Pile – Soil Interaction during Vibratory Sheet Pile Driving

a Full Scale Field Study

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Master of Science Thesis 13/05
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Stockholm 2013
Foreword

The work presented in this thesis was carried out between January 2013 and September 2013 at NCC Teknik, Stockholm, and at the Royal Institute of Technology, Division of Soil and Rock Mechanics, Department of Civil and Architectural Engineering. This MSc. thesis concludes my Double Degree of Civil Engineering from the Royal Institute of Technology, Sweden, and the École Centrale de Lyon, France. The work was supervised by Fanny Deckner (KTH/NCC) with assistance from Dr. Kenneth Viking (Grontmij), and was examined by Professor Staffan Hintze (KTH/NCC).

First and foremost I would like to thank my advisors Fanny Deckner and Dr. Kenneth Viking as well as my examiner Professor Staffan Hintze for their guidance and valuable input which helped improve the quality of my work at every step of the way.

I would like to express my gratitude to Kent Allard and Kent Lindgren, retired respectively from Geometrik i Stockholm AB and the KTH Wallenberg Laboratory, for their outstanding work with the field test instrumentation and acquisition systems. I would also like to thank them for taking the time to teach me the subtleties of signal acquisition and sensor calibration.

I also thank the Hercules team from the Solna construction site for being so helpful and patient during the field tests. Furthermore, I would like to thank Anders Rosqvist from Liebherr for his help with the PDE® system.

Finally, I would like to thank the NCC Teknik Geo/Anläggning group for providing a comfortable and supportive work environment.

Stockholm, September 2013

Claire Guillemet
Summary

Urban construction sites require strict control of their environmental impact, which, for vibratory sheet pile driving, can include damage to nearby structures due to ground vibrations. However, the lack of knowledge concerning the generation of soil vibrations makes the prediction of ground vibration levels difficult. This MSc. thesis in particular, focuses on a crucial link in the vibration transfer chain: the sheet pile – soil interface, which is also one of the least documented.

The aim of this thesis is first, to carry out a full-scale field test consisting in the monitoring of sheet pile and ground vibrations during sheet pile vibratory driving. And second, to analyze a selected portion of the collected data with focus on the sheet pile – soil vibration transfer. Both aspects of the thesis work aim, more generally, to contribute to the understanding of ground vibration generation under vibratory sheet pile driving.

The full-scale field study was performed in Solna in May 2013. It consisted in the vibratory driving of seven sheet piles, out of which three were fitted with accelerometers. During the driving, ground vibrations were measured by accelerometers, the closest ones placed only 0.5 m from the sheet pile line. The design and installation of the soil instrumentation was innovative in as much as accelerometers were not only set on the ground surface but also at three different depths (~ 3 m, 5 m and 6 m).

The analysis presented in this thesis is primarily a comparison between sheet pile vibrations and ground vibrations measured 0.5 m from the sheet pile line. The principal aspects considered in the comparison are: the influence of penetration through different soil layers, the sheet pile – soil vibration transfer efficiency, the frequency content of sheet pile and soil vibrations, and differences between toe- and shaft-generated vibrations.

The main conclusions from this study are:

- Most of the vibration loss occurs in the near field: 90-99% of the sheet pile vibration magnitude was dispersed within 0.5 m from the driven sheet pile. Moreover, the sheet pile – soil vibration transfer efficiency was reduced for higher sheet pile acceleration levels and higher frequencies.

- The soil characteristics strongly influence the sheet pile vibration levels. A clear distinction could be made between “smooth” and “hard” driving, the latter being associated with an impact situation at the sheet pile toe.

- The focus of ground vibration studies should not only be the vertical vibrations. Indeed, the ground vibrations’ horizontal component was found to be of the same or even higher magnitude than the vertical component.

Keywords: Ground vibrations, vibratory driving, sheet pile, full-scale field study, soil instrumentation, sheet pile instrumentation.
Sammanfattning


Syftet med detta examensarbete är först att utföra ett fullskaligt fältförsök bestående av spont- och markvibrationsmätningar under vibrering av spont. Det andra syftet är att analysera en utvald del av de insamlade data med fokus på vibrationsöverföring mellan spont och jord. Båda dessa uppgifter syftar till att öka kunskapen kring uppkomsten av markvibrationer i samband med vibrodrivning av spont.

