SOIL-PILE-STRUCTURE SEISMIC INTERACTION CONSIDERING THE NON-LINEAR BEHAVIOUR OF THE SOIL AND THE REINFORCED CONCRETE

Pisco, Guilherme Claro; Instituto Superior Técnico, Universidade de Lisboa, Portugal

ABSTRACT

In this article, the soil-pile-structure seismic interaction is investigated. The non-linear behaviour of the materials, soil and reinforced concrete are considered in the analysis. The kinematic soil-pile interaction and inertial pile-structure interaction are studied separately and has a unique phenomenon, as well as the design of reinforced concrete elements subjected to imposed displacement. To model the soil-pile interaction the CINEMAT computational program is used. This software integrates the Beam on a Dynamic Winkler Foundation method (BDWF), a model of one-dimensional seismic wave propagation and the linear equivalent method to account for soil non-linear behaviour. The non-linear behaviour of the reinforced concrete subjected to imposed displacements is modelled with the software PIER. The effect of the kinematic interaction is evaluated for a case of a pile with low deformability and for a case of a pile with a non-reinforced length. The conclusions are presented. The BDWF model is adapted to consider the global interaction phenomenon. The new approach is calibrated and validated by comparison with a three-dimensional finite element elastic model, for several types actions and frequency relations. A case study of the pile-structure model in an alluvium formation is analysed, considering the global seismic interaction effect and both soil and reinforced concrete non-linear behaviour.

Keywords: Seismic soil-pile-structure interaction; BDWF (Beam on Dynamic Winkler Foundation); Non-linear behaviour of the soil; Non-linear behaviour of the reinforced concrete;

1 INTRODUCTION

Earthquakes are amongst the most destructive forces that naturally occur in nature. The earthquake in Japan's Kobe region in 1995, has caused extensive damage, and ever since, several studies have been conducted to better understand all the phenomenon involved. One important phenomenon is the soil-pile-structure interaction.

Despite all the scientific work produced and all the studies carried, it has yet failed, to reach a consensus for a globally accepted methodology in pile design subjected to seismic actions. Most studies conducted considered the materials' behaviour as linear elastic which is inaccurate since they assume a non-linear behaviour.

The aim of this paper is to study the seismic soil-pile-structure interaction, disregarding the effect of soil liquefaction, by analysing the inertial forces and imposed displacements given by the soil response. This phenomenon is generally known as inertial and kinematic interaction. The soil-pile-structure interaction is modelled based on the BDWF Method ("Beam on Dynamic Winkler Foundation") and the soil's non-linear behaviour is modelled through the linear equivalent method. The non-linear behaviour of reinforced concrete is considered by implementing adequate constitutive relations.

As studied in previous works, like Brito (2011), this paper proposes the verification of pile response with kinematic variables as opposed to static ones. Therefore, this paper compares the imposed curvatures and ultimate curvature for characterization of the pile's response.

2 SEISMIC SOIL-PILE-STRUCTURE INTERACTION

When an earthquake occurs, it unleashes seismic waves that cause the soil to vibrate and consequently the vibration of all founded structures on soil. When a structure is designed to be supported over piles a natural interaction between the soil, pile and structure occurs. This phenomenon results in additional horizontal forces that are important and need to be taken into consideration, as, if not, may result in extensive damage to the piles.In the elastic domain this phenomenon can be divided into two parcels. This approximation has been adopted since it leads to a better understanding and simplification of the problem. These two parcels are known as kinematic and inertial interaction.

2.1 KINEMATIC SOIL-PILE INTERACTION

This phenomenon occurs when, due to the rigidity difference between soil and pile, results in an alteration to the free field displacements profile. This event results in an imposed displacements profile to the pile which is a key factor because of the high curvatures in the pile near the transitions between soil layers with a large rigidity difference. A relevant aspect that this interaction conditions is the structure's base action, fundamental to its design.

2.2 INERTIAL PILE-SOIL INTERACTION

Inertial interaction is a result of imposed acceleration to the base of the structure caused by the kinematic interaction phenomenon. This originates in a set of horizontal inertial forces, proportional to the structure's masses and imposed accelerations. These loads are transmitted to the building's foundation level, resulting in concentrated horizontal forces and momentum on the pile's head. The resistance to these forces is assured by the soil's lateral reaction that opposes to the pile's movement, generating additional interaction loads.

2.3 BDWF MODEL (Beam on Dynamic Winkler Foundation) FOR AN ISOLATED PILE

The BDWF Model, Figure 1, is used in the calculations developed throughout this article. It is a discrete model and simulates the soil reaction with the use of springs, k(x) and dampers, c(x) to replicate the movement effect of the soil over the pile during an earthquake. By relating the free field displacements of the soil, the above-mentioned springs and dampers, it is possible to obtain the pile's displacements and therefore to simulate the kinematic soil-pile interaction.

