

EVALUATION OF DYNAMIC SOIL-PILE INTERACTION BASED ON BACK CALCULATED P-Y CURVES

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ABSTRACT

Soil-pile interaction constitutes an important parameter in predicting the seismic response of pile-supported structures. Towards a computationally attractive investigation of the interaction mechanism, the p-y method, where the soil is represented as a series of independent springs distributed along the pile shaft, has been extensively used. Along these lines several p-y curves for different soil-pile systems have been proposed which are mainly based on in-situ pile tests under static or low frequency cyclic loading conditions. However, soil-pile interaction under seismic excitation becomes more complex due to the incident seismic waves scattered by the pile, thus modifying the p-y relationship and introducing an additional damping mechanism at the soil-pile interface. In this paper, dynamic soil-pile interaction is estimated based on back-calculated p-y curves, disregarding in a first stage any potential soil-pile gapping mechanism. Pile displacements and soil reactions are derived through double integrating and differentiating respectively the bending moments obtained along the pile shaft. The p-y curves generated at each depth are utilized to derive frequency dependent springs and dashpots, which may then be implemented within the framework of a Beam on Dynamic Winkler foundation modeling of the soil-pile system. The proposed procedure is validated through centrifuge tests results of a coupled soil-pile-structure system under real earthquake excitation and is also compared to existing analytical formulas of frequency dependent springs and dashpots under steady state harmonic excitation applied on the pile head. Analysis results reveal that for each one of the loading scenarios considered, soil-pile interaction mechanism is adequately captured while utilizing existing analytical expressions for the computation of dynamic soil spring supports may under certain conditions lead to an overestimation of pile and structural response when the coupled soil-pile-structure system is analyzed.

Keywords: soil-pile interaction, p-y curves, frequency dependent springs and dashpots

INTRODUCTION

Several procedures have been developed during the last two decades in order to determine the dynamic response of piles subjected to horizontal or vertical loads. Most of these procedures adopt the Winkler approach where the pile is modeled as a beam on elastic foundation and the soil is represented as a series of independent springs attached to the pile shaft. The determination of the spring stiffness is usually based on certain p-y curves that have been established for particular soil types (Matlock, 1970, Reese et al. 1974, Reese and Welch, 1975, Janoyan et al. 2001). Although the non linear relationship between the pile displacements and the lateral soil reactions may be accounted for, the p-y curves obtained under monotonic or cyclic loading conditions do not directly describe neither the reduction of the soil stiffness with the increasing amplitude of the seismic motion (more pronounced for soft soil conditions) nor the soil inertia and radiation damping effects which play a fundamental role in the soil-

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pile interaction mechanism (Angelides and Roesset, 1981). Consequently, the implementation of a Winkler model for analyzing soil-pile interaction under strong ground motion needs to take into consideration the modification of the p-y relationship due to the incident seismic waves that are scattered by the pile. Furthermore the presence of hysteretic and radiation damping affects soil-pile interaction, introducing the complex valued soil stiffness as a function of the frequency content of the seismic motion. In order to incorporate these dynamic effects in the framework of a Winkler approximation, simplified expressions of frequency dependent springs and dashpots (Dobry et al., 1982, Gazetas and Dobry, 1984, Kavvadas and Gazetas, 1993) as well as more rigorous analytical p-y models (Boulanger et al., 1999, Lok et al. 1998) have been proposed for seismic soil-pile interaction analysis. However, little research has so far taken place towards the correlation of the dynamic stiffness at the soil-pile interface with the p-y loops that are generated under seismic loading. Nevertheless, the fundamental properties of the dynamic p-y loops such as the slope and the area of the loop provide a straightforward estimation of the springs and dashpots that could be utilized to model dynamic soil-pile interaction.

In this paper a generalized procedure is presented where dynamic soil-pile interaction is evaluated based on back-calculated p-y loops, disregarding in a first stage any potential soil-pile gapping mechanism. Pile displacements and lateral soil reactions are initially computed separately, based on the bending moments developed along the pile shaft. Frequency dependent springs and dashpots are then estimated utilizing the basic characteristics of the respective p-y loops obtained at each depth. The adopted procedure is validated through centrifuge tests results of a coupled soil-pile-superstructure system and is further compared to existing analytical formulas of frequency dependent springs and dashpots that have been proposed for pilehead harmonic loading. Finally, a comparative analysis is performed where the effect of the springs and dashpots coefficients on the seismic response of the coupled system is investigated.

OUTLINE OF THE ADOPTED PROCEDURE

The procedure adopted herein comprises of two discrete parts of analysis. The first concerns the derivation of the dynamic p-y loops from the bending moments generated along the pile shaft, while the second one deals with the estimation of frequency dependent springs and dashpots based on the p-y loops derived in the previous stage. Therefore, the main scope of the procedure is to obtain stiffness and damping components of the dynamic soil-pile interaction mechanism that could be used in a simplified beam on dynamic Winkler foundation model of the soil-pile system. Each one of the analysis steps is explained further in the following paragraphs.

