Earthq Eng & Eng Vib (2023) 22: 105-115 **DOI**: https://doi.org/10.1007/s11803-023-2165-0

Ground-borne vibrations induced by impact pile driving: experimental assessment and mitigation measures

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Abstract: Impact pile driving is an interesting technique for the construction of deep foundations from a practical and economical point of view. However, the generalization of this technique can be restricted due to the excessive vibration levels that can be generated, which can be especially problematic in residential areas. However, different mitigation measures can be applied to prevent excessive vibration levels inside buildings located near construction sites. To compare its efficiency through a numerical prediction tool, two experimental test sites are first presented and characterized. From the results obtained, it was found that the construction of an open trench near the impact source can be used as an efficient mitigation measure to reduce the maximum vibration levels evaluated in this study.

Keywords: impact pile driving; ground-borne vibrations; experimental and numerical analysis; mitigation measures

1 Introduction

Deep foundations are an ancient solution, with reports of the use of timber piles in the year 800 BC (FHWA, 2016). This type of foundation is currently used in engineering practice to solve problems induced by poor geotechnical soil characteristics, typically shallow soil layers with low stiffness or low bearing capacity. Given the broad range of solutions that have emerged over this long period, with different materials and construction techniques, a possible way to classify the piles is based on their installation method, whereby piles are usually classified into two categories: displacement and non-displacement piles (Matos Fernandes, 2020). In particular, pre-fabricated driven piles are included in the first category, which is characterized by the occurrence of soil movements to allow the penetration of the pile into the ground, without the need for prior removal of the soil, which characterizes the second category.

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Supported by: Programmatic funding - UIDP/04708/2020 of the CONSTRUCT - Instituto de I&D em Estruturas e Construções - funded by national funds through the FCT/MCTES (PIDDAC); Project PTDC/ECI-CON/29634/2017 - POCI-01-0145-FEDER-029634 - funded by FEDER funds through COMPETE2020 - Programa Operacional Competitividade e Internacionalização (POCI) and by national funds (PIDDAC) through FCT/MCTES

Received January 4, 2022; **Accepted** June 28, 2022

For the specific case of the impact pile driving technique, some advantages can be pointed out, such as lower cost and reduced construction time. However, during driving operations performed close to buildings, some negative environmental consequences are expected, essentially associated with the generation of vibrations. The generated seismic waves can disturb people and activities, cause damage in existing equipment, and, in extreme scenarios, cause damage to surrounding structures (Athanasopoulos and Pelekis, 2000; Massarsch and Fellenius, 2014; Rahman *et al*., 2017; Hindmarsh and Smith, 2018). Thus, the prediction and quantification of these pernicious phenomena during the design stage of the foundations is essential. However, in many cases, the difficulty of its prediction limits the applicability of the method, resulting in harmful technical and economic effects.

For the quantification of the effects of vibration on humans and buildings, particle vibration velocity is usually used as the primary indicator. The main reason for that is the existing correlation between the damage sustained by buildings and the vibration velocity experienced by the soil (Chameau *et al*., 1998). In particular, and given the complexity of the vibration signal, single estimators, such as peak particle velocity (PPV), are often used. The vast majority of standards and guidelines related to this phenomenon specify the permitted vibration levels based on this vibration indicator (SWEDISH STANDARD - SS 25211, German standard DIN 4150-3, British standard BS 7385-2, Portuguese standard NP 2704 and FTA recommendations (Quagliata *et al*., 2018)).

For situations where it is expected to reach the vibration levels preconized by different standards, it is essential to limit their impact. To maximize the effectiveness, the first efforts should be concentrated at the source. From a practical standpoint, one of the first measures corresponds to the limitation of driving energy. However, in most cases, this limitation is not possible due to the necessity of driving the pile. Thus, alternatively, mitigation measures of a temporary nature applied at the propagation path can be considered. In this context, the present study presents, in a first step, a summary of the experimental measurements of ground-borne vibrations induced by impact pile driving performed on two different test sites. The recorded vibration levels are subjected to the thresholds preconized by the Portuguese national standard NP2074. Given the infringement of the standard thresholds, a general numerical study about the effectiveness of two possible mitigation measures is performed: an open trench solution and a continuous distribution of a heavy mass resting on the ground surface.

2 Experimental assessment of ground-borne vibrations

2.1 General description

Experimental measurements of ground-borne vibrations induced by impact pile driving were performed on two different test sites, with distinct geotechnical properties. The first one, called the INEGI test site, is located near the city center of Porto (Portugal). The second one (called the Aveiro test site) is located near Aveiro (Portugal). A general view of both test sites at the time of pile driving is depicted in Fig. 1.

Supporting the building design, a detailed geological-geotechnical characterization is available for both test sites. According to the geological-geotechnical report from the INEGI test site, a total of seven boreholes, with SPT tests spaced every 1.50 m in depth, were carried out. Essentially, this test site is geologically formed by a granite residual soil, with the stiffness and strength increasing with the depth, as can be seen in Fig. 2. The groundwater table was found at approximately 8 m deep.

Regarding the Aveiro test site, and according to the geological-geotechnical report that supports the building design, a total of 22 boreholes, covering an area above 27,000 m^2, were carried out. SPT tests spaced every 1.50 m in depth were performed. The boreholes have a total length from 6 to 9 m. From the results observed, the building implantation area is geologically formed by alluvial deposits of clays. In some boreholes, a sand layer belonging to the deposits of ancient beaches was detected. Superficially, the ground is covered by a layer of landfill. Moreover, an embankment with a thickness of about 1 m was built over the sand/clay layers before the pile driving operation. According to the geological geotechnical report, the embankment material should be gravel or sand belonging to classes A-1 or A-2 of the classification system in AASHTO.