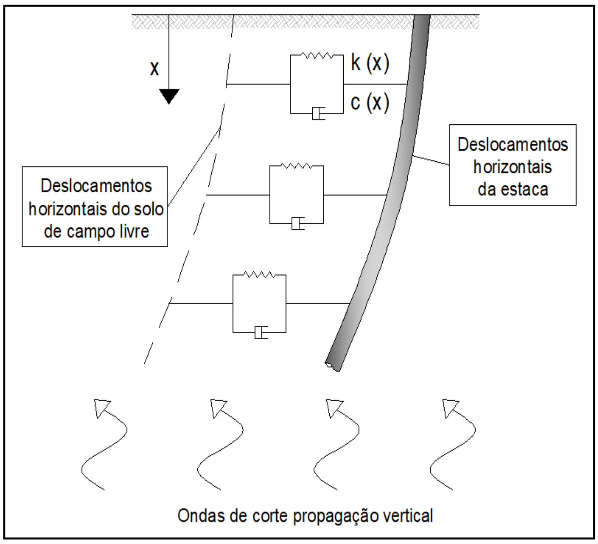


Figure 1 –BDWF Model

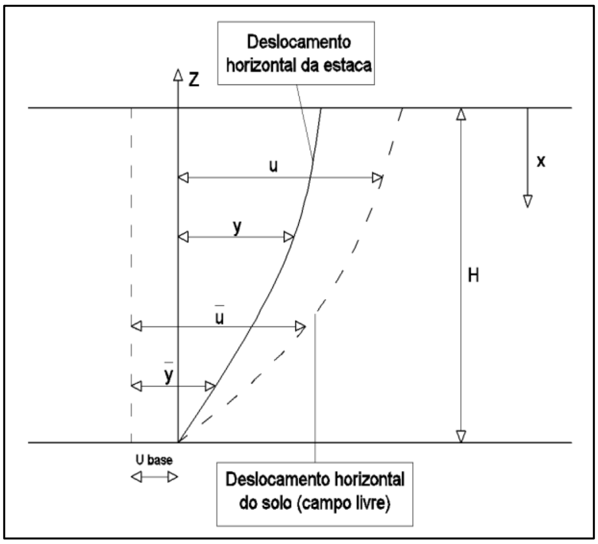


Figure 2 – Flores-Berrone e Whitman Model (1982)

The initial model was proposed by Flores-Berrones and Whitman (1982), Figure 2. At a first approach the damping was not considered. This model was then enhanced by different authors that factored the soil damping (Makris and Gazetas, 1992), soil stratification (Kavvadas and Gazetas, 1993) and analysed the domain's frequency problem. Santos (1999) introduced the soil's non-linear behaviour by incorporating the equivalent linear method.

From the work of the above-mentioned authors, the following equations were deducted in the calculation model used in this article. The first in the time domain and the second in the frequency domain.

$$E_p I_p \frac{\partial^4 y}{\partial x^4} + \bar{m}\frac{\partial^2 \bar{y}}{\partial t^2} + c\frac{\partial(\bar{y}-\bar{u})}{\partial t} + k(\bar{y}-\bar{u}) = 0 \quad \textit{(2.1)}$$

$$E_p I_p \frac{\partial^4 y}{\partial x^4} + (k - \bar{m}\omega^2 + ic\omega)\bar{y} - (k + ic\omega)\bar{u} = 0 \quad (2.2)$$

In which $E_p I_p$ is the pile's bending rigidity, $\bar{m}$ is the distributed mass, c the soil's damping coefficient e k the spring's rigidity of the model.

The value of the soil damping is obtained by the sum of hysteretic soil damping parcel and radiant damping

$$c(x) \approx c_m(x) + c_r(x) \tag{2.3}$$

To determine the radiant damping coefficient, it was used Gazetas e Dobry, (1984a e 1984b) formulation. The hysteretic damping coefficient is obtained by the following expression.

$$c_m = \frac{2k(x)\xi}{\omega} \tag{2.4}$$

To determine the spring's rigidity the values proposed by Makris and Gazetas (1992) and Makris (1994), were used in the following expression

$$k(x) \approx \delta E_s(x) \tag{2.5}$$

- Free head pile: $\delta = 2.1$
- Fixed head pile: $\delta = 1.2$

Note the formulations developed by Santos (1993) to obtain k parameter of the phenomenon.
To apply the model to general cases Santos (1999) developed the program CINEMAT, to account the soil's non-linear behaviour. This program was utilized throughout this work.

3 DESIGN OF REINFORCED CONCRETE STRCTURAL ELEMENTS SUBJECTED TO IMPOSED DISPLACEMENTS

Usually, the concrete elements verification is based on the material linear elastic constitutive laws, affected by a behaviour factor to account for the non-linear real response. This approach is done by verification of static variables. However, this type of formulation as suffered several critics. Recent studies (Brito, 2011) point out that when an element is subjected to high displacements, as in the case of seismic action, it behaves as nonlinear instead of as elastic linear material. So, the element verification must be done not by means of static values (internal forces) but with kinematic ones (displacements, deformations and specially curvatures). The make this changed in the process, suitable models must be used to account with the non-linear behaviour of the material.
The verifications of a reinforced concrete element must be done by comparing the imposed curvature with the ultimate curvature of the section. Consequently, the maximization of the ultimate curvature of the section is a key factor to achieve the appropriate behaviour of a concrete section when subjected to imposed displacement, as the case of a pile subjected to seismic action.
Base on Brito (2011), the fundamental parameters to influence the ductility of the section are described:

- Section confinement – The ultimate concrete compressive strain is proportional to the section confinement;
- Section dimensions – The yielding and ultimate curvatures are inversely proportional to the dimension in the flexural direction. The perpendicular one as a small effect on the above-mentioned characteristics.
- Material resistance – To maximize the reductions of the section area, in accordance with the last point, both concrete and steel resistance should be higher to have the necessary resistance to the applied loads, apart from the seismic one;
- Axial Forces – The axial forces have a high impact in the ultimate curvature. The bigger the axial forces the lower is the section deformability and consequently the ultimate curvature. This aspect is contrary for the yielding, but the impact is much lower.
- Reinforcing steel bars' design – The amount of reinforcement of a section as almost no effect in the yielding curvature. For the ultimate one, higher levels of steel results in a decreased of this parameter.

