Ground Movements and Vibrations Caused by Impact Pile Driving of Prestressed Concrete Piles in Central Florida

Jorge Enrique Orozco Herrera

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GROUND MOVEMENTS AND VIBRATIONS CAUSED BY IMPACT PILE DRIVING OF PRESTRESSED CONCRETE PILES IN CENTRAL FLORIDA

by

JORGE ENRIQUE OROZCO HERRERA
B.S. National University of Colombia, 2019

A thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in the Department of Civil, Environmental and Construction Engineering in the College of Engineering and Computer Sciences at the University of Central Florida Orlando, Florida

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Major Professor: Luis G. Arboleda-Monsalve
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ABSTRACT

The process of pile driving has been a commonly used method for the installation of deep foundations in Central Florida due to the soil conditions that consist mainly of medium-dense silty sands. Pile driving can generate large vibration levels that might potentially trigger ground deformations in the surrounding soils and cause damage on nearby structures. Currently, design and construction standards provide guidance in terms of ground vibration levels expected during pile driving and establish vibration thresholds to avoid damage on important infrastructure. However, little insight has been given into the amount of ground deformations that soils experience due to pile driving induced vibrations. This phenomenon becomes important when repetitive and cumulative loading cycles are applied in sandy soils.

The main goal of this thesis is to investigate numerical modeling alternatives capable of predicting ground deformations caused by pile driving performed in Central Florida soils. Field data obtained from different construction sites in Central Florida are used to understand the expected ground deformations and their relationships with ground vibration levels. Common construction practices in the area are also analyzed from the reported field data. Two numerical modeling approaches previously used in the literature are compared with data measured in the field to determine the most suitable alternative to numerically analyze and predict ground deformations. Subsequently, a numerical study of the effects of the different variables involved in this problem on expected ground vibrations and deformations is presented. These variables include the type of pile and its dimensions, the driving hammer and its transmitted energy to the pile, and the dynamic properties of the soils in terms of attenuation characteristics and densification potential.

It is concluded that in cases where vibration levels comply with the thresholds defined by the Florida Department of Transportation (FDOT) large ground deformations can still occur.
depending on the above-mentioned site-specific variables. In terms of numerical modeling alternatives, a continuous modeling approach offered a better estimation of the stress field generated by pile driving than a discontinuous approach. This allows for better determination of the strains within the soil continuum leading to better ground deformation predictions.
To my grandparents Luis Carlos and Marta for being my role models. My deepest wish is that you are watching over me from heaven, and that you are proud of everything I do.
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1. INTRODUCTION

1.1. Motivation

Driven piles are a suitable deep foundation alternative in geotechnical engineering designs to transfer structural demands to competent strata and to avoid serviceability issues associated with shallow compressible soil layers. This foundation type is commonly used in Central Florida due to the soil conditions in the area that consist mainly of sandy soil deposits allowing a relatively fast pile installation. However, the pile driving process generates ground vibrations and ground deformations which can affect nearby structures. The local geotechnical practice and design standards focus mainly on the ground vibrations (i.e., velocities and accelerations in the soil) induced by pile driving, limiting such vibrations to a fixed threshold. The Florida Department of Transportation (FDOT) establishes an acceptable vibration threshold in terms of peak particle velocity (PPV) of 0.5 in/s for road and bridge construction projects (FDOT, 2021). These vibration criteria are not linked to the amount of settlement/heave the soils will experience due to pile driving induced vibrations.

Pile driving can cause damages to nearby structures by other components not directly linked to the source of ground vibrations. Massarsch and Fellenius (2014) defined four types of damage categories due to pile driving. Figure 1 presents a graphical description of the four categories. Damage Category (I) consists of static movements caused by differential settlements and the heave commonly seen after installation of large displacement piles in cohesive soils. Damage Category (II) is linked to ground distortions generated by the propagation of the waves along the ground surface that generates cycles of hogging and sagging movements in the structures. This damage category is related to the wavelength and the number of cycles of the propagating...
waves. Damage Category (III) is related to the ground deformations caused by dynamic effects in the soil due to ground vibrations and cyclic effects, which is problematic in loose granular materials. Finally, Damage Category (IV) is the only damage type directly associated with vibrations in the structure and their dynamic effects. The last category is the only one considered in construction vibration standards which exemplifies the need for a better understanding of wave propagation and its effects on the ground surface and structures.

![Damage mechanisms due to pile driving](image.png)

Figure 1. Damage mechanisms due to pile driving (after Massarsch and Fellenius, 2014).

According to Dowding (1996) pile-driving induced vibrations can cause ground deformations by different mechanisms such as i) excess pore water pressure dissipation that can cause settlement; ii) particle rearrangement, which can also cause settlement in loose and loose to medium-dense sands and heave in dense sands; iii) resedimentation from localized liquefaction
around the pile, and iv) downdrag effects of nearby deep foundations and distortions of structures supported in deep foundations as a product of consolidation. Drabkin et al. (1996) found that ground deformations in sands do not solely depend on the vibration amplitude experienced during pile driving; more variables have to be considered for the determination of the expected deformations. These variables include: the vibration amplitude and the number of vibration cycles, deviatoric stresses generated during the driving process, in-situ confining pressures, the grain size distribution of the soil, saturation of the soil, and the initial relative density of the sand. All these variables plus the pile and hammer properties make pile driving a complex dynamic process that cannot be simplified by considering only the peak vibration amplitude as the main design criterion.

The purpose of this thesis is to develop numerical models and field tests and propose semi-empirical methods to predict the amount of dynamic ground movements arising from pile-driving operations in Central Florida soils by analyzing most of the variables involved in this process. Field data from pile driving projects across Central Florida are presented in this thesis to elucidate wave propagation characteristics in the area and collect important information regarding the state of the practice in the state. The field data are then used to validate numerical models performed in the finite element (FE) software PLAXIS 2D that are used to parametrically extrapolate the results to other cases and configurations of soils, piles, and driving accessories (i.e., hammer type, cushions properties, etc.). Pile driving numerical modeling approaches are compared by using the collected field data to determine the most suitable modeling approach to use for further analyses. Subsequently, a parametric study is performed to elucidate the effects of soil properties and hammer characteristics into the final response in terms of ground deformations. The results of this parametric study are presented in this thesis alongside recommendations to determine ground deformations due to pile driving based on site-specific characteristics.
1.2. Objectives

The main objective of this thesis is to present models that can be used to predict ground deformations due to pile driving in Central Florida soils. In order to accomplish this goal, the specific objectives of this thesis include: i) performing a comprehensive literature review and developing a reported case histories database that can be used to understand the dynamic effects of pile driving in surrounding soils; ii) collecting field data from pile driving projects in Central Florida to validate modeling hypotheses and elucidate site-specific wave propagation characteristics; iii) selecting the most appropriate pile driving modeling approach by comparing different alternatives presented in the technical literature; iv) modeling pile driving process by using advanced constitutive soil models capable of simulating hysteretic (i.e., cyclic or dynamic) behavior of the soils including a realistic groundwater model to elucidate pile-driving excess pore water pressure; and v) performing an analysis of the variables involved in the wave propagation from the driving hammer, passing through the pile and the soil continuum until reaching the ground surface.

1.3. Organization of the Thesis

The thesis is organized as follows in order to present in a reasonable manner the work done during this research:

Chapter 2 includes the literature review for this thesis. Vibration criteria from different standards and design codes are summarized to establish vibration thresholds used by practitioners. Different ground vibration attenuation formulas are presented to be used in the following chapters. A summary of different estimation methods developed by different authors is presented since the
main goal of this thesis is to evaluate ground deformations due to pile driving. A compilation of different case histories where either ground deformations or ground vibrations were measured is also presented in this chapter. Different numerical modeling approaches to the pile driving process are also presented. Basics of the wave equation analysis are presented in this chapter since this method is used in the following chapters.

Chapter 3 presents the field data collected from three different bridge construction sites in the Central Florida area. These field data consisted of soil profiles, laboratory and in-situ testing performed to determine soil properties, dynamic pile tests performed at the site to determine pile capacities and forcing functions applied to the top of the pile, and ground vibrations and deformations measurements during the pile driving process.

Chapter 4 presents the different numerical models performed to analyze the ground deformations caused by pile driving activities. A comparative analysis of pile driving numerical approaches and an analysis of the main variables involved in the final ground response are presented in this chapter.

Finally, Chapter 5 presents the summary and conclusions of this research.
2. LITERATURE REVIEW

2.1. Vibration Criteria

Ground vibrations induced by human activities can vary greatly in intensity depending on the type of source. Generally, man-made vibrations have a much lower vibration intensity compared to earthquakes, thus in most cases, they cannot cause serious structural damage and their effects are normally related to cosmetic cracks in the structure (Athanasopoulos and Pelekis, 2000). The waves travel through the soil and potentially can interact with above-ground or buried structures, thus disturbing the people occupying that structure or in some cases even threatening its serviceability and integrity. As mentioned in Section 1.1, according to Massarsch and Fellenius (2014) a large number of standards and design codes focus on limiting the man-made vibration levels to a certain threshold to minimize their impact on humans and structures.

Standards and design codes define threshold values for man-made vibrations depending on the type of receiver (i.e., people, structures, or sensitive equipment), thus there is not a unique vibration threshold value (Gkrizi, 2017). The most commonly used term to measure ground vibrations is the Peak Particle Velocity (PPV), which is the maximum velocity value measured during the vibrations time history at a certain point. Generally, the allowable ranges for PPV vary from 3.0 to 70.0 mm/s, with the lower values given for old residential buildings and the higher values pertaining to modern large-size commercial or industrial buildings (Athanasopoulos and Pelekis, 2000). Figure 2 presents a comparison of PPV thresholds between four frequently used codes such as the U.S Office of Surface Mining (OSM), the German Institute of Standards (DIN), the British Standards Institution (BSI), and the Swiss Association of Highway Engineers (SN). Notice that the threshold values depend not only on the PPV values and the type of buildings but
on the frequency of the vibration source. Athanasopoulos and Pelekis (2000) explained that in the case of pile driving, effects of induced vibrations are normally limited within one pile length distance from the point of installation of the pile.

Figure 2. Comparison of various threshold vibration criteria to cause structural damage (after Athanasopoulos and Pelekis, 2000).

Whiffin and Leonard (1971) presented the effects of man-made vibrations depending on the PPV value measured at the ground surface. These PPV values are shown in Table 1. For bridge foundation constructions, FDOT requires that when detecting settlement or heave of 0.005 ft (1.5 mm) or vibration levels reaching 0.5 in/s (12.5 mm/s), the source of vibration must be immediately stopped (FDOT, 2021). This research defines that PPV threshold as the reference value for further analyses.
Table 1. Reaction of people and damage to buildings from ground vibrations (after Whiffin and Leonard, 1971).

<table>
<thead>
<tr>
<th>PPV, mm/s</th>
<th>Human Reaction</th>
<th>Effects on Buildings</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.15-0.5</td>
<td>Threshold of perception; possibility of intrusion</td>
<td>Unlikely to cause damage of any type</td>
</tr>
<tr>
<td>2.0</td>
<td>Readily perceptible</td>
<td>Virtually no risk of “architectural” damage</td>
</tr>
<tr>
<td>2.5</td>
<td>Threshold of annoyance</td>
<td>Recommended upper level for “ruins and ancient monuments”</td>
</tr>
<tr>
<td>5</td>
<td>Annoying to people in buildings</td>
<td>Threshold risk of “architectural” damage to normal buildings</td>
</tr>
<tr>
<td>10-15</td>
<td>Considered unpleasant</td>
<td>Causes “architectural” damage and possible minor structural damage</td>
</tr>
</tbody>
</table>

Since it is not feasible for most construction projects to monitor the expected levels of vibrations, several methods have been developed to predict vibration levels. Hendricks (2002) proposed Equation (1) to predict vibration levels based on the distance from the pile ($D$):

$$PPV = PPV_0 \left( \frac{D_0}{D} \right)^k$$

where $PPV$ is the peak particle velocity at a distance $D$ from the pile, $PPV_0$ is the peak particle velocity at a reference distance $D_0$, and $k$ is a soil attenuation parameter that must be determined experimentally for site-specific conditions. Alternatively, Bornitz (1931) proposed Equation (2) to account for both soil and geometric damping for the attenuation of the PPV induced by pile driving:

$$PPV = PPV_0 \left( \frac{D_0}{D} \right)^n e^{-\alpha(D-D_0)}$$

where $n$ is the geometric damping coefficient and $\alpha$ is the material damping coefficient. The value of the coefficient $n$ depends on the type of waves generated from the source of vibrations. Notice that the PPV attenuation depends only on the distance from the pile based on Equations (1) and
However, it has been found that there is a better correlation between predicted and measured data when the distance from the pile is normalized by the energy of the hammer. Wiss (1981) introduced the concept of the scaled distance to account for this normalization:

\[ PPV = k \left( \frac{D}{\sqrt{W_r}} \right)^{-n} \]  

where \( D \) is the distance from the pile, \( W_r \) is the energy of the source, \( k \) is the value of the PPV at a unit value of scaled distance \( (D/\sqrt{W_r}) \) and \( n \) is the soil attenuation factor. For sites where there is no information about wave propagation, it can be assumed that the coefficient \( n \) lies between 1.0 and 2.0.

2.2. Vibration-Induced Settlements Estimation Methods

The problem of pile driving-induced settlements is a complex dynamic process that is affected by many variables such as the soil dynamic properties, the type of pile and its dimensions, type of hammer and transferred energy to the pile, and the vibration amplitude and cycles, among others. This section presents a summary of settlement risk assessment methods associated with pile-driving. The methods used to develop settlement estimation approaches ranged from empirical methods to laboratory-based methods and/or semi-empirical methods.

Massarsch (2004) presented a simplified method based on experience from several soil compaction projects. Figure 3 shows a sketch of the geometrical considerations for this method to assess settlements on homogeneous sand deposits. The author assumed that the vibration-induced sand densification process occurs within a zone of three times the diameter of the pile (i.e., 3D). The settlement profile caused by the volume reduction consists of an inverted 2V:1H triangular region around the pile with the tip of the cone at a depth of six times the diameter of the pile (i.e.,
6D). Therefore, the affected area will extend up to a distance of 3D+L/2 from the center of the pile with a maximum settlement \( S_{\text{max}} \) at the center of the pile. The maximum settlement \( S_{\text{max}} \) and the average settlement \( S_{\text{avg}} \) within the influence zone can be estimated as:

\[
S_{\text{max}} = \alpha (L + 6D) \tag{4}
\]

\[
S_{\text{avg}} = \frac{\alpha (L + 6D)}{3} \tag{5}
\]

where, \( L \) is the effective length of the pile (i.e., length in the compressible layer), \( D \) is the diameter of the pile, and \( \alpha \) is a compression factor introduced by Massarsch (2004) that can be estimated from Table 2. The driving energy depends on the pile installation method and the pile type. The displaced volume of the installed pile was neglected in this method, thus its effects on the final settlement were not considered. Settlements can occur outside the influence zone, but they are often negligible.