In addition to the results provided by the geological-geotechnical report, non-intrusive geophysical tests were performed at the INEGI test site: Seismic Refraction Testes and SASW tests (Spectral Analysis of Surface Waves). From these tests, it was possible to obtain the P and S-wave profiles, as presented in Fig. 3.

Concerning the foundations of the buildings, the INEGI test site comprises 160 quadrangular piles with two distinct sections of 400 mm × 400 mm and 350 mm × 350 mm. These piles are made of precast concrete and have a total length varying from 8 to 15 meters. The Aveiro test site is composed of 64 quadrangular piles (section 270 mm × 270 mm) with 6 m length. On both sites, the driving equipment corresponds to a Junttan PMx25 equipped with a hydraulic impact hammer SHK110-7. The mass of the hammer ram is equal to 7 tons.

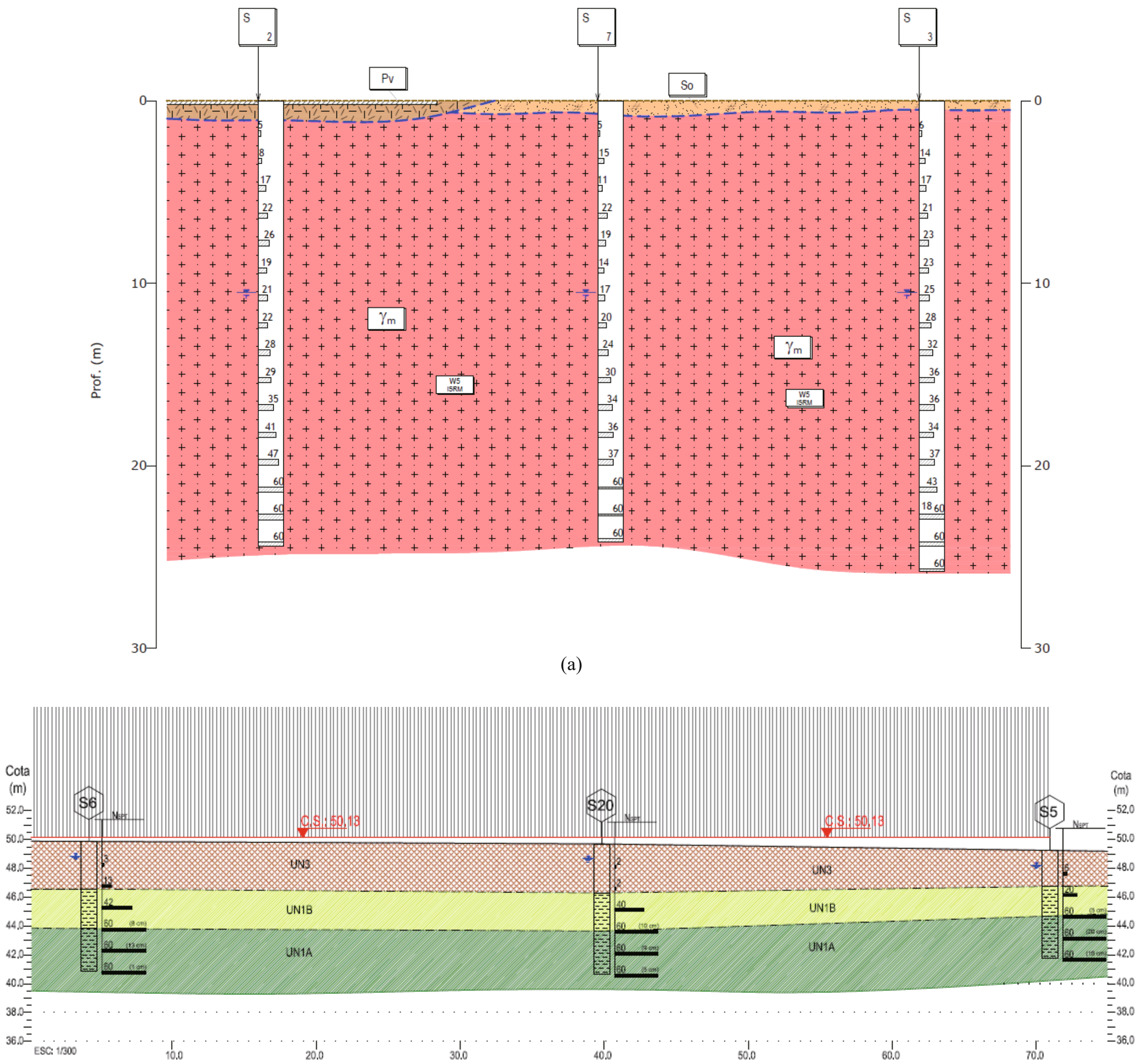
2.2 Experimental setup

The experimental assessment of ground-borne vibrations induced by impact pile driving at both test sites includes different observation points, placed at the ground surface and at different distances from the pile, where sensors were installed for the measurement of the physical system response. Based on the experimental setup, the vertical acceleration component was recorded by means of a set of unidirectional piezoelectric accelerometers with reference PCB603C01, with two different measurement ranges: ± 0.5 g (sensitivity of 10 V/g) and ± 5 g (sensitivity of 1 V/g). Higher sensitivity accelerometers were placed close to the impact source

(a)

(b)