As mentioned, since the phenomenon studied in this article implies the non-linear behaviour of the reinforced concrete, adequate constitutive models must be to use in order to compute the real behaviour of the material. To achieve it, in this study are going to be used the constitutive models developed by Mander, Priestley & Park (1988), for the reinforced concrete, and Pipa (1993), for the longitudinal steel reinforcement.
The non-linear behaviour of this materials, as its respective constitutive models, are programmed in the software FLEXAO, that is part of the program PIER developed for the calculation of structures subjected to imposed displacement. Both softwares are going to be used in the present study.

4 KINEMATIC SOIL-PILE SEISMIC INTERATION – ANALYSIS OF THE BEHAVIOUR OF A SINGLE PILE IN AN ALLUVIAL FORMATION

Following the investigations developed by Santos (1999) e Lagareiro (2015), whom study the kinematic interaction effects on piles for several different cases of longitudinal and confinement reinforcement, additional scenarios are going to be analysed in this work. Lagareiro (2015) concluded, for a total of 27 cases with variable diameters and reinforcements, that the impose curvatures are much smaller that the ultimate curvature of the section, for the kinematic interaction effect. To, to close the analysis of this effect two hypothesis were study to understand the behaviour of the pile in limit situations. A case where all the variables were chosen to minimize the deformability of the pile and a second case where a pile with a non-reinforced length is simulated to understand the deformation capacity of this type of sections.

4.1 Geotechical model

The geotechnical model adopted is the same used by Santos (1999) e Lagareiro (2015) and is described in the Table 1. In Figure 3 is represented the G/G_0-γ ξ-γ curves that characterize the non-linear behaviour of the soil.

Table 1 – Geotechnical parametrization [adapted from Santos (1999)]

Layer	Thickness (m)	behaviour	γ(kN/m³)	v	G_0 (MPa)
1 - Fill or overconsolidated layer	5	Non-Linear	19	0,3	80
2 - Aluvionar clayed layer	10	Non-Linear	17	0,5	Variable 20 to 30
3 - Degraded Miocenic Layer	5	Linear w/ ξ=1%	22	0,3	200 (vs≈300m/s)
4 - Miocenic layer	-	Rigid	-	-	-

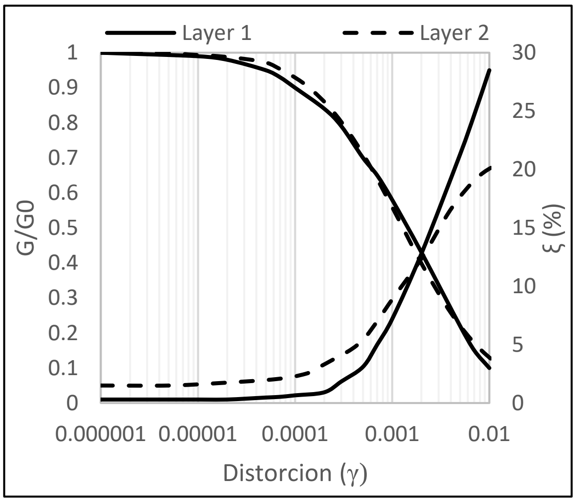


Figure 3 - G/G_0-γ ξ-γ curves [adapted from Santos (1999)]

