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NONLINEAR SEISMIC RESPONSE ANALYSIS OF PILES IN NONLINEAR VISCOELASTIC FOUNDATION

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Abstract. *Dynamic response of piles has been the subject of extensive investigations during the past few decades as in high seismicity regions pile foundation is widely used to support superstructures such as buildings, bridges and offshore platforms. In this investigation, a boundary element method is developed for the nonlinear kinematic seismic interaction of piles of arbitrary doubly symmetric simply or multiply connected constant cross section, embedded in viscoelastic foundation, undergoing moderate large deflections under general boundary conditions, taking into account the effects of rotary inertia and shear deformation by employing the concept of shear deformation coefficients. The soil deposits are modeled as a layered Winkler-type medium, while the seismic excitation motion is obtained by means of one dimensional wave propagation analysis. Five boundary value problems are formulated with respect to the transverse displacements, to the axial displacement and to two stress functions and solved using the Analog Equation Method, a BEM based method. Application of the boundary element technique yields a system of nonlinear Differential – Algebraic Equations, which is solved using an efficient time discretization scheme, from which the transverse and axial displacements are computed. The evaluation of the shear deformation coefficients is accomplished from the aforementioned stress functions using only boundary integration. The proposed model takes into account the coupling effects of bending and shear deformations along the member as well as the shear forces along the span induced by the applied axial loading. Numerical examples employing recorded accelerograms of well-known earthquakes are worked out in order to illustrate the efficiency, the range of applications and wherever possible the accuracy of the developed method.*

1 INTRODUCTION

The seismic response of piles during earthquake excitation is an area of extensive active research, as in high seismicity regions pile foundation is widely used to support superstructures such as buildings, bridges and offshore platforms. Piles, which are subjected to both axial and lateral loading that result from the supported structures during and after seismic event, develop a nonlinear dynamic response. Thus, the study of nonlinear effects on the dynamic analysis of structural elements is essential in civil engineering applications, wherein weight saving is of paramount importance. This non-linearity results from retaining the square of the slope in the strain–displacement relations (intermediate non-linear theory), avoiding in this way the inaccuracies arising from a linearized second – order analysis.

During seismic excitations, piles undergo stresses due both to the motion of the superstructure (inertial interaction) and to that of the surrounding soil (kinematic interaction). Despite the documented cases of kinematically induced damage to piles [1-4], this mode of response does not commonly receive the proper attention by engineers who take into account stresses induced by the inertial interaction, which may be responsible for pile head failure, but neglect the effects of the kinematic interaction that may be responsible for pile failure in the case of layered soils with highly contrasting mechanical characteristics [4-7].

The pile-soil kinematic interaction was first studied by Tajimi [8] and Penzien [9] who proposed an analytical and a numerical solution, respectively. Since then, many authors have proposed simplified procedures and analytical solutions for the evaluation of the bending moments due to the kinematic interaction. Margason [10] suggested computing pile kinematic bending moments from the evaluation of the free-field soil curvatures by means of a finite difference approach without accounting for soil–pile interaction and radiation damping. Under the same perspective numerous studies have been presented [11-15] focusing on the dynamic response of the pile head.

Lately, El Naggar and Novak [16] studied the lateral response of singe piles and pile groups accounting the nonlinear behaviour of the soil adjacent to the pile and discontinuity conditions at the pile-soil interface. Nikolaou et al. [5] studied the kinematic interaction implementing the beam-on-dynamic-Winkler-foundation (BDWF) assuming free-head pile. A analytical solution was proposed for the case of homogeneous soil, while a parametric investigation was performed on the bending strains in a pile embedded in a two- layered soil deposit subjected to harmonic steady-state shear waves and proposed a closed-form expression for the evaluation of the maximum bending moment at the interface between layers. Moreover, real case study was performed, executing analyses using real accelerograms and real soil profiles in order to find a correlation between the steady-state and a real transitory response. The formulas proposed may be used to calculate the bending moment at the cross-section placed at the interface of two layers with a sharp change of stiffness but is not valid for calculating bending moments at the pile head. Moreover, Padron et. al. [17] studied a BEM–FEM coupling model for the time harmonic dynamic analysis of piles and pile groups embedded in an elastic half-space where piles are modelled using finite elements as a beam according to the Bernoulli hypothesis, while the soil is modelled using boundary elements as a continuum, semi-infinite, isotropic, homogeneous or zoned homogeneous, linear, viscoelastic medium. Hu et. al. [18] presented the nonlinear partial differential equation governing the nonlinear transverse vibration of pile under the assumption that both the materials of the pile and the soil obey nonlinear elastic and linear viscoelastic constitutive relations while the frequency and the response of the system have been obtained by the complex mode method and the method of multiple time scales. Finally, Dezi et. al. [19] studied the kinematic seismic interaction of single pile embedded in soil deposits focusing the attention on the bending moments induced by the transient motion. The analysis was performed by modeling the pile like an Euler–Bernoulli beam embedded in a layered Winkler-type medium. The excitation motion is obtained by means of a one-D propagation analysis.