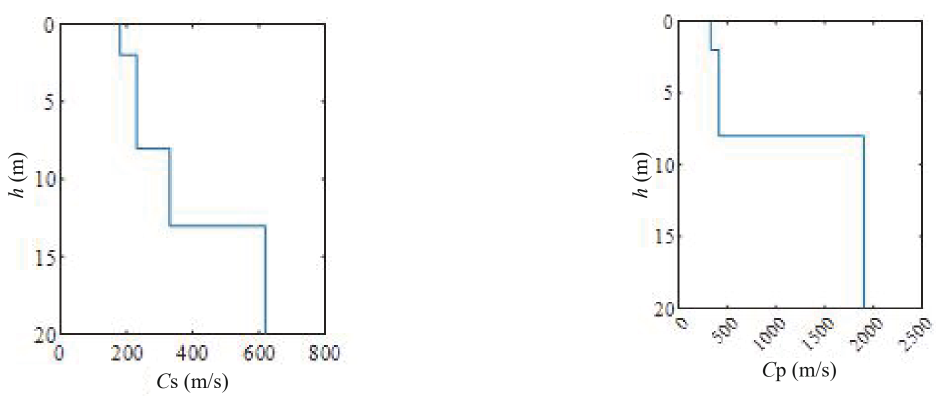
Fig. 1 General view of the experimental test sites: (a) INEGI; (tb) AVEIRO



(a)

(b)

Fig. 2 Example of a geological-geotechnical profile from: (a) INEGI test site; (b) Aveiro test site



(a)

(b)

Fig. 3 Dynamic properties of the soil along the depth: (a) S-waves velocity profile; (b) P-waves velocity profile

(distance up to 12 m). The collected signals are conditioned by an electronic system composed of a laptop, connected to an acquisition system with reference NI CDAQ-9172.

With regard to the INEGI test site, the ground motion measurements include the excitation provided by the driving of four piles: three of them with a section of 400 mm × 400 mm and another one with 350 mm × 350 mm. All the piles had a total length equal to 12 meters and the final installation depth was around 11 m. During the driving operations, the hammer height-of-fall was variable, increasing with the penetration depth (from 15 cm to a maximum of 60 cm). A total of 1841 hammer impacts were needed to reach the final penetration depth reported above (mean value for the four piles). The transient signal was recorded at the ground surface for a wide range of points, placed between 1 and 32 meters from the pile.

With regard to the Aveiro test site, the experimental activities include the measurement of the ground motion provided by the driving of four piles, with a total length equal to 6 m and with a cross-section of 270 mm × 270 mm. The final installation depth was around 5.5 m and a total of 202 hammer impacts were needed to reach this final penetration depth. As evidenced for the previous test site, the hammer height-of-fall was also variable, from 30 cm to 60 cm. The observation points considered were placed between 2 and 36 meters from the pile. Figure 4 presents photographic records of the experimental activities.

In an effort to find a systematic way to organize the extended set of recorded information, Figure 5 presents a compilation of the maximum peak particle velocity according to the horizontal distance to the pile. The results from both sites are represented on this figure. This type of graphical representation has a high practical value, allowing the maximum vibration level that can be expected at the base (foundations level) of any building located in the vicinity to be identified (if the soil-structure interaction conditions allow to neglect the kinematic soil-structure interaction). Note that a separation of the impacts was made for each meter of driving, allowing average values to be adopted for each driving interval.

In a general way, the results from both sites present a typical behavior characterized by a continuous attenuation of vibration levels as the distance to the pile increases. This effect is expected given the increase of the geometric damping with the distance. On the other hand, a clear difference on the vibration amplitude levels between both sites is observed, with the vibration levels recorded at the INEGI test site being much higher. Among others, it can be pointed out as the main reason for the differences observed for the geotechnical properties of the ground. In fact, the Aveiro test site is located in an alluvial deposit, characterized by the presence of soft soils in the first meters. On the other hand, the INEGI test site site is geologically formed by a granite residual soil with higher stiffness.

Note that in function of the propagation characteristics of Rayleigh waves, the maximum vibration levels occur for a certain depth, close to the ground surface (about 0.5 to 1 m), as can be seen in previous works (Masoumi *et al*., 2009; Homayoun Rooz and Hamidi, 2019). However, the deviation between this value and that one recorded at the ground surface is not very significant.

Additionally, the frequency content of the soil response is presented. In this kind of phenomenon, the PPV values should be analyzed taking into account the most relevant frequency content of the response. As previously expressed, the limits on the peak vibration values imposed by the vast majority of the standards/guidelines are variable according to the dominant frequency in the velocity spectrum. Thus, Fig. 6 shows the frequency content of the ground considering the driving of two representative piles (one from the INEGI test site and another from the Aveiro test site), addressing two variables: i) variation of the distance from the pile (8, 20 and 32 m); and ii) variation of the pile penetration depth.

As evidenced by the curves illustrated in the previous figure, and in addition to the expected attenuation of peak velocity values with increasing distance, there is an attenuation of the response at higher frequencies with the increase of the distance between the observation point and the pile. The larger influence of the material damping in the higher frequency content of the response justifies this observation. With regard to the evolution of the frequency content with the increase of the pile

(a) (b) (c)

Fig. 4 Pictures of the experimental setup: (a) pile driving rig Junttan PMx25 equipped with the hydraulic impact hammer SHK110-7; (b) acquisition system; (c) unidirectional accelerometer placed at the ground surface

penetration depth, although with a variable hammer height-of-fall, no significant variation of the frequency content can be observed with increasing driving depth.

