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LONG PILES INTEGRITY TROUGH IMPACT ECHO TECHNIQUE

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Abstract. *The main objective of this paper is to evaluate the capabilities of the non-destructive impact-response method in detecting the existence of defects in long piles.*
The impact-echo technique, based on the use of transient elastic waves, was developed many years ago for non-destructive detection of defects in piles and structures. Moreover, is recognised in the specialised literature that this method could be applied confidently for piles with length/diameter relationship up to 20. However, with the development of hardware in computers and new sensors and dynamic equipment, it is possible now to use this method for long piles. In this paper, piles of length/diameter relationship of approximately 40 are studied.
A numerical-experimental study was carried out. Finite element models 1D and 3D were performed for piles including the surrounding soil and defects type necks and bulbs. Subsequently, experimental studies were carried out to verify the finite element models using similar piles. These experimental studies were carried out in the air and soil, and impact responses were analyzed in both time and frequency domains. It was shown that the results of the experiment were in agreement with those of numerical studies, and the accuracy of the impact-echo method was influenced by the type, size and location of defects. In addition, it was also revealed that the method could be applied for long piles.

1 INTRODUCTION

Many civil structures, such as buildings, bridges, towers, dams and other massive structures sometimes need special foundations using piers and piles, built alternatively with precast and cast in place techniques. In the first case, the piles may be damaged under the pile driving impact process, in which long and deep cracks may appear. In the other hand, “necks” or “bulbs” may be created in the process of drilling. In both cases, these defects may affect considerably the bearing capacity of the piles.

The importance of assessing the actual quality of a pile foundation has long been recognized. To achieve this goal, various kinds of destructive (load tests) and nondestructive tests were developed. In this aspect, nondestructive testing (NDT) was considered to possess high potential for application because it is featured as cost-effective, damage-free, time-saving, and thus can cover the whole population to avoid the disadvantages of sampling and destruction of the piles tested.

Many researchers have been working on the NDT of piles. Most of the tests can be classified into two categories, i.e., (1) reflection and (2) direct transmission methods (Liao et al.[1]). Among others, tests as categorized in reflection methods could be Impulse Response (IR) (Davis[2]; Liao and Roesset[3]) or Transient Dynamic Response (TDR) (Davis and Dunn[4]), Sonic Echo (SE) (Rausche et al.[5]), Impedance Log (IL) (Paquet[6]; Rix et al.[7]), Receptance theory (Lilley[8]) and Impact Echo (IE) (Lin et al.[9]; Kim et al.[10]; Kumar et al.[11]). Tests as categorized in direct transmission methods could be Cross-hole Sonic Logging (CSL), gamma-gamma testing method, Accoustic Emission (AE) (Shiotani et al.[12], Luo et al.[13]) and Parallel Seismic (PS) (Liao et al.[1]). In reflection methods, the force to introduce a disturbance to a pile and the receiver to collect the response of the pile are basically applied at the pile head, whereas in direct transmission methods the receiver is placed at intervals in a tube or borehole which must be cored or drilled before testing. Therefore reflection methods are in general faster and more cost-effective. However, direct transmission methods can obtain the information in a more direct way and they can be applied to piles with much larger length. Finally, must be mentioned a class of methods that analyze the response and capacity of piles meanwhile are being driving in the soil. This method, normally named CASE Method (Rausche et al.[14]). Duan and Oweis[15] used dyadic wavelet transform is used to analyze PDA measured signals in order to identify the CASE-damping factor,

In connection with theoretical and numerical studies, most papers used 1D models, however, some authors presented more sophisticated models. Lin et al.[9] who conducted analyses of piles using the DYNA2D y DYNA3D codes, Liao[16] compared models 1D, 2D y 3D. R.V. Rao and N.S.V. Rao[17] studying pile machine foundations in the time domain using the Lanczos vectors in 3D FEM models. Liao and Roesset[18] analyze the dynamic response of intact piles to impulse loads. They show that 3D model results are very similar in general terms to those predicted by the usual 1D solutions. Finally, Palacz and Krawczuk[19] introduce a new finite spectral element of a cracked rod for damage detection.

The impact-echo technique, based on the use of transient elastic waves, was developed many years ago for non-destructive detection of defects in piles and structures. Moreover, is recognised in the specialised literature that this method could be applied confidently for piles with length/diameter relationship up to 20 (Davis and Dunn[4]) or 30 (Lilley[8]). However, with the development of hardware in computers and new sensors and dynamic equipment, it is possible now to use this method for long piles. In this paper, piles of length/diameter relationship of approximately 40 are studied. Moreover, experimental evidence is presented about the importance of the soil in the damping and spread of the waves propagating along the pile.

2 PROBLEM DEFINITION

Impact echo techniques involves introducing a transient stress pulse into a test object by an mechanical impact with a hammer and monitoring the surface displacements, velocities or accelerations causes by the arrival of reflections of the pulse from internal defects and external boundaries (Figure 1). The pulse consists of stress waves (compression stress waves) propagating along the body of the bar by spherical wavefronts. These waves are reflected by defects and boundaries and return back to the surface. At the top surface, the waves are reflected again and they propagate into the object. This, a transient resonant condition is set up by multiple reflections of waves between the top surface and internal flaws and external boundaries.

A transducer that measures displacement, velocities (geophones) or accelerations (accelerometers) is located close the impact point is used to monitoring the response of the surface caused by the arrival of these reflected waves.

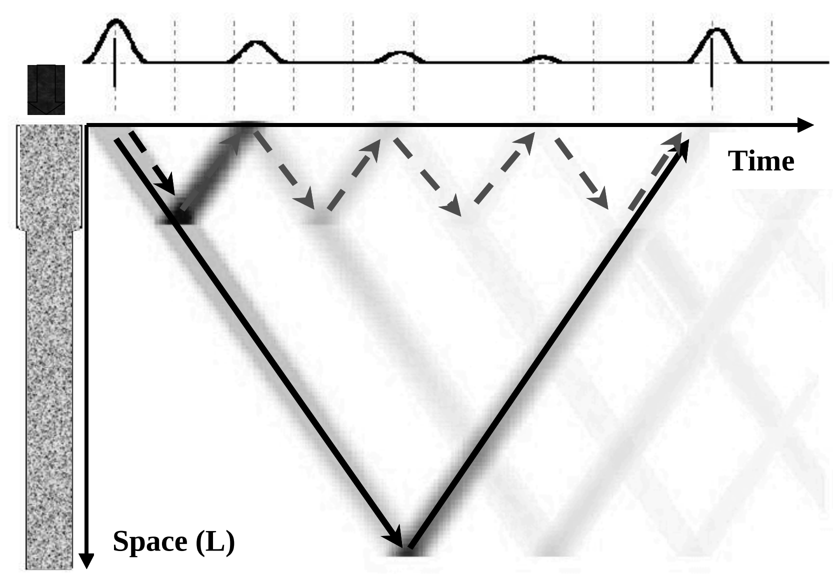

Figure 1: Physical problem analyzed

After tests, a numerical model of the piles-soil system must be made and by mean a dynamic analysis in the time and/or frequency domains the length of piles and possible defects are detected. The determination of the substantial length of a pile exhibits a high value in practical application, particularly when evaluating the integrity of existing foundations such as piles with cap after earthquakes[1].