In all of the aforementioned literature, the geometrical nonlinearity has not been taken into account. Having in mind that piles are often heavily loaded from the superstructure the nonlinear effects arising from the influence of the axial load should be considered. Thus, the aforementioned study takes into account the influence of the action of axial, lateral forces and end moments on the deformed shape of the structural element. Moreover, due to the intensive use of materials having relatively high transverse shear modulus and the need for beam members with high natural frequencies the error incurred from the ignorance of the effect of shear deformation may be substantial, particularly in the case of heavy lateral loading. The Timoshenko-Rayleigh beam theory, which includes shear deformation and rotary inertia effects, has an extended range of applications as it allows treatment of short piles (depth is large relative to length) and piles where higher modes are excited.

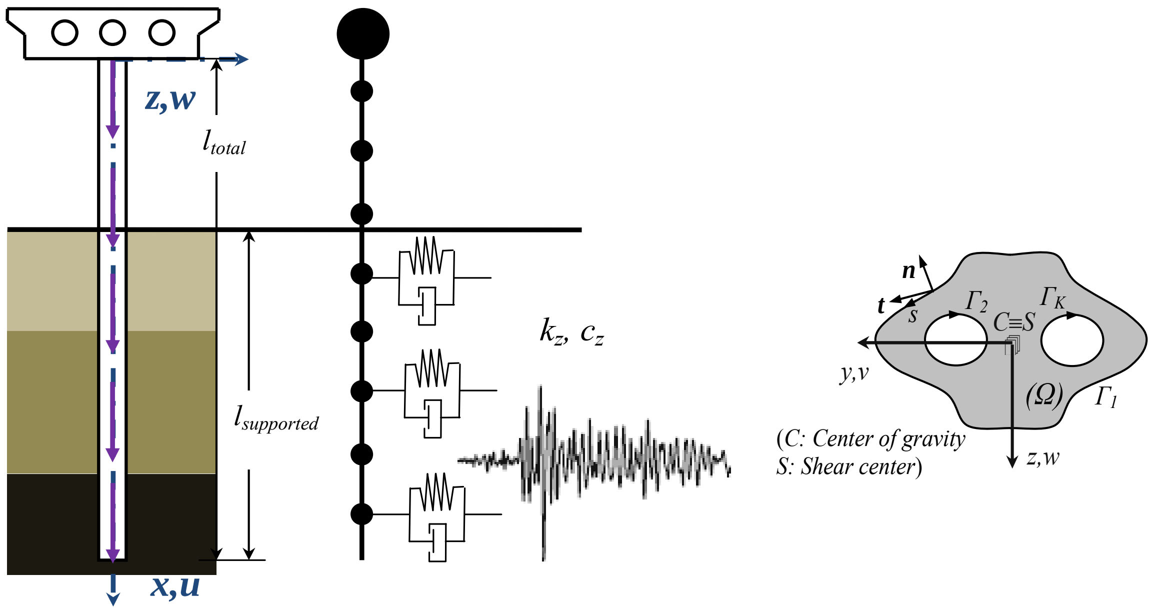
In this paper, a boundary element method is developed for the nonlinear kinematic seismic interaction of piles of arbitrary doubly symmetric simply or multiply connected constant cross section, partially embedded in viscoelastic foundation, undergoing moderate large deflections under general boundary conditions, taking into account the effects of shear deformation and rotary inertia. The pile is subjected to the combined action of the arbitrarily distributed or concentrated time dependent axial loading and to transverse seismic loading taking into account the kinematic soil-pile interaction. To account for shear deformations, the concept of shear deformation coefficients is used. The soil deposits are modeled as a layered Winkler-type medium, while the seismic excitation motion is obtained by means of one dimensional wave propagation analysis. Five boundary value problems are formulated with respect to the transverse displacements, to the axial displacement and to two stress functions and solved using the Analog Equation Method [20], a BEM based method. Application of the boundary element technique yields a nonlinear coupled system of equations of motion from which the transverse and axial displacements are computed by an iterative process.The evaluation of the shear deformation coefficients is accomplished from the aforementioned stress functions using only boundary integration. The essential features and novel aspects of the present formulation compared with previous ones are summarized as follows.