2.3 Cosmetic and structural damage thresholds

As initially set, the reported phenomenon has

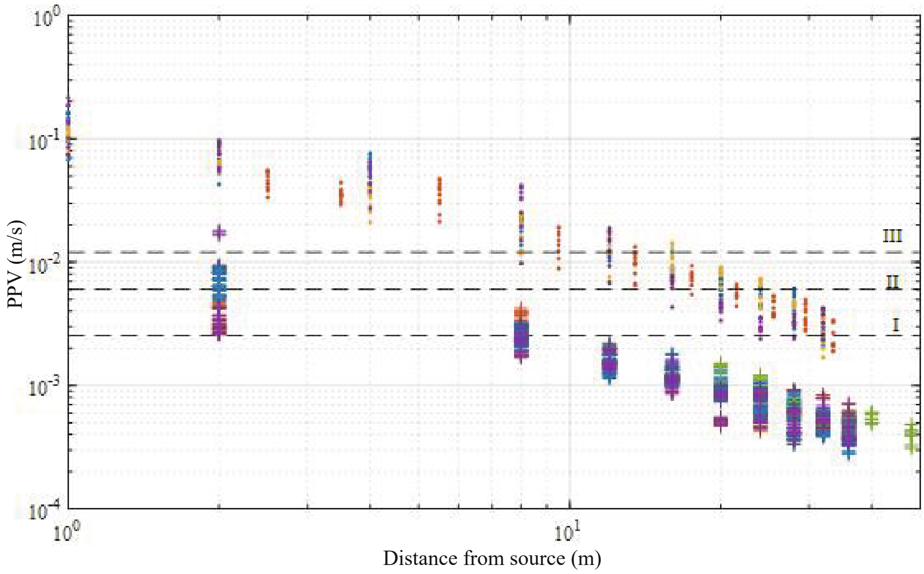
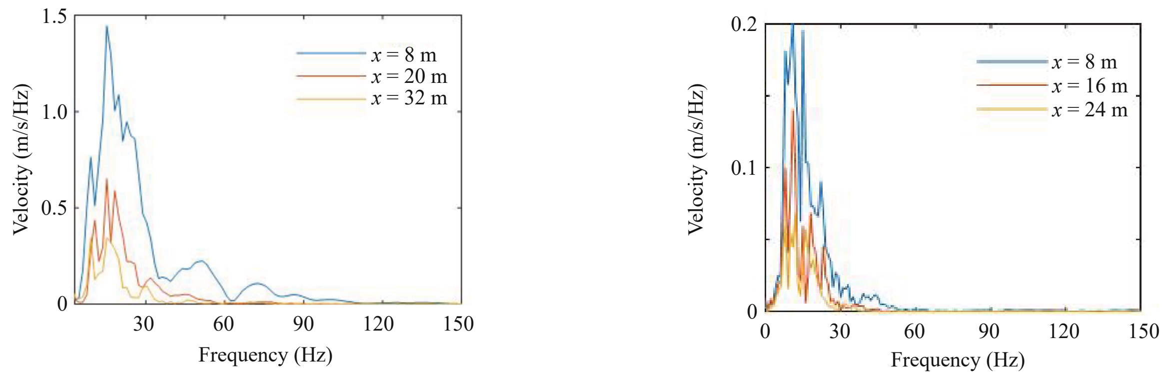
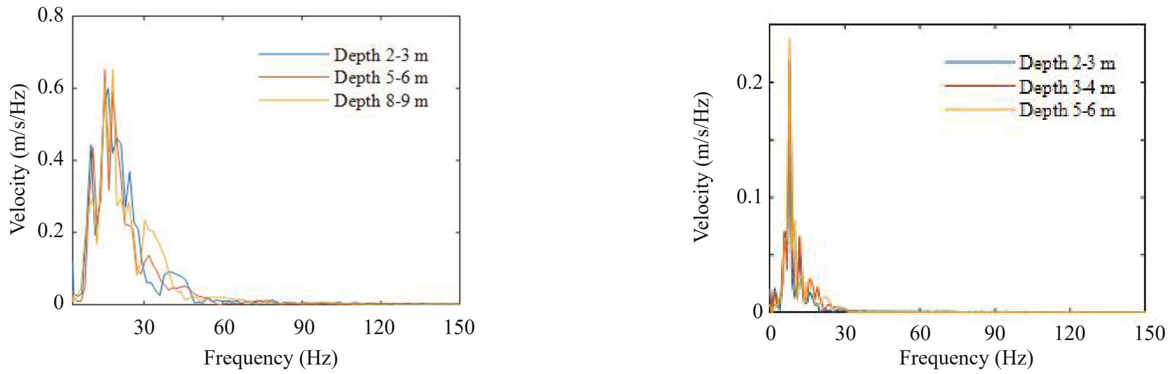


Fig. 5 Maximum peak particle velocity versus distance from the pile (circle marker – INEGI test site: Pile 1 - o; Pile 2 - o; Pile 3 - o; Pile 4 - o; cross marker – Aveiro test site: Pile 1 - +; Pile 2 - +; Pile 3 - +; Pile 4 - +) and maximum permitted PPV values for transient vibrations to prevent damage defined in the Portuguese Standard NP2074 (I-sensitive structures; II-current structures; III-reinforced structures)



(a)



(b)

Fig. 6 Frequency content of the experimental vertical vibration velocities considering: (a) different distances from pile and for a penetration depth in the interval 5-6 m; (b) considering different pile penetration depths and an observation point at 20 m from pile (upper: INEGI test site; down: Aveiro test site)

the potential to induce some damage to structures, especially when the distances between the pile and the receiver are relatively small. This condition is more critical for structures in poor conditions, as can be seen from Table 1, where the maximum permitted PPV values for transient vibrations to prevent damage defined in the Portuguese Standard NP2074 are presented.