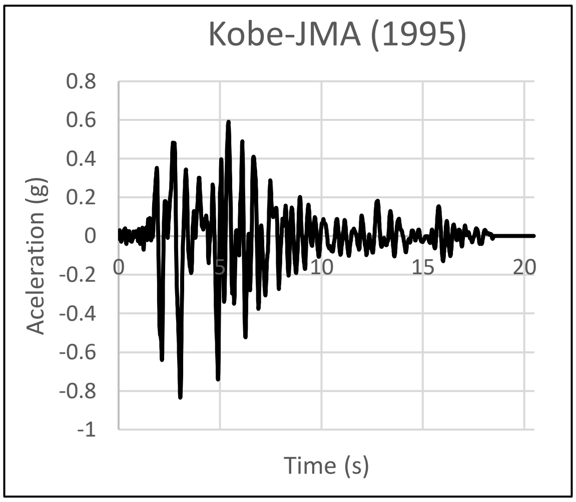


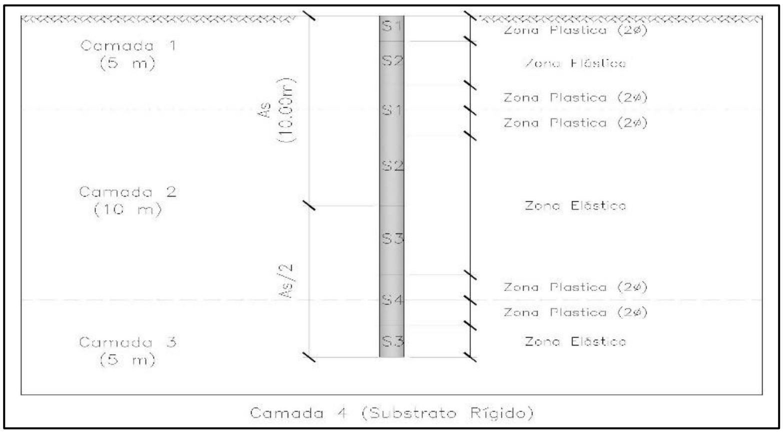
Figure 4 – Kobe JMA aceleration profile [adapted from Santos (1999)]

4.2 Seismic action

The seismic action considerer in the calculation was the acceleration profile of Kobe JMA and is represented in Figure 4

4.3 Reinforcement distribution of the piles

In Figure 5 is represented the distribution of steel reinforcement of the piles considered and in the Table 2 the curvature values. In the second case of study the pile has 12m of section S1/S4 and the remainig length has no reinforcement.



- S1/S4

Longitudinal – 18Ø25

Transversal – Plastic zone Ø12//0.10

- S2/S3

Longitudinal – 18Ø25

Transversal – Elastic zone Ø12//0,175

Figure 5 – Distribution of reinforcement in the pile

Table 2 – Concrete reinforced sections

	χ_c (‰/m)	χ_u (‰/m)	M_{ced} (KNm)	M_u (KNm)
Section S1/S4	3,66	23,28	3837	4485
Section S2/S3	3,77	16,05	3702	4164
No reinforcement	3,29	6,98	2224	2282

4.4 Case studies

4.4.1 Pile with low deformability capacity

For this case were combined all the possible variables that contributed to the decreased of the ultimate curvature of the pile. A large diameter pile (1,3m) was adopted, low levels of confinement, high axial forces (Compressive stress of 7MPa) and a high value of maximum seismic acceleration (Amax=0.5g). In Figure 6 is the represented the diagram of impose curvatures of the pile for linear elastic behaviour and nonlinear elastic behaviour, which is divided in superior and inferior interface worst case scenario. In Table 3 are resumed all the results of the calculation in terms of curvatures.

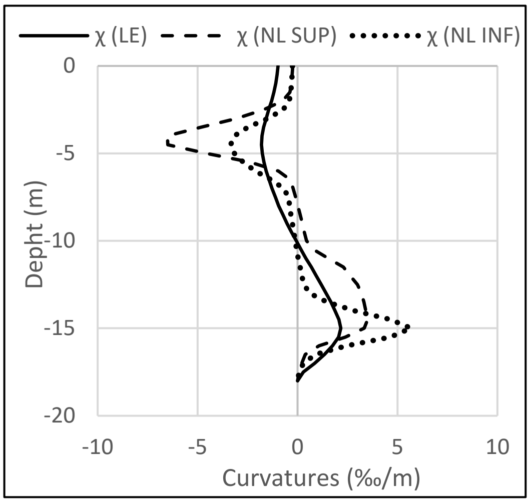


Figure 6 – Pile's Curvature diagram

Table 3 – Resume of the results in terms of curvatures

	$\chi_{imposed}$ (‰/m)	χ_y (‰/m)	χ_u (‰/m)	$\frac{\chi_{imposed}}{\chi_c}$	$\frac{\chi_{imposed}}{\chi_u}$
Secção S1/S4	6.51	3.66	23.28	1.78	0.28
Secção S2/S3	2.30	3.77	16.05	0.61	0.14

For the case of a pile with low deformability capacity is possible to concluded that, despite the plastic zone has been reached, the imposed curvature is still much smaller than the ultimate capacity of the pile. The conclusion to take from this analysis is that even for adverse conditions a well design pile may be subject to kinematic effects without collapsing. However, due to the level of plastification, damaged are expect.

4.4.2 Pile with non-reinforced length

In this second case of study the objective is to understand the response of a pile executed with a continuous flight auger. With this type of executive technique, the reinforced length is limited to 12 m but since is a less expensive solution is commonly used. So is important how is the behaviour of the non-reinforced length to the kinematic seismic effect since this is the only effect in this depth of the pile.