<table>
<thead>
<tr>
<th>Driving Energy</th>
<th>Low</th>
<th>Average</th>
<th>High</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil Density</td>
<td>Compression factor, ( \alpha )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Very Loose</td>
<td>0.02</td>
<td>0.03</td>
<td>0.04</td>
</tr>
<tr>
<td>Loose</td>
<td>0.01</td>
<td>0.02</td>
<td>0.03</td>
</tr>
<tr>
<td>Medium</td>
<td>0.005</td>
<td>0.01</td>
<td>0.02</td>
</tr>
<tr>
<td>Dense</td>
<td>0.00</td>
<td>0.005</td>
<td>0.01</td>
</tr>
<tr>
<td>Very dense</td>
<td>0.00</td>
<td>0.00</td>
<td>0.005</td>
</tr>
</tbody>
</table>
Mohamad and Dobry (1987) applied a shear strain-informed approach for the prediction of liquefaction potential due to earthquakes (Dobry et al., 1982) to also assess the susceptibility of permanent ground deformations due to man-made vibrations (e.g., pile driving-induced vibrations). This method uses the shear wave velocity of the soil to determine the maximum cyclic shear strains ($\gamma_{\text{max}}$) induced by the vibrating source and then compares them with a threshold shear strain ($\gamma_t$). This threshold shear strain is defined as a value of cyclic shear strain where shear strains less than $\gamma_t$ will not cause any densification on unsaturated sandy soils or pore water pressure build-up in saturated soils. Dobry et al. (1982) reported that for most sands the value of $\gamma_t$ can be assumed to be 0.01%. Mohamad and Dobry (1987) stated that a large amount of the energy transmitted through the soil from the pile is carried by cylindrical Rayleigh waves. However, the method
assumed that the cylindrical Rayleigh waves can be approximated to plane Rayleigh waves. Thus, the maximum shear strain can be expressed as:

$$\gamma_{max} = m_z \frac{PPV}{V_s \left( \frac{G}{G_{max}} \right)^{1/2}}$$  (6)

where PPV is the peak particle velocity, $V_s$ is the shear wave velocity at small strains, $(G/G_{max})$ is the effective modulus reduction factor at a shear strain equal to $\gamma_{max}$, and $m_z$ is the relevant maximum shear strain factor obtained from Figure 4.

Figure 4 presents the variation of the shear strain factor ($m_z$) with the dimensionless depth $z/L$. In this case, L is the wavelength and $\nu$ is the Poisson’s ratio of the soil. Massarsch (2000) suggested that as a first approximation the shear strain factor for a homogeneous soil layer can be assumed to be $m_z = 0.5$. It is noticeable from Equation (6) that an iterative procedure is needed to find the $(G/G_{max})$ corresponding to the specific $\gamma_{max}$. Then, a value of $\gamma_{max}$ is assumed and $(G/G_{max})$ is calculated from shear modulus degradation curves available in the literature (e.g., Hardin and Drnevich, 1972; Seed and Idriss, 1970). The next value of $\gamma_{max}$ can be calculated from Equation (6) at a certain distance (i.e., implicitly considered with the PPV) and compared with the assumed value. When both values are close the iterative procedure ends.
As mentioned before, this method compares the maximum shear strain ($\gamma_{\text{max}}$) against the threshold of 0.01%. For that reason, Equation (6) can be rearranged into Equation (7) by substituting $\gamma_{\text{max}}$ for $\gamma_t$ to calculate the peak particle velocity at which the strain threshold value is reached ($\text{PPV}_t$). The value of $\text{PPV}_t$ provides the susceptibility of the soils to permanent ground deformations when combined with the \textit{in-situ} PPV attenuation curve. A case history presented by Clough and Chameau (1980) was also used by the authors to validate the proposed method. Mohamad and Dobry (1987) computed the expected $\text{PPV}_t$ by using Equation (7) obtaining a value of 16.8 mm/s. Based on vibration measurements performed at the site, it was concluded that the distance at which the $\text{PPV}_t$ occurred was approximately 3.4 m, which matched well with the settlement measurements performed at the site.

$$
\text{PPV}_t = \frac{\gamma_t V_s \left( \frac{G}{\bar{G}_{\text{max}}^t} \right)^{1/2}}{m_z}
$$

(7)
Drabkin et al. (1996) developed a mathematical model based on laboratory testing to predict the settlement of sandy soils caused by low-level construction vibrations. This mathematical model was developed by using the multifactorial experimental design method by considering factors such as vibration amplitude (i.e., peak particle velocity, PPV), amount of vibration cycles, deviatoric stress, confining pressure, grain size distribution, relative density ($D_r$), and moisture content ($w$). The testing ranges used for each variable are shown in Table 3. It is important to note that according to the authors, pile driving operations can apply up to 500,000 vibration cycles when a pile group is driven. This is a key variable when the long-term impact of pile driving is analyzed. The laboratory testing program consisted of a vibratory frame designed to shake a 150 mm-tall soil specimen inside a triaxial apparatus. The tests conducted for the development of the mathematical model were performed under drained conditions.

<table>
<thead>
<tr>
<th>Factor</th>
<th>Factor Code</th>
<th>Tested Ranges</th>
<th>Coding of Factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Particle Velocity (PPV)</td>
<td>$x_1$</td>
<td>2.5-18 mm/s</td>
<td>$x_1 = -1 + \frac{PPV - 0.1}{0.3}$</td>
</tr>
<tr>
<td>Deviatoric Stress (s)</td>
<td>$x_2$</td>
<td>14-104 kPa</td>
<td>$x_2 = -1 + \frac{s - 2}{6.5}$</td>
</tr>
<tr>
<td>Confining Pressure (p)</td>
<td>$x_3$</td>
<td>69-207 kPa</td>
<td>$x_3 = -1 + \frac{p - 10}{10}$</td>
</tr>
<tr>
<td>Sand Mixture</td>
<td>$x_4$</td>
<td>Coarse, Medium, or Fine</td>
<td>$x_4$ ranges from -1 for coarse sand to 1 for fine sand</td>
</tr>
<tr>
<td>Number of vibration cycles (N)</td>
<td>$x_5$</td>
<td>60-500,000 cycles</td>
<td>$x_5 = -1 + \frac{N - 60}{26,997}$</td>
</tr>
</tbody>
</table>
For the prediction of the settlement, the previously mentioned variables must be first converted to their coded values shown in Table 3. The following equation can be used by substituting such values:

\[
\ln Y = 2.27 + 1.19x_1 - 0.71x_1^2 + 0.49x_2 - 0.68x_2^2 - 0.80x_3 + 1.09x_3^2 - 0.46x_4
\]

\[
+ 0.06x_4^2 + 0.45x_5 - 0.38x_5^2 - 0.19x_6 - 0.10x_7
\]

where \( Y \) is the settlement expressed in 0.0254 mm (0.001 in), and \( x_i \) are the variables expressed in terms of their coded values. It is important to note that if any variable exceeds the ranges specified in Table 3, it should be coded as 1. Since the laboratory specimen used by the authors was 150 mm tall, the *in-situ* settlement (\( \Delta \)) for a vulnerable layer thickness \( H \) can be directly extrapolated by Equation (9):

\[
\Delta = \frac{Y}{150}H
\]  

where input values are in millimeters. Additionally, at construction sites where the *in-situ* soil conditions are highly non-homogeneous, the authors proposed dividing the vulnerable layers into 10 equal thickness layers.
2.3. **Case Histories**

2.3.1. **Reported Settlements**

Drabkin et al. (1996) presented five projects where vibration-induced settlements were measured to validate the settlement-prediction polynomial model presented in Section 2.2. Two of these projects were located in New York City, and the rest of them were in Boston, Wantagh (NY), and Northern Spain.

The first project was located in the middle of existing buildings at the Back Bay section in Boston. A total of 180 precast 360 mm-width square concrete piles were driven by using an ICE 640 diesel hammer with rated energy of 54 kN-m. The piles were driven to depths ranging from 29 to 39 m. Figure 5 presents the site-specific soil conditions, the measured PPV, and settlements. The measuring plan consisted of vibration measurements at two adjacent buildings and settlement measurements at different site locations on the ground surface and the top of the sandy layer. Notice that the peak particle velocity ranged from 6.4 to 15.0 mm/s, and the corresponding measured settlements ranged from 18 to 54 mm, demonstrating that even values of PPV less than the threshold of 0.5 in/s (12.5 mm/s) can generate significant settlements. The observed settlements occurred only during pile driving but did not continue once driving ended. Notice also that the polynomial model, extrapolated by using one layer and discretizing the sand stratum into 10 layers, matched accurately the measured settlements.
Figure 5. Project soil conditions and vibration-induced settlement at Back Bay Section in Boston (after Drabkin et al., 1996).

The second project was located in Southern Brooklyn in New York City. This case history mostly consisted of pile driving-induced settlements on aeration tanks supported on timber piles. Figure 6 summarizes the results from this case history. More than 100 close-ended 273 mm pipe piles were driven close to the tanks more than 40 m deep through a medium dense, fine to coarse sand, by using a Vulcan 08 impact hammer. A maximum settlement of 70 mm was measured after the installation of the piles. The vibration levels exceeded the 12.5 mm/s threshold which leads to the conclusion that ground vibrations have a significant effect on the final ground surface settlements.
Figure 6. Project soil conditions and vibration-induced settlement at Southern Brooklyn Site in New York City (after Drabkin et al., 1996).

The third case history was first discussed by Picornell and del Monte (1985) and consisted of pile driving-induced settlements of a pier foundation in Lesaka, Northern Spain. H-piles were driven up to bedrock adjacent to cast-in-place concrete piers of 1.08 m diameter embedded to a depth of approximately 20 m. One of the pier foundations settled 250 mm as a result of the driving process (see Figure 7). Several static load tests were conducted at the site to evaluate the causes of the measured settlement. The static settlement was less than 9 mm indicating that the cause of settlement could have been the dynamic compaction induced by pile driving on the sandy layer.
Drabkin et al. (1996) also discussed the vibration-induced settlements at the Tri-Beca tower site in Manhattan. The tower was a 52-story residential building constructed near a historic building and a two-story building. The foundation of the tower consisted of 178 mm-diameter open-ended pipe piles with a length of 30 m, respectively. Figure 8 presents the site conditions, which consisted of medium sand that was expected to densify due to vibrations. The measured settlements and vibrations at the 2-story building are also shown. Vibrations ranged from 2.5 to 18 mm/s while the settlement ranged from 38 to 69 mm at different stages of construction.
Chen et al. (1997) presented the results of a full-scale free-field pile driving test performed at the Chang-Hua Coastal Industrial Park near the Taichung Harbor in central Taiwan. The soil conditions at the site consisted of a 4.0 m-thick man-made loose gravely and sandy fill underlain mainly by sandy soils interbedded with some silty sand layers. The groundwater table fluctuated at the site between 2.5 m and 5.0 m below the ground surface. Five (5) 800 mm precast concrete piles were driven up to a depth of 24.0 m by using a KOBELO 80 Diesel hammer. Figure 9 presents the ground surface settlements induced by the driving of the first three piles (i.e., P1, P2, and P3). These settlements were measured along an axis parallel to the line of the pile (X-axis) and perpendicular to the piles (Y-axis). Most of the settlement occurred due to driving of pile P3 which was in this case the pile closer to the settlement points. Furthermore, the settlement was still considerable (i.e., approximately 2 cm) at distances up to 7.5 m parallel to the piles and 3.0 m perpendicular to the line of the piles. Chen et al. (1997) also presented results in terms of pore water pressure build-up due to pile driving measured by three piezometers installed at different
depths. The authors concluded that most of the excess pore water pressure was generated when the tip of the pile was above the piezometers, which indicated that the effects of the spherical waves emanating from the tip of the pile are the main triggering factor compared with the conical wavefront emanating from the shaft.

Lewis and Davie (1993) presented a case history where structural response due to pile driving was measured at a U.S government facility. The project was located in the coastal plain of the eastern United States. In this case history, 355 mm precast prestressed concrete piles were installed by using an ICE 640 close-ended diesel hammer with a rated energy of 54.2 kN-m. Based on cone penetration tests (CPT) performed at the site, the soil conditions consisted of alternating layers of loose to very dense fine sand and silty fine sand. Most of the sand had a relative density of approximately 50%. Figure 10 presents the ground movements measured at different distances from each pile. Ground displacements ranged from +0.5 in. (12.7 mm) of heave to -3 in. (76 mm) of settlement. An interesting conclusion from the authors is that no movement was experienced at distances beyond the length of the piles, supporting earlier results presented by Dowding (1991).
Linehan et al. (1992) presented the effects of pile driving on a 1.2 m deep pressurized natural gas pipeline located near the construction of a railroad bridge foundation. Both vibratory sheet pile driving and impact H-pile driving were performed at the site. The 6.0 m long PZ40 sheet piles were driven up to a depth of 4.5 m by using a vibratory hammer with rated energy of 0.451 kN-m. The 18.0 m long HP 14X73 H-piles were driven by a diesel impact hammer with rated energy ranging from 313.2 to 40.7 kN-m. The site consisted of a surficial layer of soft organic soils underlain by very dense sandy and gravelly soils. Figure 11 presents the evolution of vertical displacements at the pipeline during the driving of both the sheet piles and H-piles. The settlement caused by the vibratory sheet pile driving was approximately 0.5 in (12.5 mm) and according to

Figure 10. Ground displacement versus distance from the pile (after Lewis and Davie, 1993).
the authors it can be attributed to vibration-induced densification of the soils. The driving process of the center pier H-piles caused a settlement of approximately 0.75 in (19.0 mm), while the driving of additional H-piles at the east abutment of the bridge foundation triggered settlements ranging from 0.5 to 1.0 in (12.0 to 25.0 mm). A total settlement of 2.0 in (50.0 mm) was measured at the end of construction. Linehan et al. (1992) concluded that extensive monitoring programs are required when pile driving is performed near sensitive structures. Pile driving-induced settlements should be a greater concern than ground vibrations since there are fewer documented failures from vibrations effects than from excessive displacements. Additionally, as a requirement of the project, vibrations levels were limited to a threshold of 2.0 in/s (50 mm/s) but still large vertical displacements were caused by the construction activities.

Figure 11. Vertical displacements of the pipeline during construction (from Linehan et al., 1992).
Hwang et al. (2001) presented an extensive field monitoring program from full-scale driving tests at the Chiayi-Taipo County in Taiwan. The project consisted of thirteen 1.5 m-diameter bored concrete piles spaced at 4.5 m. Additionally, thirteen 0.8 m-diameter precast concrete piles spaced at 2.4 m were driven at the site. Both bored and driven piles had a length of 34.0 m. The subsurface exploration consisted of Standard Penetration Tests (SPT), CPT, Seismic Cone Penetration Tests (SCPT), Dilatometers (DMT), and laboratory tests including unconfined compression tests (UC), consolidation undrained tests (CU), and quick direct shear tests (QDS). The soil profile consisted mostly of medium-dense to dense sandy soils interbedded by some soft clay layers up to a depth of 40.0 m. The field measurements included pore water pressures, lateral movements, settlements, and ground vibrations during the driving process of the first three driven piles (i.e., DP1, DP2, DP3).

Figure 12a presents the lateral displacements measured at distances of 3, 6, and 9 times the diameter of pile DP1 (i.e., 3d, 6d, and 9d) after complete driving of DP1. The maximum lateral displacements occurred at a distance of 3d with an average value of 20 mm (i.e., 2.5% of the diameter of the pile). Figure 12b presents the vertical displacements measured at 12 different settlement posts during driving of piles DP1, DP2, and DP3 (e.g., DP1-9M means that DP1 pile reached 9.0 m below the ground surface). Notice that most of the settlement posts experienced heave during the driving of the three piles, which might indicate that for dense sandy soils and/or clayey soils heave can be expected rather than settlement. A maximum heave of approximately 3.6 cm was observed when the DP1 pile tip reached a depth of 9.0 m at settlement post M0 located at a distance of 1.5d from DP1.
Clough and Chameau (1980) presented a case history of pile driving-induced settlements due to vibratory sheet pile driving in the San Francisco Bay area. Extensive measurements were conducted at two sites (i.e., E1 and E2). These measurements included peak particle accelerations and settlements at various distances from the piles. The soil conditions consisted of a surficial medium dense rubble fill made out of dune sand underlain by sand pockets up to a depth of 9.0 m. Below the sand pocket, soft bay muds followed by alternating layers of dense sand and firm clay were found. Figure 13 presents the settlement measured at both sites. The maximum settlement experienced was approximately 12.7 cm at a distance from the pile of 1.0 m. Notice that the settlements became negligible at a distance of approximately 12.0 m, which corresponds again to a distance of about the length of the piles.
2.3.2. Reported Peak Particle Velocity

Lewis and Davie (1993) presented vibration measurements at different sites besides the pile driving-induced ground movements case history explained in Section 2.3.1. Table 4 presents a summary of the site conditions, pile type, and hammer specifications for each project site. All projects were located at a power plant except site 1 which was located at a U.S government facility and corresponds to the case history explained in Section 2.3.1. Notice that the soil conditions at the sites consisted mostly of sandy soils with varying densities interbedded by clay layers. Vibration measurements at sites 2 through 7 were performed by seismographs, accelerometers, and velocity transducers that were used at site 1.
Table 4. Case histories summary (modified from Lewis and Davie, 1993).

