Simulating the effect of vibratory pile driving on far field of a driven pile

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ABSTRACT

The rapid increase in construction activities in past few decades has made the land a scarce resource. Thus, many new construction projects in urban environment are carried out in the vicinity of existing buildings. The construction induced vibrations can inflict significant forces on the substructures of adjacent buildings. This paper investigates the effect of vibratory pile driving on far field of the driven pile. First, the effectiveness of wave transmitting boundaries used to truncate the finite element mesh in the dynamic finite element modelling is studied. Pile installation is simulated from the ground surface and the finite element model is verified using the field data available in the literature. Then, the same model is used for a dynamic analysis with wave transmitting boundaries to study the ground vibration propagation around a driven pile.

Keywords: boundary conditions, wave propagation, pile driving, peak particle velocity

1 INTRODUCTION

The increasing demand for infrastructure due to rapid population growth has made suitable land for construction a scarce resource. Constructions are therefore carried out in vertical space, rising hundreds of meters above ground level and below ground level to accommodate other necessities such as car parks. Piles are incorporated either at construction stage as support structures due to poor soil conditions in ground, or as foundations to transfer very large design loads to ground. Depending on the construction method, piles can be placed into two general categories: driven piles and drilled piles. Driven piles are prefabricated piles of comparatively smaller diameter and are driven into the ground with a pile driver. Drilled piles have larger diameters compared with driven piles and are cast in-situ, using drilled boreholes in the ground. Three widely used techniques for driven piles are vibratory pile driving, impact pile driving and pile jacking. The present study focuses on vibratory pile driving.

Vibratory pile driving uses counter-rotating eccentric masses to generate the vibratory force necessary to drive the piles. Rodger and Littlejohn (1980) mentioned that the vibrations reduce the ground resistance even though the amplitude of the vibration is comparatively small. Woods (1997) proposed an illustration of wave propagation from driven piles in homogeneous soils. Accordingly, the shear stress waves generated from the friction between the pile and soil particles first generate from the upper contact point and propagate out in a conical shape as shown in Figure 1. Woods stated that a driven pile generates different types of stress waves: compression, shear and surface waves. Compression waves are believed to transmit from the area of pile toe expanding over a spherical wave front. Vertical shear waves are expected to generate from the friction between pile shaft and soil, which will expand around a conical surface. Due to the velocity of the vibratory pile driving, the compression wave velocity of the pile is usually ten times or more than the shear wave velocity of the soil. The shear wave front is considered cylindrical in practice. When these waves reach the ground surface, some of them will be converted to surface waves, such as Rayleigh waves, and the rest will be reflected back to the medium.

The intensity of the vibration propagation can be suitably described with respect to the peak particle velocity (PPV). Athanasopoulos and Pelekis (2000) stated that the strains induced in the ground by the propagation of vibration are proportional to the particle velocity of the medium. The particle velocity can be measured in three directions: vertical, radial and transverse. These values are used to describe the intensity of the vibration using different approaches. It can either be expressed as the peak single value of three measured components or as the peak value of true vector sum of the measured components. Peak value of the vertical component is also used to describe the intensity of the vibration. Another approach uses the square root of the sum of squares of peak values of each component. From these approaches, the true vector sum is identified as the most realistic approach to determine the intensity of ground vibrations (Athanasopoulos and Pelekis, 2000).

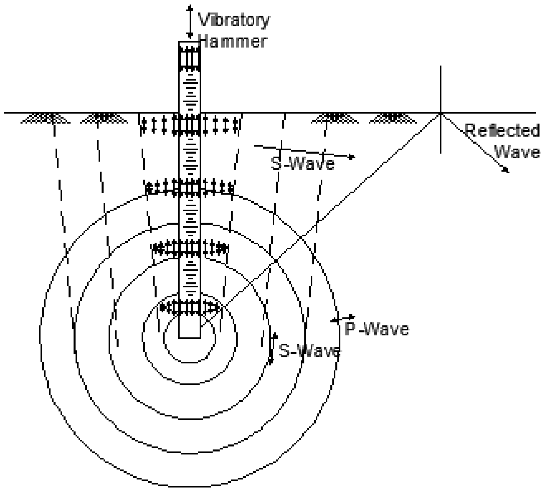

Figure 1. Wave propagation during pile driving (Woods, 1997)

2 NUMERICAL MODEL

Vibratory pile driving is simulated as an axisymmetric finite element model using the ABAQUS/Explicit finite element programme. First, the selected finite element modelling technique for pile driving was verified using field data produced by Cooke et al. (1979) for pile jacking. Then, a non-reflecting boundary condition was selected to truncate the soil domain beyond the region of interest, which is further discussed later in this section.