Derivation of p-y curves from recorded pile bending moments

Under seismic excitation the magnitude of the pile bending moment is generally a function of time as well as of the vertical distance along the pile shaft z . Therefore, for each time instance, t_i , a different bending moment distribution $M(z, t_i)$ is generated along the pile. Using simple beam theory, the seismic p-y behaviour can be back calculated from the bending moment distribution according to the equations:

$$p(z, t_i) = \frac{\partial^2 M(z, t_i)}{\partial z^2} \quad (1)$$

$$y_{pile}(z, t_i) = \frac{1}{EI} \cdot \iint M(z, t_i) \quad (2)$$

where p is the lateral soil reaction on the pile, y_{pile} is the absolute lateral displacement of the pile and EI is the flexural stiffness of the pile. Having obtained the p and y_{pile} distribution along the pile at each time instance, the time histories of the lateral soil reaction and the pile displacement are then computed at each depth z . The relative displacement between pile and soil, y , is finally obtained by y_{pile} after subtracting the free field soil deflected shape y_{soil} . Hence p-y loops are generated at a given depth. It

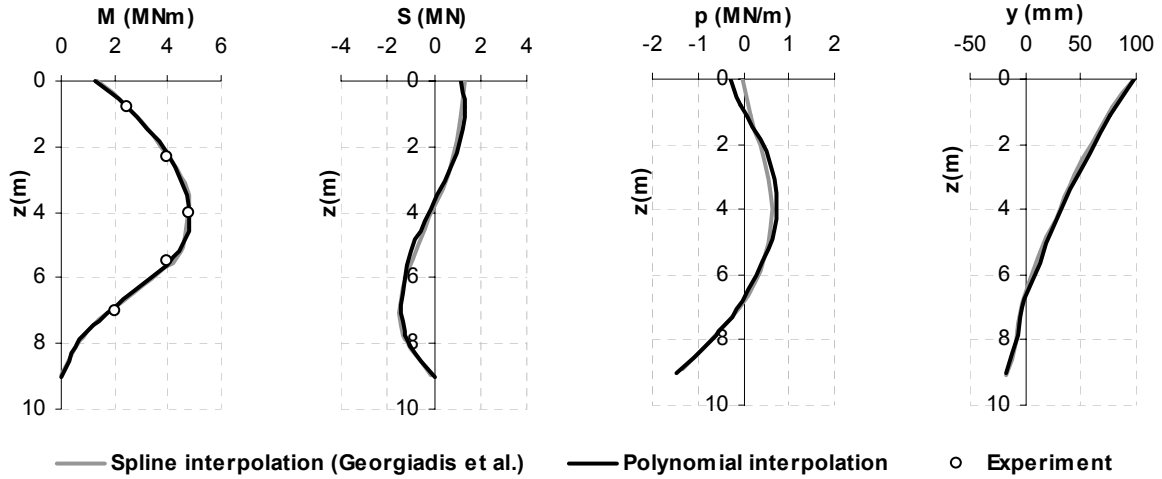


Figure 1. Effect of interpolation function on the pile bending moments, shear forces, soil reactions and pile displacement distribution

should be mentioned though that one of the main tasks in estimating $p(z, t_i)$ and $y_{pile}(z, t_i)$ is the selection of a proper interpolation function to describe pile bending moment distribution.

Interpolation functions

Two of the most common interpolation techniques usually employed to describe bending moments distribution along the pile involve fitting either cubic splines between successive data points (Finn et al., 1983) or higher order polynomial functions to all data points (Ting, 1987). In the present study a 4th order polynomial is adjusted on the bending moments profile at each time instance t_i . Thus the bending moment distribution is given by the equation:

$$M(z, t_i) = A_{t_i} \cdot z^4 + B_{t_i} \cdot z^3 + C_{t_i} \cdot z^2 + D_{t_i} \cdot z + E_{t_i} \quad (3)$$

where $A_{t_i}, B_{t_i}, C_{t_i}, D_{t_i}, E_{t_i}$ are the constant terms of the polynomial. Inclusion of the quadratic term in the polynomial function may result in large lateral resistance values at the ground surface (Wilson, 1998). Thus, the ability of the adopted interpolation function to adequately reproduce lateral soil reactions was further checked with experimental results from a statically loaded single pile obtained by Georgiadis et al. 1992. In the aforementioned study, a fourth order spline function that interpolates between successive experimental points was employed. For the particular pile test, the comparison between spline and polynomial interpolation based on the experimental bending moment measurements is depicted in Fig.1. The relative effect of the interpolation type on the pile shear forces, S , the soil reactions, p , as well as on the pile displacements y is also shown in Fig.1. The satisfactory agreement observed between the different interpolation techniques verifies further the ability of the polynomial approximation to describe the pile response, justifying therefore its use during the adopted procedure.

Estimation of frequency dependent springs and dashpots based on p-y curves

The force (per unit length of the pile) p to displacement y ratio of the Winkler medium at every depth, defines the complex valued frequency-dependent impedance (Kavvas and Gazetas, 1993):

$$S_X = \frac{P}{y} = k_X(\omega) + i \cdot \omega \cdot c_X(\omega) \quad (4)$$

where the real part $k_X(\omega)$ represents the stiffness of the supporting soil and the imaginary component $\omega c_X(\omega)$ reflects the hysteretic and radiation damping generated in the system. Having obtained the p-y loop at each level along the pile length, the damping coefficient is computed via the expression (Badoni & Makris, 1995):

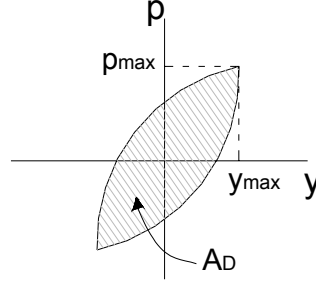


Figure 2. Schematic illustration of the fundamental p-y characteristics

$$c_X(\omega) = \frac{A_D}{\pi \cdot \omega \cdot y_{\max}^2} \quad (5)$$

where A_D is the area of the p-y loop which corresponds to the sum of the hysteretic and radiation damping, ω is the frequency of the input motion and y_{\max} corresponds to the maximum displacement observed in the p-y loop. The parameters A_D and y_{\max} are schematically clarified in Fig.2. The combination of equations 4 and 5 leads to the computation of the real part $k_X(\omega)$ of the complex valued impedance S_X :

$$k_X(\omega) = \left(\left(\frac{p_{\max}}{y_{\max}} \right)^2 - \left(\frac{A_D}{\pi \cdot y_{\max}^2} \right)^2 \right)^{1/2} \quad (6)$$

where p_{\max} is the maximum soil reaction obtained by the double differentiating of the bending moment. A similar approach was adopted by Nogami et al. 1992, utilizing though frequency independent parameters for the description of the soil-pile interaction model that was proposed.