i. Both kinematic and inertia interaction can be employed.
ii. The multilayered soil deposits have been models as a Winkler-type viscoelastic foundation.
iii. The seismic excitation is obtained at every point of the pile by means of one dimensional shear wave propagation analysis.
iv. Shear deformation effect and rotary inertia are taken into account on the nonlinear dynamic analysis of piles subjected to arbitrary loading.
v. The pile is supported by the most general nonlinear boundary conditions including elastic support or restrain, while its cross section is an arbitrary doubly symmetric one.
vi. The proposed model takes into account the coupling effects of bending and shear deformations along the member as well as shear forces along the span induced by the applied axial loading.
vii. The shear deformation coefficients are evaluated using an energy approach, instead of Timoshenko's [21] and Cowper's [22].
viii. The effect of the material's Poisson ratio v is taken into account.
ix. The proposed method employs a BEM approach (requiring boundary discretization) resulting in line or parabolic elements instead of area elements of the FEM solutions (requiring the whole cross section to be discretized into triangular or quadrilateral area

elements), while a small number of line elements are required to achieve high accuracy.

2 STATEMENT OF THE PROBLEM

Let us consider a prismatic pile of length l (Fig.1a), of constant arbitrary doubly symmetric cross-section of area A. The homogeneous isotropic and linearly elastic material of the pile cross-section, with modulus of elasticity E, shear modulus G and Poisson's ratio v occupies the two dimensional multiply connected region Ω of the y,z plane and is bounded by the $\Gamma_j\left(j=1,2,...,K\right)$ boundary curves, which are piecewise smooth, i.e. they may have a finite number of corners (Fig.1b). Consider Cyz to be the principal bending coordinate system through the cross section's centroid.



(a) (b)

Figure 1: x-z Plane of a pile partially embedded in nonlinear viscoelastic foundation (a) with an arbitrary cross-section occupying the two dimensional region Ω (b).

The pile is partially embedded in soil deposits which are modeled as a layered Winkler-type viscoelastic medium. The foundation model is characterized by the linear Winkler moduli k_y, k_z and the damping coefficients c_y, c_z corresponding to the directions y, z respectively. Thus, the foundation reaction is written as

$$p_{sy}\left(x,t\right)=k_{Ly}v\left(x,t\right)+c_y\frac{\partial v\left(x,t\right)}{\partial t} \tag{1}$$

$$p_{sz}\left(x,t\right)=k_{Lz}w\left(x,t\right)+c_z\frac{\partial w\left(x,t\right)}{\partial t} \tag{2}$$

The pile is subjected to the combined action of the arbitrarily distributed or concentrated time dependent axial loading $p_x = p_x(x,t)$ and to transverse seismic loading taking into account the kinematic soil-pile interaction $p_y = p_y(x,t)$, $p_z = p_z(x,t)$ acting in the y, z directions, respectively.

Under the action of the aforementioned loading, the displacement field of the pile taking into account shear deformation effect is given as

$$\bar{u}(x,y,z,t) = u(x,t) - y\theta_z(x,t) + z\theta_y(x,t) \tag{3}$$

$$\bar{v}(x,t) = v(x,t) \tag{4}$$

$$\bar{w}(x,t) = w(x,t) \tag{5}$$

where $\bar{u}$, $\bar{v}$, $\bar{w}$ are the axial and transverse pile displacement components with respect to the Cyz system of axes; $u(x,t)$, $v(x,t)$, $w(x,t)$ are the corresponding components of the centroid C and $\theta_y(x,t)$, $\theta_z(x,t)$ are the angles of rotation due to bending of the cross-section with respect to its centroid.

Employing the strain-displacement relations of the three - dimensional elasticity for moderate displacements, the following strain components can be easily obtained

$$\varepsilon_{xx} = \frac{\partial \bar{u}}{\partial x} + \frac{1}{2}\left[\left(\frac{\partial \bar{v}}{\partial x}\right)^2 + \left(\frac{\partial \bar{w}}{\partial x}\right)^2\right] \tag{6}$$

$$\gamma_{xz} = \frac{\partial \bar{w}}{\partial x} + \frac{\partial \bar{u}}{\partial z} + \left(\frac{\partial \bar{v}}{\partial x}\frac{\partial \bar{v}}{\partial z} + \frac{\partial \bar{w}}{\partial x}\frac{\partial \bar{w}}{\partial z}\right) \tag{7}$$

$$\gamma_{xy} = \frac{\partial \bar{v}}{\partial x} + \frac{\partial \bar{u}}{\partial y} + \left(\frac{\partial \bar{v}}{\partial x}\frac{\partial \bar{v}}{\partial y} + \frac{\partial \bar{w}}{\partial x}\frac{\partial \bar{w}}{\partial y}\right) \tag{8}$$