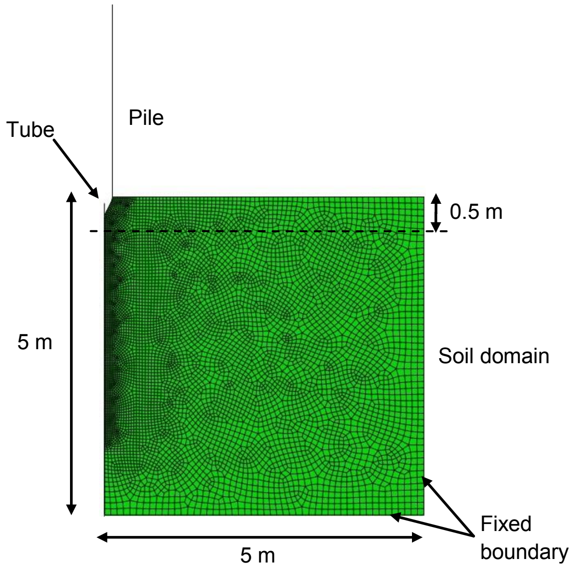

Figure 2. Axisymmetric model for pile jacking

The axisymmetric model used to simulate pile jacking is restricted to a depth of 5 m and a radius of 5 m. As shown in Figure 2, the mesh was generated having finer elements towards the driven pile and coarser elements towards the boundary. The finer elements had a minimum dimension of 0.05 m and the coarser elements had a maximum dimension of 0.2 m. Adaptive mesh technique was incorporated in the mesh region near the jacked pile to avoid element distortion during the analysis. Since this technique supports linear elements with reduced integration, Four-node quadratic elements with reduced integration were used for the soil domain. Since the adaptive mesh technique changes the mesh irrespective of the material movement, tracer particles were used to track the material movement due to pile jacking. The soil domain was modelled as a homogeneous continuum with a density of 1947 kg/m^3, a Young's modulus of 50x10^6 N/m^2 and a Poisson's ratio of 0.49. The undrained shear strength varied with the depth from 35x10^3 N/m^2 at surface to 78x10^3 N/m^2 at 4.6 m below the surface level. The boundary was fixed at the far end of the soil domain in both radial and vertical directions, when simulating the pile jacking problem.

2.1 Model verification

The technique presented by Henke and Grabe (2006) to simulate pile driving using the finite element method was adopted to develop the axisymmetric finite element model used in the present study. A tube of 1.0 mm radius, which was in frictionless contact with the soil, was modelled along the axis of symmetry. As the pile was driven to the soil, pile was allowed to slide over the tube, separating the soil from the tube, generating the pile-soil contact. Both tube and pile were modelled using the analytical rigid surface function available in ABAQUS/Explicit. The pile was in frictionless contact with the tube and has a single reference point to apply the loads and the boundary conditions. The tip of the pile was modelled as a cone and the tip was embedded in the soil at the beginning of the analysis to prevent excessive element deformation at the tip as the pile starts to penetrate the soil.

Field data published by Cooke et al. (1979) were used to verify the axisymmetric model simulated in this study. They presented vertical ground movements induced by jacked piles, conducting a series of field tests in North London, where London Clay can be found from surface to a depth of approximately 30 m. A set of laboratory tests were performed to determine the properties for the soil deposit. Three tubular steel piles of approximately 5 m in length, each having a diameter of 0.168 m, were jacked to a depth of 4.6 m. The vertical ground movements were recorded using inclinometers positioned at different depths below the surface. Vertical ground movements extracted about 0.5 m below ground level are compared with the finite element model results.

Figure 3 shows field data from Cooke et al. (1979) and finite element results from the numerical model for the pile jacking. It is apparent that after the pile is jacked 4.6 m into the ground, the finite element prediction of the ground settlement at 0.5m below the surface level is fairly in agreement with the field data. The maximum heave predicted by the numerical model is 8.5 mm whilst the maximum heave measured at the site is 10.1 mm. The difference of 1.6 mm is acceptable considering the non-uniform field conditions at the site. Thus, the proposed numerical model is expected to simulate the different types of pile driving methods with a reasonable accuracy.