3 DYNAMIC TESTS

3.1 Experimental set-up and instrumentation

As was mentioned before, the impulse response and impact-echo methods use a low-strain impact to send stress waves through the tested element (Figure 2). The impactor is usually a hammer with or without a built-in load cell in the hammerhead. Response to the input stress is measured using a geophone or accelerometer. The voltage signals generated by the transducer and eventually by the hammer are normally conditioned and amplified by an amplifier and the resulting signal is sent to a notebook computer for data acquisition and storage (Fig. 2). After that, the results are analyzed using a numerical model and processing data techniques.

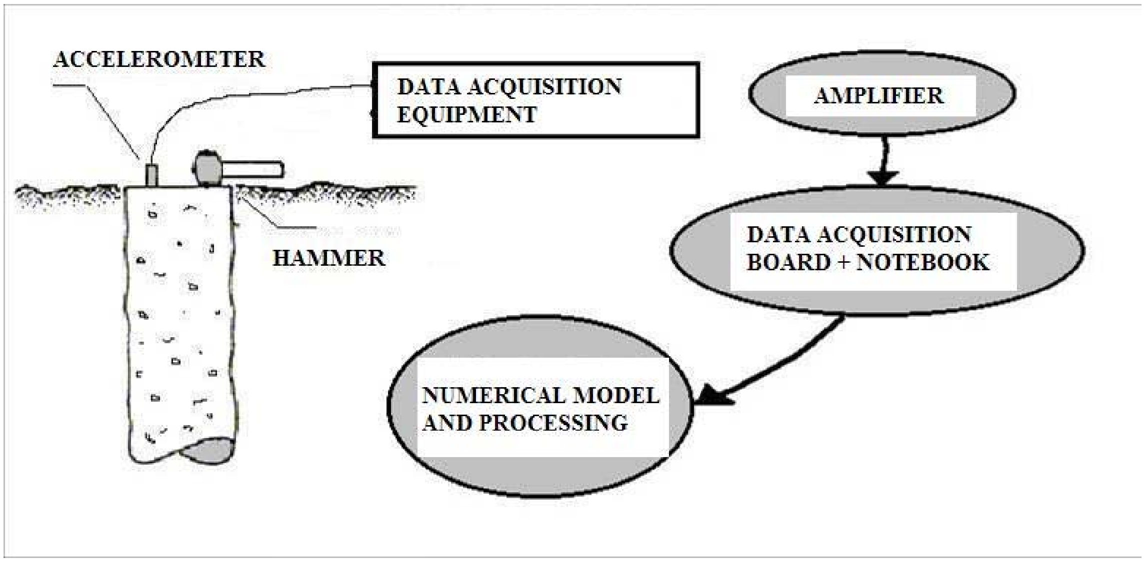

Figure 2. Experimental methodology of impact-echo method

In connection with this paper, the following equipments and instruments were used:

Accelerometers KYOWA AS-GB, with sensibility 100 mV/g, were used to measure the dynamic response of the piles. A dynamic strain amplifier KYOWA DPM-612B amplified the signal generated by the accelerometer. Moreover, the amplifier has a low-pass filter applied to avoid aliasing. A data acquisition board Computerboards PCM-DAS16D/16 of 16 bit of resolution and a maximum conversion time of 10 μs (100 KHz) was mounted on a notebook computer in order to record and process the signals by means of the program HP VEE 5.0[17]. The piles were excited with a hammer blow with different head weights from 590 g to 3 kg in order to excite different frequency contents. Rausche et al.[5] estimates the applied force as

1000 times the weight of the hammer. On the other hand, Davis[2], states that typical peak stress levels range from 5 MPa for hard rubber tips to more than 50 MPa for aluminum tips.

The signals were sampled, for all tests, with the following parameters: N = 75000 (total number of points), n = 15000 (sampling rate or number of points per second), T = 5 s (total time of the sample), Δt = 6.6710^{-5} s, (time interval), Δf = 0.2 Hz (frequency interval), $f_{máx}$ = 1500 Hz (maximum frequency).

An algorithm to obtain and process the data was programmed in the environment HPVEE[20]. After applying the Fast Fourier Transform, the spectrum was calculated using the Welch method (Peeters[21]). The signal processing includes corrections using square minimum techniques, low-pass filter and Ormsby filter.

3.2 Case study

Precast tube concrete piles were tested both in air (Figure 3) and inserted in the soil. External and internal diameters were 0.33 m and 0.19 m respectively and the pile length is 13.27 m giving a length/diameter relationship of approximately 40. The field measured wave velocity was 3706.1 m/s with a coefficient of variation very low of $9\ 10^{-4}$. Considering the mass density of concrete of 2400 kg/m^3, an elasticity modulus E = 33.0255 GPa was estimated.

Figure 3. Concrete annular piles tested in CTNOA S.A.

4 COMPUTATIONAL MODEL

4.1 Dynamic loading

The dynamic load $p(t)$ used in the analysis was define as an sinusoidal impact of a duration of half cycle and a period $2T_d$:

$$p(t)=\begin{cases} p_0 sen(\overline{\omega} t) & for \quad 0 \le t \le T_d \\ 0 \quad for \quad t > T_d \end{cases} \tag{1}$$

In a previous paper [6] the influence of shape, duration and magnitude of the dynamic load was studied.

4.2 Finite element model

Although some previous studies (ref) were performed using 1D and 3D models, the same qualitative conclusions than Liao and Roesset[18] were obtained in the sense that the response obtained with the 3D model is very similar in general terms to those predicted by the 1D solution. For this reason, a 1D model is used in this paper.