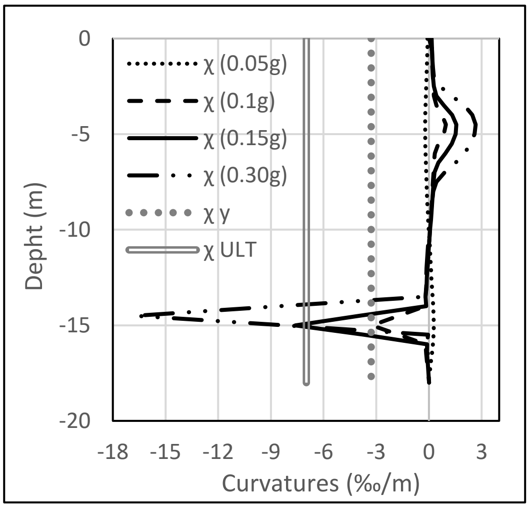


Figure 7 - Pile's Curvature diagram

Table 4 - Resume of the results in terms of curvatures

	$\chi_{imposed}$ (‰/m)	$\frac{\chi_{imposed}}{\chi_c}$	$\frac{\chi_{imposed}}{\chi_u}$
Amax=0,05 g	0,28	0,09	0,04
Amax=0,10 g	3,25	0,99	0,47
Amax=0,15 g	7,61	2,31	1,09
Amax=0,30 g	16,73	5,10	2,40

For the case of continuous flight auger, the impose curvatures are bigger that the ultimate curvature of the non-reinforced length, even for small to medium accelerations levels. The difference between elastic behaviour, yielding and collapse, in terms of accelerations intervals, is very small. In seismic zones this type of technique must not be used since the pile doesn't have the capacity to support the impose curvatures will collapse.

5 VALIDATION OF THE BDWF MODEL APPLIED TO THE GLOBAL SEISMIC INTERACTION SOIL-PILE-STRUCTURE PHENOMENON

With the conclusion of the kinematic interaction study, it is necessary to understand the response of a pile to the global seismic interaction phenomenon. The first step is to generalise the BDWF to consider the inertial effect, by simulating the structure in the top of the piles. The structure was represented by single degree of freedom oscillator with a concentrated mass at the top of a unique beam finite element with 9 m length. To simulate this additional aspect some modification had to be introduced in the formulation of BDWF model and in the software CINEMAT codex:

1. Introduction of concentrated masses in the nodal points to simulate the structure mass;
2. Introduction of a parameter to cancel the effect of the model springs, Fk, the soil damping, Fc, and soil mass, Fm. These three parameters range from 0 to 1. For zero value the soil action is not considered in the nodal points behaviour and for the value one the soil is computed without any alteration in its parameters.
3. Variation of the proportionality factor, δ, between the spring model stifness and deformability modulus of the soil. As mentioned, this factor depends of the boundary conditions at the pile head. Since the structure is now implemented in the problem, the pile head is between a free and fixed condition. This parameter is a new variable in the model.

Based on these changes, the equation 2.1 assumes a new form:

$$E_p I_p \frac{\partial^4 y}{\partial x^4} + F_m \times \bar{m}\frac{\partial^2 \bar{y}}{\partial t^2} + F_c \times c\,\frac{\partial(\bar{y}-\bar{u})}{\partial t} + F_k \times k(\bar{y}-\bar{u}) = 0 \tag{5.1}$$

The concentrated masses are simulated as applied nodal forces that depends of the acceleration of the node.

The new version of CINEMAT program was validated by comparation with a three-dimensional finite element model of a system composed by an elastic soil layer, a pile and the single degree of freedom oscillator. Since the response of the system depends on the relation between the frequencies of the soil and the structure, the last parameter was calculated based on the theoretical solution for presented by Clough and Penzien (1995).

In the Figure 8 – Model properties and finite element model mesh is defined the model used in the validation process.

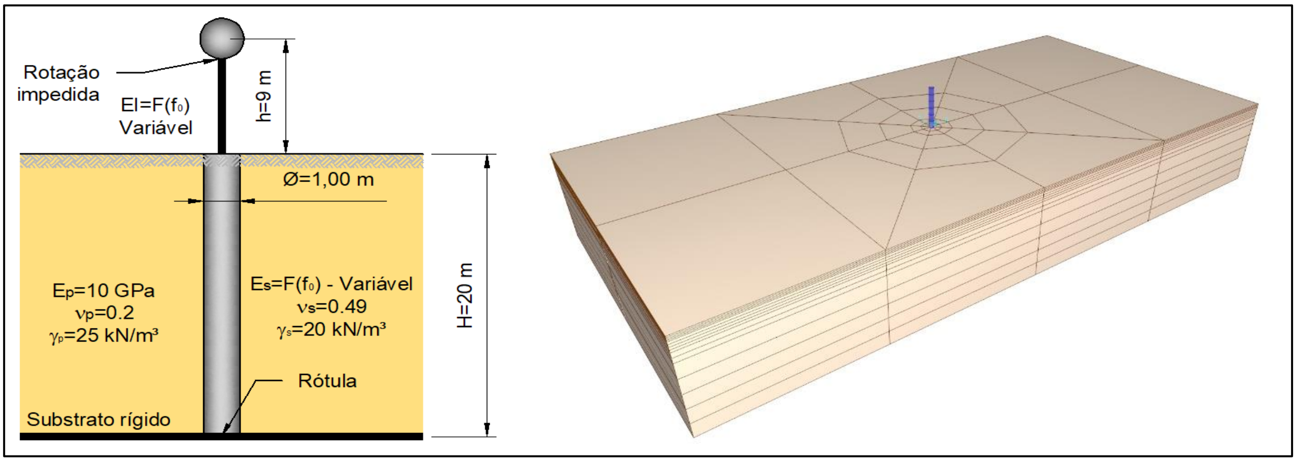


Figure 8 – Model properties and finite element model mesh

The model was validated based on the next procedures:

1. Comparation of the kinematic effect with masses totally concentrated at the nodes and distributed along the bars;
2. Comparation of the response of a single degree of freedom oscillator in both CINEMAT and 3D model
3. Comparation of the response of the pile to the global effect in both CINEMAT and 3d model for different soil-structure frequencies relations.