In Fig. 5, the vibration levels recorded were plotted against cosmetic and structural damage thresholds preconized by the Portuguese national standard NP2074 for various types of structures and for a dominant frequency in the interval between 10 and 40 Hz (most relevant frequency range, as can be seen from Fig. 6).

As can be seen from Fig. 5, there is a clear distinction between the potential impacts preconized for facilities located in the different test sites. Given the higher levels of vibrations verified at the INEGI test site, the potential pernicious effects are more relevant. For the specific case of sensitive structures, even from distances higher than 25 m from the source of impact, the recorded levels of vibrations are larger than the standard limits. Concerning to the Aveiro test site, the vibration levels are significantly lower, as discussed before, resulting in no potential for negative impacts at distances higher that 10 m. For lower distances, the impact is not significant for current structures.

Given the conclusions reached, for similar conditions to the INEGI test site and in the presence of buildings in the vicinity, it is important to carefully analyze possible mitigation measures to make the application of the pile driving technique possible.

3 Numerical study of a mitigation measure

3.1 General overview of the numerical model

In situations where there is potential to exceed the legally established limits of vibrations, the design of simple and practical mitigation measures can be an alternative to the use of a given technique such as pile driving. Numerical modelling can be a powerful tool to perform this task. Thus, the numerical model previously presented in Colaço *et al*. (2021a) is briefly reviewed.

Given the complex system involved in the prediction of ground-borne vibrations induced by pile driving, composed of distinct components (hammer device, pile and soil), a modular numerical model is considered. This numerical model is split into two main modules: one comprises the pile-ground model, modelled by FEM-PML (finite element method-perfectly matched layer) approach in axisymmetric conditions; the other refers to the dynamic simulation of the hammer device. Since there is a dynamic interaction problem between the hammer device and the remaining system, both models are coupled, fulfilling the equilibrium and compatibility requirements. A schematic illustration of the problem is presented in Fig. 7.

The axisymmetric FEM-PML approach is formulated in the frequency domain. From the literature (Masoumi *et al*., 2009), it is clear that the nonlinear behavior of the soil near the pile greatly influences the generated vibration field. In this way, the nonlinear behavior of the ground is addressed by means of an equivalent linear methodology, where an iterative method that compensates for the inelastic behavior by adjusting the parameters of the elastic material to significant strain levels is considered, rendering a simple but reliable methodology. With regard to the impact hammer device, an analytical model was applied to determine the hammer impact force generated by an impact at the top of the embedded pile. A complete description of the entire model and the experimental validation can be found in Colaço *et al*. (2021a and b).

With the aid of the equivalent linear FEM-PML numerical model, the efficiency of different mitigation measures is analyzed. For that, and given the higher vibration levels found at the INEGI test site, the present case study adopts its geomechanical characteristics. For numerical modelling purposes, the elastodynamic

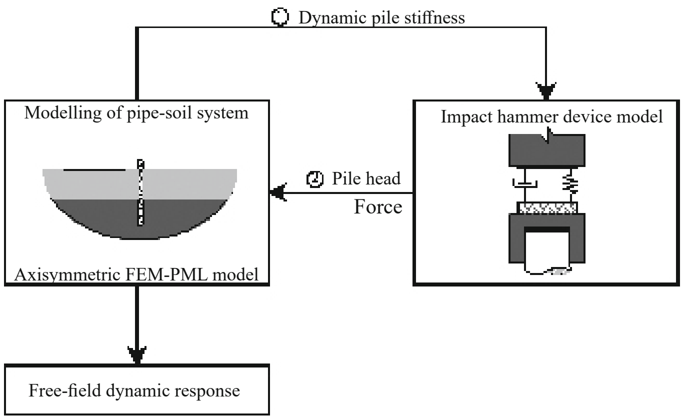


Fig. 7 Schematic illustration of the problem

Table 1 Maximum permitted PPV values (mm/s) for transient vibrations to prevent damage defined in the Portuguese Standard NP2074

Structure type	Dominant frequency (f)		
	$f \leq 10$ Hz	10 Hz $< f \leq 40$ Hz	$f > 40$ Hz
Sensitive structures	1.5	3.0	6.0
Current structures	3.0	6.0	12.0
Reinforced structures	6.0	12.0	40.0

properties of the ground were obtained from non-intrusive geophysical tests, as presented before. Laboratorial characterization of the soil indicates a mass density (ρ) close to 1900 kg/m^3. According to the sensitivity study preconized in Colaço *et al*. (2022), a hysteretic damping factor (ξ) of 5% is used for the first layer and a value of 2.5% is used for the remaining layers. According to the geotechnical characterization, the elastodynamic properties of the ground are expressed in Table 2, where the variable h stands for the thickness of the layers.

In addition to soil small-strain stiffness data, the consideration of an equivalent linear methodology demands information about the stress-strain soil behavior when the strains are larger than the elastic threshold. As such, the mathematical laws proposed by Ishibashi and Zhang (1993) describing stiffness degradation and damping increase with the increase of the strain levels are considered. These are plotted in Fig. 8 for different confining stresses and considering non-plastic soil.