<table>
<thead>
<tr>
<th>Site</th>
<th>Pile Type</th>
<th>Driven Length (m)</th>
<th>Hammer Type</th>
<th>Rated Energy (kN-m)</th>
<th>Soil Conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>350 mm-Square Concrete Pile</td>
<td>24.4</td>
<td>ICE 640</td>
<td>54.2</td>
<td>Loose to dense sands and silty sands</td>
</tr>
<tr>
<td>2</td>
<td>Raymond Step Taper</td>
<td>23.8</td>
<td>Vulcan 80c</td>
<td>33.1</td>
<td>Fill, Soft Clayey Silt and medium clayey Sand</td>
</tr>
<tr>
<td>3</td>
<td>PZ-27 Sheet Pile</td>
<td>9.1</td>
<td>Delmag D-15</td>
<td>36.6</td>
<td>Medium to dense sands</td>
</tr>
<tr>
<td>4</td>
<td>Raymond Step Taper</td>
<td>12.2</td>
<td>Vulcan 80c</td>
<td>33.1</td>
<td>Fill, Soft Silts and clay, dense to medium dense sands</td>
</tr>
<tr>
<td>5</td>
<td>Close-end Pipe pile 270 mmx265 mm</td>
<td>9.1</td>
<td>Vulcan 06</td>
<td>26.4</td>
<td>Loose to Medium Sand, Soft Clay, very dense sand</td>
</tr>
<tr>
<td>6</td>
<td>H-Pile 14x117</td>
<td>9.1</td>
<td>Vulcan 06</td>
<td>26.4</td>
<td>Medium dense to dense sand</td>
</tr>
<tr>
<td>7</td>
<td>Raymond Step Taper</td>
<td>24.4</td>
<td>Vulcan 06</td>
<td>26.4</td>
<td>Loose sand, soft clayey silt, and medium dense to dense sand</td>
</tr>
</tbody>
</table>

Peak particle velocity measurements versus distance from the pile and scaled distance are presented in Figures 14a and 14b, respectively. The scaled distance used for the charts was defined following Equation (3) presented by Wiss (1981). The charts were developed using the transmitted energy to the pile, which the authors assumed was approximately 30% to 40% of the rated energies of the hammers, thus an average transmitted energy of 10,000 lbf-ft (13.6 kN-m) was used. Notice that the attenuation coefficients \( n \) and \( k \) from Equation (3) in imperial units were computed as 1.0 and 0.1, respectively. The authors reported that for distances greater than 10 ft (3.0 m) the PPV values were less than 50 mm/s, and no structural damage was reported on nearby structures.
Brunning and Joshi (1989) monitored ground vibrations due to driving of six 300 mm x 300 mm HP-piles spaced 2 m center to center in a construction project in Calgary, Italy. The piles were driven at a distance of 2.1 m from an existing 400 mm gas pipeline buried at a depth of 1.0 m below the ground surface. A single-acting D-22 diesel hammer with a rated energy of 54 kJ/blow was used to drive the 11.0 m-long piles. The soil conditions consisted of a loose silty sand and gravelly fill underlain by a very dense coarse gravel mixed with boulders. The deepest stratum was defined as a low plasticity very stiff clay found at a depth ranging from 6.0 m to 8.0 m below the ground surface. Figure 15a presents the vibration levels at the gas pipeline during driving of piles 97, 99, and 100 in terms of the depth of the pile tip. The authors noted that peak particle velocity was not recorded as the piles penetrated through the loose granular fill and afterward the PPV values ranged from 19 mm/s to 22 mm/s. The maximum PPV occurred when the pile tip reached 1.0 m of penetration through the dense gravel layer, which indicates that a dense material might attenuate less than a loose material. Figure 15b presents the ground peak particle velocities measured at a distance of 1.2 m and 1.5 m during driving of piles 100 and 102, respectively. As the piles penetrated through the loose granular fill, the values of PPV were 38 mm/s and 25 mm/s.
at 1.2 m and 1.5 m, respectively. Similar to the case of vibrations at the pipeline, the maximum values of PPV were recorded when the pile tip penetrated through the dense gravel layer.

Figure 15. Vibration levels: (a) at the pipeline during driving of piles 97, 99, and 100; and (b) at the ground surface during driving of piles 100 and 102 (after Brunning and Joshi, 1989).

Figure 16 presents the maximum measured PPV at different distances from the pile. Notice that for distances up to 1.5 m from the pile the ground vibrations exceeded a limiting value of 50 mm/s. The authors concluded that ground vibrations induced by pile driving are strongly correlated to pile penetration resistance and that vibrations decreased with increasing horizontal distance from the source of vibration.
Grizi et al. (2016) presented ground vibration measurements at different sites in the state of Michigan during driving of 360 mm x 109 mm H-piles in granular soils. A Pileco D30-32 and a Delmag D30-32 diesel hammers were used to drive the 16.8 m-long H-piles. Penetration depths varied between 13.1 m and 16.1 m. The soil conditions consisted of predominantly loose sands underlain by layers of medium dense to very dense sands. The measurements were performed at different depths below the ground surface and at different distances from the piles. Figure 17 presents the variation of PPV values with pile tip elevation during driving of the H-piles at three different embedment depths of the sensors. The dimensions in parenthesis correspond to the distance from the pile where the sensor was located. Notice that the ground vibrations increased significantly when the tip of the pile penetrated below the sensor depth. The authors concluded that the sensors only recorded the waves coming from the tip of the pile when the pile tip was still above them. When the tip of the pile penetrated below the sensors, they measured both waves coming from the tip and the shaft of the pile.
Figure 17. Peak particle velocity versus pile penetration depth at depths of: (a) 7.8 m, (b) 4.9 m, and (c) 10.8 m (after Grizi et al., 2016).

Figure 18 presents the attenuation curves fitted at different embedment depths of the sensors. Equation (2) developed by Bornitz (1931) was used to fit the attenuation curves to the measured data. A high rate of attenuation was observed near the pile, but it decreased dramatically when the distance from the pile increased.

Figure 18. Attenuation curves fitted to in-depth measurements at depths of: (a) 7.8 m, (b) 4.9 m, (c) 10.8 m (after Grizi et al., 2016).

Cleary et al. (2015) presented an investigation of ground vibrations induced by pile driving near the Mobile River in Mobile, Alabama. The purpose of this study was to understand the factors affecting the level of vibrations during construction processes such as distance from the source,
site-specific conditions, and pile installation method. A 900 mm-wide square precast concrete pile, and HP14X117 and HP12X53 H-piles were driven at the site. The pile lengths were 27 m, 32 m, and 21 m, respectively. A single-acting Delmag D62-22 diesel hammer was used to drive the precast concrete pile, while an APE D30-42 diesel hammer was used to drive the H-piles. The soil conditions consisted of loose to medium and medium dense sands interbedded by a thin stiff to very stiff clay layer. Geophones located at 15 m, 21 m, 30 m, and 45 m away from the pile were used to measure the ground vibrations. Figure 19 presents the PPV attenuation curves for both precast concrete pile and H-Piles. The regression lines from the field data were computed by using Equation (1) proposed by Hendricks (2002). Notice that higher vibration levels are expected when precast concrete piles are driven than in the case of the H-piles, with maximum PPV values of 20.8 mm/s (0.8 in/s) and 5.8 mm/s (0.23 in/s), respectively. This can be explained due to the volume displaced by the piles and the effort required to drive them into the ground.

![Figure 19. Peak Particle Velocity relationships for precast concrete pile and H-piles (after Cleary et al., 2015).](image-url)
2.4. Numerical Modeling

The main objective of this research is to present numerical models capable of predicting ground deformations due to pile driving and analyze the variables involved in this soil dynamics process. This section presents numerical modeling approaches reported in the technical literature and the basic theory behind the constitutive soil models used in this research.

2.4.1. Wave Equation Analysis

The wave equation analysis is a numerical method for assessing pile capacities. It considers the pile, soil, and hammer properties and analyzes them as compliance of masses, springs, and dashpots. It was first developed by Smith (1960) to overcome the deficiencies of the commonly used pile driving formulas, which were at the time mostly empirical and only applied to site-specific conditions. Figure 20 presents the first representation of the hammer, pile, and soil system assumed by Smith (1960). Notice that the hammer is represented as a system of masses and springs on top of the pile, while the pile is divided into segments attached with each other by other springs, thus simulating the stiffness of the element.

For the purpose of the wave equation analysis, the pile is divided into segments of the same length (normally 1.0 m), and these segments are connected with each other by springs, thus modeling the stiffness of the pile. The forces acting on each segment can be expressed by Equation (10):

\[ Z_i = F_{i-1} - F_i - R_i \]  

(10)

where,

\( Z_i \): Acceleration (net) force at the ith segment.

\( F_{i-1} \): Force exerted by the spring at the start of the element.
F_i: Force exerted by the spring at the end of the element.

R_i: Soil resistance in the i^{th} element at the time n.

Figure 20. Discretization of the pile driving process by the wave equation analysis (from Smith, 1960).

Notice that the force exerted by the springs will depend on the spring displacement, thus it is a function of the time interval chosen to analyze the problem. Smith (1960) recommended a time step of 1/4000 or half the time it takes the compression waves to travel the entire length of one segment to avoid misleading results. The soil resistance (R_i) at any time would be a combination of a static resistance (R_s) modeled as a spring and a dynamic resistance (R_d) modeled as a dashpot accounting for damping in the soil. Figure 21 presents the stress-strain behavior of the soil assumed.
by Smith (1960). The soil was assumed to behave linearly up to a displacement $Q$ defined as the “quake”, which is the displacement where the ultimate static resistance ($R_u$) is reached. Beyond this point the soil would behave perfectly plastic, thus the static resistance would remain as $R_u$.

![Stress-strain behavior of the soil model](image)

Figure 21. Stress-strain behavior of the soil model for the wave equation analysis (from Smith, 1960).

Furthermore, the dynamic resistance can also be expressed in terms of the static resistance using the static damping coefficient ($J$) that can be applied to the velocity of the pile ($v_p$) at the instant $x$ as shown in Equation (11). Notice that the coefficient $J$ and the quake might be different for the end bearing and shaft resistances.

$$R_d = Jv_p R_x$$  \tag{11}

This method has already been developed for various computer programs. The ones that are going to be considered in this research are GRLWEAP (PDI, 2005) and CAPWAP developed by GRL Engineers, Inc. The former is used at the design stage to estimate the driving criterion of the pile and select the hammer system to be used. The latter uses measurements from the Pile Driving Analyzer (PDA), which is used to interpret pile accelerations and strains, to determine the capacity of the element in the field and control the desired resistance.
2.4.2. Pile Driving Numerical Approaches

Pile driving is a complex dynamic soil-structure interaction problem that induces vibrations and ground deformations in surrounding soils. Numerical models must predict accurately the pile and soil dynamics so that the response of soil during the pile installation can be properly assessed. As explained before, wave equation analysis programs such as GRLWEAP (PDI, 2005) can use the wave equation analysis to estimate engineering demands triggered during the driving process (e.g., hammer forcing function) and are also used to analyze dynamic testing of piles for the determination of in situ capacity. However, these programs do not provide insight into the effects of pile driving on the surrounding soil or nearby structures (e.g., deformations and vibrations). Finite element (FE) software such as PLAXIS 2D (Brinkgreve et al., 2010) have been used by several authors to overcome this issue. This research compares two commonly used pile driving FE modeling approaches. A “discontinuous” modeling approach can be performed by installing the pile at different “wished-in-place” depths and applying a single hammer blow at the top of the pile for each depth (e.g., Grizi et al., 2018; Mabsout et al., 1995). This approach has been commonly used to understand ground vibration levels and excess pore water pressure build-up at different depths and distances from the pile. On the other hand, a “continuous” modeling approach consists of a continuous pile driving process, in which the pile is driven without any interruption up to a final depth (e.g., Khoubani and Ahmadi, 2014; Farshi Homayoun Rooz and Hamidi, 2017). This approach has been used mostly to analyze vibrations generated as the pile is driven.

Grizi et al. (2018) conducted a reduced-scaled laboratory pile test and also modeled the test in the FE program PLAXIS 3D to compare the accuracy of a discontinuous modeling approach to predict ground vibrations. The laboratory test consisted of the installation of a 2.5 m-long S3X5.7 beam through a cylindrical sandpit filled with silica sand. The silica sand was modeled in PLAXIS
3D by using the Hardening Soil (HS) model. A material data set for the pile-soil interface with reduced parameters was employed since default interface elements in PLAXIS did not work well. This interface was extended in a cylindrical shape with a diameter of 0.15 m around the pile and 0.15 m below the pile tip. A strength reduction factor \((R)\) and a shear wave velocity reduction factor \((R_s)\) were used for this interface to affect the strength parameters and stiffness moduli of the HS model, respectively. A total of seven hammer blows at seven different penetration depths were selected. Figure 22 presents the comparison between the PPV values at different pile penetration depths measured in the laboratory and the values computed in the numerical model at different depths and distances from the pile. The dimensions in parenthesis correspond to the distance from the pile where the sensor was located. There was a good agreement between the laboratory data and the computed values. However, the numerical model underestimated the measured PPV values depending on the pile penetration depth. It should be noted that the authors recognized that this methodology cannot capture changes in stresses and strains during pile driving, thus making it virtually impossible to get soil deformations by using discontinuous pile driving.
Mabsout et al. (1995) modeled a discontinuous concrete pile driving through a normally consolidated clay under axisymmetric conditions. The constitutive soil model used by the authors was a bounding-surface plasticity model for isotropic undrained cohesive soils developed by Kaliakin and Dafalias (1989). A linear elastic formulation was used to model the pile elements. The soil-pile interaction was modeled by a slide-line formulation that allows large relative sliding between pile and soil. Additionally, absorbing boundaries were introduced in the far-field to transmit the waves and prevent wave reflection. In order to understand the effects of applying more than one blow at a certain depth, the authors analyzed the case of a pile pre-drilled at a depth of 17.0 m driven to an additional penetration of 0.33 m by applying 8 hammer blows (B8-D17) compared with the response of the same pile under a single hammer blow but pre-drilled at a depth of 18.0 m (B1-D18). Figure 23 presents the displacement time history of the top of the pile under a single hammer blow for the cases B8-D17 and B1-D18. Both displacement responses are similar which leads to the conclusion that if the pile in case B8-D17 was driven beyond the 8\textsuperscript{th} hammer blow (i.e., 1.0 m of penetration instead of 0.33 m of penetration as defined) the pile response should
be stiffer than B1-D18. Thus, a discontinuous analysis under a single blow could lead to more flexible and unrealistic responses than an analysis under multiple blows.

![Figure 23. Comparison of pile tip displacements for the cases of 1 and 17 blows at a depth of 17.0 m and 1 blow at a depth of 18.0 m (after Mabsout et al., 1995).](image)

Farshi Homayoun Rooz and Hamidi (2017) stated that a discontinuous pile driving can lead to misleading conditions of the soil around the pile since it does not consider soil distortion around the pile, thus it cannot properly model stresses, strains, and pile-soil interaction. The importance of modeling the pile element as an elastic element instead of a rigid element was also highlighted by the authors.