Den fullskaliga fältstudien genomfördes i Solna i maj 2013. Den omfattade vibreringen av sju spontprofiler, varav tre var utrustade med accelerometrar. Under neddrivningen uppmättes markvibrationer med nio accelerometrar, varav de närmsta var placerade endast 0,5 m från spontlinjen. Sanningsättning och installation av markinstrumentering var nyskapande eftersom accelerometrarna inte bara satt på markytan utan också på tre olika djup (~ 3 m, 5 m och 6 m).

Analysen som presenteras i detta examensarbete är framförallt en jämförelse mellan spontvibrationer och markvibrationer uppmätta 0,5 m från spontlinjen. De viktigaste parametrarna som betraktas är: påverkan av olika jordlager, vibrationsöverföring mellan spont och jord, frekvensinnehåll i spont- och markvibrationer, samt skillnader mellan tä- och mantelorsakade vibrationer.

De viktigaste slutsatserna från denna studie är:

- De största vibrationsförlusterna inträffar i närområdet: 90-99% av spontvibrationer förlorades inom 0,5 m från sponten. Dessutom minskades vibrationsöverföring mellan spont och jord med ökande spontaccelerationsnivåer och högre frekvenser.
- Geotekniska förhållanden på platsen påverkar starkt spontvibrationsnivåerna. I fältstudien fanns det en tydlig skillnad mellan ”mjuk” och ”hård” drivning där ”hård” drivning förknippades med störta vid tån.
- Fokus i vibrationsanalyser bör inte alltid ligga på de vertikala vibrationskomponenterna. I denna fältstudie var de horisontella vibrationskomponenterna lika stor eller större än de vertikala.

Nyckelord: Markvibrationer, vibrodrivning, spont, fullskaligt fältförsök, markinstrumentering, spontinstrumentering.
## Symbols and Abbreviations

### Roman letters

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Amplitude</td>
<td>[m]</td>
</tr>
<tr>
<td>$A_p$</td>
<td>Sheet pile cross-section area</td>
<td>[m$^2$]</td>
</tr>
<tr>
<td>$a$</td>
<td>Acceleration</td>
<td>[m/s$^2$]</td>
</tr>
<tr>
<td>$c$</td>
<td>Wave propagation velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$c_p$</td>
<td>Wave propagation velocity of P-waves</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$c_R$</td>
<td>Wave propagation velocity of R-waves</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$c_S$</td>
<td>Wave propagation velocity of S-waves</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$D$</td>
<td>Depth</td>
<td>[m]</td>
</tr>
<tr>
<td>$d_{crit}$</td>
<td>Minimal distance for R-wave formation</td>
<td>[m]</td>
</tr>
<tr>
<td>$E$</td>
<td>Young/Elasticity modulus</td>
<td>[Pa]</td>
</tr>
<tr>
<td>$F_c$</td>
<td>Centrifugal force</td>
<td>[N]</td>
</tr>
<tr>
<td>$F_d$</td>
<td>Driving force</td>
<td>[N]</td>
</tr>
<tr>
<td>$F_h$</td>
<td>Horizontal component of the centrifugal force</td>
<td>[N]</td>
</tr>
<tr>
<td>$F_S$</td>
<td>Suspension force</td>
<td>[N]</td>
</tr>
<tr>
<td>$F_v$</td>
<td>Vertical component of the centrifugal force</td>
<td>[N]</td>
</tr>
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<td>$F_{V,TOT}$</td>
<td>Dynamic driving force</td>
<td>[N]</td>
</tr>
<tr>
<td>$F_0$</td>
<td>Static surcharge force</td>
<td>[N]</td>
</tr>
<tr>
<td>$f$</td>
<td>Frequency</td>
<td>[Hz]</td>
</tr>
<tr>
<td>$f_d$</td>
<td>Driving frequency</td>
<td>[Hz]</td>
</tr>
<tr>
<td>$f_{max}$</td>
<td>Vibrator unit maximal driving frequency</td>
<td>[Hz]</td>
</tr>
<tr>
<td>$f_n$</td>
<td>Natural frequency</td>
<td>[Hz]</td>
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<tr>
<td>$G$</td>
<td>Shear modulus</td>
<td>[Pa]</td>
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<td>[Pa]</td>
</tr>
<tr>
<td>$g$</td>
<td>Acceleration of earth’s gravity</td>
<td>[m/s$^2$]</td>
</tr>
<tr>
<td>$H$</td>
<td>Soil layer thickness</td>
<td>[m]</td>
</tr>
<tr>
<td>$I_y$</td>
<td>Second moment of inertia</td>
<td>[cm$^4$]</td>
</tr>
<tr>
<td>$k$</td>
<td>Spring stiffness</td>
<td>[N/m]</td>
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<tr>
<td>$k_d$</td>
<td>Dynamic calibration factor</td>
<td>[mV/g]</td>
</tr>
<tr>
<td>$L$</td>
<td>Sheet pile length</td>
<td>[m]</td>
</tr>
<tr>
<td>$M$</td>
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<td>[Pa]</td>
</tr>
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<td>[kg.m]</td>
</tr>
<tr>
<td>$M_{ei}$</td>
<td>Eccentric moment of one eccentric weight</td>
<td>[kg.m]</td>
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<tr>
<td>$m$</td>
<td>Mass</td>
<td>[kg]</td>
</tr>
<tr>
<td>$m_{dyn}$</td>
<td>Dynamic mass</td>
<td>[kg]</td>
</tr>
<tr>
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<td>Mass of one eccentric weight</td>
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</tr>
<tr>
<td>$m_0$</td>
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</tr>
<tr>
<td>$N$</td>
<td>Number of cycles</td>
<td>[-]</td>
</tr>
<tr>
<td>$n$</td>
<td>Mode number</td>
<td>[-]</td>
</tr>
<tr>
<td>$OCR$</td>
<td>Over Consolidation Ratio</td>
<td>[-]</td>
</tr>
<tr>
<td>$P_I$</td>
<td>Plasticity Index</td>
<td>[-]</td>
</tr>
<tr>
<td>$PPV$</td>
<td>Peak particle velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$R_e$</td>
<td>Dynamic shaft resistance</td>
<td>[kN]</td>
</tr>
<tr>
<td>$R_T$</td>
<td>Dynamic toe resistance</td>
<td>[kN]</td>
</tr>
<tr>
<td>$r_{ei}$</td>
<td>Eccentricity radius</td>
<td>[m]</td>
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Pile – Soil Interaction during Vibratory Sheet Pile Driving