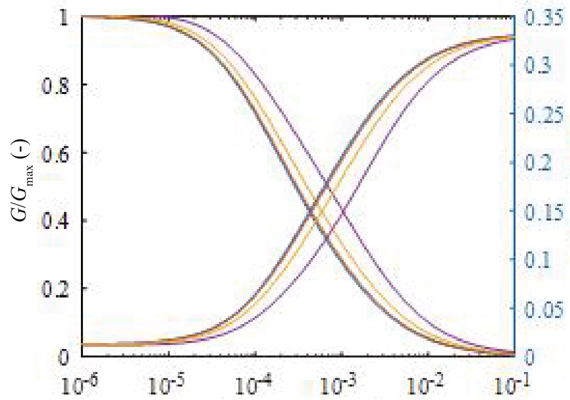


Fig. 8 Evolution of the dynamic properties of a granite residual soil with shear strain - Ishibashi and Zhang curves (1993)): stiffness degradation and damping increase for different mean effective stresses (from 53.3 kPa to 283.3 kPa)

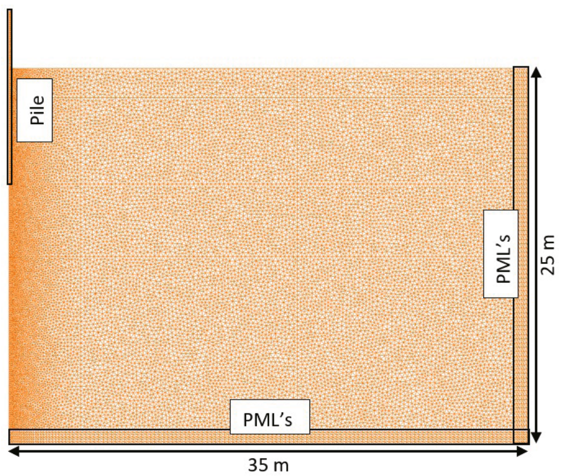


Fig. 9 FE–PML mesh adopted to model the pile-ground system

As a general overview, the pile-ground medium is discretized by a FE-PML mesh, corresponding to a discretized cross-section of 35 m × 25 m. The PML layers are bounding the FEM region, as illustrated in Fig. 9. Given the limitations of an axisymmetric formulation, in which it is not possible to model the true geometry of the pile (rectangular section 400 mm × 400 mm), an equivalent circular pile section was considered. The elastodynamic properties of the pile can be found in Table 2.

3.2 Trench solution

Aiming to compare the response of the soil free surface before and after the construction of an open trench, two variables were analyzed: variation of the trench depth (D) and distance fom the pile to the trench (x). Note that the intention is to design a measure that can be easily implemented on the site, with no significant cost to the project. A schematic representation of the trench solution is presented in Fig. 10.

Considering the particular case of a pile penetration depth equal to 2 meters, Fig. 11 presents the experimental results of one of the monitored piles with a cross-section of 400 mm × 400 mm. In terms of experimental results, the curve shown in this figure was obtained for pile penetration depths in the interval [1.5-2.5] m, assuming a mean value. In the same figure, the numerical results

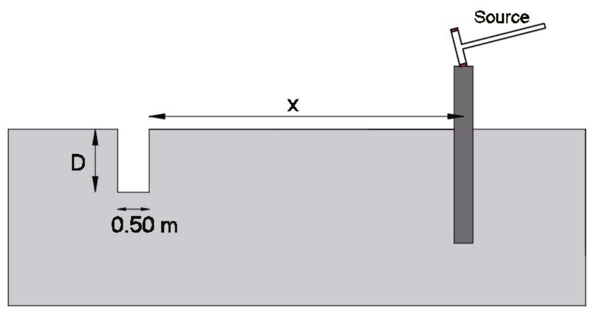


Fig. 10 Geometry of the trench solution

Table 2 Elastodynamic properties of the pile-ground system

Element	h (m)	C_s (m/s)	C_p (m/s)	ρ (kg/m^3)	ξ (%)
Soil	2	180	330	1900	5
	6	230	415		2.5
	5	330	1900		
	inf	620			
Pile	L= 12 m	2284	3560	2500	1

provided by the equivalent linear approach are also plotted for different cases: Case 0 – Reference case (without trench); Case 1 – Trench (D = 1 m; x = 5 m); Case 2 – Trench (D = 2 m; x = 5 m); Case 3 – Trench (D = 1 m; x = 10 m).

Analyzing the results depicted in Fig. 11, it is possible to point out two main conclusions: i) a very good match between the experimental data and the numerical results is provided by the equivalent linear approach. The overall behavior of the system is satisfactorily captured by the numerical model. ii) The introduction of a trench induces a significant attenuation on the PPV. This attenuation appears immediately after the trench and increases with the increase of the trench depth. However, from a practical standpoint, the maximum depth of open trenches is conditioned by the geotechnical properties to avoid collapse of its walls. Given the temporary nature of this measure, and taking into account the characteristics of each site, adopting sloped walls for the trench could be a strategy to allow the depth to be increased without compromising the trench′s performance.

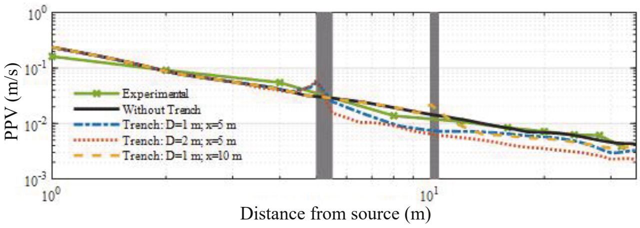
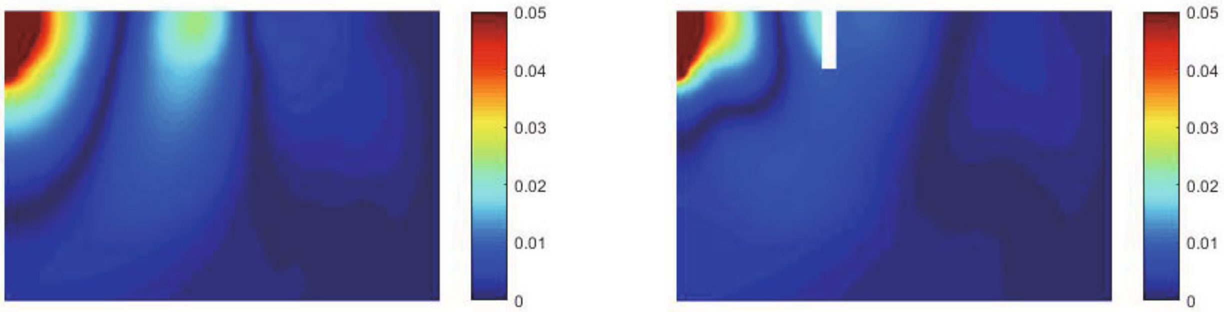