$$\varepsilon_{yy} = \varepsilon_{zz} = \gamma_{yz} = 0 \tag{9}$$

where it has been assumed that for moderate displacements $\left(\partial \bar{u}/\partial x\right)^2 << \partial \bar{u}/\partial x$, $\left(\partial \bar{u}/\partial x\right)\left(\partial \bar{u}/\partial z\right) << \left(\partial \bar{u}/\partial x\right) + \left(\partial \bar{u}/\partial z\right)$, $\left(\partial \bar{u}/\partial x\right)\left(\partial \bar{u}/\partial y\right) << \left(\partial \bar{u}/\partial x\right) + \left(\partial \bar{u}/\partial y\right)$. Substituting the displacement components to the strain-displacement relations, the strain components can be written as

$$\varepsilon_{xx}(x,y,z,t) = u' + z\theta_y' - y\theta_z' + \frac{1}{2}\left(v'^2 + w'^2\right) \tag{10}$$

$$\gamma_{xy} = v' - \theta_z \tag{11}$$

$$\gamma_{xz} = w' + \theta_y \tag{12}$$

where γ_{xy}, γ_{xz} are the additional angles of rotation of the cross-section due to shear deformation .

Considering strains to be small, employing the second Piola – Kirchhoff stress tensor and assuming an isotropic and homogeneous material, the stress components are defined in terms of the displacement ones as

$$S_{xx} = E\left[u' + z\theta_y{}' - y\theta_z{}' + \frac{1}{2}\left(v'^2 + w'^2\right)\right] \tag{13}$$

$$S_{xy} = G \cdot \left(v' - \theta_z\right) \tag{14}$$

$$S_{xz} = G \cdot \left(w' + \theta_y\right) \tag{15}$$

On the basis of Hamilton's principle, the variations of the Lagrangian equation defined as

$$\delta\int_{t_1}^{t_2}\left(U - K - W_{ext}\right)dt = 0 \tag{16}$$

and expressed as a function of the stress resultants acting on the cross section of the pile in the deformed state provide the governing equations and the boundary conditions of the pile subjected to nonlinear vibrations. In eqn. (16), $\delta(\cdot)$ denotes variation of quantities while U, K, W_{ext} are the strain energy, the kinetic energy and the external load work. Moreover, the stress resultants of the pile using the expressions of the stress components are given as

$$N = EA\left[u' + \frac{1}{2}\left(v'^2 + w'^2\right)\right] \tag{17}$$

$$M_y = EI_y\theta_y{}' \tag{18}$$

$$M_z = EI_z\theta_z{}' \tag{19}$$

$$Q_y = GA_y\gamma_{xy} \tag{20}$$

$$Q_z = GA_z\gamma_{xz} \tag{21}$$

where A is the cross section area, I_y, I_z the moments of inertia with respect to the principle bending axes and GA_y, GA_z are its shear rigidities of the Timoshenko's beam theory, where

$$A_z = \kappa_z A = \frac{1}{a_z}A \qquad A_y = \kappa_y A = \frac{1}{a_y}A \tag{22}$$

are the shear areas with respect to y, z axes, respectively with κ_y, κ_z the shear correction factors and a_y, a_z the shear deformation coefficients. Substituting the stress components and the strain resultants to the strain energy variation and employing eqn. (16), the equilibrium equations of the pile are derived as

$$-EA\left(u''+w'w''+v'v''\right)+\rho A\ddot{u}=p_x \tag{23}$$

$$\begin{aligned}&EI_z v'''' + \rho A\ddot{v} + p_{sy} + \frac{EI_z}{GA_y}\left(\left(Nv'\right)''' - \rho A\frac{\partial^2\ddot{v}}{\partial x^2} - p_{sy}'' + p_y''\right) - \left(Nv'\right)' \\ &-\rho I_z\frac{\partial^2\ddot{v}}{\partial x^2} - \frac{\rho I_z}{GA_y}\left(\frac{\partial^2\left(Nv'\right)'}{\partial t^2} - \rho A\ddot{\ddot{v}} - \ddot{p}_{sy} + \ddot{p}_y\right) = p_y\end{aligned} \tag{24}$$

$$\begin{aligned}&EI_y w'''' + \rho A\ddot{w} + p_{sz} + \frac{EI_y}{GA_z}\left(\left(Nw'\right)''' - \rho A\frac{\partial^2\ddot{w}}{\partial x^2} - p_{sz}'' + p_z''\right) - \left(Nw'\right)' \\ &-\rho I_z\frac{\partial^2\ddot{w}}{\partial x^2} - \frac{\rho I_y}{GA_z}\left(\frac{\partial^2\left(Nw'\right)'}{\partial t^2} - \rho A\ddot{\ddot{w}} - \ddot{p}_{sz} + \ddot{p}_z\right) = p_z\end{aligned} \tag{25}$$

where the transverse seismic loading taking into account the kinematic soil-pile interaction can be written as $p_y\left(x,t\right)=k_y u_{ff}+c_y\dot{u}_{ff}$, $p_z\left(x,t\right)=k_z u_{ff}+c_z\dot{u}_{ff}$ with $u_{ff},\dot{u}_{ff}$ being the displacement and velocity of the free filed motion.