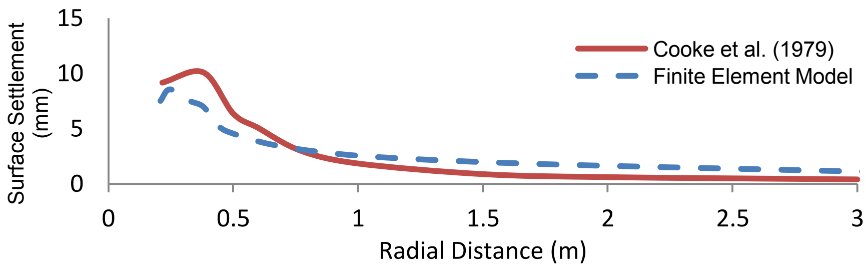

Figure 3. Ground settlement at 0.5m below the surface.

2.2 Selection of boundary condition

When analysing a wave propagation problem such as vibratory pile driving using the finite element method, it is necessary to avoid the reflection of the waves from the boundary of the finite element mesh back to the domain of interest, during the time of the analysis. This can be achieved by using a larger domain, thus the waves travel longer distances before reflect back from the boundaries. A lager domain can provide adequate time to extract the results from the region of interest but it will increase the computational cost of the analysis significantly.

Ekanayake et al. (2011) discussed about non-reflecting boundary conditions which can be used in finite element analysis. They observed that the infinite elements, which are available in ABAQUS as a non-reflecting boundary condition, are not as effective as the wave transmitting boundaries proposed by Deeks and Randolph (1994) and Du and Zhao (2010) for axisymmetric wave propagation. This was proved by analysing an axisymmetric vibratory pile driving model using the wave transmitting boundaries proposed by Deeks and Randolph (1994) for shear and dilation wave propagation and infinite elements . The finite element model used to verify the pile driving model in section 2.1 was used to compare the different boundary conditions. Instead of the jacked pile, a vibratory pile was modelled applying a sinusoidal load with an amplitude of 200 $x10^3$ N and a frequency of 30 Hz. Du and Zhao (2010) boundary condition has not been selected in this study as it involves extra nodes outside the finite element mesh, which increases the computational time compared to the Deeks and Randolph (1994) boundary.

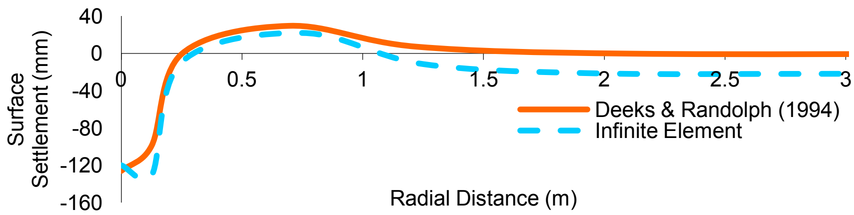

Figure 4. Surface settlement at the end of the vibratory pile driving for different types of non-reflecting boundary conditions

As illustrated in Figure 4, a clear difference between the surface deformations of two cases was observed far from the pile, which was a result of infinite elements producing a rigid body movement of the soil domain. Thus, the infinite element was considered inappropriate to represent the practical situation. Therefore, the wave transmitting boundaries proposed by Deeks and Randolph (1994) for shear and dilation wave transmission was selected to truncate the finite element mesh when analysing the vibratory pile driving problems in what follows.

2.3 Numerical modelling of vibratory pile driving

Vibratory pile driving was modelled using ABI vibratory pile drivers (ABI, 2011) MRZV_12S and MRZV_30S where each vibrator had a driving force of 600 kN and 1500 kN respectively. The frequencies of the driving forces are selected as 28 and 40 Hz as those were the minimum and the maximum operating frequencies specified for the selected vibrators. The driving forces were applied at the reference point of the analytical rigid surface representing the pile as a sinusoidal force. The soil domain for axisymmetric finite element model was truncated at 10 m in vertical and radial directions using the wave transmitting boundary conditions for shear and dilation waves as discussed in section 2.2. The soil properties used in this study were extracted from Hwang et al. (2001). The soil domain consists of a homogeneous layer of clay with a density of 1947 kg/m^3, a Young's modulus of $20x10^6$ N/m^2, a Poisson's ratio of 0.4 and an undrained shear strength of $25x10^3$ N/m^2.