In Figure 9 to Figure 20 is represented the response of the pile in terms of moments and shear forces, at the head node, for the Kobe acceleration profile and the transfer function, for three frequencies relations.

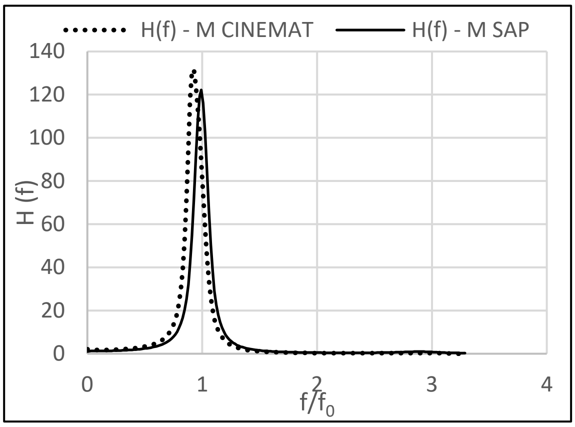


Figure 9 - Pile's head moments' transfer function – $f_{structure} = f_{soil}$

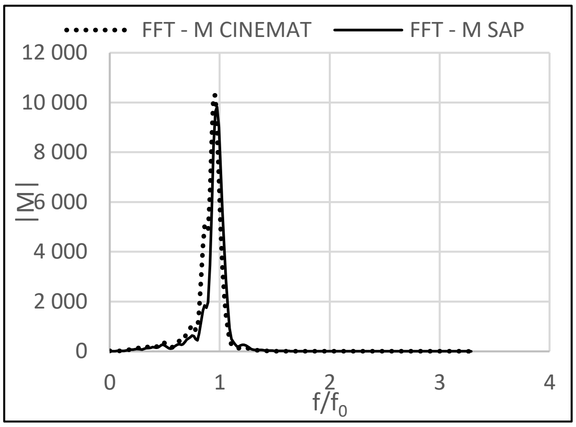


Figure 10 - |M| vs f/f0 – Pile's head response to Kobe acceleration profile – $f_{estrutra} = f_{soil}$

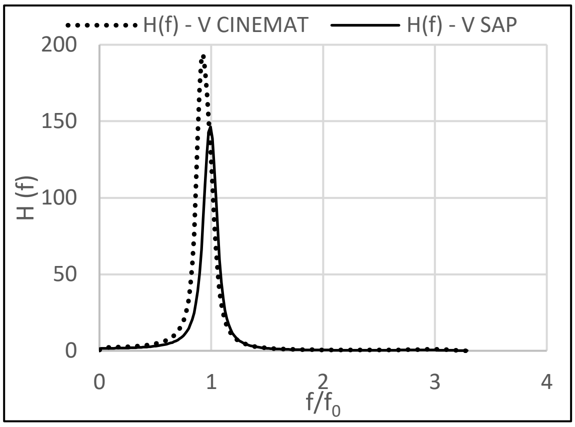


Figure 11 - Pile's head shear forces' transfer function – $f_{structure} = f_{soil}$

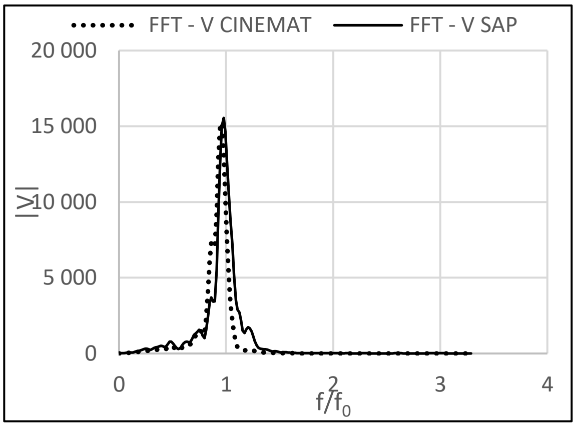


Figure 12 - |V| vs f/f0 – Pile's head response to Kobe acceleration profile – $f_{estrutra} = f_{soil}$

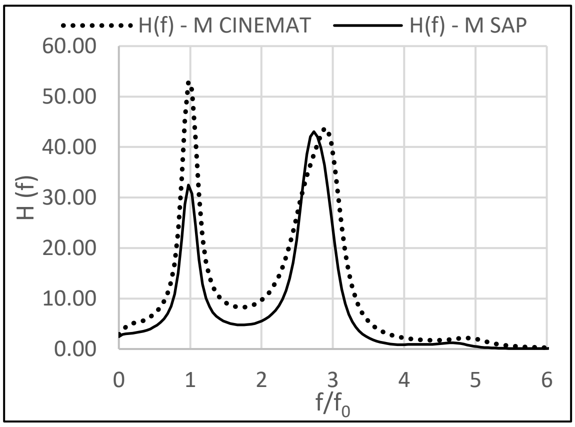


Figure 13 - Pile's head moments' transfer function – $f_{structure} = 3 \times f_{soil}$