The piles were discretized using two-node, isoparametric truss elements in the one-dimensional analysis using standard linear elements under axial deformation, with a linear variation of displacements. A schematic representation of one-dimensional finite element model is shown in Figure 4. To simulate the effect of the soil on the lateral surface of the pile distributed springs with elastic stiffness k_v and dashpots with damping coefficient c_v were applied to each segment below the ground surface. The soil at the base of the pile was modelled using a spring with elastic constant K_v, and a dashpot with the damping coefficient C_v (Liao and Roesset[18]). c_v and C_v are intended to represent the radiation (or geometric) damping. Material damping in both pile and soil are assumed to be negligible under low strain integrity testing conditions. The values of the soil parameters per unit length of pile were determined through the following equations[18]:

$$k_v = 2.3 G_s \tag{2}$$

$$c_v = 2\pi \rho_s V_s r_p \tag{3}$$

$$K_v = 4 G_s r_p / (1 - \nu_s) \tag{4}$$

$$C_V = 0.85 K_V r_p / V_S \tag{5}$$

In which the properties of the soil are used: G_s the shear modulus, ρ_s the mass density, ν_s the Poisson modulus, V_s the shear wave velocity (S-waves) and r_P the radius of the circular footing resting on top of the soil that is assumed as hypothesis and in this case is equal to the radius of the pile analyzed.

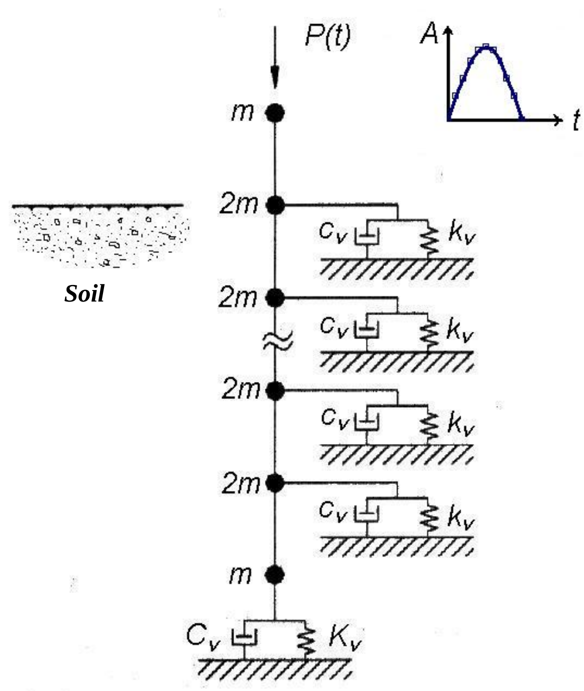

Figure 4: One dimensional model for piles and surrounding soil.

Another important factor influencing the results of the FE analysis is the element size. It is shown that in order to obtain a satisfactory result, the element size (*Δl*) should be less than about one tenth the wavelength (λ) of the highest frequency waves propagating in the shafts[10] i.e.,

$$\Delta l = \frac{\lambda}{10} \tag{6}$$

4.3 Equations of motion

The dynamic motion equations that govern the physical problem are given by:

$$\mathbf{M\ddot{U}} + \mathbf{C\dot{U}} + \mathbf{KU} = \mathbf{R} \tag{7}$$

in which **M**, **C** y **K** are the mass, damping and stiffness matrix respectively, and **R** is external loading vector; $\mathbf{U}$, $\mathbf{\dot{U}}$ y $\mathbf{\ddot{U}}$ are the displacement, velocity and acceleration vector respectively.

Both direct integration and mode superposition methods were used in order to solve numerically the system (7). Both schemes are theoretically equivalent but the computational cost is different.

The direct integration is carried out using the Hilbert, Hughes and Taylor method[22], also named α *method*, which is based in the solution of the following modified equation:

$$\mathbf{M}^{t+\Delta t}\ddot{\mathbf{U}} + (1+\alpha)\mathbf{C}^{t+\Delta t}\dot{\mathbf{U}} - \alpha\mathbf{C}^{t}\dot{\mathbf{U}} + (1+\alpha)\mathbf{K}^{t+\Delta t}\mathbf{U} - \alpha\mathbf{K}^{t}\mathbf{U} = (1+\alpha)^{t+\Delta t}\mathbf{R} - \alpha^{t}\mathbf{R} \qquad (8)$$

The system (8) could be considered as an improved trapezoidal rule. The parameter α adds an fictitious damping to the model that gives stability to the method damping strongly the higher modes. The value of α is between 0 y –1/3. When α equals zero, this method is equal to linear acceleration method (Newmark method[22] with β = 1/4), meanwhile α = –1/3, exists an important amount of fictitious damping. The parameter γ is the same that used in Newmark method, being recommendable to use 1/2.

In the mode superposition method, integration algorithms are used that assumed a linear variation of temporal functions between the points defined by the temporal increments. This method is extremely efficient in cases in which only few modes are necessaries in order to obtain a right dynamic response.

5 RESULTS OBTAINED

5.1 Piles in air

In order to calibrate as the numerical as the experimental procedures, a set of tests with piles isolated in the air were performed. A typical frequency spectrum is presented in Figure 5, in which up to 9 peaks could be distinguished associated to the first 9 axial vibration modes of the piles.

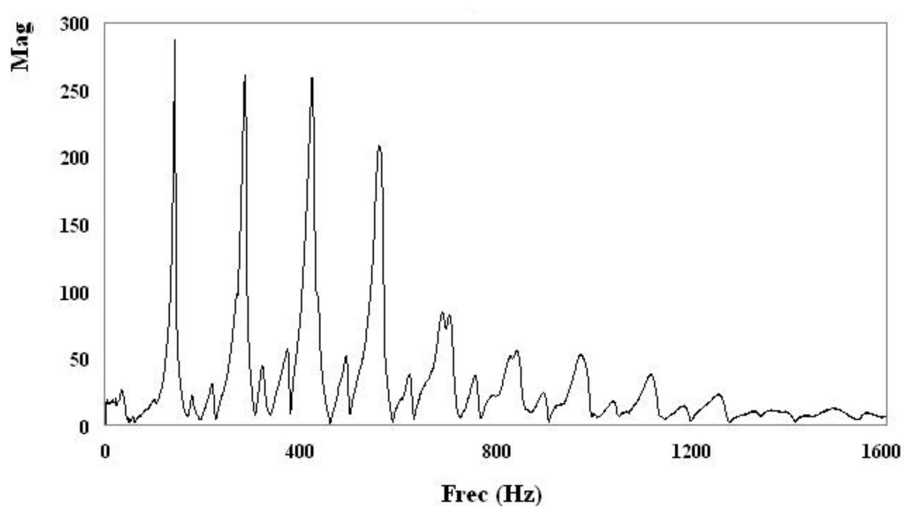

Figure 5. Frequency spectrum from a test.