Fig. 11 Comparison between experimental and numerical results (before and after the introduction of the trench) - PPV vs distance - for a pile penetration depth of 2 m

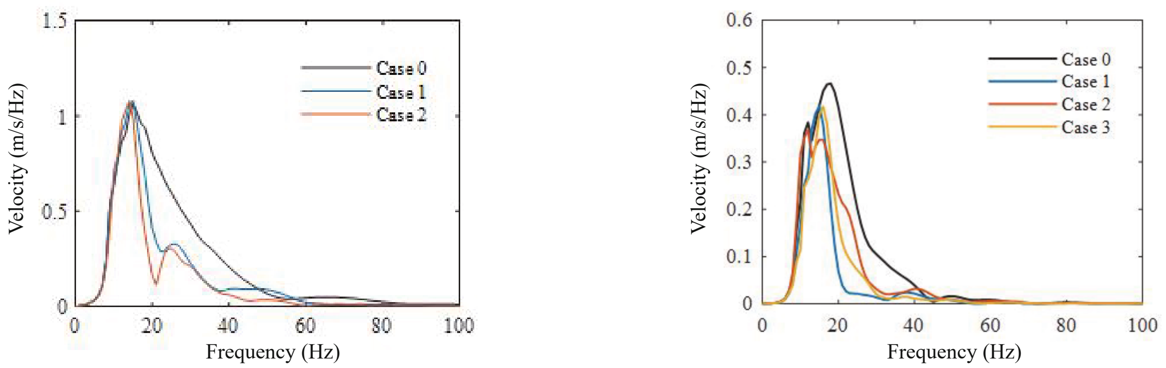
To visualize the effect of the trench on the wave′s propagation, Fig. 12 shows a comparison of the norm of the particle velocity for a given instant temporal (75 milliseconds) for Case 0 (without trench) and Case 2 (D = 2 m; x = 5 m) under analysis. As can be seen, the presence of the open trench introduces a wave barrier, with a clear perception of the interference in the waves′ propagation. Note that the amplification of the particle vibration just before the trench, as shown in Fig. 11, is motivated by the Rayleigh wave reflection on the trench wall, leading to a concentration of energy around this location, as can be seen from Fig. 12(b).

Complementary to the previous analysis, an analysis in the frequency domain is mandatory for a deeper understanding of the problem. In fact, as defined by the different standards, it is not only necessary to take into account the maximum levels of vibration, but also the most relevant frequency range of the response. Following that goal, Fig. 13 presents a comparison of the soil free surface response before and after the construction of the trench. As can be seen, with the presence of the trench there is just an attenuation of the medium/higher



(a) (b)

Fig. 12 Norm of the particle velocity at the time step t=75 ms for two different scenarios: (a) Case 0; (b) Case 2



(a) (b)

Fig. 13 Comparison of the soil surface response before and after the construction of the trench in the frequency domain for an observation point located at the ground surface and at a distance from the pile of: (a) 8 m; (b) 24 m

frequencies. This behavior can be easily explained, since low frequencies are associated with high wavelengths, which are insensitive to local discontinuities, such as a trench.

3.3 Continuous distribution of a heavy mass resting on the ground surface next the pile

A different mitigation measure with practical application corresponds to a colocation of a continuous distribution of a heavy mass resting on the ground surface next to the pile. This type of mitigation was studied and implemented in other areas involving vibrations, as the vibrations induced by railway traffic (Krylov, 2005, 2007; Dijckmans *et al.*, 2015).

Figure 14 gives a schematic view of the study configuration, where a concrete block (mass equal to 2500 kg/m^3), with the geometrical dimensions indicated in Fig. 14, is placed over the ground, 5 m away from the pile. To evaluate its efficiency on vibration screening, the calculations are performed with the axisymmetric FEM-PML approach, considering the geo-mechanical properties of the INEGI test site.

To understand the physical mechanisms responsible for the wave impeding effect, the behavior of the continuous distribution of a heavy mass resting on the ground surface can resemble that of a mass–spring system. The effectiveness of this kind of mitigation approach is related to the introduction of a new resonance frequency of the system, where the stiffness of the ground and the mass of the wall play a relevant role (Dijckmans *et al.*, 2015). Theoretically, at low frequencies, the mass follows the vertical displacement of the soil's surface, and no reduction of the dynamic response is expected. Around the mass-soil resonance frequency, an amplification of the response is expected. Above this frequency, the introduction of the mass on the system can lead to a reduction of the dynamic response due to the inertia of the mass (Dijckmans *et al.*, 2015). This theoretical behavior can be observed from the results shown in Fig. 15, considering the response of the observation point located 8 m away from the pile (at the free surface), where it is possible to identify a resonant frequency around 16.5 Hz. In a simplified way, this resonance frequency, f_0, can be estimated from $f_0 = \frac{1}{2\pi}\sqrt{\frac{K_{\text{soil}}}{m}}$, where m corresponds to the total mass of the element and K_{soil} stands for the vertical dynamic stiffness of the soil under the continuous mass. The computation of the K_{soil} was performed through the axisymmetric FEM-PML numerical approach. From the calculations performed, a value of f_0 equal to 19 Hz was found. The slight difference found can be justified by the damping, once it was considered the undamped natural frequency through this simplified methodology.