Both Farshi Homayoun Rooz and Hamidi (2017), and Khoubani and Ahmadi (2014) used an Arbitrary Lagrangian-Eulerian (ALE) adaptive meshing to deal with significant mesh distortions expected due to the continuous pile driving. This method consists of three domains (i.e., material, spatial, and referential domains) allowing an arbitrary movement between material points and spatial mesh. The stages of this method at each time increment in the numerical analysis consisted of i) material nodes moving to new positions; ii) a new spatial mesh generated to best match the material nodes, and iii) transferring the solution from the old mesh to the new one.
According to Khoubani and Ahmadi (2014), adaptive meshing can be used to obtain faster, more accurate, and more robust solutions than with pure Lagrangian analyses.

2.4.3. Hardening Soil (HS) Small Model

The Hardening Soil model enhanced with small strain relationships (HS small) developed by Benz (2006) is used in the FE models presented in this section. This model is an advanced version of the standard Hardening Soil (HS) model proposed by Schanz et al. (1999), which can reproduce basic behavior observed in soils such as stress-dependent stiffness, soil stress history, plastic yielding, and dilatancy based on an isotropic hardening rule (Obrzud, 2010). The HS model represents the stress-strain behavior of the soil using three different stiffness moduli defined as the triaxial unloading-reloading stiffness \( E_{ur} \), oedometer loading modulus \( E_{oed} \), and the triaxial loading stiffness \( E_{50} \). Shear hardening caused by plastic strains due to deviatoric stresses is considered with \( E_{50} \). Compression hardening caused by plastic strains due to primary compression in oedometer loading and isotropic loading is considered with \( E_{oed} \). Figure 24 presents the typical stress-strain behavior computed with the HS model for a drained triaxial test. This model aims to reproduce the axial strain and deviatoric stress relationship with a hyperbolic nonlinear function similar to the commonly used approach by Duncan and Chang (1970).

The confining pressure-dependent behavior of the soil is introduced as a power-law by using an additional parameter \( m \), and the Mohr-Coulomb strength parameters \( c \) and \( \phi \) as follows:

\[
E_{50} = E_{50}^{ref} \left( \frac{ccos(\phi) - \sigma_3'sin(\phi)}{ccos(\phi) - p^{ref}sin(\phi)} \right)^m
\]  

(12)
\[ E_{ur} = E_{ur}^{ref} \left( \frac{ccos(\phi) - \sigma_3^{\prime} \sin(\phi)}{ccos(\phi) - p^{ref} \sin(\phi)} \right)^m \]  

(13)

where, \( E_{50}^{ref} \) and \( E_{ur}^{ref} \) are the reference stiffness modulus and Young’s unloading-reloading modulus at a reference pressure \( p^{ref} \), respectively. Normally, the reference pressure is set to 100 kPa in PLAXIS 2D (Brinkgreve et al., 2010b). Notice that the standard HS model assumes that the soil behaves as a linear elastic material during unloading-reloading cycles. Therefore, the HS small model overcomes this issue by introducing additional expected behavior such as strong stiffness variation and hysteretic, nonlinear elastic stress-strain relationships applicable in the small strains range (Obrzud, 2010).

Figure 24. Typical stress-strain hyperbolic curve computed with the HS model (from Brinkgreve et al., 2010b)

Table 5 presents the description of each of the required parameters for the HS small model. Hysteretic behavior and shear modulus degradation curves are considered by adding two parameters to the standard HS model (i.e., \( G_0 \) and \( \gamma_{0.7} \)). This model considers a similar small-strain behavior to Hardin and Drnevich (1972).
### Table 5. Description of constitutive soil parameters for the HS small model.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>ϕ'</td>
<td>Internal friction angle</td>
</tr>
<tr>
<td>ψ</td>
<td>Angle of dilatancy</td>
</tr>
<tr>
<td>c'</td>
<td>Cohesion</td>
</tr>
<tr>
<td>p&lt;br&gt;ref</td>
<td>Reference pressure for stiffnesses</td>
</tr>
<tr>
<td>E&lt;sub&gt;50&lt;/sub&gt;ref</td>
<td>Secant stiffness in drained triaxial test at reference pressure</td>
</tr>
<tr>
<td>E&lt;sub&gt;oed&lt;/sub&gt;ref</td>
<td>Tangent stiffness for primary oedometer loading at reference pressure</td>
</tr>
<tr>
<td>E&lt;sub&gt;ur&lt;/sub&gt;ref</td>
<td>Unloading-reloading stiffness at reference pressure</td>
</tr>
<tr>
<td>G&lt;sub&gt;0&lt;/sub&gt;Ref</td>
<td>Reference shear modulus at small strains (ε&lt;10&lt;sup&gt;-6&lt;/sup&gt;)</td>
</tr>
<tr>
<td>m</td>
<td>Power coefficient for pressure-dependent stiffness</td>
</tr>
<tr>
<td>ν'&lt;sub&gt;ur&lt;/sub&gt;</td>
<td>Poisson’s ratio for unloading reloading</td>
</tr>
<tr>
<td>γ&lt;sub&gt;0.7&lt;/sub&gt;</td>
<td>Threshold shear strain at 0.7G&lt;sub&gt;0&lt;/sub&gt;</td>
</tr>
<tr>
<td>R&lt;sub&gt;f&lt;/sub&gt;</td>
<td>Failure ratio</td>
</tr>
</tbody>
</table>

#### 2.4.4. Hypoplasticity Constitutive Soil Model

The constitutive soil model used in these analyses was the hypoplasticity model for sands formulated by von Wolffersdorff (1996) enhanced with the intergranular strain concept by Niemunis and Herle (1997). Since this model was not available in PLAXIS 2D as a default material option, the user-defined model implemented for PLAXIS 2D by Gudehus et al. (2008) was used.

The hypoplasticity model for sands uses critical state soil mechanics and the hypoplasticity framework to reproduce the nonlinear behavior of the soil. This means that the soil behavior directly depends on the stress tensor (i.e., confining pressure and deviatoric stress) and the void ratio of the granular material to represent stress-strain relationships and stiffness degradation. The Drucker-Prager model and Matsuoka-Nakai yielding criterion were incorporated by von Wolffersdorff (1996) to represent the asymptotic behavior near failure of granular materials. The basic formulation separates the material behavior dependency on the void ratio and pressure into so-called pycnotropy and barotropy factors, respectively.
Three asymptotic states (i.e., states where the stress rate becomes zero) are defined by von Wolffersdorff (1996). These states are limited by the void ratios $e_d$, $e_c$, and $e_i$ which are defined as the lower limit, critical state, and upper limit at isotropic compression void ratios, respectively. Figure 25 presents a graphical representation of the range of admissible void ratios based on the hypoplasticity formulation. It is no possible for a soil to reach a state out of the limits defined by these three limiting void ratios. Notice that the limit void ratios depend on the mean pressure ($trT$), thus the different asymptotic states and the soil behavior vary depending on the location of the soil in the stress-void ratio plane.

Figure 25. Range of possible void ratios as a function of stress (from von Wolffersdorff, 1996).

Niemunis and Herle (1997) introduced the intergranular strain concept to overcome issues related to cyclic loading and deformations at the very small strain range. These issues consisted mainly of an overprediction of strains at small stress cycles as well as an overestimation of pore water pressure build-up. This concept considers an intergranular interface layer that accounts for an additional deformation in this layer.

Table 6 presents the description of the parameters for the hypoplasticity model for sands enhanced with the intergranular strain concept. The basic hypoplasticity parameters are described in parameters 1 to 9, while the intergranular strain parameters are described in parameters 10 to 14.
Table 6. Description of the constitutive soil parameters for the Hypoplasticity for sands enhanced with the intergranular strain concept.

<table>
<thead>
<tr>
<th>No</th>
<th>Parameter</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$\phi_c$</td>
<td>Critical state friction angle</td>
</tr>
<tr>
<td>2</td>
<td>$p_t$</td>
<td>Shift of the mean stress due to cohesion</td>
</tr>
<tr>
<td>3</td>
<td>$h_s$</td>
<td>Granular hardness</td>
</tr>
<tr>
<td>4</td>
<td>$n$</td>
<td>Exponent for pressure-sensitive of a grain skeleton</td>
</tr>
<tr>
<td>5</td>
<td>$e_{d0}$</td>
<td>Minimum void ratio at zero pressure ($p_s = 0$)</td>
</tr>
<tr>
<td>6</td>
<td>$e_{c0}$</td>
<td>Critical void ratio at zero pressure ($p_s = 0$)</td>
</tr>
<tr>
<td>7</td>
<td>$e_{i0}$</td>
<td>Maximum void ratio at zero pressure ($p_s = 0$)</td>
</tr>
<tr>
<td>8</td>
<td>$\alpha$</td>
<td>Exponent for transition between peak and critical stresses</td>
</tr>
<tr>
<td>9</td>
<td>$\beta$</td>
<td>Exponent for stiffness dependency on pressure and density</td>
</tr>
<tr>
<td>10</td>
<td>$m_R$</td>
<td>Stiffness increase for 180° strain reversal</td>
</tr>
<tr>
<td>11</td>
<td>$m_T$</td>
<td>Stiffness increase for 90° strain reversal</td>
</tr>
<tr>
<td>12</td>
<td>$R_{\text{max}}$</td>
<td>Size of elastic range</td>
</tr>
<tr>
<td>13</td>
<td>$\beta_r$</td>
<td>Material constant representing stiffness degradation</td>
</tr>
<tr>
<td>14</td>
<td>$\chi$</td>
<td>Material constant for evolution of intergranular strains</td>
</tr>
</tbody>
</table>
3. FIELD DATA

This chapter presents the field testing program designed to measure both ground displacements and ground vibrations induced by pile driving operations in Central Florida. An overall description of each project and the pile driving process are also presented. A total of eight bridge construction sites in Central Florida are included in this chapter. The field data collected is used in the following chapters to build and validate numerical models and to study the interactions among the variables involved in this problem (i.e., type of hammer, type, size and length of pile, and soil properties) and to issue guidelines on the prediction of ground surface deformations and vibration levels for similar geotechnical conditions as the ones considered herein. This chapter also includes details of the testing equipment and procedures used to install the piles.

3.1. Description of Field Equipment at University of Central Florida

3.1.1. Geophones and Data Acquisition System

Ground vibrations measurements were conducted using single component (i.e., vertical axis) geophones manufactured by Sercel Ltd. The technical specifications for these geophones are shown in Table 7. The geophones had a natural frequency of 5 Hz and worked under a wide range of temperatures. A total of nine geophones were used in this research. This number of sensors allowed measurements close to the piles and at approximately free-field conditions to define ground attenuation characteristics.
Table 7. Technical specifications of the geophones used in the field.

<table>
<thead>
<tr>
<th>Specification</th>
<th>SG-5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Natural Frequency</td>
<td>5 Hz</td>
</tr>
<tr>
<td>Coil Resistance</td>
<td>1850 Ω</td>
</tr>
<tr>
<td>Harmonic Distortion</td>
<td>&lt;0.1%</td>
</tr>
<tr>
<td>Sensitivity</td>
<td>80 V/m/s</td>
</tr>
<tr>
<td>Moving Mass</td>
<td>22.7 g</td>
</tr>
<tr>
<td>Spurious Resonance</td>
<td>&gt; 150 Hz</td>
</tr>
<tr>
<td>Diameter</td>
<td>32 mm</td>
</tr>
<tr>
<td>Length</td>
<td>43 mm</td>
</tr>
<tr>
<td>Weight</td>
<td>170 g</td>
</tr>
<tr>
<td>Operating Temperature</td>
<td>-40° to 80°C</td>
</tr>
</tbody>
</table>

The data acquisition unit was the multi-channel system RAU eX-3 manufactured also by Sercel Ltd. Each RAU unit is equipped with three slots for geophones, thus three acquisition units were used in this project for the 9 geophones. Table 8 presents the technical specifications of the system. These units were selected because they provided a wireless system offering flexibility when deploying sensors to the field, provided a good sampling rate and a wide operational temperature range.

Table 8. Technical specifications of the data acquisition system used in the field.

<table>
<thead>
<tr>
<th>Specification</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of channels</td>
<td>3</td>
</tr>
<tr>
<td>Memory</td>
<td>310 h</td>
</tr>
<tr>
<td>Timing accuracy</td>
<td>better than 20 µs</td>
</tr>
<tr>
<td>Operational temperature</td>
<td>-40 °C to +60°C</td>
</tr>
<tr>
<td>Acquisition gain</td>
<td>0 dB or 12 dB</td>
</tr>
</tbody>
</table>

3.1.2. Survey Equipment

Ground deformations were measured during pile driving by using DT209 Theodolites manufactured by Topcon Ltd. Table 9 presents the technical specification of the DT209 model. Three theodolites were available to perform ground deformation measurements during the pile driving process to obtain data on how the deformations vary during and after the installation of the
piles. The location of the nine geophones was used as the deformation points during the pile driving process. Also, 8 in-long survey nails manufactured by Bernsten International were used to collect additional deformation points in the field.

| Table 9. Technical specifications of the survey equipment used in the field. |
|---------------------------------|-----------------|-----------------|
| **Angle Measurement**           | **Accuracy**    | 9 seconds       |
|                                 | **Method**      | Absolute reading|
|                                 | **Min. Reading**| 20 seconds      |
| **Telescope**                   | **Magnification**| 26x             |
|                                 | **Minimum Focus**| 0.9m            |
|                                 | **Sighting Collimator**| Double        |
| **Optical Plummet**            | **Magnification**| 3x              |
|                                 | **Field of view**| 3°              |
| **Operating Time**              | Theodolite and Laser | 170h          |
| **Operating Temperature**       |                  | 20° to 50°C     |

3.2. Overall Project Site Descriptions

Figure 26 presents the location of the bridges considered in this research. Sites are numbered from north to south. Dynamic pile tests were performed at sites A through C. Direct measurements were taken during the tests in terms of ground deformations, ground vibrations, and forces applied to the top of the pile. Site A is located on State Road 44 (SR44) over the St. John’s River. Sensors were deployed to the site to measure ground surface deformations and PPVs caused by pile driving operations. Site B is located on the Wekiva Parkway Section 6 near the Wekiva River. Peak particle velocities and ground surface settlements were collected during a test pile installation at the project site. Site C is located at the intersection between Florida’s Turnpike and I-4 highway. This bridge consists of a ramp that connects both highways. The soil profile and field data were obtained from a dynamic pile test provided by District 5 engineers at the FDOT. Sites
D through H correspond to sites previously studied by Bayraktar et al. (2013) during the construction of Florida’s Turnpike. These sites are located relatively close to site C (i.e., 3.2 km from site D and 30.4 km from site H), thus these measurements were considered valuable for this research. Even though measurements of the ground deformations were not reported by Bayraktar et al. (2013), PPV measurements and information regarding the input energy are used in the following chapters to compare and validate the proposed numerical models.

Figure 26. Location of the project sites (Map data © 2020 Google).

Table 10 presents a summary of the measurements obtained from each project site. Ground deformation measurements were performed at sites A and B. Pile Driving Analyzer (PDA) tests performed at site C were used to obtain forces applied at the top of the pile during dynamic pile
tests. The PPV measurements performed by Bayraktar et al. (2013) at sites D through H are also presented.

Table 10. Summary of project site locations and measurements performed at the sites.

<table>
<thead>
<tr>
<th>Site</th>
<th>Location</th>
<th>PDA</th>
<th>Driving Process</th>
<th>PPV</th>
<th>Ground Deformation</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>SR 44 over St. John's River</td>
<td></td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>Wekiva Parkway Section 6</td>
<td></td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>Connection Ramp Turnpike with I-4</td>
<td>X</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>Turnpike over Shingle Creek&lt;sup&gt;a&lt;/sup&gt;</td>
<td></td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>Sand Lake Rd. over Turnpike&lt;sup&gt;a&lt;/sup&gt;</td>
<td></td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>SR 528 over Turnpike&lt;sup&gt;a&lt;/sup&gt;</td>
<td></td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>G</td>
<td>Turnpike over US 441&lt;sup&gt;a&lt;/sup&gt;</td>
<td></td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>H</td>
<td>Kissimmee Park Road&lt;sup&gt;a&lt;/sup&gt;</td>
<td></td>
<td></td>
<td>X</td>
<td></td>
</tr>
</tbody>
</table>

<sup>a</sup>Measurements previously reported by Bayraktar et al. (2013).