The damping coefficient was determined from the spectrum peaks using the half power bandwidth method. The results obtained are presented in Table 1.

Table 1: Damping coefficients.

Mode	1	2	3	4	5	6	7	8	9
ξ	0.012	0.018	0.015	0.014	0.020	0.018	0.013	0.011	0.010

In order to obtain the dynamic response, the impact force was calculated using the value suggest by Rausche[5] giving the magnitude of the pulse about 6000 N. After some numerical tests, a duration of the pulse of 1.5 ms was selected. The FEM model was built with truss elements of 0.05 m in length. The mode superposition method using the first ten modes was selected in this case in order to solve the system (7). A temporal increment similar those used in the field tests was selected ($6.6667 \cdot 10^{-5}$ sec). The numerical-experimental comparison is presented in Figure 6. An excellent correlation was founded and the numerical and experimental curves are coincident.

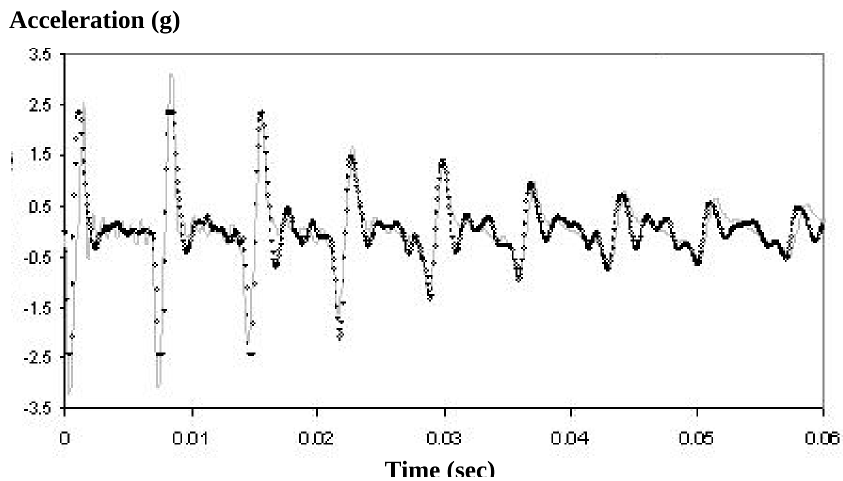

Figure 6. Numerical-experimental comparison. Piles in air.

The results in this stage were important to calibrate the numerical model, pulse, time step, etc. and shows that the real propagation phenomena could be studied confidently using the models proposed.

5.2 Piles inserted into the soil

The next stage in the analysis was the study of piles insert into the soil that is the real situation that was included in the main objective of this paper. Then, similar piles that those studied in the air were tested inserted in the soil. When the pile is within into the soil the radiation condition applies and the waves are propagated away from the system pile-soil. This effect has as main consequence a critical increase in the total damping and it is much more difficult to detect the reflection of the waves not only in the defect but in the bottom too.

In Figure 7 are presented the tests results in a pile that has a variable length free above the soil between 1.7 and 1.9 m. The mass of the hammer was 3 kg and a low-pass filter of 1500

Hz was applied to the measured data. Only the first 16 ms of the dynamic response is showed because the wave is rapidly damped. In order to detect the different reflect waves the original measured curve was amplified using an appropriate amplifying function combining constant, exponential and logarithmic functions. Both curves real and amplified are showed in Figure 7. Moreover, the function (**F_amp**) used[23] to amplify the measured data is showed in Figure 7 too. In the amplified curve four peaks are detected and named a_i (i = 1 a 4).

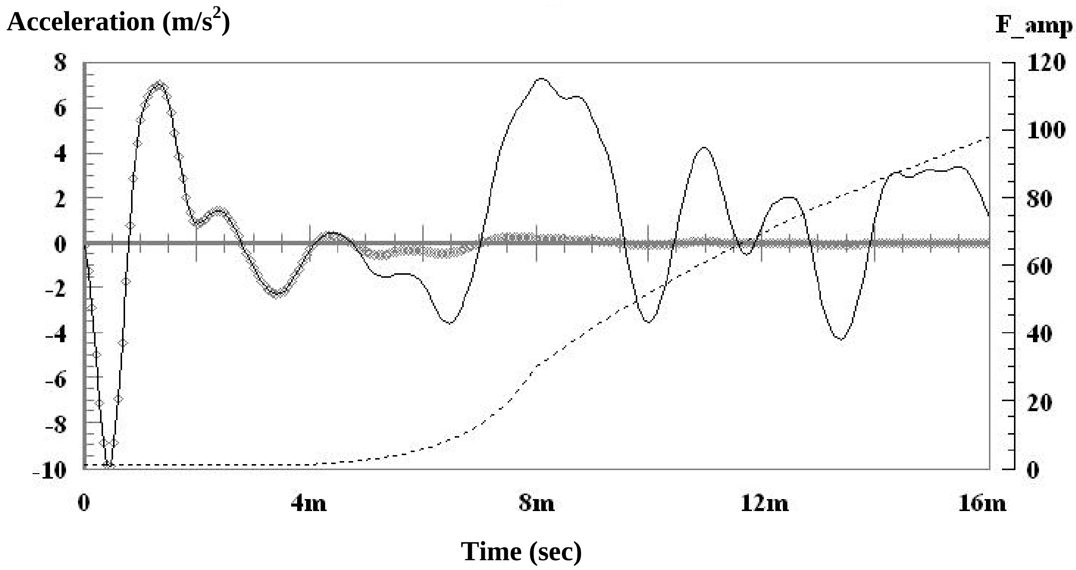

Figure 7. Tests result. Pile inserted into the soil.

The times corresponding to the reference (impact) and the four peaks are: t_0 = 1.285 ms, t_1 = 2.394 ms, t_2 = 3.4 ms, t_3 = 4.333 ms and t_4 = 8.748 ms. The distance between two peaks could be calculated simply with:

$$\Delta l = \frac{V_P \Delta t}{2} \tag{9}$$

Using (9) for the first reflection corresponding to $\Delta t = t_1 - t_0$, a length of 2.06 m is obtained, which is very close to the free portion of the pile. Moreover, the distance calculated between a_1 and a_2 is 1.86 m which could correspond to the second reflection in the grade. After that, the signal starts to be distorted by the superposition of effects. The instant a_3 is located in the mid point of the total length of the pile, which could indicates a fixed end of the pile. Finally, the distance obtained between $a_4 - a_0$, is 13.84 m, which implies only an error of 4.3 % respect to the real length (13.27 m). This was considered a very good and promissory result taking into account the very high relation length/diameter (40).