VALIDATION OF THE SOIL-PILE INTERACTION MODEL

Centrifuge testing of single pile supported structures

Centrifuge testing has the advantage of reproducing the initial stress field of soil models of significant depth since it is conducted in high gravitational environments. Moreover, pile bending moments from centrifuge and/or shaking table tests of soil-pile systems have been widely utilized to derive experimental p-y curves (Meymand, 1998, Wilson, 1998 among others). In this study, a well-documented series of dynamic centrifuge tests of pile-supported structures in soft ground was

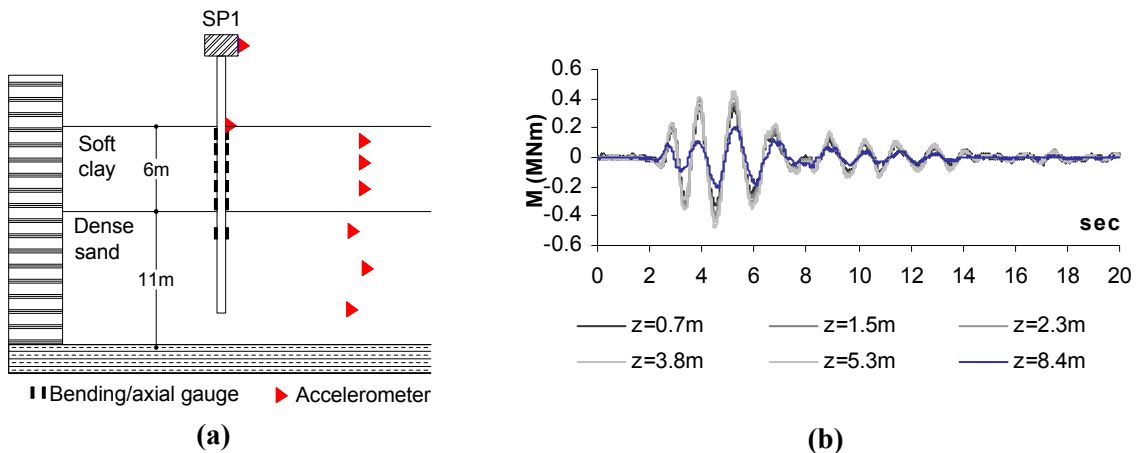


Figure 3. a) Schematic of centrifuge layout and instrumentation b) Bending moment time histories recorded along the pile shaft for the low amplitude motion (0.05g)

implemented for the validation of the procedure described above. The tests were performed in a 9-m-radius centrifuge at the University of California at Davis (Wilson et al. 1997b, Boulanger et al, 1999). The models were tested in a flexible shear beam (FSB) container at a centrifugal acceleration of 30g. The double layer soil profile comprises soft clay overlying dense sand. The lower layer was a 11m thick, dense Nevada sand layer with relative density $Dr=75-80\%$. The upper 6m thick layer was reconstituted Bay Mud with significantly low undrained shear strength, measured up to 8-10KPa at the level of -6m. The centrifuge test layout is schematically shown in Fig 3a where the instrumentation of the examined single pile supported structure (SP1) is also depicted. The instrumentation of the pile consists of six pairs of axial strain gauges attached to the pile (five in the soft clay layer and one in the dense sand layer). Furthermore, accelerometers were placed at the pilehead and the superstructure mass as well as at different levels of the soil profile in order to measure free field soil response. The fixed base period of the single-pile supported structure (SP1), which consisted of a superstructure mass attached to an extension of the pile, was measured equal to 0.29sec. The input motion applied at the base of the centrifuge apparatus was a strong motion record from Port Island in the 1995 Kobe earthquake scaled to $PGA=0.05g$ and $PGA=0.20g$ respectively. Fig.3b shows the bending moments recorded along the pile shaft for the low amplitude motion. In order to avoid high frequency noise during the differentiation process, the bending moments time histories were initially band pass filtered and were then utilized to obtain soil reactions and pile displacements.

Derivation of p-y curves

Based on the recorded bending moments for each intensity level of the base excitation, p-y curves were generated utilizing the procedure described above. In order to obtain pile displacements by double integrating bending moments, two boundary conditions were defined. The first one was determined at the pile head, where the displacement time history was calculated from the recorded acceleration, while the second one was assigned to the pile tip where the net displacement between soil and pile was assumed to be zero. The latter was based on the fact that due to the large stiffness of the sand layer, pile deformations are expected to follow the sand profile displaced shape. Thus, the second boundary