To make a general assessment of the efficiency of the mitigation measure, Fig. 16 presents a comparison of the peak particle velocity (PPV) as a function of the distance from the pile center. As can be seen, the introduction of the continuous distribution of a heavy mass resting on the ground surface next to the pile can lead to a decrease of the maximum vibration levels. However, when compared to the trench solution, the results are more modest, especially when a deeper trench is considered, as seen in Fig. 16.

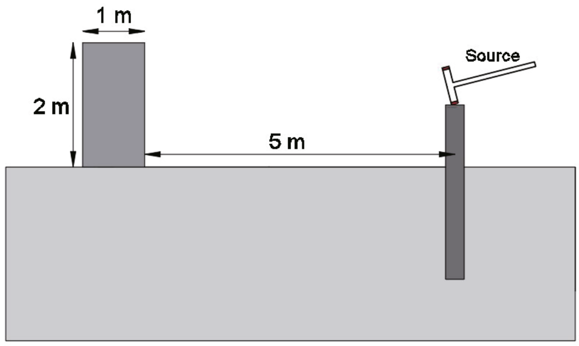


Fig. 14 Geometry of the heavy mass next to the pile

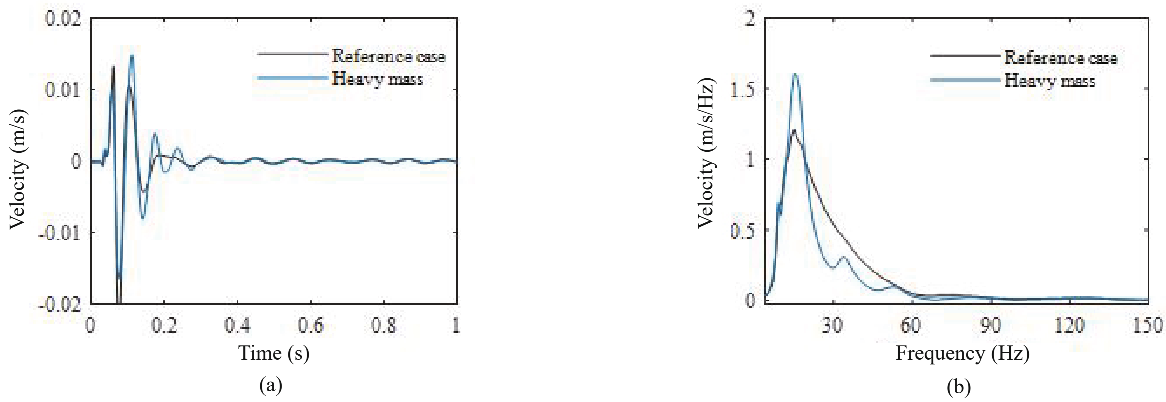


Fig. 15 Comparison of the soil response before and after the construction of the continuous distribution of a heavy mass resting on the ground surface for an observation point located at the ground surface and at a distance from the pile of 8 m: (a) time record; (b) frequency content

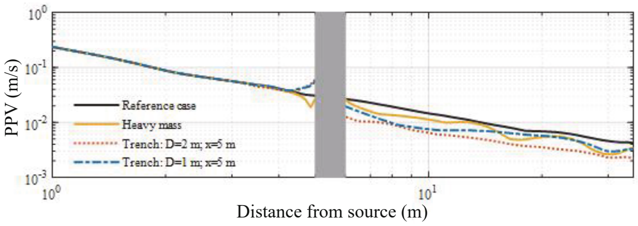


Fig. 16 Comparison of the PPV vs distance after and before the introduction of a continuous distribution of a heavy mass resting on the ground surface for a pile penetration depth of 2 m (the trench solution results are also presented)

4 Conclusions

This study summarizes the measurements performed at two different experimental test sites, and compares the vibration levels recorded against the thresholds preconized by the Portuguese national standard NP2074. Given the levels recorded, with infringement of the standard thresholds, two different mitigation measures were proposed. To analyze the effectiveness of the mitigation measures in reducing the vibrations induced by pile driving, a general parametric study was carried out.

From the results obtained, both mitigation measures, open trench and a continuous distribution of a heavy mass resting on the ground surface, allow a reduction of the vibration levels. However, the open trench is more effective and its efficiency increases with the trench depth. The technical feasibility of implementation of this solution, with temporary character, is high, and it can be a valid solution to account for the excessive vibration levels without limiting the applicability of the pile driving method.

As a final note, the sustainable evolution of construction activities is intrinsically linked with increased quality demands, decreased construction time and reduced environmental impact. The application of precast solutions in deep foundations can play a significant role in this purpose. However, the large-scale application of the technique in an urban environment can be conditioned, not by the technical impossibility of driving the piles, but mainly by the potential consequences on the surrounding environment. In this way, the consideration of mitigation measures can overcome these issues. Additionally, experimental validation of the numerical results indicates that further studies be undertaken in this area.

Acknowledgment

This work was financially supported by: Programmatic funding - UIDP/04708/2020 of the CONSTRUCT - Instituto de I&D em Estruturas e Construções - funded by national funds through the FCT/MCTES (PIDDAC); Project PTDC/ECI-CON/29634/2017 - POCI-01-0145-FEDER-029634 - funded by FEDER funds through COMPETE2020 - Programa Operacional Competitividade e Internacionalização (POCI) and by national funds (PIDDAC) through FCT/MCTES.

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