After that the wave reflected into the bottom of the pile arrives, there are more successive reflections, but the wave pattern is very complicated because the superposition of many reflections.

In figure 8, the numerical-experimental comparison is presented. The soil data were estimated from typical values in the site: $G = 3{\cdot}10^6$ N/m^2 and $\gamma = 1900$ kg/m^3 for upper layers and $G = 8{\cdot}10^6$ N/m^2 and $\gamma = 1800$ kg/m3 for lower layers. The pulse generated for the 3 kg headhammer, was estimated as 23850 N amplitude and 1.5 ms total duration.

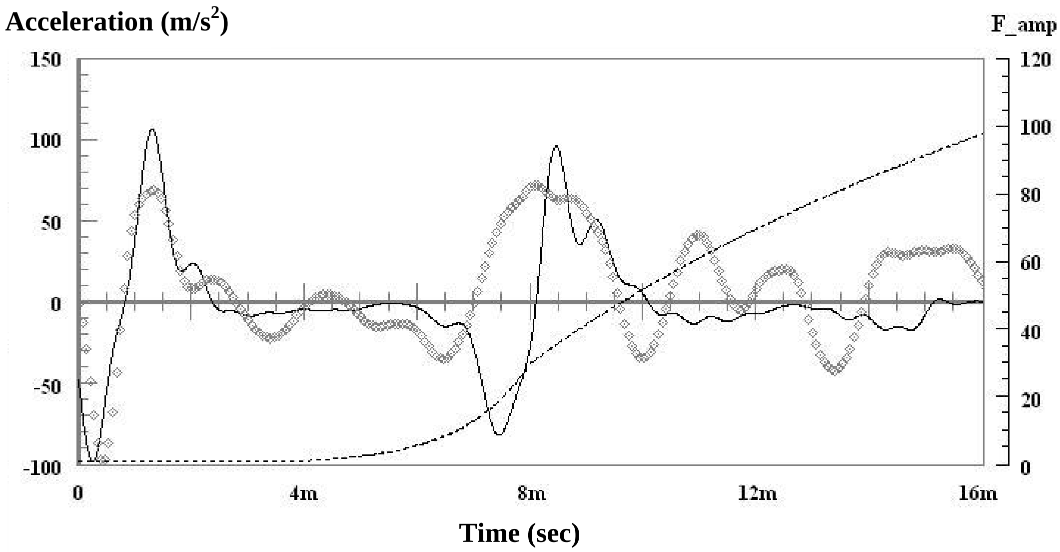

Figure 8. Numerical-experimental comparison. Pile 1 inserted into the soil.

In Figure 9, the results corresponding to other tested pile with a free distance of 0.7 m are presented. In this case, the pulse used has 13540 N amplitude and 1.6 ms of duration.

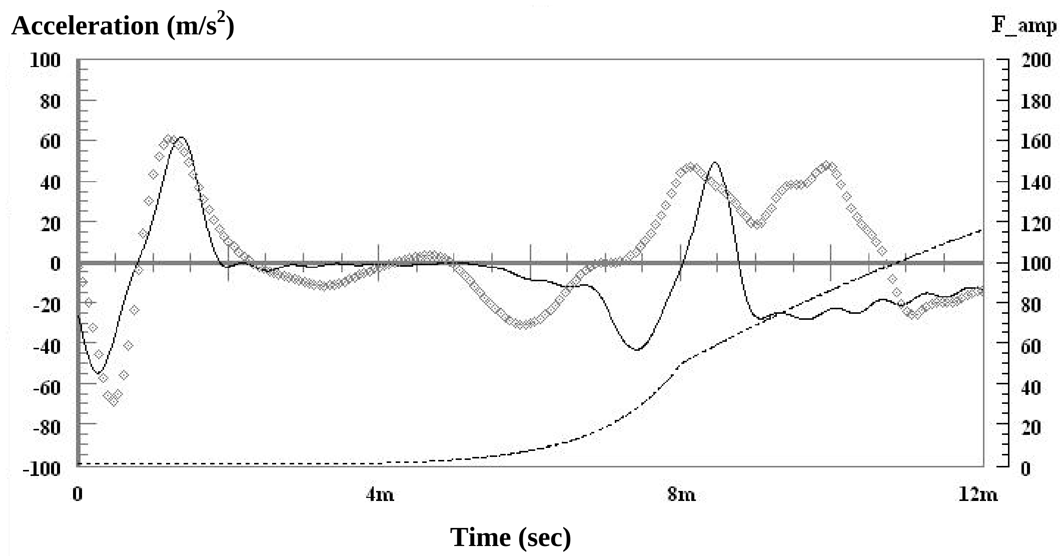

Figure 9. Numerical-experimental comparison. Pile 2 inserted into the soil.

As it was expected, when the pile is inserted into the soil more differences could be appreciated between the results from experimental tests and the numerical ones. However, taking into account the numerous uncertainties involved, especially with the soil properties, the results are considered satisfactory.

The numerical-experimental comparison reveals that it is possible to estimate the real length of long piles with minor errors and, in case there are important defect, it could be detect confidently. This possibility will be analyzed in the next points.

5.3 Neck defects

Considering the main objective of the paper, a numerical study about defects was performed and the results are presented in the next sections. A model of a pile with equal diameter and length of the test piles was made, but with solid circular section and a free distance of 0.7. The pulse used in the numerical analysis was 13,540 N of amplitude and 1.6 ms of total duration. In this section, a neck defect as presented in Figure 10 is studied. The total diameter (D_p) is reduced 10 cm at a length of 3.25 m from the head of the pile. The neck has a total length (l_c) of 5 cm.

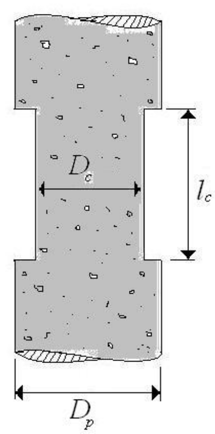

Figure 10. Neck defect.

The dynamic response is presented in Figure 11 when a low-pass filter of 1,500 Hz is used. Moreover, in Figure 12 shows the dynamic response when a low-pass filter of 8,000 Hz is used.