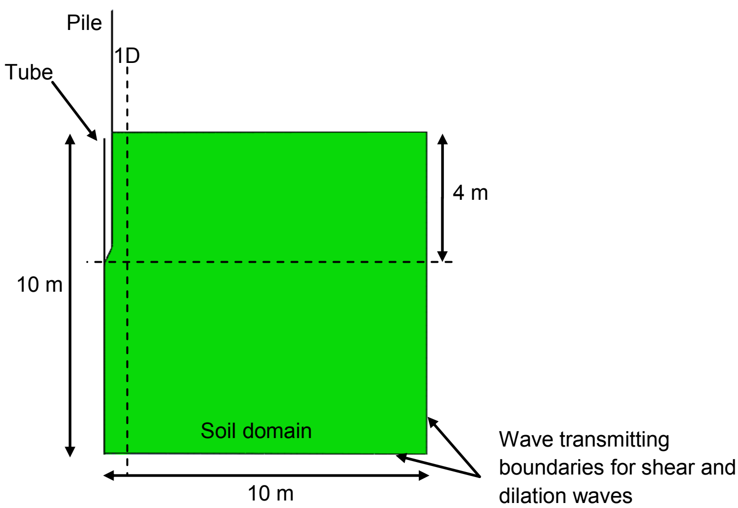

Figure 5. Axisymmetric model for vibratory pile driving.

Figure 5 illustrates the axisymmetric finite element model used for vibratory pile driving. The pile has a diameter (D) of 0.5 m. It was modelled as an analytical rigid surface. The contacts of the rigid tube with pile and the soil were frictionless while the friction coefficient between the pile and the soil was assumed to be 0.2. The pile was laterally restrained while the rigid tube of 1 mm radius was fixed in all directions. The pile penetration level was considered as 3.5 m below the surface for all the cases. The model was created having the driven pile about 3.5 m below the surface level to save the computational time required for the pile to be driven 3.5 m below the ground surface. Since the pile tip has a conical shape, the tip lies about 4 m below the surface level during the analyses. The duration of the analyses was 0.5 seconds for each case. The penetration of the driven pile during the 0.5 seconds is insignificant compared to the initial depth of penetration. The particle velocities were extracted at locations 1D away from the driven pile and also at locations 2, 3, 4, 5 and 6 m below the ground surface. Figure 6 compares the PPVs of different vibratory pile driving configurations. The PPVs are calculated using the true vector sum of the vertical and radial velocities and are plotted against r/D where r is the radial distance from the centre of the pile.