3.2.1 Site A (SR 44 over St. John's River)

This project consists of a two-lane bridge on State Road 44 (SR-44) projected over the St. John’s River close to Deland, Florida (see Figure 27a). The information about this project was provided by FDOT district 5 including soil borings and structural drawings. Figure 27b presents the location of the soil borings (B1-B3) and a nearby cone penetration test performed at the site (CPT-177).
Figure 27. Location of: (a) bridge on SR 44 over St. John’s River at site A and (b) soil borings relative to the construction site (Map data © 2021 Google).

Figure 28 presents the geometry of the proposed 505 m (1656 ft) long bridge. The structure consists of 10 spans with a maximum span length of 55.0 m (180 ft) between Piers 5 and 6. Piers 4, 5, and 6 were projected to be built inside the river and the remaining piers on land. The location of the pier used to collect pile driving data is shown in the figure (i.e., Pier 3).
Figure 28. Geometric layout of the bridge at site A. Pier 3 is highlighted.

Figure 29a presents the foundation layout for the west side of the bridge. A detailed view of the foundation layout for Pier 3 is shown in Figure 29b. Pier 3 consisted of a group of twenty-two 610 mm (24 in.) wide prestressed concrete piles with a length of approximately 38 m (125 ft). Piles 10 and 13 were used as the test piles for this pier.
Figure 29. Foundation layout at site A: (a) overall view with location of the piles and (b) Detailed plan view of foundation layout.
A shallow sheet pile was installed only for construction purposes around pile 10 prior to driving both piles. Figure 30 presents the sheet piles installed in the field. Notice that the sheet piles were located only around pile 10, thus no structural elements were installed in front of pile 13. Figure 31 and 30 present a graphical explanation of the driving process of both piles. An APE D50-52 hammer was used to drive the piles approximately 32 m (105 ft) into the ground as shown in Figure 31b. Pile driving operations of pile 13 were conducted first. Installation of pile 10 was conducted the following day.

![Sheet pile installed around pile 10 at site A: (a) general view and (b) close up view.](image)

Figure 30.
Figure 31. Pile driving process at site A: (a) hoisting of pile 13, (b) APE D50-52 used for driving piles 10 and 13, and (c) installation of plywood cushion.

Figure 32. Pile driving process at site A: hammer lift-up before driving of (a) Pile 13 and (b) Pile 10, and (c) ending of driving.
3.2.2 Site B (Wekiva Parkway Section 6)

This project consisted of a bridge at SR-429 (Wekiva Parkway). Figure 33 presents the foundation layout of the project. A total of 3 bridges were projected at this site: bridges 110118, 110119, and 110120. For the field measurements, pile 12 located at Pier 5 of Bridge 110119 was selected.

The 19.8 m-long prestressed concrete pile with a 610 mm square cross-section (see Figure 34a) was part of a group of 14 piles. Figure 34c presents a sheet pile cofferdam built by the contractor around the pile group due to the groundwater regime and soil conditions at the site. The effects of this cofferdam around the pile group on the pile driving induced vibrations and ground movements are also discussed in light of the measurements taken during the driving process. Typical photographic records during the pile driving operations, including pile hoisting, hammer lift up and ending of the pile driving process are shown in Figures 35 to 36.
Figure 33. Foundation layout at site B: (a) overall plan view of bridges 110118, 110119, and 110120 with location of test pile at pier 5 and (b) detailed typical plan view of foundation layout.
Figure 34. Conditions prior to driving process: (a) prestressed concrete test pile cross-section, (b) accelerometer installation for PDA test, and (c) cofferdam built around pier 5 at site B.

Figure 35. Pile driving process at site B: (a) prestressed concrete pile hoisting, (b) hammer lift up, and (c) APE hammer used for pile driving operation.
Figure 36. Pile driving process at site B: (a) beginning, (b) pile at final penetration depth, and (c) driving hammer during the installation of the prestressed concrete test pile.
3.2.3 Site C (Turnpike with I-4)

This project, also presented by Turkel et al. (2021), involved the construction of a connection ramp bridge at the intersection between Florida’s Turnpike and I-4 highway (see Figure 37a). The information provided by the FDOT about this project included driving records from several test piles, soil borings, and structural drawings. The location of the soil borings performed at the project site (TB-63) and at nearby locations (B1-B7) is presented in Figure 37b.

![Figure 37](image)

Figure 37. Location of: (a) the bridge at site C at the intersection between Florida’s Turnpike and I-4 highway and (b) soil borings relative to the construction site (after Turkel et al., 2021).

Figure 38 presents the structural plans for the project that consisted of a 640 m-long bridge with 13 spans built over 15 piers and 2 end bents. The foundation system for the structure consisted of groups of precast prestressed concrete piles. A dynamic test conducted at Pile 1 of Pier 11RT was selected for the analyses presented herein. The 27.4 m long, 0.61 m-wide prestressed concrete pile was pre-drilled at a depth of 9.7 m before the pile driving operations started. The pile was installed by using an APE D 70-52 open-ended diesel (OED) hammer with a ram weight of 68.7 kN and a maximum rated energy of 235.5 kJ. A 381 mm-thick plywood pile cushion was used but
it was later modified during the driving process for a 457 mm-thick plywood. The hammer cushion consisted of 25.4 mm thick Micarta and 12.7 mm-thick aluminum materials.
Figure 38. Foundation layout at site C: (a) overall view and (b) detailed view of Pier 11 RT foundation layout.
3.3. Soil Condition at the Sites

The soil conditions at the sites were defined based on SPT, CPTs and index properties: fine contents, water contents \((w)\), liquid limits \((LL)\), and plastic limits \((PL)\). The relative density \((D_r)\) of the sand layers and the undrained shear strength \((S_u)\) of interbedded clay layers were determined by using correlations with the SPT N number presented by Kulhawy and Mayne (1990). In summary, the soil conditions in the area were characterized by the presence of mostly poorly graded sands with silts \((SP-SM)\) of relative densities in the medium-dense range.

3.3.1. Site A (SR 44 over St. John's River)

Figure 39 presents the results of the subsurface exploration conducted at project site A. The summarized soil conditions shown in the figure consist of a surficial 3.0 m thick muck layer, which according to the contractor it was removed at the site before the pile driving operations started. Beneath this stratum, a silty sand layer was observed to a depth of approximately 18.3 m. This layer presented a gradual increase in relative density from approximately 20% at the shallow portion to almost 60% at a depth of 18.3 m where the soil transitioned to a fat clay layer \((CH)\) with a thickness of 3.0 m. This clay layer was underlain by a 12.2 m thick medium-dense sand stratum. Unlike the topmost silty sand layer, this medium-dense sand presented more uniform values of SPT blow counts with depth. At the bottom of the soil profile, a weathered limestone at the final boring depth of approximately 57.9 m was reached. The figure also shows the approximate location of a shallow groundwater table encountered at the project site.
Figure 39. Summarized subsurface conditions at site A.


3.3.2. Site B (Wekiva Parkway Section 6)

Soil conditions at site B were based on qualitative descriptions obtained from the FDOT soil borings database. The soil profile consisted of a surficial loose to medium fine sand with silts up to a depth of 6.1 m (20 ft) underlain by a 5.0 m-thick (15 ft) sandy clay. A gray weathered dolostone with phosphates was found at the bottom of the borings which occurred at an approximate depth of 23.0 m. The groundwater table was found approximately at the ground surface.

3.3.3. Site C (Turnpike with I-4)

Figure 40 presents the summarized subsurface conditions at site C. The medium dense sand layer, which extends from the ground surface level to a depth of 6.20 m, is underlain by a 7.0 m thick medium stiff clay layer. A 15.0 m thick loose to medium dense sand with a 45% relative density is followed by a dense sand of 85% in relative density. The predominant soil conditions at
the site consist mainly of medium dense sands with the exception of the 7.0 m thick interbedded fat clay layer (very typical of this region) and some transitional zones from silty clays to silty sands of relative densities lower than 40%. The figure also shows the approximate location of the shallow groundwater table found at the project site.

Figure 40. Summarized subsurface conditions at site C (after Turkel et al., 2021).

Note: NGVD= National Geodetic Vertical Datum.

3.4. Field Equipment Installation

This section presents details of the field equipment installation at sites A and B to measure ground deformations and ground vibrations during pile driving. As shown in Table 10, sites A and B were the projects selected in this research to collect data regarding the above-mentioned phenomena.

3.4.1. Site A (SR 44 over St. John's River)

The field measurements at this construction site consisted of ground deformations measured with the survey equipment and ground vibrations measured with the geophones. The
measurements during driving of pile 10 were performed by using survey nails at the locations of the geophones. Figure 41 presents the equipment installed at site A during driving of pile 10. A survey nail was installed very close to the sheet piles to measure the effects of the temporary sheet piles around pile 10.

Figure 41. Field equipment installed during driving of Pile 10 at site A.

Figure 42a presents the survey equipment layout during driving of pile 13. The ground deformations were measured at the location of the nine geophones available at the University of Central Florida. The closest geophone (G1) was located 3.0 m (10 ft) away from the pile. Geophones G1 through G6 were spaced at 1.5 m (5 ft). Three geophones (i.e., G7 through G9) were placed 17.4 m (57 ft) away from the pile to capture vibrations and deformations on the free-
field zone. Figure 42b presents the survey equipment layout during driving of pile 10. The survey nails were located in front of pile 13 due to restrictions from the sheet pile installed around of pile 10. In this case, all the deformation points were located close to the pile. The closest survey nail was placed 2.1 m (7.9 ft) away from pile 13 and 5.4 m (17.2 ft) away from the center of pile 10.
Figure 42. Plan view of the instrumentation layout showing location of geophones, survey nails, and survey stations used to collect data during driving of: (a) pile 13 and (b) pile 10 at site A.
3.4.2. Site B (Wekiva Parkway Section 6)

The nine 5.0 Hz geophones shown in Section 3.1.1 were used to measure ground vibrations levels outside the cofferdam installed for the construction of the bent. The location of the geophones were also used as settlement points to control the ground movements with the 3 survey equipment stations (see Figure 43a). Additionally, a settlement plate was located at the same distance as the first geophone (i.e., G1) to control potential ground surface deformations as close as possible to the cofferdam (see Figure 43b).

![Figure 43. Field equipment installed at site B: (a) survey station and (b) settlement plate and geophones installed in the field.](image)

Figure 44 presents the layout of the geophones installed in the field. The cofferdam was located approximately 3.0 to 5.0 m away from the test pile. The closest location that the geophones were allowed to be installed was 3.4 m (11.3 ft) from the face of the cofferdam to satisfy safety requirements by the contractor. The remaining geophones in the array were placed at a separation...
of either 1.0 or 2.4 m (3.0 or 8.0 ft) from each other. The spacing of 2.4 m (8 ft) was necessary to allow construction trucks and equipment to drive through the project site prior to the pile driving process.

The surface wave attenuation capabilities provided by the cofferdam and its stiffening effect during installation and pile driving process caused low levels of PPVs and negligible ground deformations. The estimated accuracy of the survey equipment stations to measure ground deformations is estimated to be approximately 1/8 of an inch.

Figure 44. Plan view of the instrumentation layout showing location of geophones, survey nails, and survey stations used to collect data during driving of pile 12 at site B.
3.5. Dynamic Test Pile Measurements at Site C

For this research, it was important to accurately quantify the transmitted forcing function and energy applied to the top of the pile since it has been shown that these two variables largely affect the pile dynamics and the adjacent soil response. Pile Driving Analyzer results were obtained from District 5 engineers at the FDOT for the construction of the bridge at site C. The force obtained from PDA results at the top of the pile during the dynamic test pile for a single hammer blow is shown in Figure 45. The peak applied force was approximately 1600 kips (7117 kN) and the complete hammer blow was applied in approximately 125 milliseconds.

![Figure 45](image)

Figure 45. Measured impact force at the top of pile 1 of pier 11RT for the 180th hammer blow at a penetration depth of 88.9 ft (27.1 m) at Site C.

Figure 46 presents the measured pile penetration during the driving process for test pile 1 at site C and the CAPWAP penetration results from the foundation reports. Only the pile penetration due to the applied hammer blows is presented, thus the pre-drilled length of 9.7 m is not considered (i.e., initial pile penetration value set to zero). The figure presents a good match between the measured data in the field and the CAPWAP model used to determine the forcing
function on top of the pile. Notice that after 1173 hammer blows and a penetration depth of approximately 14.0 m (46.0 ft) the driving process changed due to a change in the fuel settings in the hammer. This is evidenced by the change in slope at approximately 1173 hammer blows. A total of 1822 blows were necessary to drive the pile 17.1 m (56.0 ft) below the pre-drilled depth.

Figure 46. Pile driving process at pile 1, pier 11RT, in terms of hammer blows necessary to reach a penetration below the pre-drilled depth at Site C.

3.6. Measurements of Ground Deformations

3.6.1. Site A (SR 44 over St. John's River)

Figure 47 presents the ground deformation time history during driving of pile 13 at each control point (i.e., P1 through P9). Positive values express heave while negative values represent settlement. Notice that a maximum settlement of approximately 30 mm occurred at the location of P1 and P2 which were located at distances from the center of the pile of approximately 3.4 m (11 ft) and 4.9 m (16 ft), respectively. The settlement decreased after that point as the pile penetrated deeper into the ground.
Figure 47. Ground deformations time histories during driving of pile 13 at site A.

Figure 48 presents the final ground deformations measured during driving of pile 13. Notice that close to the pile a maximum settlement of approximately 20 mm occurred. Settlements are negligible at a distance of approximately 8.0 m. The attenuation properties of the soil in terms of deformations at the site are considerable. Recall that the predominant soil conditions at this site are sandy soil layer with varying relative densities and that the pile was predrilled 22 ft below the ground surface.

Figure 48. Final ground deformations induced by driving of pile 13 at site A.
Figure 49 presents the ground deformation time history during driving of pile 10 at the survey nails. Driving of pile 10 at pier 3 occurred after pile 13 was driven. First, single hammer blows were applied every 2 or 3 minutes up to 1500 seconds. This change in the hammer blow application rate can be observed in the figure at approximately 1500 to 1600 s from the start of driving. The hammer cushion was changed at approximately 4000 s, which can be noticed in the sudden increase in the magnitude of heave for settlement point P13 and P14. Mostly ground surface settlement was measured at P17 (i.e., survey nail close to the sheet piles) with a maximum settlement of approximately 5.8 mm. This can be attributed to the relative position of the settlement point P17 with the sheet piles and their attenuation characteristics.

![Figure 49. Ground deformations time histories during driving of pile 10 at site A.](image)

Figure 50 presents the final ground deformation profile after driving pile 10. The residual vertical displacements at the end of the pile driving process are shown in the figure. A maximum heave of approximately 11 mm occurred at a distance of 8.4 m away from the pile. Installation of pile 10 caused mostly heave at the ground surface in relation to the ground deformations during
installation of driving pile 13. This can be attributed to the densification process after driving pile 13, thus causing dilation (i.e., volumetric expansion) during installation of pile 10.

![Figure 50](image.png)

Figure 50. Final ground deformations induced by driving of pile 10 at site A.

3.6.2. Site B (Wekiva Parkway Section 6)

Ground deformation measurements were performed at site B at the location specified in Section 3.4.2. For that project, negligible measurements of ground surface deformations (heave or settlement) were recorded. This is attributed to the presence of the cofferdam installed prior to the beginning of the driving process. The cofferdam around the pier not only densified the soil during installation but also provided a protection barrier that caused energy absorption of the cylindrical and spherical waves emanating from the pile. This result is also consistent with the measurements of peak particle velocities conducted at the project site.
3.7. Measurements of Ground Vibration

This section presents PPV measurements performed at sites A and B during pile driving operations. The ground vibrations were measured by using the nine 5 Hz vertical geophones available at the University of Central Florida (see Section 3.1.1). Vibrations reported by Bayraktar et al. (2013) are also presented in this section for sites D through H.