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
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<td>$S_0$</td>
<td>Free hanging double displacement amplitude</td>
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</tr>
<tr>
<td>$s_0$</td>
<td>Free hanging single displacement amplitude</td>
<td>[m]</td>
</tr>
<tr>
<td>$T$</td>
<td>Period</td>
<td>[s]</td>
</tr>
<tr>
<td>$T_d$</td>
<td>Period associated to the driving frequency $f_d$</td>
<td>[s]</td>
</tr>
<tr>
<td>$t$</td>
<td>Time</td>
<td>[s]</td>
</tr>
<tr>
<td>$u$</td>
<td>Sheet pile toe displacement</td>
<td>[mm]</td>
</tr>
<tr>
<td>$v$</td>
<td>Particle velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$W_C$</td>
<td>Energy dissipated in one cycle</td>
<td>[J/m³]</td>
</tr>
<tr>
<td>$W_y$</td>
<td>Elastic section modulus</td>
<td>[cm³]</td>
</tr>
<tr>
<td>$Z$</td>
<td>Impedance</td>
<td>[N.s/m]</td>
</tr>
<tr>
<td>$z$</td>
<td>Displacement</td>
<td>[m]</td>
</tr>
<tr>
<td>$\dot{z}$</td>
<td>Velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$\ddot{z}$</td>
<td>Acceleration</td>
<td>[m/s²]</td>
</tr>
<tr>
<td>$z_s$</td>
<td>Material specific impedance</td>
<td>[N.s/m³]</td>
</tr>
</tbody>
</table>