Eqns. (23)-(25) constitute the governing differential equations of a Timoshenko- Rayleigh pile, partially embedded in viscoelastic foundation, subjected to nonlinear vibrations due to the combined action of time dependent axial and transverse loading. These equations are also subjected to the pertinent boundary conditions of the problem, which are given as

$$a_1 u\left(x,t\right)+\alpha_2 N\left(x,t\right)=\alpha_3 \tag{26}$$

$$\beta_1 v\left(x,t\right)+\beta_2 V_y\left(x,t\right)=\beta_3 \qquad \bar{\beta}_1\theta_z\left(x,t\right)+\bar{\beta}_2 M_z\left(x,t\right)=\bar{\beta}_3 \tag{27}$$

$$\gamma_1 w\left(x,t\right)+\gamma_2 V_z\left(x,t\right)=\gamma_3 \qquad \bar{\gamma}_1\theta_y\left(x,t\right)+\bar{\gamma}_2 M_y\left(x,t\right)=\bar{\gamma}_3 \tag{28}$$

at the pile ends $x=0,l$, together with the initial conditions

$$u\left(x,0\right)=\bar{u}_0\left(x\right) \qquad \dot{u}\left(x,0\right)=\dot{\bar{u}}_0\left(x\right) \tag{29}$$

$$v\left(x,0\right)=\bar{v}_0\left(x\right) \qquad \dot{v}\left(x,0\right)=\dot{\bar{v}}_0\left(x\right) \tag{30}$$

$$w\left(x,0\right)=\bar{w}_0\left(x\right) \qquad \dot{w}\left(x,0\right)=\dot{\bar{w}}_0\left(x\right) \tag{31}$$

where $\bar{u}_0\left(x\right),\bar{v}_0\left(x\right),\bar{w}_0\left(x\right),\dot{\bar{u}}_0\left(x\right),\dot{\bar{v}}_0\left(x\right)$ and $\dot{\bar{w}}_0\left(x\right)$ are prescribed functions. In eqns. (27), (28) V_y, V_z, M_z, M_y and θ_y, θ_z are the reactions, the bending moments and the angles of rotation due to bending with respect to y, z, respectively.

Finally, $\alpha_k,\beta_k,\bar{\beta}_k,\gamma_k,\bar{\gamma}_k$ ($k=1,2,3$) are functions specified at the pile ends $x=0,l$. Eqs. (26)-(28) describe the most general nonlinear boundary conditions associated with the problem at hand and can include elastic support or restraint. It is apparent that all types of the conventional boundary conditions (clamped, simply supported, free or guided edge) can be derived from these equations by specifying appropriately these functions (e.g. for a clamped edge it is $\alpha_2 = \alpha_3 = \beta_2 = \beta_3 = \gamma_2 = \gamma_3 = \overline{\beta}_2 = \overline{\beta}_3 = \overline{\gamma}_2 = \overline{\gamma}_3 = 0$, $\alpha_1 = \beta_1 = \gamma_1 = 1$, $\overline{\beta}_1 = \overline{\gamma}_1 = 1$).

The solution of the initial boundary value problem given from eqns. (23)-(25), subjected to the boundary conditions (26)-(28) and the initial conditions (29)-(31) which represents the nonlinear flexural dynamic analysis of a Timoshenko-Rayleigh pile, partially embedded in viscoelastic foundation, presumes the evaluation of the shear deformation coefficients a_y, a_z, corresponding to the principal coordinate system Cyz. These coefficients are established equating the approximate formula of the shear strain energy per unit length [23] $U_{appr.} = \dfrac{a_y Q_y^2}{2AG} + \dfrac{a_z Q_z^2}{2AG}$ with the exact one given from $U_{exact} = \int_\Omega \dfrac{(\tau_{xz})^2 + (\tau_{xy})^2}{2G} d\Omega$ and are obtained as [24]

$$a_y = \frac{1}{\kappa_y} = \frac{A}{\Delta^2} \int_\Omega \left[(\nabla\Theta) - \boldsymbol{e} \right] \cdot \left[(\nabla\Theta) - \boldsymbol{e} \right] d\Omega \tag{32}$$