3.7.1. Site A (SR 44 over St. John’s River)

Figure 51 presents the PPV measurements performed during driving of piles 10 and 13 at site A. Notice that PPVs at distances up to 10.0 m are not reported. This is attributed to issues related to the established acquisition gain, thus limiting the maximum recorded values to a threshold of 7.0 mm/s. For distances beyond 10.0 m the PPV values did not reach the threshold by FDOT of 12.5 mm/s (0.5 in/s). Notice how the recorded PPV values were larger during driving of pile 13 than during driving of pile 10. This indicates changes in the soil attenuation characteristics (i.e., changes in volumetric contractive or dilative responses of soils) due to pile driving vibrations.

![Graph showing PPV measurements at Site A during driving of piles 10 and 13.](image)

Figure 51Figure 52. PPV measurements at Site A during driving of piles 10 and 13.
3.7.2. Site B (Wekiva Parkway Section 6)

Figure 52 presents the measurements of PPV throughout the velocity time histories of each geophone during the pile driving process (see Figure 52a and Figure 52b). The driving process was divided into two stages since the contractor stopped to adjust driving settings in the middle of the process, thus the results are presented in two figures. Most of the driving process occurred during the first stage (see Figure 53a) than the second stage (see Figure 53b). Notice that higher velocity values were measured during the second stage due to changes in the fuel settings.

Figure 52. Velocity time history for: (a) first pile driving stage at a typical geophone (G1) and (b) second pile driving stage at a typical geophone (G6).

Figure 53 presents the PPV measured at different distances from the face of the cofferdam at during the two stages. The PPV values were calculated based on the maximum velocity in the time history for each geophone (e.g., in Figure 52a the PPV value is 2.1 mm/s). The maximum vibration level in terms of PPV for the first driving sequence was approximately 2.1 mm/s (0.08 in/s), measured at a distance of 3.4 m (11.3 ft) from the face of the cofferdam (i.e., G1). In the second driving sequence, the maximum recorded value was approximately 4.3 mm/s (0.17 in/s) at a distance of approximately 10.0 m (30.0 ft) from the cofferdam (i.e., G6). Larger PPV values were experienced during the second stage of the driving process due to changes in the driving
settings. The recorded measurements did not exceed the PPV threshold of 12.5 mm/s (0.5 in/s) established by FDOT. The pre-drilling operation and the presence of the cofferdam had a major attenuation effect on the final measured vibration response and ground surface deformations generated by the pile driving process. The presence of cofferdams or other underground geostructures have a major impact on the vibrations and settlements induced by pile driving operations.

Figure 53. PPV measurements for the (a) first and (b) second part of the driving process at site B.

3.7.3. Sites D through H

Bayraktar et al. (2013) used Equation (3) proposed by Wiss (1981) to fit the PPV attenuation curves for each site. Table 11 presents the rated and transferred energies of the hammers used as well as the coefficients $k$ and $n$ found for each project. It is interesting to note that the ratio between transferred and rated energy (i.e., energy efficiency) varied between 10% and 32%.
Table 11. Source energy and attenuation coefficients for selected Central Florida projects. (Adapted from Bayraktar et al., 2013).

<table>
<thead>
<tr>
<th>Site</th>
<th>Rated Energy (kip-ft)</th>
<th>Transferred Energy (kip-ft)</th>
<th>Energy Efficiency (%)</th>
<th>k</th>
<th>n</th>
</tr>
</thead>
<tbody>
<tr>
<td>Z.1</td>
<td>161.5</td>
<td>16.8</td>
<td>10.4</td>
<td>1.4</td>
<td>1</td>
</tr>
<tr>
<td>Z.2</td>
<td>100.0</td>
<td>18.0</td>
<td>18.0</td>
<td>1.3</td>
<td>1</td>
</tr>
<tr>
<td>Z.3</td>
<td>80.0</td>
<td>21.2</td>
<td>26.5</td>
<td>3.4</td>
<td>1</td>
</tr>
<tr>
<td>Z.4</td>
<td>100.0</td>
<td>16.8</td>
<td>16.8</td>
<td>3.3</td>
<td>1</td>
</tr>
<tr>
<td>Z.5</td>
<td>84.1</td>
<td>26.9</td>
<td>32.0</td>
<td>6.7</td>
<td>1</td>
</tr>
</tbody>
</table>

Figure 54 shows the PPV attenuation curves derived for each project by Bayraktar et al. (2013). These curves represent the upper limits for the PPV values recorded at each project. The scaled distance was defined using the transfer energy instead of the rated energy of the hammer. Notice that the project with the largest transfer energy of 26.9 kip-ft (i.e., site H) presented the attenuation curve with the higher PPV values. So direct proportionality between rated energy and PPV was found.

![Figure 54](image-url)  
Figure 54. Peak Particle Velocity attenuation curves measured along selected Florida's Turnpike projects corresponding to Sites D to H. (Adapted from Bayraktar et al., 2013).
4. NUMERICAL MODELING OF PILE DRIVING INDUCED VIBRATIONS AND DEFORMATIONS

This chapter consists of two sections that include: 1) a comparison between numerical modeling approaches by using the field data presented in Section 3.3 and 2) a study of the effects of the sand relative density on the ground response to pile driving based on the information gathered from the projects explained in previous chapters. The numerical approaches compared in this study are continuous and discontinuous pile driving approaches. A discontinuous modeling approach consists of installing the pile at different depth and applying a single hammer blow to compute ground vibrations in the soil continuum. A continuous modeling approach consists of driving the pile uninterrupted until the final penetration depth.

The selection of the most adequate numerical and constitutive models to study ground deformations arising from pile driving and the understanding via wave equation analysis of soil-pile interactions and dynamics are the main goals of this section. The pile driving demands applied to the models (i.e., forcing function) were numerically simulated using the wave equation analysis program GRLWEAP. This program only allows the calculation of a detailed time history of displacements, velocities, forces, and energies in the pile for a single hammer blow. Hence, the pile driving process was also modeled in the finite element (FE) platform PLAXIS 2D to draw conclusions about the relationships between input energy, ground deformations, peak particle velocities, distance from the source, and soil properties. Input parameters for the PLAXIS 2D models presented herein were estimated from subsurface exploration data and processed using GRLWEAP.
4.1. Comparative Analysis of Modeling Approaches

The pile driving process of the project at site C presented in section 3.5 was numerically modeled in order to compare both continuous and discontinuous modeling approaches. This comparison allowed to elucidate the advantages of each approach and define the most suitable for this research. The wave equation analysis program GRLWEAP and the FE program PLAXIS 2D were used in the analyses shown herein. Details about this study can also be found in Turkel et al. (2021).

4.1.1. GRLWEAP Pile Driving Model

In the wave equation analysis program GRLWEAP, the soil profile was generated based on SPT-N values for each stratum presented in Figure 40. Soil parameters such as quake for the shaft and toe (i.e., 5.59 mm and 6.60 mm., respectively) and damping for the shaft and toe resistances (i.e., 0.69 s/m and 0.29 s/m, respectively) were obtained from the CAPWAP results presented in the project foundation reports (see Figure 45). The type of hammer and pile dimensions were defined from actual pile driving conditions considering the information presented in Section 3.2.3. Since the PDA measurement during the dynamic test was performed for the 1810th blow (i.e., 27.1 m penetration depth), then the properties for the pile cushion were assigned as “used” plywood material to model the thickness reduction in the cushion at the end of driving according to the GRLWEAP manual (PDI, 2005). A thickness of 38.1 mm. for the hammer cushion and 381 mm for the pile cushion were used.

A GRLWEAP driveability analysis was performed for a penetration depth of 26.9 m for a load-bearing capacity of 8109.1 kN to obtain a forcing function similar to the measured force with the PDA shown in Figure 45. Figure 55 presents a comparison between the GRLWEAP model and
the field measurements in terms of the applied stress at the top of the pile versus time processed by CAPWAP for a single hammer blow. This input demand was applied in terms of a uniformly distributed stress acting on top of the pile and was computed by dividing the measured and computed force by the area of the pile (i.e., 0.37 m²). Since the applied stress history at the top of the pile obtained with GRLWEAP matched well the one measured with PDA, especially in terms of peak magnitude and overall shape, the forcing function was converted into a stress function to be distributed on top of the pile in PLAXIS 2D.

![Stress function time history applied at the top of the pile for a single hammer blow.](image)

A second driveability analysis was performed in GRLWEAP to compare the results of the discontinuous model with the continuous pile driving analysis. In order to define the “wished-in-place” pile penetration depth, the 667th hammer blow was selected, which corresponds to a penetration depth of 23.4 m with an ultimate capacity of 1112.1 kN. This pile penetration depth was modeled in GRLWEAP and PLAXIS 2D to compare the results of pile dynamics in light of the measured field data.
4.1.2. Finite Element Model for Driving Process at Site C

The numerical model in PLAXIS 2D was performed under axisymmetric conditions to simulate the driving of the test pile 11 at Pier 11RT of Site C. Figure 56a shows the model geometry. The model mesh was 44.2 m long and 54.0 m wide. Normally fixed boundaries were defined for the right and left boundaries, while fully fixed conditions were selected for the bottom. Additionally, viscous boundaries were used at the right and bottom ends to avoid reflected waves affecting the computed results. Fifteen-node triangular elements and a medium-mesh option were used.

The concept of a “plastic zone” with reduced stiffness and strength was introduced around the pile to model the continuous process of pile installation. The soil-pile interaction was modeled by adding a soil cluster around the pile with reduced strength ($R$) and shear wave velocity ($R_s$) parameters instead of defining an interface element between soil and pile. This method was proposed by Grizi et al. (2018), since interface elements in PLAXIS 2D led to difficulties when a dynamic stage is conducted. The radius of the plastic zone was defined to be twice the diameter of the pile (i.e., 1.2 m for a 0.61 m-prestressed concrete pile), which is the same ratio used by Grizi et al. (2018) that used a plastic zone of 0.15 m for a laboratory test performed in a pile of 76 mm diameter. However, instead of defining an $R$ value of 0.5 and an $R_s$ of 0.2 as suggested by Grizi et al. (2018), this study used factors of 0.4 and 0.12 for $R$ and $R_s$, respectively.

Figure 56b presents a detailed view of the pile and the “plastic zone” clusters defined to represent the soil-pile interaction. Since the pile was first pre-drilled up to a depth of 9.7 m before the pile driving process started, the pile cluster was activated in the model at that depth instead of beginning the driving process from the ground surface. The water table was placed at the ground
surface. For the discontinuous model, the only parameter that changed was the initial depth of pile penetration from 9.7 m to 23.2 m.

![Diagram showing pile driving model in PLAXIS 2D](image)

Figure 56. Continuous pile driving model in PLAXIS 2D: (a) model geometry and (b) detailed view of the pile initial penetration depth.

The HS small model explained in Section 2.4.3 was used as the constitutive soil model due to its capabilities to include small-strain soil stiffness, and adequate hysteretic soil behavior in the analysis. The constitutive model has been successfully used in various types of soils (e.g., Grizi et al., 2018; Obrzud, 2010). The correlations with the relative density of the sand layers presented by Brinkgreve et al. (2010) were used to calculate the parameters of the granular layers. For the clay layer that underlies the top sand layer, the parameters were based on an $S_u$ of 110 kPa corresponding to a medium-stiff clay. The remaining HS small soil parameters for the clays were defined based on similar clayey soils presented in the technical literature (e.g., Likitlersuang et al. 2013; Surarak et al. 2012).
Table 12 presents the selected parameters for both the plastic zone and the zone of soil continuum away from the plastic zone as it approaches free field conditions, labeled as “free-field zone”. The strength (i.e., \( \phi' \), \( c' \), \( S_u \), and \( \psi \)) and stiffness parameters (i.e., \( E_{50}^{ref} \), \( E_{oed}^{ref} \), \( E_{ur}^{ref} \), and \( G_0^{ref} \)) for the soil cluster corresponding to the plastic-zone were affected by the reduction factors \( R \) and \( R_s \), respectively. A uniform Rayleigh (\( \zeta \)) damping of 5% was applied in the model to supplement the constitutive model hysteretic damping. This damping was applied to both zones by means of the Rayleigh mass (\( \alpha \)) and stiffness (\( \beta \)) proportional damping coefficients. As proposed by Hudson et al. (1994), \( \alpha \) and \( \beta \) were determined by estimating the natural frequency of the soil layers.

Table 12. Soil layer properties used for the HS small model in PLAXIS 2D.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Units</th>
<th>Medium Dense Sand</th>
<th>Medium Stiff Clay</th>
<th>Loose Sand</th>
<th>Dense Sand</th>
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<th>Plastic Zone</th>
<th>Medium Stiff Clay</th>
<th>Loose Sand</th>
</tr>
</thead>
<tbody>
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<td>Thickness</td>
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<td>16.0</td>
<td>6.3</td>
<td>7.0</td>
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<tr>
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<td>(%)</td>
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<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.12</td>
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<tr>
<td>( \gamma_{at} )</td>
<td>kN/m³</td>
<td>20.0</td>
<td>19.0</td>
<td>19.7</td>
<td>20.4</td>
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<td>19.0</td>
<td>19.7</td>
<td></td>
</tr>
<tr>
<td>( \phi' )</td>
<td>°</td>
<td>35.5</td>
<td>28.0</td>
<td>33.6</td>
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<tr>
<td>( \psi )</td>
<td>°</td>
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<td>-</td>
<td>3.6</td>
<td>8.6</td>
<td>2.2</td>
<td>-</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>( c' )</td>
<td>kPa</td>
<td>1.0</td>
<td>-</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>-</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>( S_u )</td>
<td>kPa</td>
<td>-</td>
<td>110.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>44.0</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>( E_{50}^{ref} )</td>
<td>kPa</td>
<td>36000</td>
<td>9500</td>
<td>27000</td>
<td>51000</td>
<td>518</td>
<td>137</td>
<td>389</td>
<td></td>
</tr>
<tr>
<td>( E_{oed}^{ref} )</td>
<td>kPa</td>
<td>36000</td>
<td>12000</td>
<td>27000</td>
<td>51000</td>
<td>518</td>
<td>173</td>
<td>389</td>
<td></td>
</tr>
<tr>
<td>( E_{ur}^{ref} )</td>
<td>kPa</td>
<td>108000</td>
<td>30000</td>
<td>81000</td>
<td>153000</td>
<td>1555</td>
<td>432</td>
<td>1166</td>
<td></td>
</tr>
<tr>
<td>( G_0^{Ref} )</td>
<td>kPa</td>
<td>100800</td>
<td>70000</td>
<td>90600</td>
<td>117800</td>
<td>1452</td>
<td>1008</td>
<td>1305</td>
<td></td>
</tr>
<tr>
<td>( m )</td>
<td>0.5</td>
<td>0.7</td>
<td>0.6</td>
<td>0.4</td>
<td>0.5</td>
<td>0.7</td>
<td>0.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \nu_{ur} )</td>
<td>0.3</td>
<td>0.2</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
<td>0.2</td>
<td>0.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \gamma_{0.7} )</td>
<td>x10⁻⁴</td>
<td>1.40</td>
<td>9.95</td>
<td>1.55</td>
<td>1.15</td>
<td>1.40</td>
<td>9.95</td>
<td>1.55</td>
<td></td>
</tr>
<tr>
<td>( \alpha )</td>
<td>2.7</td>
<td>2.6</td>
<td>1.9</td>
<td>2.0</td>
<td>0.7</td>
<td>0.6</td>
<td>0.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \beta )</td>
<td>x10⁻⁴</td>
<td>9.4</td>
<td>9.2</td>
<td>6.7</td>
<td>6.9</td>
<td>2.6</td>
<td>1.9</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>( R_f )</td>
<td>0.9</td>
<td>1.0</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>1.0</td>
<td>0.9</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The forcing function that was obtained from GRLWEAP was applied as a stress function on top of the pile in PLAXIS 2D (see Figure 55). Three stages were applied in PLAXIS 2D to
perform the continuous analysis. The first stage was applied to initialize the stress field of the soil layers so that representative $K_0$-conditions in the field can be simulated before the pile driving process started. Subsequently, the pile cluster was activated at the pre-drilling depth of 9.7 m. The third stage included the activation of the plastic soil cluster and the application of 1824 hammer blows at the top of the pile using the stress forcing function. A time interval of 1 s between blows was implemented in the analysis. For the discontinuous model, the first two stages were also used with an installation depth of 23.2 m instead of 9.7 m, since it was the selected installation depth for the GRLWEAP analysis. However, the third stage only involved a single hammer blow.