**Greek letters**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
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</thead>
<tbody>
<tr>
<td>$\gamma$</td>
<td>Shear strain</td>
<td>[-]</td>
</tr>
<tr>
<td>$\gamma_C$</td>
<td>Cyclic shear strain</td>
<td>[-]</td>
</tr>
<tr>
<td>$\varepsilon$</td>
<td>Normal strain</td>
<td>[-]</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Rotation angle of the eccentric weights</td>
<td>[°]</td>
</tr>
<tr>
<td>$\theta_{crit}$</td>
<td>Minimal angle for R-wave formation</td>
<td>[°]</td>
</tr>
<tr>
<td>$\lambda$</td>
<td>Wave length</td>
<td>[m]</td>
</tr>
<tr>
<td>$\tau$</td>
<td>Shear stress</td>
<td>[Pa]</td>
</tr>
<tr>
<td>$\tau_C$</td>
<td>Cyclic shear stress</td>
<td>[Pa]</td>
</tr>
<tr>
<td>$\tau_{uk}$</td>
<td>Characteristic undrained shear strength</td>
<td>[kPa]</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Poisson’s ratio</td>
<td>[-]</td>
</tr>
<tr>
<td>$\xi$</td>
<td>Viscous damping ratio</td>
<td>[-]</td>
</tr>
<tr>
<td>$\pi$</td>
<td>Pi</td>
<td>[-]</td>
</tr>
<tr>
<td>$\rho$</td>
<td>Material density</td>
<td>[kg/m³]</td>
</tr>
<tr>
<td>$\sigma$</td>
<td>Normal stress</td>
<td>[Pa]</td>
</tr>
<tr>
<td>$\phi$</td>
<td>Phase angle</td>
<td>[rad]</td>
</tr>
<tr>
<td>$\phi_k$</td>
<td>Characteristic angle of internal friction</td>
<td>[°]</td>
</tr>
<tr>
<td>$\Omega$</td>
<td>Sheet pile circumference</td>
<td>[m]</td>
</tr>
<tr>
<td>$\omega$</td>
<td>Angular frequency</td>
<td>[rad/s]</td>
</tr>
<tr>
<td>$\omega_n$</td>
<td>Natural angular frequency</td>
<td>[rad/s]</td>
</tr>
</tbody>
</table>

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1 Introduction

1.1 Background

Urban construction sites require strict control of their environmental impact, which, for sheet pile driving, can include damage to nearby structures due to ground vibrations and settlements, as well as human perception of uncomfortable vibrations and noise pollution. This MSc. thesis in particular deals with the vibration transfer at the sheet pile – soil interface, in the context of vibratory sheet pile driving.

The sheet pile – soil interaction is a crucial link in the understanding of the vibration transfer from the sheet pile driver to nearby structures. And, while several models exist for propagation of ground vibrations, research performed on the pile – soil interaction is still quite limited, (Deckner et al., 2012), (Deckner, 2013).

This thesis work was supervised by F. Deckner with the help of Dr. K. Viking and examined by Professor S. Hintze. It is a part of the joint KTH – NCC research program Vibrations due to pile and sheet pile driving in urban areas which aims at developing a simple and reliable prediction model for ground vibrations arising from pile and sheet pile driving. The research in this program is financed by SBUF, NCC and KTH.

The present work followed M. Lidén’s MSc. thesis (Lidén, 2012) which compared ground vibrations measured in a trial sheet pile driving in Karlstad (Sweden) in May 2010 with existing propagation models for ground vibrations.

The relation between the different program members’ work is schematized in Figure 1.1.

![Figure 1.1: Situation of the present thesis in relation to other work performed in the joint KTH – NCC research program Vibrations due to pile and sheet pile driving in urban areas.](image-url)
1.2 Aim
The centerpiece of this thesis was a field study consisting in the monitoring of sheet pile and ground vibrations during vibratory driving of sheet piles in Solna, a suburb of Stockholm (Sweden). The aim of the thesis work was to:

- Collect experimental data from vibratory pile driving to be used in F. Deckner’s doctoral thesis. This aspect governed the extent of the field test and demanded high academic rigor in the measurement procedures.

- Observe the vibratory behavior of the soil in close proximity of the sheet pile and comparing the results with the conceptual models currently available. This aspect governed parts of the field test instrumentation and the choice of the performed data analysis.

Both aspects of the thesis work aimed, more generally, to contribute to the understanding of ground vibration generation during vibratory sheet pile driving.

1.3 Limitations
This thesis focuses on ground vibrations and does not tackle noise and settlement issues which also can arise due to vibrations caused by sheet pile installation. Moreover, only vibratory installation of sheet piles was studied here (as opposed to impact driving, drilling, jacking or other methods).

The study is also limited to the soil types encountered in the field study; the generalization of the results to other soils is left to the care of F. Deckner in her doctoral thesis.

With regard to the focus of the MSc. thesis as well as its expected duration and scope, only a limited amount of the collected data was analyzed. Out of seven measurement series, only two are discussed in this thesis. The remaining data will be exploited and presented in F. Deckner’s further publications.

1.4 Method
The thesis work was planned in three main parts: a literature review, the organization and conduct of the field study, and finally the analysis of the chosen field data.