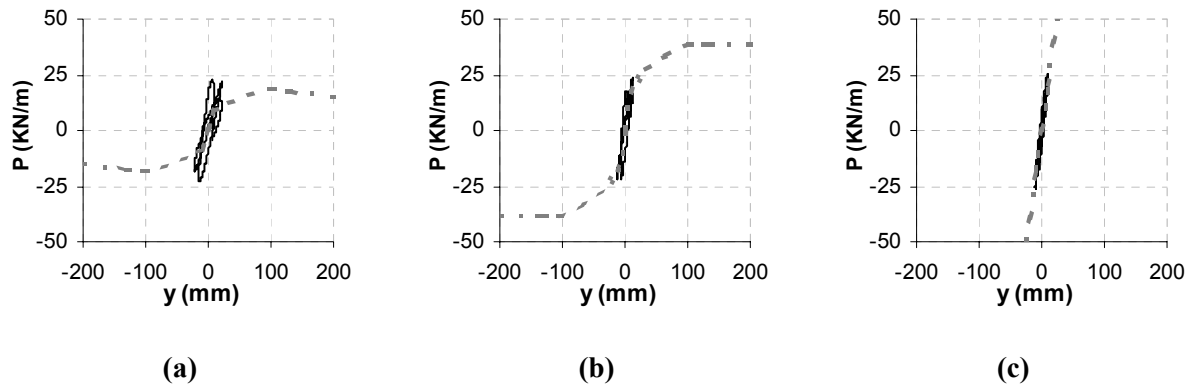


Figure 4. p-y loops computed for 0.05g amplitude of the input motion at a) 1.5m b) 4.5m c) 8.2m

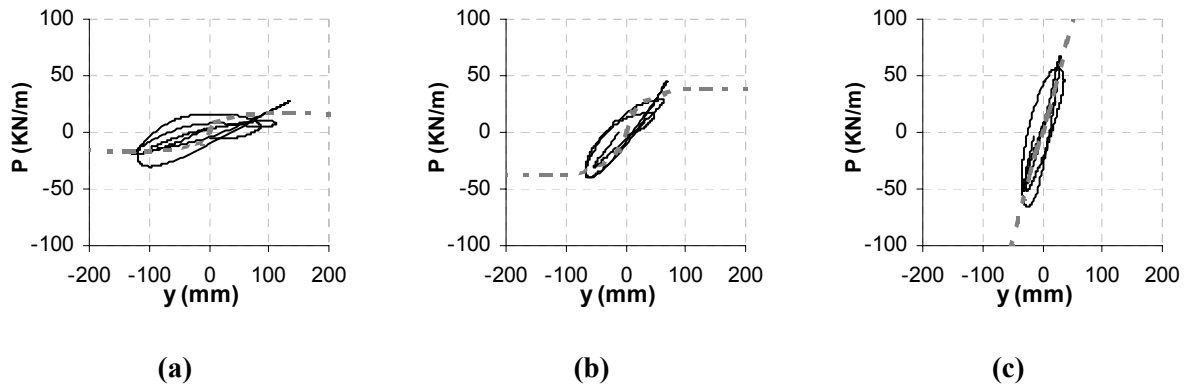


Figure 5. p-y loops computed for 0.20g amplitude of the input motion at a) 1.5m b) 4.5m c) 8.2m

condition was obtained by double integrating the soil acceleration time history recorded at the respective level. Typical results of the computed p-y loops are shown in Fig.4 and Fig.5 corresponding to 0.05g and 0.2g amplitude of the input motion respectively. The p-y loops shown in Figures 4 and 5 were obtained at three different levels along the pile shaft; at the depth of 1.5m and 4.5m in the soft clay layer and at the depth of 8.2m in the dense sand layer. Reviewing the results it is observed that the slope of the p-y loops is increasing with depth, indicating higher soil stiffness levels, while on the other hand the area of the p-y loop is decreasing corresponding to lower levels of damping generated in the soil-pile system. The above findings are related to the reduction of the pile displacements with depth and the associated linear response prevailing over the nonlinear hysteretic action in the soil. The computed p-y loops were also compared to cyclic p-y curves (shown in Figure 4 and 5 with dashed gray lines) commonly used in design practice. In the case of soft clay layer the backbone p-y curve was based upon Matlock's model (Matlock, 1970), which is defined by the equation:

$$\frac{p}{p_{ult}} = 0.5 \cdot \left(\frac{y}{y_{50}} \right)^{1/3} \quad (7)$$

where p_{ult} is the ultimate lateral reaction and y_{50} is the lateral pile displacement at one-half of the ultimate reaction, derived through the following formulas:

$$p_{ult} = c_u \cdot D \cdot N_p \quad (8)$$

$$N_p = \left(3 + \frac{\gamma' \cdot z}{c_u} + \frac{J \cdot z}{D} \right) \leq 9 \quad (9)$$

$$y_{50} = 2.5 \cdot D \cdot \epsilon_{50} \quad (10)$$

where D is the pile diameter, N_p is the lateral bearing capacity factor, γ' is the average buoyant unit weight, z is the depth, c_u is the undrained shear strength and ϵ_{50} is the strain corresponding to a stress of 50% of the ultimate stress in a laboratory stress-strain curve. This variable was taken equal to 0.005 based on published laboratory test data for the particular soil type (Boulanger et al., 1999) while the value of parameter J was selected equal to 0.5 based on Matlock's recommendations for soft clay. For the underlying dense sand layer the p-y curves proposed by Murchison and O'Neill (1984), which are also adopted in the API provisions (API, 1993) were implemented.

The comparison shows that commonly used p-y curves can predict satisfactorily the secant spring stiffness as long as the response remains in the linear elastic range. For higher levels (e.g 0.2g) of excitation though, where significant hysteretic response is activated in the soil, the cyclic p-y curves tend to overestimate soil stiffness.

Analysis of the coupled system utilizing a BDWF model

The experimental p-y loops obtained at each location along the pile length are then utilized to derive frequency dependent springs and dashpots. The computation of the stiffness and damping coefficients, $k_x(\omega)$ and $c_x(\omega)$, was performed using equations 5 and 6 where ω is taken equal to the fundamental frequency of the free field motion recorded at the respective level. It should be mentioned that the Fourier spectra peak amplitudes of the free filed motion recorded in the soft clay layer appeared at the frequency of 1Hz, which is also close to the fundamental frequency of the superstructure including the effect of soil structure interaction. The utilization of the predominant frequency of the input motion has also been adopted in relevant soil-pile-structure interaction studies (Makris et al. 1994) resulting in accurate predictions of the system response. The distribution of the real and imaginary part of the impedance S_x (as defined in equation 4) is shown in Fig. 6a corresponding to the low (0.05g) amplitude excitation.