$$a_z = \frac{1}{\kappa_z} = \frac{A}{\Delta^2} \int_\Omega \left[(\nabla\Phi) - \boldsymbol{d} \right] \cdot \left[(\nabla\Phi) - \boldsymbol{d} \right] d\Omega \tag{33}$$

where $(\tau_{xz})_j, (\tau_{xy})_j$ are the transverse (direct) shear stress components, $(\nabla) \equiv \boldsymbol{i_y}(\partial/\partial y) + \boldsymbol{i_z}(\partial/\partial z)$ is a symbolic vector with $\boldsymbol{i_y}, \boldsymbol{i_z}$ the unit vectors along y and z axes, respectively. Moreover, $\Delta = 2(1+\nu) I_y I_z$ where ν is the Poisson ratio of the cross section material, $\boldsymbol{e} = \left(\nu I_y \dfrac{y^2 - z^2}{2} \right) \boldsymbol{i_y} + \nu I_y yz \boldsymbol{i_z}$ and $\boldsymbol{d} = \nu I_z yz \boldsymbol{i_y} - \left(\nu I_z \dfrac{y^2 - z^2}{2} \right) \boldsymbol{i_z}$ while $\Theta(y,z)$ and $\Phi(y,z)$ are stress functions which are evaluated from the solution of the following Neumann type boundary value problems [24]

$$\nabla^2 \Theta = -2 I_y y \quad \text{in } \Omega \tag{34}$$

$$\frac{\partial\Theta}{\partial n} = \boldsymbol{n} \cdot \boldsymbol{e} \quad \text{on } \Gamma = \bigcup_{j=1}^{K+1} \Gamma_j \tag{35}$$

$$\nabla^2 \Phi = -2 I_z z \quad \text{in } \Omega \tag{36}$$

$$\frac{\partial\Phi}{\partial n} = \boldsymbol{n} \cdot \boldsymbol{d} \quad \text{on } \Gamma = \bigcup_{j=1}^{K+1} \Gamma_j \tag{37}$$

where $\boldsymbol{n}$ is the outward normal vector to the boundary Γ. In the case of negligible shear deformations $a_z = a_y = 0$. It is also worth here noting that the boundary conditions (37), (39) have been derived from the physical consideration that the traction vector in the direction of the normal vector $\boldsymbol{n}$ vanishes on the free surface of the pile.

3 INTEGRAL REPRESENTATIONS – NUMERICAL SOLUTION

According to the precedent analysis, the nonlinear flexural dynamic analysis of Timoshenko-Rayleigh pile, partially embedded in viscoelastic foundation, undergoing moderate large deflections reduces in establishing the displacement components $u(x,t)$ and $v(x,t)$, $w(x,t)$ having continuous derivatives up to the second order and up to the fourth order with respect to x, respectively, and also having derivatives up to the second order with respect to t (ignoring the inertia terms of the fourth order [25]). These displacement components must satisfy the coupled governing differential eqns. (23)-(25) inside the pile, the boundary conditions (26)-(28) at the pile ends $x=0,l$ and the initial conditions (29)-(31). Eqns. (23)-(25) are solved using the Analog Equation Method [20] as it is developed for hyperbolic differential equations [26].

4 NUMERICAL EXAMPLE

On the basis of the analytical and numerical procedures presented, a computer program has been written and a representative examples employing recorded accelerograms of well-known earthquakes are worked out in order to illustrate the efficiency, the range of applications and wherever possible the accuracy of the developed method. In this example, the results have been obtained using L=41 nodal points along the pile while both the analysis duration and the time step are considered so to follow the recorder data.

4.1 Example

A fully embedded pile of length $l=6.0m$ of circular cross section of diameter $D=0.5m$ ($E=25GPa$, $\nu=0.2$, $\rho=2.5Mg/m^3$, $A=0.196\text{m}^2$, $I_y=I_z=30.68\cdot10^{-4}m^4$) is studied. The pile is embedded in soil deposits with shear velocity $V_s=100m/s^2$ and density $\rho_s=1.5Mg/m^3$. According to its boundary conditions, the embedded pile end is fixed, while the other end is free according to its displacements and rotations. The accelerogram of the Athena 1999 earthquake as shown in Figure 2, is used as excitation motion. In order to avoid residual displacement the accelerogram has been base line corrected and one-dimensional shear wave propagation analysis has been performed.

In Figures 3,4 the time histories of the head transverse displacement w_{top} and velocity $\dot{w}_{top}$ of the pile embedded in a viscoelastic foundation is presented taking into account the rotary inertia and the shear deformation effects. Moreover, in Table 1 the extreme values of the head displacements are presented taking into account or ignoring the aforementioned effects. From the nature of the problem (relatively long pile) and from the obtained results, it can be observed that these effects are not important in this particular example.