The literature review is presented in Chapter 2 and focuses on current understanding of the soil vibration generation during sheet pile driving and on earlier research concerning pile – soil interaction. It aims at providing a theoretical background to the field study and prompting appropriate reflection around the collected data in order to draw educated conclusions.
The field study is described in Chapter 3. The chapter covers the site conditions as well as the specifications of the driving equipment and the execution of the field study. Lastly, the data collection, acquisition and processing methods are explained.

Results from the chosen measurement series are presented in Chapter 4, along with the corresponding performed analyses. A discussion based on the comparison with the dominant conceptual models concludes the chapter.

General conclusions drawn from the thesis work are presented in Chapter 5, which also contains proposals for future research.
2 Literature study

2.1 Introduction
The literature review presented here aims at providing a theoretical background for the preparation of the field study and the analysis of its results. Basic parameters and concepts of dynamic and geodynamics are explained in section 2.2. Vibratory sheet pile driving is described in section 2.3, followed by the current understanding of the vibration transfer at the sheet pile – soil interface in section 2.4. Section 2.5 describes conventional methods for data acquisition, processing and presentation. It also gives a brief summary of a selection of previous field studies whose experience can be of interest in this thesis.

Several sections of the literature study strongly overlap with M. Lidén’s and F. Deckner’s respective literature studies, (Lidén, 2012), (Deckner, 2013). Their work was therefore used as a basis and largely referred to, especially in sections 2.2.1-2.2.3 and 2.3.

2.2 Vibrations and dynamic soil behavior

2.2.1 Description of vibratory motion
A vibration is defined as the oscillatory motion of a particle around a position of equilibrium, (Holmberg et al., 1984). The main parameters generally used for the description of vibratory motions are presented in this section, along with a short classification of different vibration types.

A particle’s oscillation is described by its displacement $z$, velocity $\dot{z}$, and acceleration $\ddot{z}$ but only one of these quantities is needed to define the vibration, (Richart et al., 1970). They are indeed linked by time derivation and integration as shown in Table 2.1.

<table>
<thead>
<tr>
<th>Table 2.1: Derivation and integration relations between the three quantities of motion.</th>
</tr>
</thead>
<tbody>
<tr>
<td>... displacement</td>
</tr>
<tr>
<td>Displacement in function of ...</td>
</tr>
<tr>
<td>Velocity in function of ...</td>
</tr>
<tr>
<td>Acceleration in function of ...</td>
</tr>
</tbody>
</table>
### Deterministic vibrations

A deterministic vibration can be described by a mathematical equation which makes it theoretically possible to predict the future displacement. The simplicity of these vibrations enables their characterization by a small number of parameters. *Harmonic vibrations* are the simplest deterministic vibrations, (Richart et al. 1970). They are pure sine/cosine functions, see Figure 2.1, and can be fully described by the parameters of Table 2.2.

#### Table 2.2: Parameters describing a harmonic motion.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Expression</th>
<th>Unit</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td></td>
<td>[m]</td>
<td>Amplitude – peak displacement from equilibrium</td>
</tr>
<tr>
<td>$T$</td>
<td></td>
<td>[s]</td>
<td>Period – length of a cycle</td>
</tr>
<tr>
<td>$\omega$</td>
<td>$2\pi/T$</td>
<td>[rad/s]</td>
<td>Angular frequency – radians per second (rotation analogy)</td>
</tr>
<tr>
<td>$f$</td>
<td>$1/T$</td>
<td>[s$^{-1}$] or [Hz]</td>
<td>Frequency – number of cycles per second</td>
</tr>
<tr>
<td>$\varphi$</td>
<td></td>
<td>[rad]</td>
<td>Phase angle – time lag compared to a pure sine function</td>
</tr>
</tbody>
</table>

**Figure 2.1:** Description of a harmonic vibration, from Deckner (2013) modified after Richart et al. (1970).

The phase angle is often not interesting in practical cases and the motion can be described by its amplitude $A$ and frequency $f$, or angular frequency $\omega$, (Holmberg et al., 1984).

A periodic vibration is a displacement cycle which repeats itself after a time period $T$, (Richart et al., 1970), see Figure 2.2. The French mathematician and physicist Jean Baptiste Joseph Fourier discovered that all periodic signals could be described as a sum of a series of sinusoids of...
different amplitude, frequency and phase, (Kramer, 1996). This is the basis for the Fourier Transform mentioned in section 2.5.1.