Comparing Figures 11 and 12, it is clear that the defect only is detected when high frequencies are included in the analysis. From a practical point of view, it is very important taking into account this observation when the accelerometers that will be used in the test were selected. The frequency range from the accelerometers must cover high frequencies but defects could pass unnoticed in the analysis.

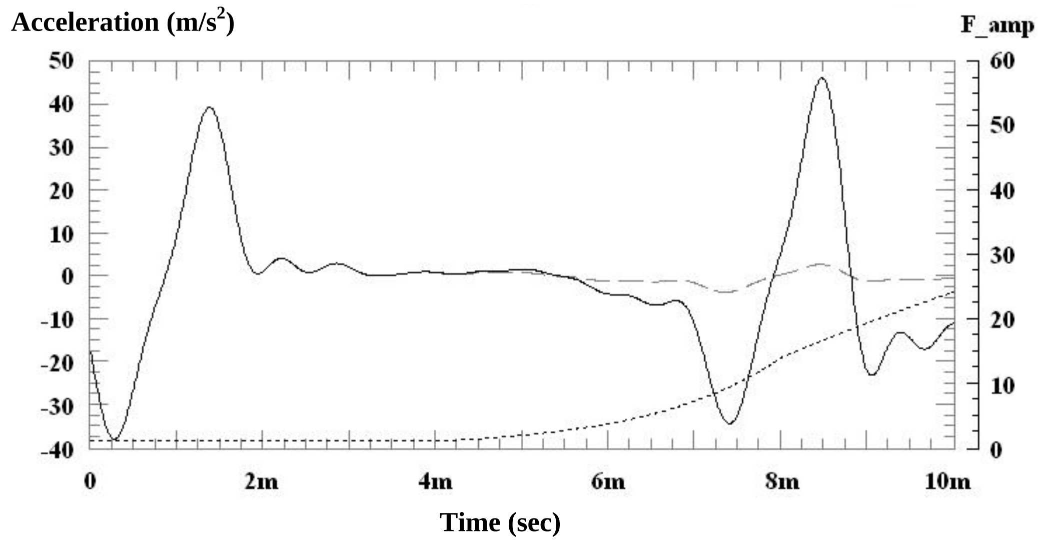

Figure 11. Numerical analysis. Neck defect. Low-pass filter of 1,500 Hz. Location 3.25m.

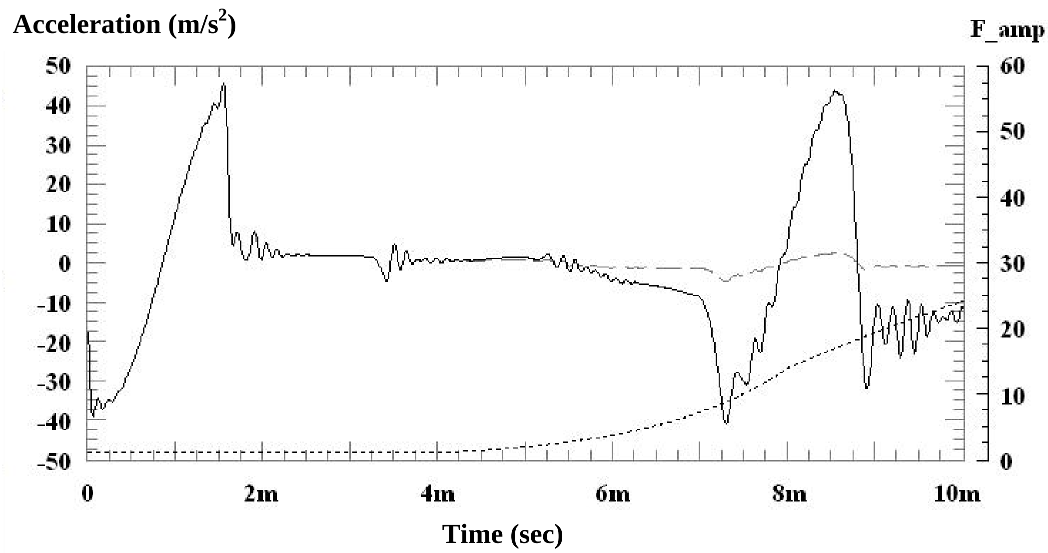

Figure 12. Numerical analysis. Neck defect. Low-pass filter of 8,000 Hz. Location 3.25m

It is must be noted that is very important the magnitude and duration of the stress pulse induced with the hammer. It would be very difficult to detect a defect located to a distance equal to $V_p \cdot T_p$. For this reason, the impact is better when introduces high levels of energy in a time as low as possible.

In Figure 13 the dynamic response is presented to a similar neck but located to a distance of 9.75 m from the head of the pile. It is used the same pulse and a low-pass filter to 8,000 Hz.

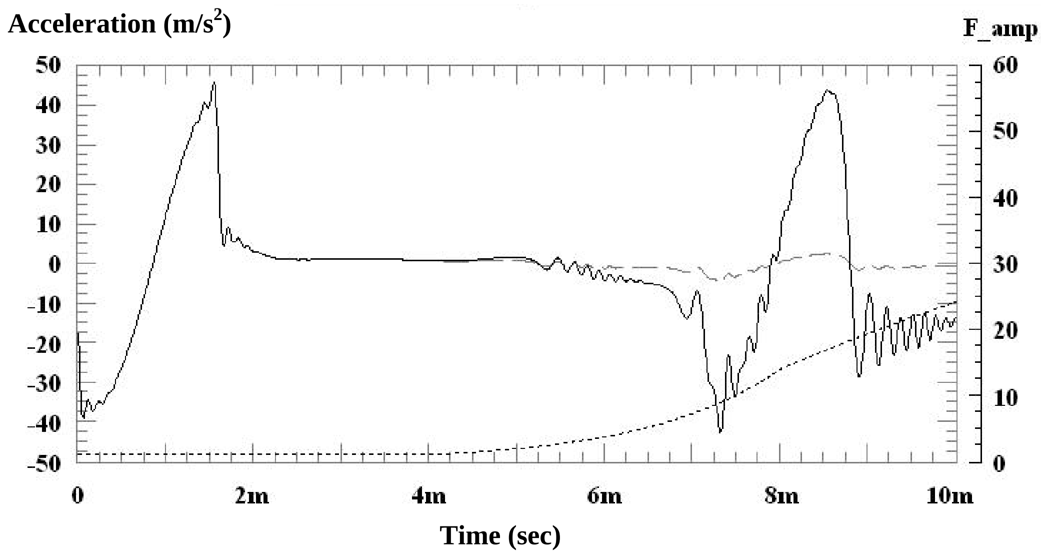

Figure 13. Numerical analysis. Neck defect. Low-pass filter of 8,000 Hz. Location 9.75m

It can be seen that the defect is detect accurately.