5 CONCLUDING REMARKS

The main conclusions that can be drawn from this investigation are

- The numerical technique presented in this investigation is well suited for computer aided analysis for piles of arbitrary simply or multiply connected doubly symmetric cross section.
- The proposed method is developed for general seismic analysis, while the pile is subjected to the most general boundary conditions.

- The soil deposits are modeled as layered Winkler-type viscoelastic foundation.
- The proposed model can take into account both the kinematic and inertial soil-pile-structure interaction.
- The base line correction for the excitation motion and the one dimensional shear wave propagation are incorporated in this investigation.
- In some cases, the effect of shear deformation is significant, increasing the transverse displacements and decreasing the bending moments in both linear and nonlinear analysis.
- The discrepancy between the results of the linear and the nonlinear analysis may be significant in case of heavily loaded piles.

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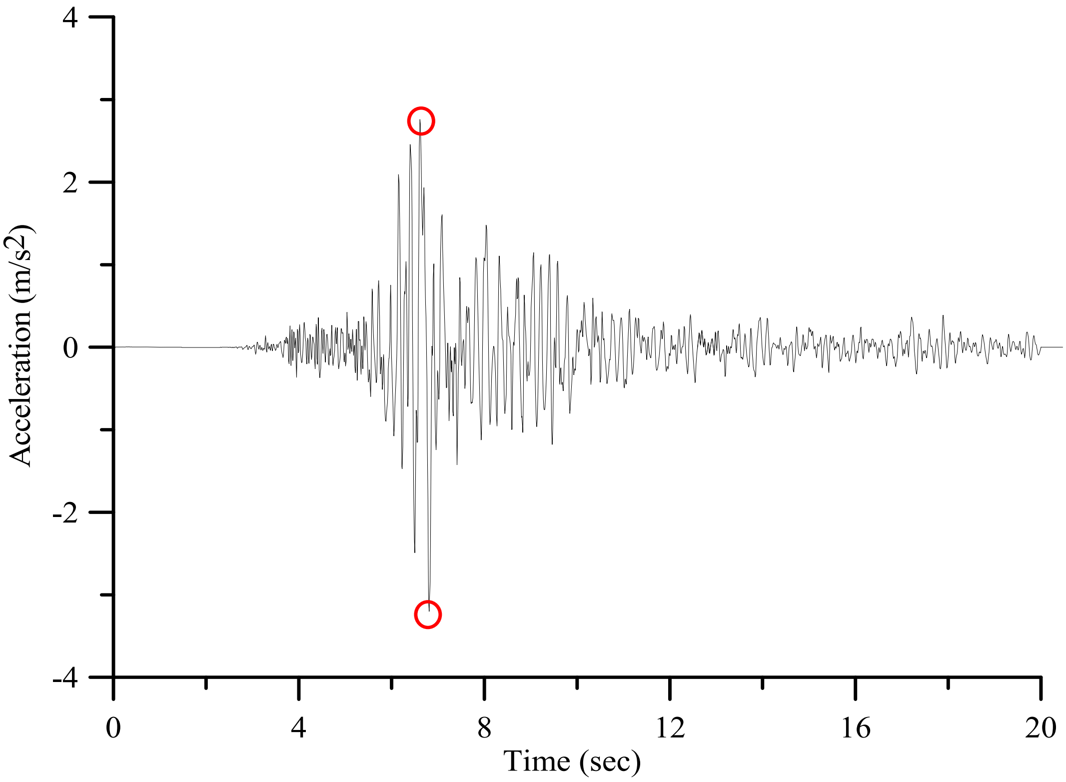

Figure 2: Athena 1999 earthquake accelerogram.

Accounting rotary inertia and shear deformation		Ignoring rotary inertia and shear deformation	
$w_{max}\,(cm)$	$w_{min}\,(cm)$	$w_{max}\,(cm)$	$w_{min}\,(cm)$
1.23	-2.69	1.19	-2.58

Table 1: Extreme head displacement of the pile.