![Figure 2.2: Periodic vibration, from Holmberg et al. (1984).](image)

A transient vibration is a vibratory motion of decreasing amplitude, see Figure 2.3. Transient vibrations are almost never fully deterministic but in many practical cases the vibration can be approximated by an exponentially decreasing sine vibration, (Holmberg et al., 1984). Transient soil vibrations are generally associated with impulse-type disturbances like blasting or impact pile driving, (Richart et al., 1970), (Head & Jardine, 1992), (Svinkin, 2008).

![Figure 2.3: Transient vibration, from Holmberg et al. (1984).](image)

Random motion
A random motion has no pattern, see Figure 2.4. The simple parameters listed above do not apply and only statistical methods can describe random motions. Wind and traffic are common sources of random vibrations, (Holmberg et al., 1984).

![Figure 2.4: Random vibration, from Holmberg et al. (1984).](image)
2.2.2 Wave propagation in elastic media

The vibratory motions defined above describe the oscillation of an individual particle. In the soil the particles are in contact, and the motion of one particle excites the neighboring particles thus transmitting the vibratory motion, (Head & Jardine, 1992). This is the basis of wave propagation which is the transport of energy without transport of particles. The parameters usually used to describe waves in elastic media are listed in Table 2.3.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Expression</th>
<th>Unit</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>c</td>
<td>(c)</td>
<td>[m/s]</td>
<td>Wave velocity – speed at which the wave travels</td>
</tr>
<tr>
<td>f</td>
<td>(f)</td>
<td>[s⁻¹] or [Hz]</td>
<td>Frequency – frequency of the particle motion</td>
</tr>
<tr>
<td>(\lambda)</td>
<td>(c/f)</td>
<td>[m]</td>
<td>Wave length – distance between two particles in the same state (eg. between two wave crests)</td>
</tr>
</tbody>
</table>

It is important to distinguish the local particle velocity \(\dot{v} = \dot{z}\) which is the speed at which a particle oscillates around an equilibrium position (see section 2.2.1) and the wave velocity \(c\) which is the speed at which the wave travels away from the source.

The wave velocity depends on the stress-strain relationship of the soil which is defined by a constitutive model. A common approximation is to consider the soil as a homogeneous isotropic linear elastic material governed by the generalized Hooke’s law, (Barkan, 1962), with the following stress-strain relationships:

\[
(2.1) \quad \sigma_x = 2G \left( \varepsilon_x + \frac{\nu}{1-\nu} (\varepsilon_x + \varepsilon_y + \varepsilon_z) \right)
\]

\[
(2.2) \quad \sigma_y = 2G \left( \varepsilon_y + \frac{\nu}{1-\nu} (\varepsilon_x + \varepsilon_y + \varepsilon_z) \right)
\]

\[
(2.3) \quad \sigma_z = 2G \left( \varepsilon_z + \frac{\nu}{1-\nu} (\varepsilon_x + \varepsilon_y + \varepsilon_z) \right)
\]

\[
(2.4) \quad \tau_{xy} = G \cdot y_{xy}
\]

\[
(2.5) \quad \tau_{yz} = G \cdot y_{yz}
\]

\[
(2.6) \quad \tau_{zx} = G \cdot y_{zx}
\]

Where \(G\) = shear modulus [Pa], \(\nu\) = Poisson’s ratio [-], \(\sigma\) = normal stress [Pa], \(\varepsilon\) = normal strain [-], \(\tau\) = shear stress [Pa], \(\gamma\) = shear strain [-].

\(\sigma_{xx} \: \sigma_{yy} \: \sigma_{zz} \: \tau_{xx} \: \tau_{yy} \: \tau_{zz} \: \tau_{xy} \: \tau_{yz} \: \tau_{zx}\)

\(\sigma_x \: \sigma_y \: \sigma_z \: \tau_{xy} \: \tau_{yz} \: \tau_{zx}\)

Figure 2.5: Components of stress, from Barkan (1962).
The oedometer modulus $M$ is defined for a uniaxial deformation, i.e. $\varepsilon_x \neq 0 \; ; \varepsilon_y = \varepsilon_z = 0$:

\[(2.7) \quad \sigma_{xx} = M \cdot \varepsilon_{xx}\]
\[(2.8) \quad M = \frac{2G(1-\nu)}{1-2\nu}\]

Where $G =$ shear modulus [Pa] 
$\nu =$ Poisson’s ratio [-] 
$\sigma =$ normal stress [Pa] 
$\varepsilon =$ normal strain [-]

**Body waves**

The equation of motion in an infinite homogeneous, isotropic, elastic medium has two solutions which describe two body waves of different nature which propagate from the source independently from each other, (Barkan, 1962). The compressional wave (or P for primary) describes a volume change and the distortional wave, also called shear wave (or S for secondary), describes a shape change, (Richart et al., 1970).