Having obtained the distributed springs and dashpots for each intensity level of the base motion, the validation of the p-y procedure was then performed utilizing a Beam on Dynamic Winkler Foundation model of the coupled soil-pile-structure system (Fig.6b). The seismic response of the coupled system

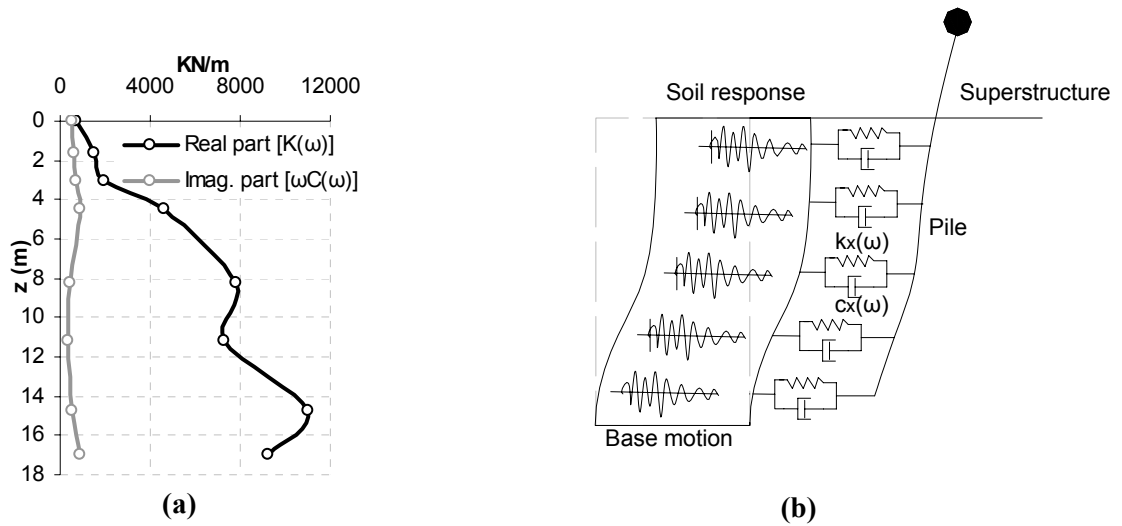


Figure 6. a) Distribution of the spring and dashpots along the pile for the low amplitude motion
b) Schematic illustration of the coupled system used for the validation of the p-y procedure

was analyzed with the general purposed FE code ANSYS (Ansys, 2000). The pile structure system was modeled with linear elastic beam elements while each one of the pile nodes was connected to a Kelvin element having spring and dashpot properties as computed above. Soil displacement time histories that were calculated by double integrating the recorded free field acceleration time histories, were the imposed input motion at the ends of the Kelvin elements. The numerical results obtained from the analysis of the BDWF model were compared to the recorded data in terms of pile bending moments as well as pilehead and superstructure acceleration time histories and response spectra. Typical results are shown in Fig.7 for the two examined intensity levels of the input motion. It is worth noting that, notwithstanding the sharp increase in soil stiffness at the level of the clay-sand interface, the peak bending moment occurs close to the pilehead indicating the effect of the inertial loading transmitted to the pile due to the oscillation of the superstructure (Mylonakis et al, 1997). The very good agreement that is observed between the numerical results and the experimental recordings both in time and frequency domain, proves the ability of the BDWF model to adequately reproduce soil-pile-structure interaction effects. The procedure adopted is therefore validated based on the particular coupled system response under real earthquake excitation. The application of the procedure to different soil-pile systems under different loading considerations as well is examined in the following paragraph.

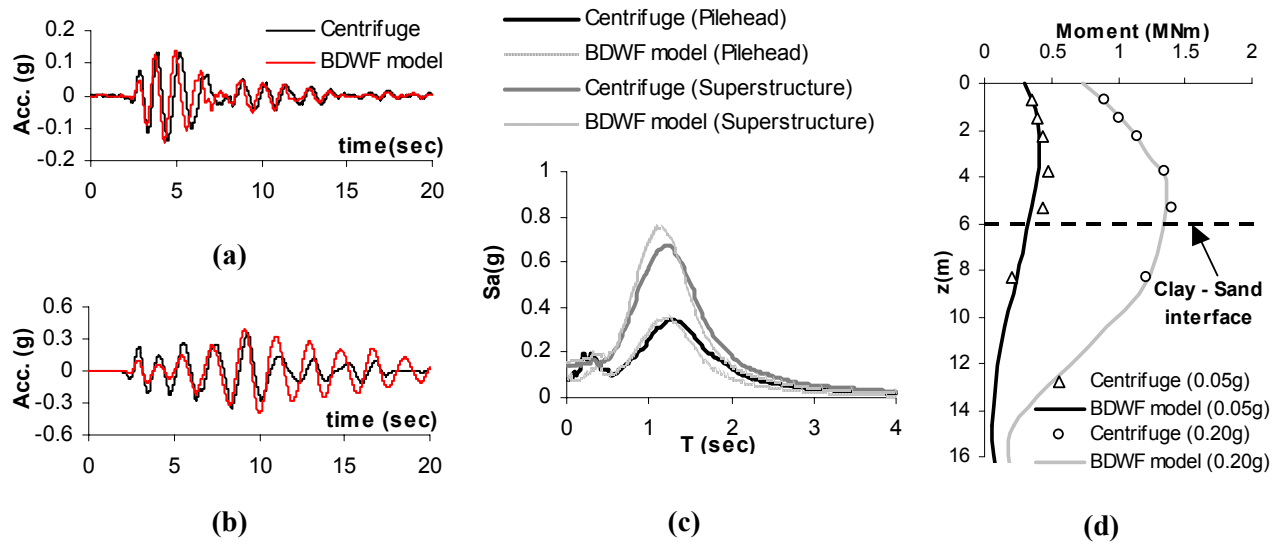


Figure 7. Comparison of the BDWF model response to centrifuge recordings regarding a) Superstructure acceleration (0.05g) b) Superstructure acceleration (0.20g) c) Response spectra (0.05g) d) Peak bending moments

