). The values of the masses are \( m_1 = 6 \) lb-sec/ft and \( m_2 = 5.2 \) lb-sec/ft. Two different springs were tried; their spring constants are 1150...
Fig. 3. Comparison of Experimental Data with Results from BNWF Method Analysis
lb/in (medium stiff) and 1890 lb/in (stiff) respectively; only the results from test with spring of stiffness 1150 lb/in are considered for further discussion here. LVDTs (Linear voltage differential transformers) were used to measure the horizontal pile-head displacement and the pile displacement at the ground level. However, the range of the LVDT was inadequate to record the pile-head displacement. The pile head was also furnished with an accelerometer for measuring the horizontal acceleration.

RESULTS AND ANALYSIS

Some of the key results are presented in Fig. 3. Using the BNWF described in the preceding section, the problem was analyzed, and the results are compared in Fig. 3. The modifiers used with $C_s$, $\phi$ and $k_1$ are: $F_c=0.83$, $F_\phi=0.5$, and $F_k=1.0$. In this analysis, the measured load time history at the pile-head level was directly used as the input into the analysis. It is seen that the agreements between the measured and calculated acceleration and displacement time histories are reasonably good, indicating that BNWF analysis method does have the potential for capturing the dynamic response of piles. When the modifiers are applied, the values of $C_s$, $\phi$ and $k_1$ should represent the specific site conditions, and may then be used to analyze a pile of different diameter or a pile group. These aspects are the subjects of the on-going research.

While the method described above is suitable for a site consisting of a uniform soil, a different method is needed for calculating the parameters when site consists of layers of different soils. This can be achieved from the results of strains on the pile at different depths; this is also an objective of the on-going study.

ANALYSIS OF A LIQUIFACTION PROBLEM USING BNWF METHOD AND A FULLY-COUPLED ELASTO PLASTIC FINITE ELEMENT METHOD

One of the centrifuge tests conducted by Wilson (Wilson, 1998; Boulanger, et al., 1999) is analyzed using HOPDYNE (Anandarajah, 1990), and the BNWF method described above, and the results compared and discussed. A two-surface plasticity model was developed for describing the stress-strain behavior of liquefying sand (Anandarajah, 1994a, 1994b); a version of this model that involves 9 parameters was used to represent the soil in this study. The coupled finite element analysis is based on the fully-coupled formulation of Zienkiewicz and Shiomi (1984). These procedures are implemented into HOPDYNE. Some recent verification studies may be found in Anandarajah (1993) and Anandarajah et al. (1995).

The centrifuge model of interest here consisted of two horizontal layers of Nevada sand containing a number of model structures, including single piles and pile groups; here the behavior of one of the single piles is analyzed. The bottom and top layers were 11.4m thick dense Nevada sand ($Dr=80\%$) and 9.3m thick medium-dense ($Dr=60\%$) Nevada sand respectively. The model is referred to as Csp3 and the reader may find the full details in Wilson (1998). Here the structure in prototype dimensions is considered for analysis. The pile has a diameter of 0.667m and a wall thickness of 7.2cm. The depth of embedment is 16.8m. The free length of the pile above the ground level is 3.81m. A weight of 500 kN was placed at the top of the pile. The cross sectional area, second
moment of inertia and Young's modulus of the pile are 0.135 m$^2$, 0.006 m$^4$, and 70,000 kPa respectively. An equivalent, one-foot thick domain was used in the 2-D, plane strain finite element analysis. The values of moment of inertia and the cross sectional area were increased by a factor of 1/0.667=1.5; that is, the pile is assumed to extend all the way in the z-direction, and a one-foot thick segment is chosen for the 2-D analysis. Therefore, the influence of that part of the soil in the z-direction that does not contain the pile is ignored.

![Graph showing acceleration over time](image1)

**Fig. 4. Comparison of Centrifuge Results with that Predicted Using a Fully-Coupled Elasto-Plastic Finite Element Method (HOPDYNE)**

Full details of analyses may be found in Anandarajah (2000). The acceleration time history of the pile head obtained from the finite element analysis is compared in Fig. 4 with the measured acceleration time history. It is evident that the results compare reasonably well. The pore water pressure that developed at various points around the pile also compared reasonably well with the finite element results, and the details may be found in Anandarajah (2000).

The problem was then analyzed using the BNWF method (which has also been incorporated into the same computer program HOPDYNE). The results are compared in Fig. 5. While the magnitude of the predicted accelerations at various points along the length of the pile was reasonably close to that of the measured ones, the frequency
content of the predicted accelerations deviated significantly from that of the measured ones (compare the top and bottom figures in Fig. 5). The predominant period of the calculated motion is about 0.35s, whereas that of the measured acceleration is about 1.0s. In examining the cause of the discrepancy, it was discovered that the Berger at al.’s (1977) model over damped the system. The primary reason for the stiff response obtained from the analysis was, however, found to be the result of not allowing the springs to “soften” as a function of the rising pore water pressure. To delve further into these aspects, the Reese et al.’s p-y relation was replaced by a simple elastic-perfectly plastic relation, with the slope of the elastic part matched with the initial slope of the Reese et al.’s relation. The radiation damping was reduced to a negligible value. Analyses were repeated, each time with different values for the initial stiffness of the p-y relationship. The acceleration response predicted using 5% of the earlier initial stiffness value is presented in the middle of Fig. 5. It may now be seen that a new peak begins to develop on the response spectra at a period very close to 1.0s, indicating that the pore water pressure induced softening of the p-y springs must be accounted for in the analysis for accurately capturing the behavior of the pile-soil system; a suitable method of achieving this is currently being developed. The results show that (a) fully-coupled elasto-plastic finite element procedures have the capability to simulate the response of earthquake pile-soil interaction, accounting for the pore-pressure induced softening in a natural manner, and (b) the BNWF methods have the potential to simulate the earthquake response of piles provided that the pore pressure effects are properly accounted for.
CONCLUSIONS

It is shown that by conducting a suitable pile-head shaking test on a single pile in the field, the parameters needed for the analysis of the earthquake pile-soil interaction problem by the Beam of Nonlinear Winkler Foundation method may be back calculated. The proposed test is easy to carry out in the field, and therefore, may be adopted as part of the routine in situ tests to be conducted when designing pile foundations for earthquake loading. To further explore the potential of BNWF method for analyzing pile-soil interaction problems involving liquefaction, one such problem was analyzed using both the BNWF method and a fully-coupled elasto-plastic finite element method (incorporated into HOPDYNE) and the results compared and discussed. It was found that when the spring and dashpot coefficients are allowed to soften as a function of the developed pore water pressure, the results from BNWF, HOPDYNE and experiments begin to agree with each other.

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REFERENCES