![P-wave and S-wave shapes and associated particle motions](image)

*Figure 2.6: P- and S-wave shapes and associated particle motions, from Deckner (2013) modified after Kramer (1996).*

The P-wave is the propagation of a local volume change (or local density change) of the soil mass. The particle motion associated with this wave is a longitudinal push-pull motion, see Figure 2.6. The P-wave’s propagation is based on the material’s resistance to uniaxial deformation and thus depends on the oedometer modulus $M$:

\[(2.9) \quad c_p = \frac{M}{\sqrt{\rho}} = \frac{G}{\rho} \cdot \frac{2(1-\nu)}{1-2\nu}\]

Where $c_p =$ P-wave velocity [m/s]
Pile – Soil Interaction during Vibratory Sheet Pile Driving

\[ M = \text{oedometer modulus} \quad \text{[Pa]} \]
\[ \rho = \text{soil density} \quad \text{[kg/m}^3\text{]} \]
\[ G = \text{soil shear modulus} \quad \text{[Pa]} \]
\[ \nu = \text{Poisson’s ratio} \quad [-] \]

The \textit{S-wave} is the propagation of a local transversal distortion (without volume change) in the soil mass. The corresponding particle motion is a transversal oscillation, see Figure 2.6. The S-wave’s propagation is based on the ability to transmit shear forces between particles and thus depends on the shear modulus. Since soil is weaker in shear than in axial loading, the S-wave is slower than the P-wave. The S-wave velocity is defined by:

\[ c_S = \sqrt{\frac{G}{\rho}} \]

Where
\[ c_S = \text{S-wave velocity} \quad \text{[m/s]} \]
\[ G = \text{soil shear modulus} \quad \text{[Pa]} \]
\[ \rho = \text{soil density} \quad \text{[kg/m}^3\text{]} \]

Typical propagation velocities for P- and S-waves in sand and clay can be found in Table 2.4.

\textbf{Table 2.4:} Typical propagation velocities for P- and S-waves, summarized from Bodare (1998).

<table>
<thead>
<tr>
<th>Material</th>
<th>P-wave velocity ( c_P ) [m/s]</th>
<th>S-wave velocity ( c_S ) [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>Dry 100-600, Saturated 1450</td>
<td>40-300, 40-250</td>
</tr>
<tr>
<td>Sand</td>
<td>Dry 150-1000, Saturated 1450</td>
<td>100-500, 80-450</td>
</tr>
</tbody>
</table>

\textbf{Influence of the water table:} water is capable of transmitting the P-waves at a higher velocity than the soil structure which means that \( c_P \) increases with the soil water content. The velocity of the P-wave in saturated soils is about 1450 m/s which is the velocity of P-waves in water. As water has no shear strength, the S-wave velocity is not as affected by the degree of saturation and even tends to be lower in saturated soils as the S-waves can only propagate through the solid structure (Richart et al., 1970), (Dowding, 1996).

\textbf{Surface waves}

At the interface between two materials with very different elastic properties, a stress free surface can be considered for the stiffest material, i.e. the soil at the ground-air interface. Different types of \textit{surface waves} are developed at a free surface but only the Rayleigh wave (or R-wave), which is the most important (Holmberg et al., 1984), is described here. According to Svinkin (2008), the R-waves are the most harmful ground vibrations as they have large displacements, low frequencies, low wave velocity and carry most of the vibration energy.
The R-wave is a combination of P- and S-waves which is associated to an elliptical particle motion, see Figure 2.7. The shape and size of the ellipse is dependent on depth and the Poisson’s ratio as shown in Figure 2.8.