5.4 Bulb defects

In this point, bulb-type defects are analyzed. In this case, the bulb diameter (D_b) is 10 cm greater than the diameter of the pile. The other geometrical and physical properties are the same that in point 5.3. In figure 14, the dynamic response is presented.

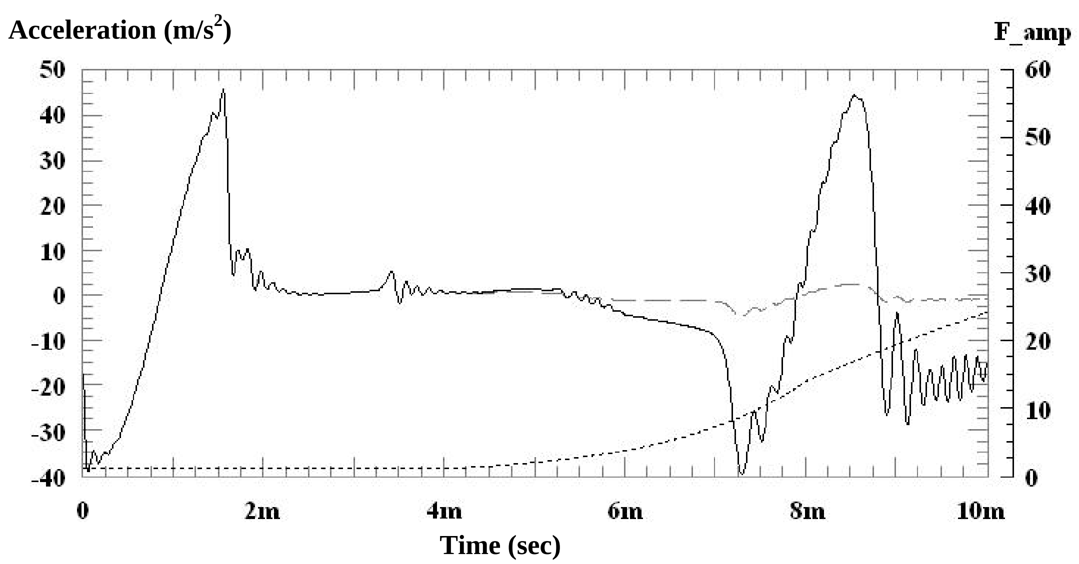

Figure 14. Numerical analysis. Bulb defect. Low-pass filter of 8,000 Hz. Location 3.25m

The accuracy obtained is very good, with an error minor to 5 %.

In figure 15, the dynamic response is presented to bulbs having different lengths: 0.05, 0.1 and 0.2 m. It can be seen a considerable increase in the amplitude of the reflection.

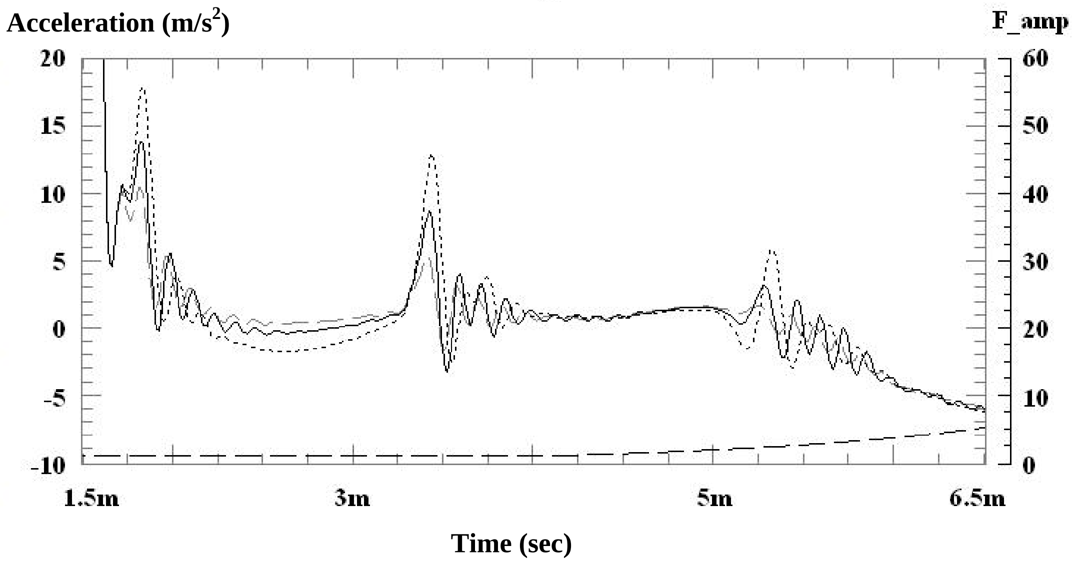

Figure 15. Numerical analysis. Bulb defect. Low-pass filter of 8,000 Hz. Location 3.25m

In Figure 16 the dynamic response is presented for different diameter bulbs: 0.38, 0.43 and 0.48 m. In this case, the length of the bulb is adopted as 0.2 m