4.1.3. Numerical Model Results

The results obtained from the GRLWEAP and PLAXIS 2D numerical analyses of the driving process are presented in this section. It was possible to analyze the influence of the geometry and the parameters of the plastic zone on the pile driving process, since the pile driving log was provided by the FDOT (see Figure 46). Table 13 presents four different sets of reduction factors defined in this study to investigate such influence. Model $A$ is considered as a baseline model in this study. In order to analyze the separate effects of $R$ and $R_s$, models $B$ and $C$ were created by varying each factor separately. Model $D$ used the same reduction factors proposed by Grizi et al. (2018).

<table>
<thead>
<tr>
<th>PLAXIS 2D Model</th>
<th>Strength Reduction Factor ($R$)</th>
<th>Shear Wave Velocity Reduction Factor ($R_s$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model $A$</td>
<td>0.40</td>
<td>0.12</td>
</tr>
<tr>
<td>Model $B$</td>
<td>0.40</td>
<td>0.20</td>
</tr>
<tr>
<td>Model $C$</td>
<td>0.50</td>
<td>0.12</td>
</tr>
<tr>
<td>Model $D$</td>
<td>0.50</td>
<td>0.20</td>
</tr>
</tbody>
</table>
The continuous numerical model was validated by comparing the results versus the actual pile driving records from the foundation reports in terms of vertical displacements at the top of the pile as a result of 1,824 hammer blows. Figure 57 presents the computed and measured number of blows versus pile penetration. As mentioned in Section 3.5, the measured data shows after 1173 blows that the penetration depth increased suddenly due to changes in the fuel settings of the hammer reported in the pile driving record log of the project. However, the input forcing function in the numerical models was not modified to allow for changes in the fuel setting affecting the stress forcing function to perfectly match the measured pile penetration process. A comparison of the numerical results is presented in terms of pile penetration for the different sets of parameters adopted for the plastic zone. This is to highlight its importance in the numerical modeling framework, in particular when a model like HS small is used. It is observed that the model A, selected as the base model, matches very well the measured data up to the point of change in the fuel setting. As expected, it is found that as the reduction factors increased, the pile penetration decreased. Comparing model D with models B and C, it is concluded that the shear wave velocity factor has a greater effect on the driveability of the pile than the strength reduction factor.

Figure 57b presents the results obtained after analyzing the influence of the size of the plastic zone (r) on the pile penetration process. Since model A matched well the pile penetration process, this set of parameters was used for the rest of the comparisons. Notice how an increase in the width of the plastic zone increased the pile penetration as well. The assumption of having a plastic zone radius of twice the diameter of the pile is in good agreement with the measured penetration and also matches the values proposed by Grizi et al. (2018). The selection of numerical input parameters for this highly disturbed zone near the pile, idealized in this first finite element
model as a plastic zone, must be performed as a function of the type of soil, pile properties (i.e., geometric and material), and characteristics of the input source.

![Graphs showing pile penetration and hammer blow number for different models and plastic zone sizes.](image)

Figure 57. Effects of: (a) plastic zone parameters on the pile driving process and (b) size of the plastic zone on the pile penetration.

The discontinuous model was also performed in PLAXIS 2D at the desired depth of 23.2 m since the continuous model was validated by means of the pile penetration process. The set of parameters corresponding to model A and the size of the plastic zone of 1.2 m were used. The comparison between the two FE modeling approaches and GRLWEAP regarding the pile dynamics for a single blow applied at the top of the pile is presented in Figure 58. The time history of vertical velocities at the top of the pile for the 667th hammer blow is shown in Figure 58a. The three models obtained approximately the same peak velocity of 2.5 m/s at the top of the pile. However, a better representation of the GRLWEAP time history was obtained using the continuous modeling approach as opposed to the discontinuous model. Figure 58b presents the vertical displacement time history computed with GRLWEAP and both modeling approaches in PLAXIS 2D. The continuous model was able to represent the residual vertical displacements as a result of
a single hammer blow, while the discontinuous approach resulted in larger displacements. These differences in the discontinuous approach can accumulate and provide misleading results when the entire pile driving process is modeled. Despite differences in the shape of the time history results of vertical displacements, the continuous model provides very similar results that GRLWEAP in terms of both displacements and velocities. This is attributed to the accuracy in the numerical representation of the state of stresses generated during the pile driving process when continuous pile driving models are used.

![Graph showing computed pile dynamics from GRLWEAP](image)

**Figure 58.** Comparison of continuous and discontinuous numerical approaches with computed pile dynamics from GRLWEAP in terms of: (a) vertical velocity at the top of the pile and (b) vertical displacement at the top of the pile.

Figure 59 presents the PPV values at various distances on the ground surface away from the pile obtained with the continuous numerical analysis. This data is compared with the historical records of PPV data collected by Bayraktar et al. (2013) which were presented in Section 3.7.3. For the calculation of scaled distance, the maximum transferred energy of the hammer provided in Table 11 was used. The figure shows how the computed PPVs from the numerical model reasonably matched the attenuation curve boundaries provided by Bayraktar et al. (2013).

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Figure 59. Peak Particle Velocity attenuation curves computed along selected Florida's Turnpike projects (Adapted from Bayraktar et al., 2013).

4.2. Analysis of Variables Involved

A study analyzing the effects of the main variables involved in the pile driving problem (i.e., soil properties, peak particle velocity, distance from the pile, and input energy) was performed by gathering information from the projects described in Chapter 3. This analysis was performed to investigate the predominant soil conditions and commonly used driving hammers in Central Florida. Based on the soil profiles summarized in Chapter 3, it was observed that the soils in the area are mainly characterized by the presence of granular soils with relative densities varying from loose to medium dense. A parametric study was performed to investigate the effect of various relative densities for these sandy soils and various hammer types commonly used in Florida on the continuous pile driving process of a prestressed concrete pile. The analyses are presented using a combination of numerical models conducted in GRLWEAP and PLAXIS 2D. The hypoplasticity model for sands was used in this parametric finite element model instead of modeling the soil cluster with the HS small constitutive model and using a “plastic zone” around the pile, as
previously detailed for the comparative analysis. In this new modeling strategy, the “plastic zone” was not necessary. It was found possible to directly evaluate the pile-driving induced effects using the hypoplasticity model for sands since soil dilative and contractive behaviors under dynamic loadings are more accurately captured with this model.

4.2.1. Different Type of Hammers used in Central Florida

Information regarding typical hammers in Florida was collected for the analysis of the input energy during pile driving operations to estimate sources of energies and applied forces/stresses in the numerical models. A total of 25 pile driving projects along Florida’s Turnpike were collected by Heung et al. (2007). Information regarding hammer type and number of projects where the hammer was used are presented in Table 14. The hammers used were mostly diesel-type hammers. The DELMAG D36-32 and the ICE 120-S were the most common hammers surveyed. It is important to note that most of the case histories involved the use of large displacement prestressed concrete piles (PCP) with sizes ranging from 457 to 762 mm (18 in. to 30 in). Furthermore, only two projects used small-displacement piles (i.e., HP-piles) as shown in the table. However, information of the transmitted energy to the pile and appurtenances such as cushions or other appurtenances are not reported by Heung et al. (2007).
Table 14. Typical hammer types used in Florida projects based on Heung et al. (2007).

<table>
<thead>
<tr>
<th>Hammer Type</th>
<th>Number of Projects</th>
<th>Rated Energy (kip-ft)</th>
<th>Type of pile used</th>
</tr>
</thead>
<tbody>
<tr>
<td>D36-32</td>
<td>9</td>
<td>90.56</td>
<td>PCP</td>
</tr>
<tr>
<td>ICE 120-S</td>
<td>4</td>
<td>120.00</td>
<td>PCP</td>
</tr>
<tr>
<td>ICE 100-S</td>
<td>3</td>
<td>100.00</td>
<td>PCP</td>
</tr>
<tr>
<td>ICE 80-S</td>
<td>3</td>
<td>80.00</td>
<td>PCP and HP-pile</td>
</tr>
<tr>
<td>D30-02</td>
<td>3</td>
<td>66.20</td>
<td>PCP</td>
</tr>
<tr>
<td>D46-32</td>
<td>2</td>
<td>122.19</td>
<td>PCP</td>
</tr>
<tr>
<td>D62-22</td>
<td>1</td>
<td>164.60</td>
<td>PCP</td>
</tr>
<tr>
<td>D30-32</td>
<td>1</td>
<td>75.44</td>
<td>PCP</td>
</tr>
<tr>
<td>ICE I-19</td>
<td>1</td>
<td>43.24</td>
<td>HP-pile</td>
</tr>
</tbody>
</table>

4.2.2. GRLWEAP Pile Driving Model

Driveability analyses were performed in GRLWEAP prior to running the finite element models so that forcing functions for the various hammer models could be obtained and input in PLAXIS 2D. The same cushion and pile properties of the comparative analysis (see Section 4.1.1) were defined in the numerical models. The soil profile was defined in GRLWEAP using two layers consisting of a 30.5 m thick dense granular layer underlain by a 53.4 m thick very dense competent granular soil strata for the analyses. The driveability analyses were performed for the same penetration depth (i.e., 9.7 m) for the piles with the same load-bearing capacity (i.e., 8109.1 kN) as they were used in Section 4.1.1. Thus, forcing functions were obtained for the typical hammer types in Florida which were introduced in Table 14. In this thesis, the two most commonly used hammers (i.e., DELAMG D36-32 and the ICE 120-S) were analyzed.

Figure 60a presents the forcing functions at the top of the pile created by a single blow of these hammers. The hammer used in the comparative analysis (i.e., APE D70-52) is also presented in the figure. Based on this analysis, the APE D70-52 hammer applies the highest peak force at the top of the pile while the DELMAG D36-32 applies the lowest peak force at the top of the pile.
Figure 60b presents the energy functions transmitted to the top of the pile by a single hammer blow for each type of hammer. Similar to the results for the forcing function at the top of the pile, the APE D70-52 and DELMAG D36-32 hammers transmitted the highest and lowest energies at the top of the pile, respectively. The forcing function of the hammer blows that were obtained from GRLWEAP was input in PLAXIS 2D as a stress function distributed on top of the pile by dividing the forcing function by the area of the pile (see Figure 60a).

![Figure 60](image.png)

Figure 60. Analysis of commonly used hammers in Central Florida in terms of: (a) forcing functions at the top of the pile, and (b) transmitted energy at the top of the pile.

4.2.3. Finite Element Model for Parametric Study

The selected hammers were used to conduct finite element analyses in PLAXIS 2D and investigate the pile driving induced effects on the surrounding soils in terms of ground surface deformations and vibrations. Since the subsurface conditions in Central Florida are mainly characterized by the presence of granular materials varying with relative densities from loose to medium dense conditions (i.e., based on the field data shown in Chapter 3), the numerical studies presented in this chapter were enhanced by investigating the effects of various relative densities
of the soils (i.e., 25%, 40%, 55%, 60%, and 70%) on the final response of the ground in terms of ground vibrations and deformations.

The numerical model was performed under axisymmetric conditions in PLAXIS 2D. Figure 61a shows the model geometry, mesh and idealized soil profile. The finite element mesh had a height and width of 84.2 m and 94.0 m, respectively. Boundary conditions in the model were the same as those used for the comparative analysis in Section 4.1. Fifteen-node triangular elements and a medium-mesh option were used. A soil cluster with a refined mesh having a height of 21.2 m and width of 20.0 m was created around the pile to improve the accuracy of the numerical results close to the pile.

Figure 61b presents a detailed view of this refined soil cluster which had a mesh coarseness factor of 0.25 in PLAXIS 2D. The mesh coarseness factor describes the ratio of the mesh refinement at the determined soil cluster to the overall mesh coarseness of the model (i.e., in this model the soil cluster was 4 times more refined). The large deformation of the mesh given the continuous nature of the pile driving process was modeled by enabling the updated mesh option in PLAXIS 2D. A similar staged construction process to the comparative analysis in section 4.1 was performed. In the last stage, the driving operation was initiated by applying a total of 1400 hammer blows at the top of the pile. The water table was kept constant at the ground surface during the entire simulation.
Figure 61. Pile driving model used in the parametric study in PLAXIS 2D: (a) model geometry and (b) detailed view of the refined zone and initial pile penetration depth.

The model consists of a 31.2 m thick idealized sand layer with variable relative density as defined in this parametric study on top of a 53.0 m thick very dense competent sand layer. The finite element model matched the conditions defined also in the GRLWEAP model. A relative density of approximately 90% was assigned to the very dense sand layer. A high relative density was selected for the bottom layer to represent a firm layer where the pile driving processes finished since the pile reached a competent bearing stratum.

The forcing function of the hammer blows that were obtained from GRLWEAP was input in PLAXIS 2D as a distributed stress function on top of the pile. Several hammer types, and relative densities of the upper sand layer were considered in the analysis that was performed in a continuous manner as previously discussed in Section 4.1 since that type of analysis was able to better represent the dynamic compressive wave propagations within the pile. The analyses were finalized when the pile reached the bottom competent stratum or because of the large computational effort when approximately 1400 blows were applied. The pile driving process
defined in terms of the number of hammer blows necessary to install the pile in place varied for the different relative densities assigned to the topmost soil layer.

4.2.4. Definition of Soil Parameters

The critical-state based hypoplasticity model for sands presented in Section 2.4.4 was used in this section for the proposed analyses. The hypoplasticity model for sands is preferred in this section over the HS small constitutive model presented in Section 2.4.3, since accurate relationships between the variables involved can be established given its capabilities to perform dynamic analyses (Gudehus et al., 2008). Additionally, the formulations of the hypoplasticity model consider the influence of void ratio ($e$) that enhances the computational capabilities and response of soils subjected to dynamic loadings. This allows the models to provide a better understanding of the mechanical behavior under a wide range of relative densities and confining pressures (Wichtmann et al., 2019).

Relative densities varying between 25% and 70% were assigned to the upper soil layer in this parametric study. Void ratios at a pressure of 0 kPa (i.e., $e_0$ for hypoplasticity model) were calculated corresponding to the selected relative densities by using Equation (14). A maximum void ratio ($e_{max}$ or $e_{d0}$) of 1.10 and a minimum void ratio ($e_{min}$ or $e_{c0}$) of 0.58 were defined for similar soil conditions in terms of relative densities and critical state void ratios based on Zapata-Medina et al. (2019). Table 15 summarizes computed $e_0$ corresponding to each relative density.

$$e_0 = e_{max} - \frac{D_r}{100\%} \ast (e_{max} - e_{min})$$
Table 15. Calculated $e_0$ values corresponding to each relative density.

<table>
<thead>
<tr>
<th>$D_r$ (%)</th>
<th>$e_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>0.97</td>
</tr>
<tr>
<td>40</td>
<td>0.89</td>
</tr>
<tr>
<td>60</td>
<td>0.79</td>
</tr>
<tr>
<td>70</td>
<td>0.73</td>
</tr>
</tbody>
</table>

The methodology conducted by Kim (2011) was followed to calculate the secant shear modulus degradation curves of the upper sand layer based on monotonic triaxial compression tests for each relative density. Figure 62 presents the computed secant shear modulus degradation curves with the selected parameters for the upper sand layer at selected relative densities (i.e., 25%, 40%, 60%, and 70%). The nonlinear behavior of the upper sand layer was studied to match expected dilative or contractive responses. Undrained triaxial compression tests consolidated to $K_0$ conditions ($CK_0U-TXC$) on the soil test module available in PLAXIS 2D were conducted to define hypoplasticity model parameters. An initial cell pressure of 100 KPa and a $K_0$ of 0.5 were applied based on monotonic undrained triaxial tests conducted by Hyodo et al. (1994) on saturated loose Toyoura sand. Thus, a mean confining pressure ($p_s$) of 133 kPa was applied. Based on the numerically simulated triaxial test results using the hypoplasticity sand model for the upper sand layer, the secant shear modulus degradation curves at each relative density were computed.