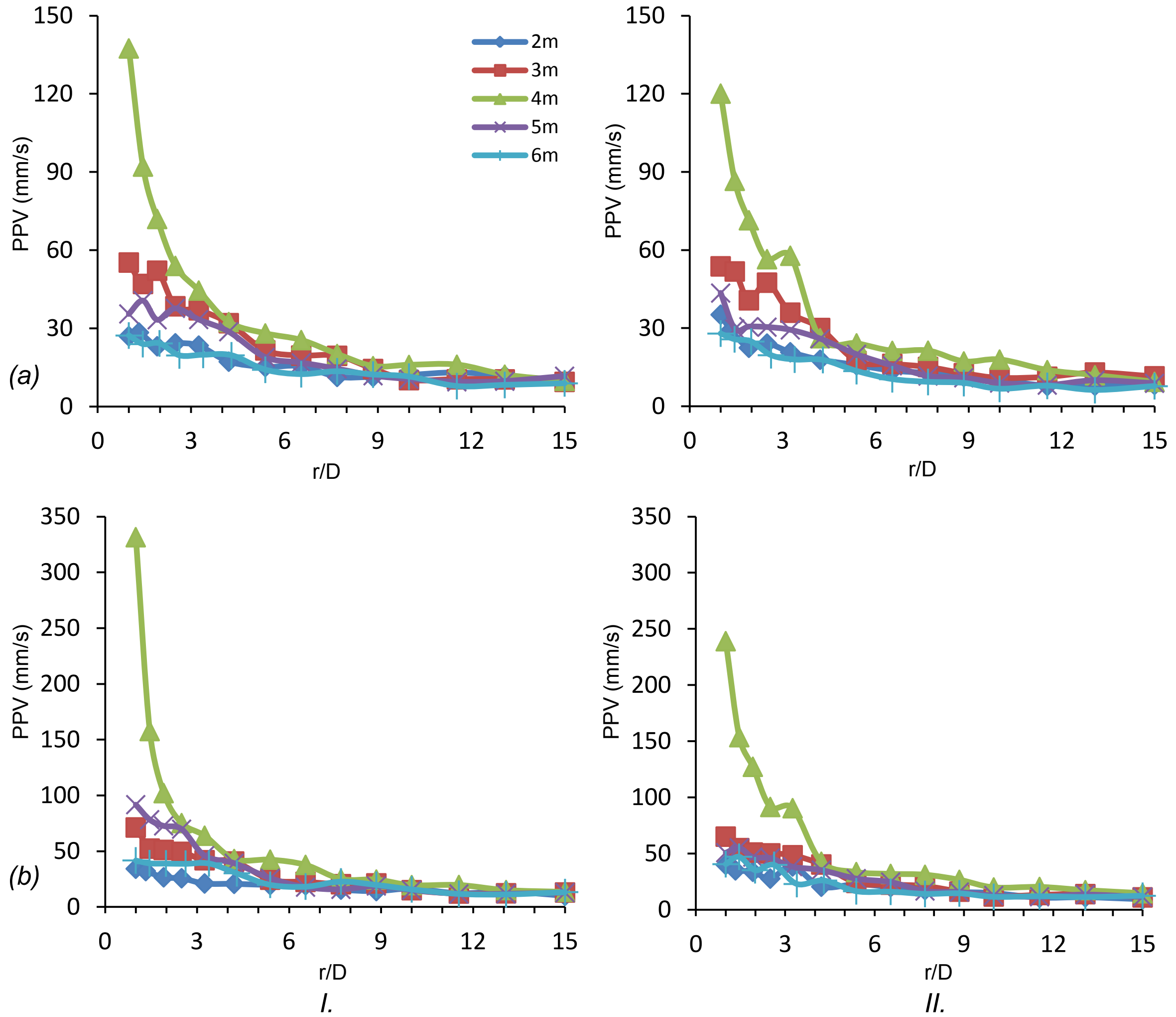

Figure 6. Peak particle velocity vs. r/D curves at different depths and locations for driven forces (a) 600 kN (b) 1500 kN and for driving frequencies I. 28 Hz II. 40 Hz

3 DISCUSSION

It can be clearly seen that the peak values of particle velocities decrease significantly with the increasing distance from the tip of the driven pile. The PPVs at different depths have significant differences around the pile, closest to the pile tip being the highest. As the radial distance increases, the PPVs decrease and have values around 12 mm/s about 15D away from the pile. According to the National Roads Authority (2004), for frequencies ranging from 10-50 Hz, maximum allowable PPV near any sensitive property is 12.5 mm/s. Thus, the influence zone can be recognised to extend about 15D in the radial direction. Along the depth, at one D away from the pile, the values decrease with the increasing vertical distance from the pile tip. The results indicate that the influence zone is around +/- 4D from the pile tip in the vertical direction. Though, the influence is significant throughout the depth as the PPV is always above 20 mm/s. It is also observed that for a given frequency, increasing the amplitude of the driven force increases the PPV. Simultaneously, increase in frequency decreases the PPV. Significant decrease of peak values of the PPV curves can be identified as the frequency increases from 28 Hz to 40 Hz and this is common for all the amplitudes of the driving force. Increasing both amplitude and frequency of the driven force will also increase the PPV. Nevertheless, the increase in PPV in this case is smaller than the PPV increase when only the amplitude of the driven force is increased.

4 CONCLUSION

The effect of vibrartory pile driving around a driven pile is investigated. A finite element model, which has the ability to drive a pile from the ground surface, is first verified with field data available for pile jacking and the same model is used to simulate vibratory pile driving with wave transmitting boundaries for shear and dilation waves at the boundary of the mesh using the dynamic finite element analysis program ABAQUS/Explicit. Vibratory forces are applied to the pile with varying amplitude and frequency. Based on the results, it can be concluded that the influence zone of a vibratory driven pile extends about 15D in the radial direction and +/- 4D in the vertical direction, away from the pile tip. An increase in the amplitude of the driven force increases the size of the influence zone and the opposite occurs when the frequency of the driven force is increased.

5 ACKNOWLEDGEMENT

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