COMPARISON WITH ANALYTICAL MODELS

Harmonic excitation at the pile head

In order to investigate further the applicability of the p-y procedure, the case of a single fixed-head, end bearing pile embedded in a uniform soil profile was also examined (Fig.8a). Therefore, a 30m thick homogeneous soil layer over rigid bedrock was considered, with shear wave velocity $V_s=200\text{m/sec}$, Poisson ratio $\nu=0.33$ and hysteretic damping ratio $\beta_s=0.05$. The first, second and third fundamental frequency of the soil deposit equals to 1.67Hz, 5Hz and 8.33Hz respectively. The pile cross section was considered circular while the relative soil-pile stiffness was taken equal to $E_p/E_s=1000$ and the pile slenderness, i.e the ratio of the pile length L to the pile diameter D , was selected equal to $L/D=20$.

The excitation motion introduced at the pile head consists of a steady state horizontal force, $P=P_0 e^{i2\pi f t}$, with amplitude $P_0=1\text{KN}$ and frequency varying within the range of 1-10Hz. The analysis of the soil-pile system was performed in the frequency domain utilizing a 3D FE model. It should be noted that soil response analyses were also performed in order to verify that wave propagation effects are adequately captured (Rovithis et al, 2007).

Due to the particular type of the excitation force, pile bending moments are generally described by the equation:

$$M(z, f) = M_0(z, f) \cdot e^{i[(2\pi \cdot f) \cdot t + \phi(f, z)]} \quad (11)$$

where the moment amplitude $M_0(f, z)$ and the phase angle $\phi(f, z)$, which are both functions of the excitation frequency f and of the depth z , are obtained from the numerical analyses of the soil-pile system. Fig. 8b shows the distribution with depth of the amplitudes of the bending moments computed at the fundamental frequencies of the soil deposit after being normalized to the maximum bending moment that occurred at the pile head. It is observed that bending moments are practically insensitive to the excitation frequency. Utilizing equation 11 the pile bending moments can be computed in the time domain for a given excitation frequency. Having calculated the time histories of the pile bending moments at each depth, the distribution of moments along the pile's length can be obtained at each time instance and may then be utilized to derive the p-y loops and the distributed springs and dashpots following the procedure described above. The p-y loops obtained at the pilehead that correspond to the fundamental frequencies of the soil deposit are shown in Fig. 8c. It should be mentioned that the results of Fig. 8c are normalized to the maximum p and y values observed in the first natural frequency of the soil deposit. It is noted that the shape of the loop resembles that of an ellipse due to the harmonic type of loading that was employed. Furthermore, when the pilehead is subjected to higher excitation frequencies the pile displacements are getting smaller and the (secant) stiffness of the distributed springs increases.

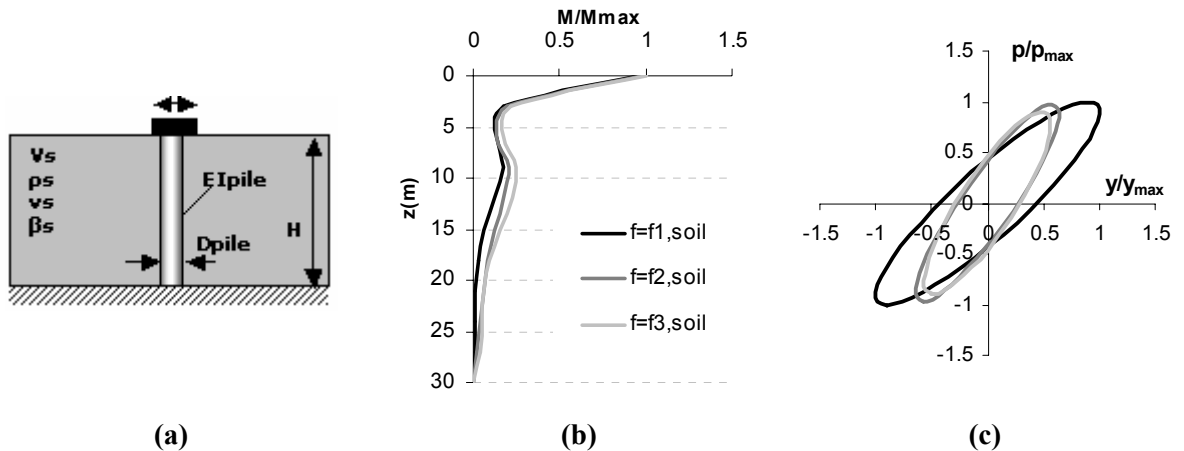


Figure 8. a) Examined soil-pile system b) Normalized distribution with depth of the amplitudes of bending moments computed at the fundamental frequencies of the soil deposit
 (c) Normalized pilehead p-y curves computed at the fundamental frequencies of the soil deposit