The definition of soil parameters was conducted aiming to match the computed secant shear modulus degradation curves at each void ratio with other published methodologies presented by Hardin and Drnevich (1972), and Seed and Idriss (1970). Since the void ratio at the $p_s$ is required to plot the soil stiffness degradation of the reference curves, Equation (15) developed by Bauer (1996) was used to calculate void ratio and applied pressure relationships. In the equation, $h_s$ and $n$ represent the granular hardness and an exponent for the grain skeleton, respectively.
\[ e = e_0 \times \exp \left[ -\left(3 \times \frac{p_s}{h_s}\right)^n \right] \]  

Figure 62. Secant shear stiffness degradation curves for the relative densities of: (a) 25\% and 60\%; and (b) 40\% and 70\%.

The adopted set of parameters for the upper sand layer based on the numerically simulated triaxial tests are listed in Table 16. The same reference values proposed by Zapata-Medina et al. (2019) for minimum void ratio at zero pressure (\(e_{d0}\)), critical void ratio at zero pressure (\(e_{c0}\)), and maximum void ratio at zero pressure (\(e_{i0}\)) were used. The remaining basic hypoplastic model parameters (i.e., \(h_s, n, \alpha\) and \(\beta\)), and the size of the elastic range (\(R_{\text{max}}\)) and material constant representing stiffness degradation (\(\beta_r\)) were obtained by fitting the secant shear modulus degradation curves to the reference curves. The remaining intergranular strain concept parameters (i.e., \(m_R, m_T, \chi\)) were also proposed by Zapata-Medina et al. (2019). An input Rayleigh damping ratio of 5\% was defined throughout the analyses.
Table 16. Soil properties used for the Hypoplasticity sand model in PLAXIS 2D.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi_c$</td>
<td>31</td>
<td>°</td>
</tr>
<tr>
<td>$p_t$</td>
<td>0</td>
<td>kPa</td>
</tr>
<tr>
<td>$h_s$</td>
<td>1200</td>
<td>MPa</td>
</tr>
<tr>
<td>$n$</td>
<td>0.37</td>
<td></td>
</tr>
<tr>
<td>$e_{d0}$</td>
<td>0.58</td>
<td></td>
</tr>
<tr>
<td>$e_{c0}$</td>
<td>1.096</td>
<td></td>
</tr>
<tr>
<td>$e_{i0}$</td>
<td>1.315</td>
<td></td>
</tr>
<tr>
<td>$\alpha$</td>
<td>0.05</td>
<td></td>
</tr>
<tr>
<td>$\beta$</td>
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<td></td>
</tr>
<tr>
<td>$m_R$</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>$m_T$</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>$R_{\text{max}}$</td>
<td>$5.00 \times 10^{-5}$</td>
<td>-</td>
</tr>
<tr>
<td>$\beta_r$</td>
<td>0.1</td>
<td></td>
</tr>
<tr>
<td>$\chi$</td>
<td>1.0</td>
<td></td>
</tr>
</tbody>
</table>

Numerically simulated $CK_0U-TXC$ results using the selected parameters are presented in Figure 63. Deviatoric stress ($\Delta q$) and excess pore water pressures ($\Delta u$) are presented versus axial strains ($\epsilon_a$) for various relative densities. A dilative response to soil shearing was the main characteristic for the medium-dense sands (i.e., $D_r=60\%$ and $70\%$). A more contractive response was computed for the loose sands (i.e., $D_r=25\%$ and $40\%$). The overall computed sand response to shearing investigated herein generally matches the results of $CK_0U-TXC$ tests conducted by Hyodo et al. (1994) for saturated Toyoura sands.
Figure 63. Computed Triaxial Test Results ($CK_0U - TXC$): a) $\Delta q$ versus $\epsilon_a$; and b) $\Delta u$ versus $\epsilon_a$.

The constitutive soil model used for the very dense competent sand layer was the HS small model. Correlations with the $D_r$ presented by Brinkgreve et al. (2010) were used to calculate HS small parameters of this bearing stratum. The selected parameters are given in Table 17.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>$\phi'$</th>
<th>$\psi$</th>
<th>$c'$</th>
<th>$E_{s0}^{\text{ref}}$</th>
<th>$E_{oed}^{\text{ref}}$</th>
<th>$E_{ur}^{\text{ref}}$</th>
<th>$G_0^{\text{ref}}$</th>
<th>$m$</th>
<th>$v'_ur$</th>
<th>$\gamma_{0.7}$ ($\times 10^{-4}$)</th>
<th>$R_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value</td>
<td>$39.3$</td>
<td>$9.3$</td>
<td>$1$</td>
<td>$54.0$</td>
<td>$54.0$</td>
<td>$160.0$</td>
<td>$120.0$</td>
<td>$0.42$</td>
<td>$0.3$</td>
<td>$1.1$</td>
<td>$0.89$</td>
</tr>
</tbody>
</table>

4.2.5. Numerical Results

The computed pile penetration process for various relative densities and hammer types is shown in Figure 64. The pile penetration is shown versus the amount of hammer blows required to reach that depth. Only the pile penetration due to the applied hammer blows is presented, thus the pre-drilled length of 9.7 m is not considered (i.e., initial pile penetration value set to zero). As expected, the effort required to install each pile is highly dependent on the relative density of the
soil. Figure 64a shows the results obtained using an APE D70-52 hammer. A total of 770 hammer blows were necessary to drive the pile completely through the soil having a $D_r$ of 25%. Conversely, 1400 hammer blows were necessary to drive the pile to reach the target 18.3 m depth when the $D_r$ was 70%. Figure 64b presents the same type of analysis of pile penetration versus hammer blow when the ICE 120-S hammer was used. When comparing the analyses versus those obtained with the APE D70-52 hammer, more hammer blows were required to drive the pile to reach the same vertical penetration since the input energy of the ICE 120-S hammer is lower than the one from the APE D70-52 hammer (see Figure 64b). Figure 64c presents pile penetrations when the D36-32 hammer was used. Similar results were reached with this hammer than those computed with the ICE 120-S hammer.

![Graphs showing pile penetration vs hammer blow number for different $D_r$ and $e_0$ values for three different hammers: APE D70-52, ICE 120-S, and D36-32.]

Figure 64. Comparison of the vertical penetration during the pile driving process for different hammer types and relative densities: (a) APE D70-52 (b) ICE 120-S (c) D36-32.

Figure 65 presents the PPV values computed at the ground surface versus distance away from the pile. The PPV attenuation curves shown in the figure were compared with the historical
records of measured PPV data performed by Bayraktar et al. (2013). The normalization factor for the scaled distance in the horizontal axis was defined using the maximum transferred energy of the hammers. Additionally, the FDOT threshold of 12.7 mm/s is shown in the figure as a horizontal red dashline. Notice how the computed PPVs reasonably matched the attenuation curve boundaries provided by Bayraktar et al. (2013). Maximum computed PPV values of approximately 1230 mm/s occurred very close to the pile. Observe that a threshold scaled distance of approximately 1.0 \( m/\sqrt{kJ} \) can be defined regardless of the hammer type and relative density considered in the numerical model since PPVs beyond that point lie within the maximum acceptable threshold.

![Graph showing PPV versus scaled distance values compared to the reported boundaries by Bayraktar et al. (2013).](image)

Figure 65. Computed PPV versus scaled distance values compared to the reported boundaries by Bayraktar et al. (2013).

A study of how much ground surface settlement occurs due to the pile driving process, even if PPV requirements by FDOT are followed is presented in this section. Figure 66 presents the maximum computed settlements \( S \) during the pile driving at distances where PPV values satisfied the 12.7 mm/s (0.5 in/s) FDOT threshold (FDOT, 2021) for different relative densities.
The threshold value was met at different distances from the pile, input energies, and relative densities since typical PPV attenuation curves vary as a function of those variables. Those maximum settlements were obtained from the computed settlement time history during the pile driving operations. A linear trendline of settlement for each hammer type is shown in the figure. APE D70-52 and D36-32 trends were similar. Lower settlements were computed with ICE 120-S. Approximately 38 mm to 76 mm of maximum settlement associated with a PPV 12.7 mm/s were observed at the relative density of 25%, which decreased to approximately 25 mm for denser soil profiles (i.e., $D_r = 70\%$).

![Graph showing maximum computed settlement associated with PPV of 12.7 mm/s for various relative densities and input energies.]

Figure 66. Maximum computed settlement associated with PPV of 12.7 mm/s for various relative densities and input energies.

The selected relative densities were grouped into three categories to facilitate the use of the charts and the understanding of the variables involved in the problem. Relative densities were grouped as loose (i.e., 25\%-40\%), medium-dense (i.e., 55\%-60\%), and dense (i.e., 70\%) sands. A total of 107 data points were used for this computation corresponding to 15 numerical simulations reported in this thesis.

Figure 67 presents only the maximum settlement trends created for the loose sand group. Figure 67a provides settlement attenuation curves by plotting settlement versus scaled distance.
Linear regression and maximum computed envelope lines are also shown in the figure. The settlement envelope attenuated from 115 mm for a scaled distance very close to the pile to approximately a negligible 5 mm at a scaled distance of \(4.0 \text{ m}/\sqrt{kJ}\). Figure 67b presents the maximum settlement versus PPV. The threshold level of 12.5 mm/s (0.5 in/s) is indicated with a red dashed vertical line. The settlements located left of that threshold line indicate computed maximum settlements that occurred below the FDOT threshold. Notice that for PPV values less than 0.5 in/s, and regardless of the hammer used for the loose sandy soils, a maximum settlement of approximately 75 mm was computed.

Figure 67. Settlement-PPV-scaled distance trend lines for loose sands in terms of: (a) settlement versus scaled distance and (b) settlement versus PPV.

Figure 68 presents the same type of analysis for the medium-dense sand group. Figure 68a provides the settlement attenuation curves by plotting settlement versus scaled distance. The settlement envelope attenuated from 200 mm for a scaled distance very close to the pile to approximately a negligible 5 mm at a scaled distance of \(4.0 \text{ m}/\sqrt{kJ}\). Figure 68b presents the maximum settlement versus PPV. Note that for PPV values less than 12.5 mm/s (0.5 in/s) and regardless of the hammer used for the medium-dense sandy soils, a maximum settlement of
approximately 50 mm was computed, which is less than the maximum settlement presented for loose sandy soils.

Figure 68. Settlement-PPV-scaled distance trend lines for medium-dense sands in terms of: (a) settlement versus scaled distance and (b) settlement versus PPV.

Figure 69 presents the maximum settlement trends computed for the dense sand group. Figure 69a provides the settlement attenuation curves by plotting settlement versus scaled distance. The settlement envelope attenuated from 70 mm for a scaled distance very close to the pile to approximately a negligible 2.5 mm at a scaled distance of \(4.0 \frac{m}{\sqrt{kJ}}\). Figure 69b presents the maximum settlement versus PPV. Note that for PPV values less than 12.5 mm/s (0.5 in/s) for the dense sandy soils, a maximum settlement varying from approximately 10 to 25 mm was computed which is significantly less than the maximum settlement presented for the loose and medium-dense sandy soil conditions. The computed ground deformation for the dense sandy soils at the PPV threshold of 12.5 mm/s (0.5 in/s) is very sensitive to the hammer used in the numerical model.
Figure 69. Settlement-PPV-scaled distance trend lines for dense sands in terms of: (a) settlement versus scaled distance and (b) settlement versus PPV.
5. SUMMARY AND CONCLUSIONS

5.1. Summary

Field data corresponding to three bridge construction projects across Central Florida were presented in this document. The field data consisted of i) soil profiles based on laboratory, and field testing (e.g., SPT and CPT); ii) dynamic load tests performed at the sites where pile penetration, pile capacities and forcing functions applied to the top of the pile were measured; iii) ground vibrations induced by pile driving measured by using vertical geophones; and iv) ground deformations measured by means of survey equipment. A similar soil profile was observed for all construction sites consisting of sandy layers with varying relative densities interbedded by a thick fat clay layer.

A comparison between two pile driving numerical modeling approaches was performed by using wave equation analysis software (e.g., GRLWEAP and CAPWAP) and the FE program PLAXIS 2D. A continuous pile driving approach consisted of driving the pile uninterrupted up to a desired penetration depth. A discontinuous pile driving approach was performed by activating the pile at different depths and applying a single hammer blow for each depth. Field data corresponding to pile penetration and forces applied to the top of the pile were used to validate both GRLWEAP and FE models. The constitutive soil model used for this comparison was the HS Small available in PLAXIS 2D. The soil parameters were defined based on published correlations that use the relative densities of the sandy soils as the main variable. The soil-pile interaction was modeled by introducing a soil cluster with reduced parameters (i.e., plastic zone) in the vicinity of the pile. A parametric study based on the properties of this plastic zone was also presented.
A continuous numerical pile driving model was also presented to analyze the effects of different variables on the final ground response to pile driving operations (i.e., ground deformations). Such variables included the type of hammer and the transferred energy to the pile, the relative density of the sandy soils, and the vibration levels experienced during pile driving. In order to analyze the effects of the hammer types and their transmitted energies to the pile, a compilation of different hammers used in previous construction projects was presented. The constitutive soil model used for this comparison was the Hypoplasticity model for sands available in PLAXIS 2D. The soil parameters were defined by matching expected cyclic behaviors published in the literature depending on the void ratios of the sandy soils. A total of 107 data points were used for this computation corresponding to 15 numerical simulations reported in this research.

Based on the numerical analyses, different types of charts able to estimate ground deformations are given in this document. Variations of ground deformations with the PPV induced by the pile, relative densities of the soils, and scaled distance from the pile (i.e., normalized with the transmitted energy of the hammer) were also presented.

5.2. Conclusions

Based on the comparative analysis of pile driving numerical approaches, it was concluded that a continuous modeling approach provides a better approximation to the pile response during pile driving than a discontinuous modeling approach. This was attributed to the fact that the former explicitly considers the accumulation of stresses during the pile driving, while the latter always starts in at-rest conditions, thus providing misleading results.

Additionally, it was concluded that adding a plastic zone around the pile can replace the soil-pile interface in PLAXIS 2D. Special attention has to be paid to the reduction factors applied
to this plastic zone, since these are site-specific. The shear wave velocity factor $R_s$ has a larger impact on the pile penetration than the strength reduction factor ($R$), since it directly modifies the stiffness of the soil-pile interface. It was also concluded from this analysis that the appropriate size of the plastic zone should be twice the diameter of the pile in the case of large displacements pile like the ones shown herein.

Based on the analysis of the variables involved in the dynamic process, it can be concluded that larger ground deformations can be expected for loose sandy soils. This might be attributed to a densification process due to pile-driving induced vibrations. The hammer type can affect the final ground deformations by means of the transmitted energy to the pile and the shape of the forcing function. It was concluded that the larger the transmitted energy the larger the computed ground deformations. It has to be noted that the transmitted energy depends also on the driving accessories (e.g., hammer and pile cushions) and the dynamic properties of the soils in contact with the pile.

Furthermore, in cases where vibration levels comply with the thresholds defined by the Florida Department of Transportation (FDOT) large ground deformations can still occur depending on the site-specific conditions. It was interesting to note that regardless of the relative density or the hammer used in the analyses, the PPV threshold of 12.7 mm/s (0.5 in/s) was always computed at a scaled distance of $1.0 \, m/\sqrt{kJ}$. 

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LIST OF REFERENCES