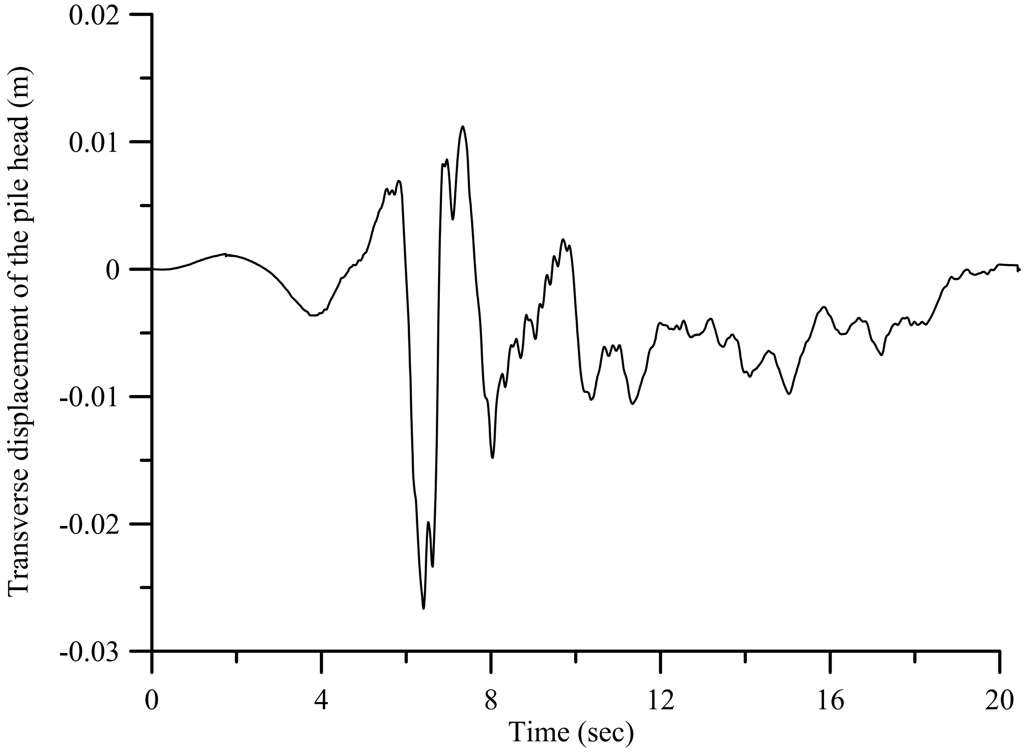

Figure 3: Time history of the transverse displacement $w_{top}(0,t)$ of the pile head.

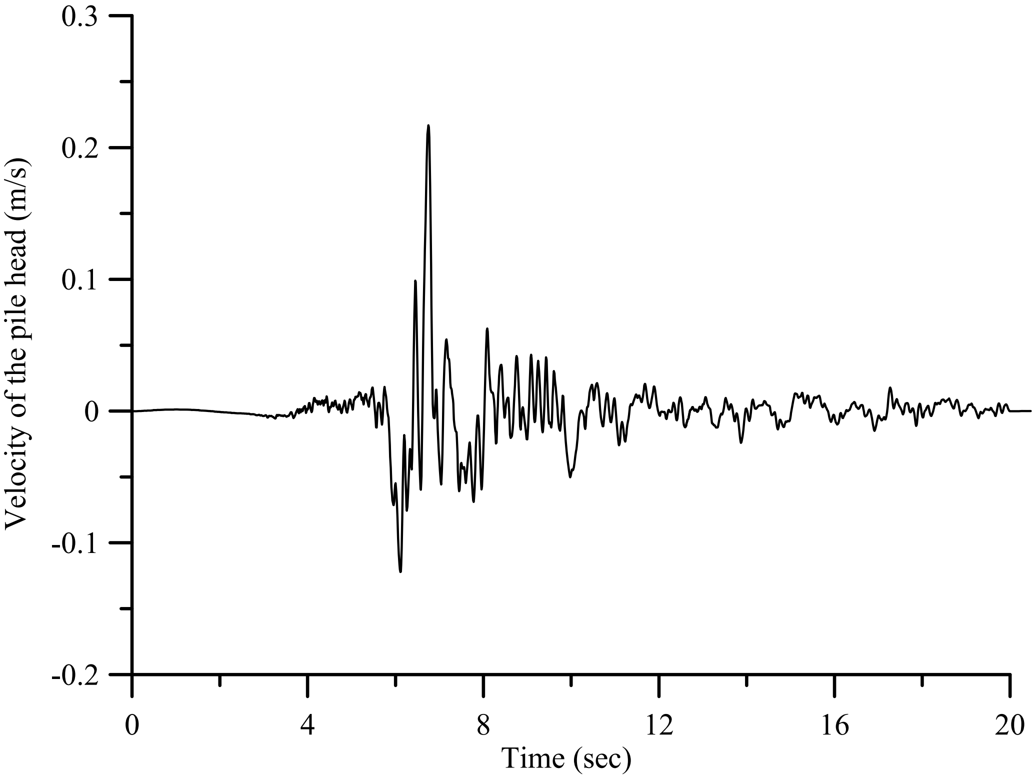

Figure 4: Time history of the velocity $\dot{w}_{top}(0,t)$ of the pile head.

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