This effect has also been observed in [Badoni & Makris, 1995](#) where the nonlinear response of single piles is investigated. The frequency dependent springs and dashpots computed from the generated p-y loops, utilizing [equations 5 and 6](#), were then compared to the following closed form expressions, which are based on the 2D plane strain model of [Gazetas and Dobry, 1984](#):

$$k_x \approx 1.2 \cdot E_s \quad (12)$$

$$c_x = (c_x)_{\text{radiation}} + (c_x)_{\text{hysteretic}} \approx 2 \cdot a_0^{-1/4} \cdot \rho_s \cdot V_s \cdot D \cdot \left[1 + \left(\frac{V_{La}}{V_s} \right)^{5/4} \right] + 2 \cdot \beta_s \cdot \frac{k_x}{\omega} \quad (13)$$

The above formulas have also been used in various soil-pile interaction studies ([Makris and Gazetas, 1992](#), [Gazetas et al., 1992](#), [Mylonakis, 1995](#), [Ahmad et al., 2006](#)). The parameter V_{La} represents the Lysmer's analog velocity and is given by the equation:

$$V_{La} = \frac{3.4 \cdot V_s}{\pi \cdot (1 - \nu)} \quad (14)$$

At very shallow depths ($z \leq 2.5D$), [equation 13](#) is replaced by:

$$c_x = (c_x)_{\text{radiation}} + (c_x)_{\text{hysteretic}} \approx 2 \cdot a_0^{-1/4} \cdot \rho_s \cdot V_s \cdot D \cdot \left(\frac{\pi}{4} \right)^{3/4} + 2 \cdot \beta_s \cdot \frac{k_x}{\omega} \quad (15)$$

in order to incorporate the generation of surface waves due to the presence of the stress-free ground surface. Furthermore, the c_x values obtained from [equations 13 and 15](#) apply for frequencies ω higher than the first natural frequency, ω_s , of the soil deposit. For frequencies lower than ω_s radiation damping is significantly small compared to the hysteretic type of soil damping. On the other hand the expression for the spring coefficient ([Eq.12](#)) has been determined by matching the horizontal stiffness at the head of the pile embedded in the Winkler soil model and the continuum (finite element) soil model. However, an approximation introduced in obtaining [equation 12](#) is to neglect the influence of pile slenderness L/D and relative soil-pile stiffness E_p/E_s ([Makris et al. 1996](#)). This effect is, in contrast, incorporated in the analytical expression proposed by [Kavvas and Gazetas, 1993](#), which is given by the following equation corresponding to a pile embedded in a two layered soil profile.

$$k_x \approx \left[\frac{2}{1 - \nu_s^2} \cdot \left(\frac{E_{sa} \cdot D^4}{E_p \cdot I_p} \right)^{1/8} \cdot \left(\frac{L}{D} \right)^{1/8} \cdot \left(\frac{H_a}{H_b} \right)^{1/12} \cdot \left(\frac{V_{sa}}{V_{sb}} \right)^{1/15} \right] \cdot E_s \quad (16)$$

and which for the case of a homogeneous soil and a pile of circular cross-section simplifies to:

$$k_x \approx \frac{3}{1 - \nu_s^2} \cdot \left(\frac{E_s}{E_p} \right)^{1/8} \cdot \left(\frac{L}{D} \right)^{1/8} \cdot E_s \quad (17)$$

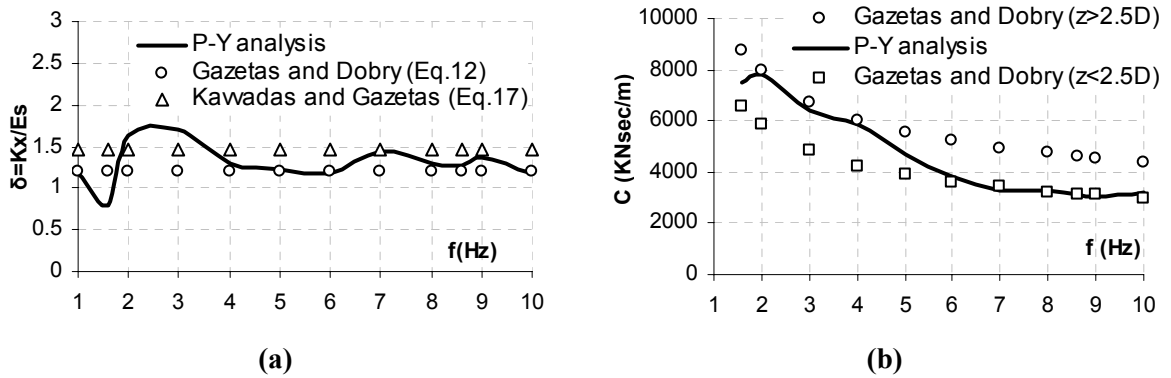


Figure 9. Comparison of analytical methods and p-y analysis results regarding real (a) and imaginary (b) part of the pilehead dynamic stiffness (Homogeneous soil profile, fixed head pile, $E_p/E_s=1000$, $L/D=20$, $\beta_s=0.05$)

Fig.9a and Fig.9b show the comparison between the springs and dashpots obtained by the p-y procedure and by utilizing the analytical expressions described above. In particular, Fig.9a shows the frequency dependency of the real part $k_x(\omega)$ of the impedance S_x normalized to the soil modulus of elasticity E_s while Fig.9b depicts the variation with frequency of the dashpot coefficient C_x (given in equation 5) corresponding to the sum of material and radiation damping. The satisfactory agreement that is observed throughout the examined frequency range verifies further the applicability of the adopted procedure. Indeed the damping term is reduced with increasing frequency while the real part of the dynamic soil stiffness is practically insensitive to the frequency content of the excitation force with an exception at the low frequency range where a slight difference is observed. This should be attributed to resonance effects that dominate the response at the first natural frequency of the soil deposit.