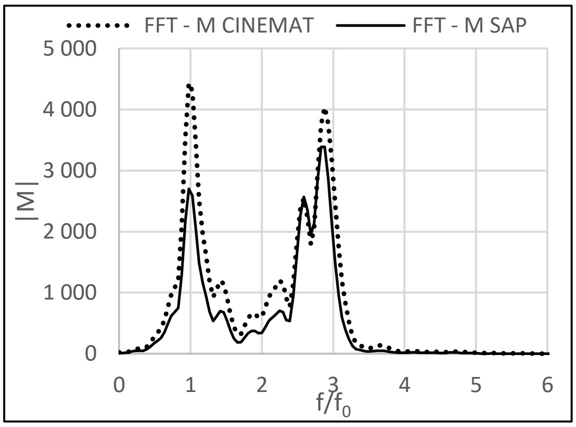
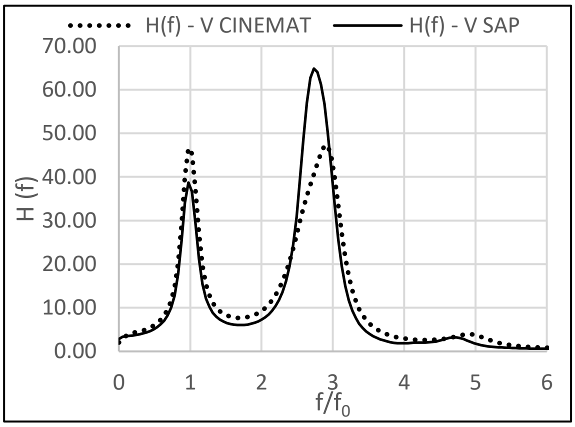


Figure 14 |M| vs f/f0 – Pile's head response to Kobe acceleration profile – $f_{estrutra} = 3 \times f_{soil}$



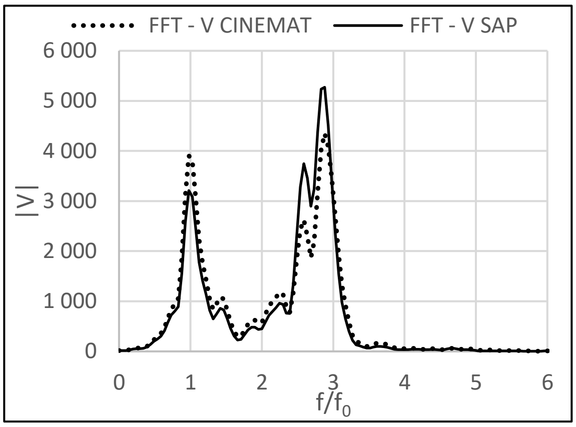
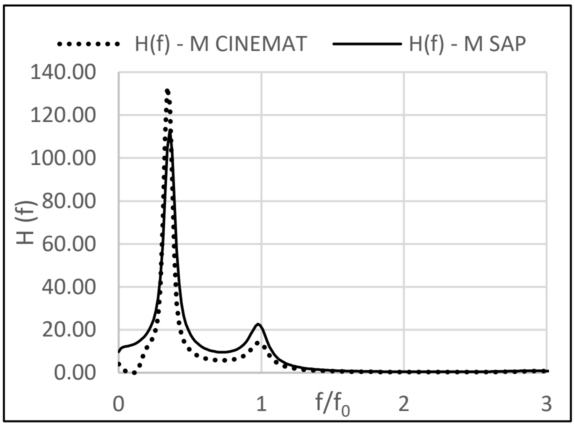


Figure 15 - Pile's head shear forces' transfer function – $f_{structure} = 3 \times f_{soil}$

Figure 16 - |V| vs f/f0 – Pile's head response to Kobe acceleration profile – $f_{estrutra} = 3 \times f_{solo}$



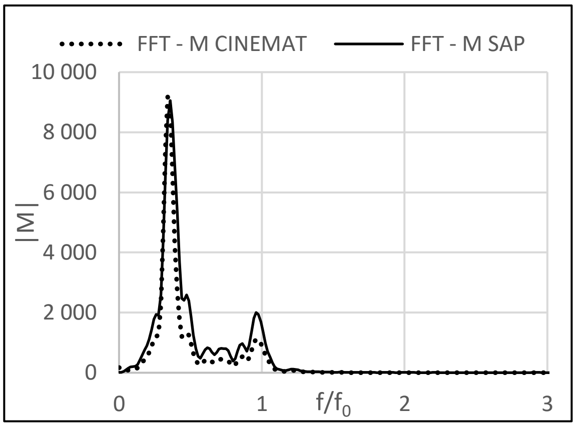
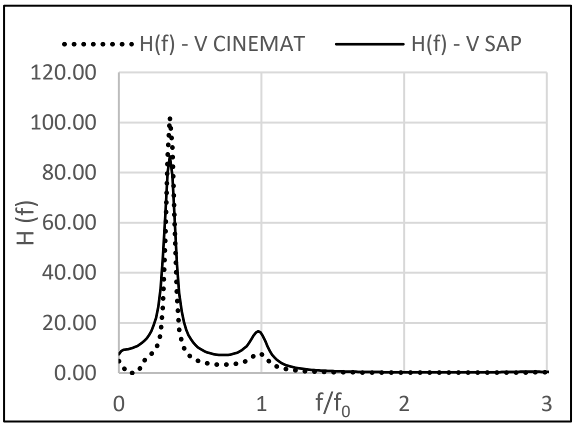