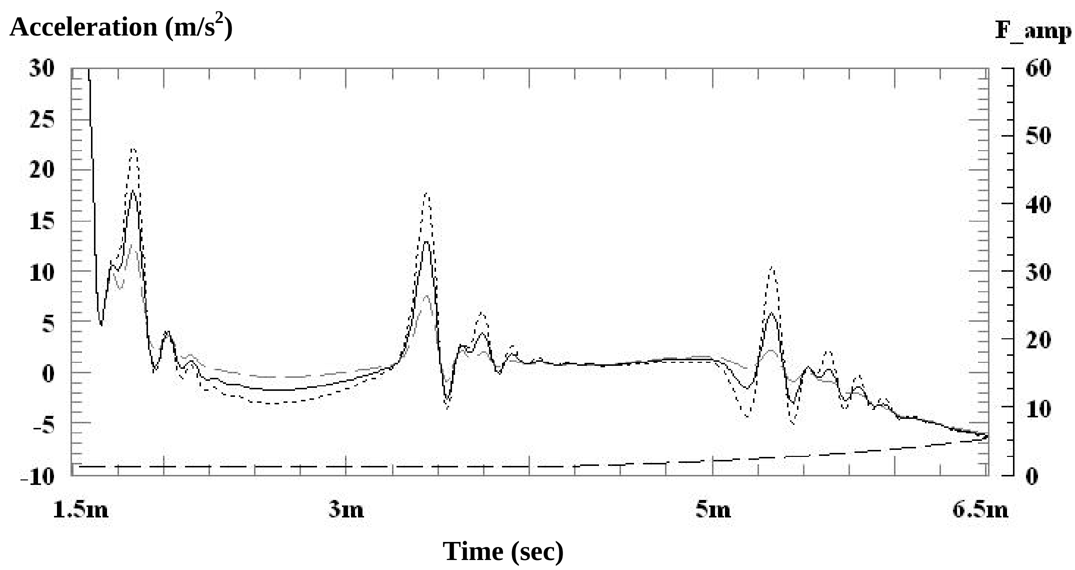

Figure 16. Numerical analysis. Bulb defect. Low-pass filter of 8,000 Hz. Location 3.25m

Again, it can be seen a considerable increase in the amplitude of the reflection, but must be pointed out that it is not possible to distinguish if the defect has greater length or diameter.

5.5 Neck-Bulb defects comparison

Finally, it is possible to identify if the defect is neck or bulb type. For this purpose, the boundary condition in a pile is studied. In figure 17, the dynamic response is presented for a pile of 13 m length, 0.33 m diameter, a velocity of wave propagation of 3709 m/s and a mass density of 2400 kg/m^3. There are two curves corresponding to a fixed-end pile and a pile with a low-stiffness spring similar to free-end condition.

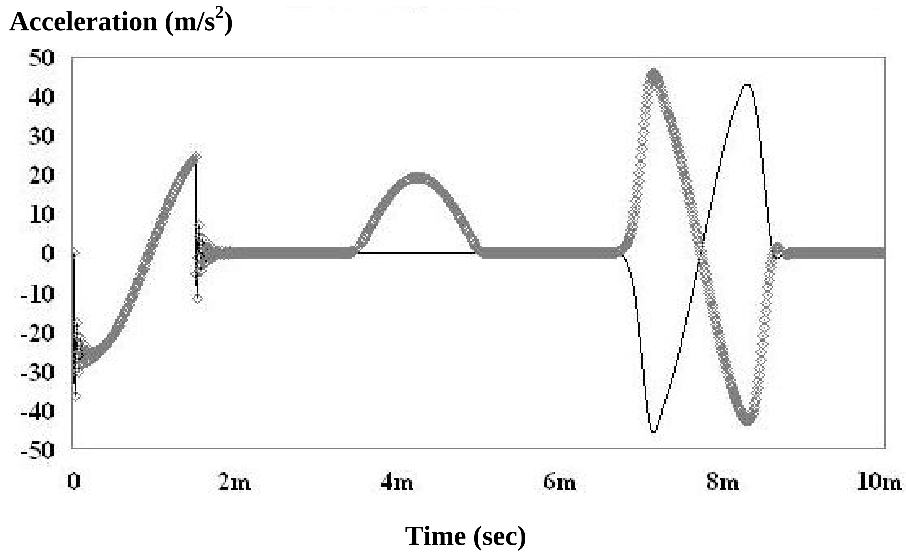

Figure 17. Numerical analysis. Neck-Bulb defect comparison.

Due to the elasticity modulus of the soils are lower than concrete (pile), the boundary condition is like free-end. In this case, taking into account the theory of propagation and reflection of waves in bars, the wave is reflected with the same shape that the impulse. When a defect neck-type exists, there is a weakening of the transverse section and the reflection is like a free-end. When a defect bulb-type exists, the inverse occurs and the reflection is like a fixed-end. This behaviour is observed clearly in Figures 12 to 17.

6 CONCLUSIONS

In this paper a dynamic analysis of the integrity of long piles in the time domain is presented. The impact-echo method was used and consequently a low-strain impact is applied to send stress waves through the tested element. Then, the superposition of effects and the Hooke's law could be applied. Comparing the theoretical (virgin) and real results, information about the length and integrity of the pile could be obtained.

Initially, a set of tests with piles isolated in the air were performed. The results in this stage were important to calibrate the numerical model, pulse, time step, etc. and shows that the real propagation phenomena could be studied confidently using the models proposed

The piles were discretized using two-node, isoparametric truss elements in the one-dimensional analysis using standard linear elements under axial deformation, with a linear variation of displacements. To simulate the effect of the soil on the lateral surface of the pile distributed springs and dashpots were applied to each segment below the ground surface. The soil at the base of the pile was modelled using a spring and a dashpot. The dashpots are intended to represent the radiation (or geometric) damping.

The next stage in the analysis was the study of piles insert into the soil that is the real situation that was included in the main objective of this paper. Then, similar piles that those studied in the air were tested inserted in the soil. In this case, more differences could be appreciated between the results from experimental tests and the numerical ones. However, taking into account the numerous uncertainties involved, especially with the soil properties, the results are considered satisfactory. The numerical-experimental comparison reveals that it is possible to estimate the real length of long piles with minor errors. It is important to note that, in case that driven precast piles are analyzed, the soil properties change and are strongly distorted as consequence of the driving impact process. In the case of cast in place piles, this change in the properties of the soil is not present and the analysis could have higher accuracy.

Finally, neck and bulb-type defects were studied numerically and, based in the obtained results, must be pointed out the following conclusions:

a) It could be detect defects with errors minor to 5%

b) It could be distinguish between neck and bulb defects

c) In connection with the hammer used, it is very important the magnitude and duration of the stress pulse induced. The impact is better when introduces high levels of energy in a time as low as possible.

d) The use of low-pass numerical filters leads to improvements in the visualization of results, filtering the high frequency components that generally have lower accuracy. However, the appropriate value of the cut frequency must be analyzed carefully in each case because sometimes could exist defects that cannot be detected (see figures 11 and 12)

e) According to the said before, accelerometers with high frequency range are required. However, it is necessary high sensibility too. Normally, these two properties are opposite.

f) The use of amplifying functions in the time domain results invaluable in order to appreciate the reflections in piles inserted into the soil. In this paper, an amplifying function is used which combines constant, exponential and logarithmic function. It must be pointed out that the logarithmic function is used for limiting the high grow due to exponential function, which leads to distortions in the signal.

Summarizing, the most important result presented in this paper is the experimental and numerical evidence that it is possible detect as the length as defects in long piles with length/diameter relationship up to 40.

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