Seismic excitation at the bedrock level

The experimental recordings from the centrifuge test used in the validation process were further utilized in order to investigate the effect of the soil springs and dashpots coefficients on the coupled soil-pile-structure system response where both kinematic and inertial soil-pile-structure interaction is incorporated. For this reason, the BDWF model shown in Fig. 6b was reanalyzed using spring and dashpot coefficients calculated from the existing analytical expressions and the computed response was compared to the one obtained utilizing the p-y loops for the determination of the springs and dashpots. Since the examined coupled system consists of a pile-supported structure founded on a two-layered soil deposit, the analytical expressions given by equation 16 and equations 12 or 13 were employed as more appropriate to the particular case. The soil shear wave velocity, V_s , and the hysteretic damping ratio, β_s introduced in the analytical equations, were properly modified according to the developed shear strains during the experiment. These strain compatible values were determined from an equivalent linear soil response analysis which results in shear wave velocity profiles of decreasing values with the intensity of the input earthquake motion. Further details on the calibration procedure of the soil dynamic properties can be found in Pitilakis et al. 2004, Kirtas et al. 2006 and Rovithis et al. 2006. The coupled system response obtained by utilizing the two different approaches of calculating the soil springs and dashpots

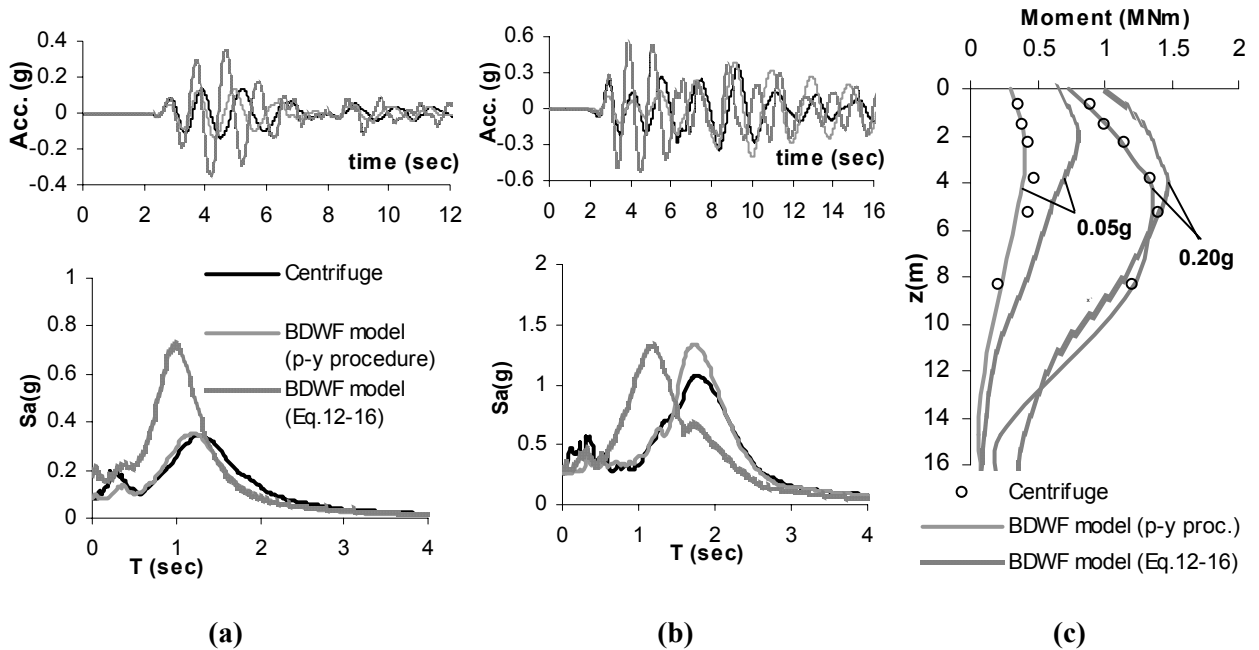


Figure 10. Comparison of the coupled system response utilizing analytical expressions and p-y procedure to estimate springs and dashpots a) Pilehead response spectra and superstructure acceleration time history (0.05g) b) Pilehead response spectra and superstructure acceleration time history (0.20g) c) Peak pile bending moments

is compared in Fig.10 in terms of structural and pile response. It is observed that when the analytical expressions are implemented, structural accelerations and pile bending moments seem to be overestimated with respect to the recorded response. Especially for the low amplitude motion the computed structural response resulted approximately two times larger than the recorded one, while higher peak bending moments were also observed. Furthermore the superstructure responded at a higher frequency than the actual recordings show. On the contrary the proposed p-y model succeeds in capturing correctly the basic characteristics of the soil-pile system response. Although the above remarks should not be generalized, they indicate that a more rigorous estimation of distributed springs and dashpots, as it is proposed and developed herein, may be required when the coupled soil-pile-structure system with a particular soft foundation soil is investigated.

CONCLUSIONS

The present paper focuses on the evaluation of dynamic soil – pile interaction based on back calculated p-y loops emphasizing on the computation of frequency dependent springs and dashpots that could be employed within the framework of a beam on Dynamic Winkler foundation modeling. The adopted procedure was validated with centrifuge tests results and was further compared to existing analytical formulas of soil springs and dashpots commonly utilized in seismic soil-pile interaction studies. The application of the procedure to different soil-pile systems as well as to different loading scenarios revealed that p-y loops provide a satisfactory estimation of frequency dependent springs and dashpots which in turn can adequately reproduce soil-pile interaction mechanism. On the other hand, utilizing existing analytical springs and dashpots formulas to the examined coupled system showed that under certain conditions (i.e presence of sharp soil stiffness discontinuities combined with strong inertial interaction) pile and structural response may be overestimated and hence further improvement of the existing analytical expressions is needed.

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