Figure 17 - Pile's head moments' transfer function – $f_{structure} = f_{soil}/3$

Figure 18 |M| vs f/f0 – Pile's head response to Kobe acceleration profile – $f_{estrutra} = f_{soil}/3$



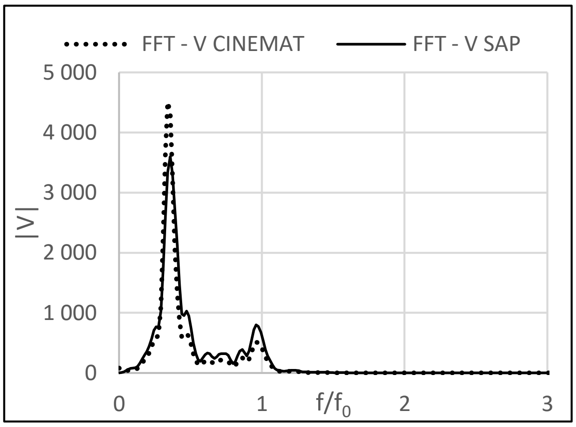


Figure 19 - Pile's head shear forces' transfer function – $f_{structure} = f_{soil}/3$

Figure 20 - |V| vs f/f0 – Pile's head response to Kobe acceleration profile – $f_{estrutra} = f_{solo}/3$

For the case of $f_{structure} = f_{\mathrm{soil}}$ the parameter that resulted in better approximation with the 3d model was δ equals to 1,2. For the case of $f_{structure} = 3 \times f_{soil}$ the value was 2.1, like expected due to the stiffness of the structure, the pile head rotate and exhibit a free type behaviour . For the last case $f_{structure} = f_{soil}/3$ since the structure had a small stifness it was according with a fixed head pile, so δ equals to 1,2. The results obtained demonstrated that the CINEMAT program simulated successfully the soil-pile-strcture-interation. Another important observation is related with the high sensibility that the program revealed to the variation of parameter δ.

6 SOIL-PILE-STRUCTURE I NTERACTIO GLOBAL PHENOMENEN – ANALYSIS OF A PILE BEHAVIOUR IN AN ALUVIONAR SOIL FORMATION

The present study is concluded with the analysis of the response of a pile subjected to the seismic global interaction considering both non-linear behaviour of the soil and the reinforced concrete. The structure was simulated with elastic behaviour. The pile teste is the same of the point 4.4.1, with a mas equal to 0,05g and 0,1g and a axial stress of 5 MPa. In the Figure 21 is represented the diagram of impose curvatures of the pile and it respective ultimate and yielding curvature. The soil and the structure have the same frequency because in the validation procedure this was the worst case for the pile

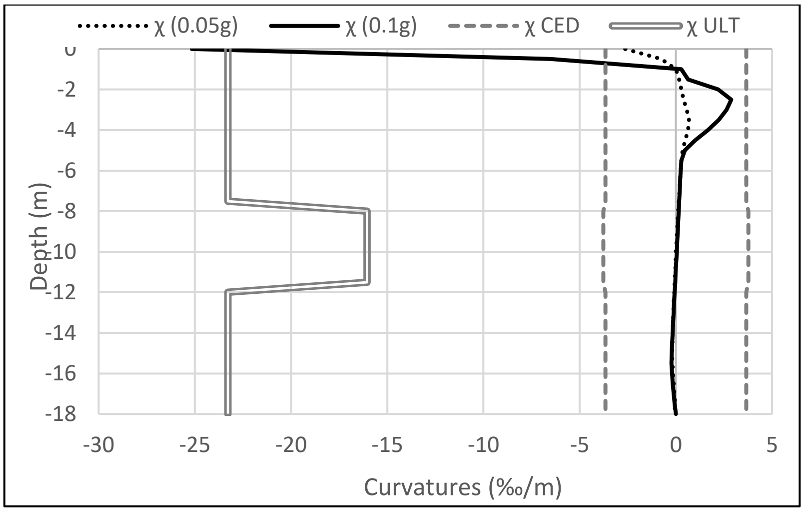


Figure 21 - Pile's Curvature diagram

From the observation of the graphic it is possible to conclude that the pile collapse for a max of 0,1g.

7 CONCLUSIONS

- Piles reinforced in all its length don't collapse for the kinematic interaction but suffer some degree of damage due to plastic deformations;
- Piles with non-reinforced length collapse for the kinematic interaction for low to medium seismic accelerations. This type of elements should not be used in zones with medium to high seismic potential
- The CINEMAT program successfully simulates the soil-pile-structure interaction. However, this program is to sensible to the δ parameter and his variation need to be more study.
- When a pile is subjected to soil-pile-interaction it collapses. The value of maximum acceleration may be a conservative since the structure was modelled elastic behaviour, condition that may overestimate the forces transmitted to the pile head in a seismic situation.

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