According to Bodare (1998), the velocity of the R-wave can be approximated by:

\[
(2.11) \quad c_R = \frac{0.87 + 1.12\nu}{1 + \nu} c_S
\]

Where \( c_R = \text{R-wave velocity} \quad \text{[m/s]} \)
\( c_S = \text{S-wave velocity} \quad \text{[m/s]} \)
\( \nu = \text{Poisson ratio} \quad \text{[-]} \)
Which for a Poisson ratio of $\nu = 0.3$ gives:

$$c_R = 0.93 \ c_S$$

### 2.2.3 Resonance

Mechanical systems exposed to vibrations present a certain “response”, i.e. motion pattern, which depends on their mass and stiffness as well as on the exciting vibration, (Chopra, 1995). When the response is plotted for a given system, certain frequencies seem to induce amplified displacements. This phenomenon is known as resonance and occurs when the exciting frequency corresponds with the system’s natural frequency and a stationary wave can occur in the system, (Möller et al., 2000).

A system’s natural frequency is the frequency at which it would oscillate freely after being disturbed from its equilibrium state, (Chopra, 1995). The natural mode of vibration is the shape of the corresponding harmonic motion.

The natural frequency of a linear elastic single degree of freedom mass-spring system is the simplest to determine and is:

$$f_n = \frac{\omega_n}{2\pi} = \frac{1}{2\pi} \sqrt{\frac{k}{m}}$$

Where $f_n$ = natural frequency [Hz]

$\omega_n$ = natural angular frequency [rad/s]

$k$ = spring stiffness [N/m]

$m$ = mass [kg]

For a system with multiple degrees of freedom, there are as many natural frequencies as degrees of freedom. Continuous systems thus have an infinity of natural frequencies, but usually, only the lowest frequencies are of concern as they produce the largest amplitudes (Richart et al., 1970).

At resonance, a continued excitation theoretically leads to infinite displacement amplitudes. This is however not the case for physical systems as energy is drained from the system in the form of friction, plastic deformation, etc. This phenomenon is known as damping and is quantified by the damping ratio $\xi$, (Chopra, 1995).

### 2.2.4 Soil dynamic models

As for all dynamic systems, the parameters governing the soil’s dynamic response are its stiffness, mass and damping. For small deformations elastic wave propagation can be assumed, i.e. constant stiffness and damping. For large deformations, high soil strains lead to soil degradation, i.e. decreased stiffness and increased damping, (Kramer, 1996).

Classification of soil behavior in the context of construction related vibrations has been proposed as a function of shear strain levels, see Table 2.5, and as a function of particle acceleration in the case of granular materials, see Table 2.6.
Table 2.5: Soil behavior corresponding to different shear strain levels, from Whenham (2011) after Ishihara (1996).

<table>
<thead>
<tr>
<th>Shear strain</th>
<th>$10^{-6}$</th>
<th>$10^{-5}$</th>
<th>$10^{-4}$</th>
<th>$10^{-3}$</th>
<th>$10^{-2}$</th>
<th>$10^{-1}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Elasto-plastic</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Effect of load repetition</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Effect of loading rate</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Model</td>
<td>Linear elastic</td>
<td>Visco-elastic</td>
<td>Load history tracing</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 2.6: Granular soil behavior corresponding to different acceleration levels, based on shaft resistance reduction described by Whenham (2011), from research performed by Rodger and Littlejohn (1980).

<table>
<thead>
<tr>
<th>Particle acceleration</th>
<th>0.6g</th>
<th>1.5g</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic state</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear strength reduction</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fluidized state</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Vibrations in the linear elastic range
Far enough from the source, the soil strain is smaller than $10^{-5}$ and the wave propagation is considered elastic, see section 2.2.2. For modeling purposes it is common to place the far field (zone of purely elastic deformations) at four wavelengths from the source, (Whenham, 2011). In the far field, the wave velocity and vibration amplitude are governed by the parameters listed in Table 2.7.

Table 2.7: Parameters governing vibrations in an elastic field.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Expression</th>
<th>Unit</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$G$</td>
<td>$G = \tau/\gamma$</td>
<td>[Pa]</td>
<td>Shear modulus – stress strain behavior for elastic deformations</td>
</tr>
<tr>
<td>$\nu$</td>
<td>$\nu = -\varepsilon_{yy}/\varepsilon_{xx}$</td>
<td>[-]</td>
<td>Poisson’s ratio – ratio of transversal and axial strain</td>
</tr>
<tr>
<td>$\rho$</td>
<td>$\rho = m/V$</td>
<td>[kg/m$^3$]</td>
<td>Bulk density</td>
</tr>
</tbody>
</table>

Vibrations in the visco-elastic range
For medium strains ($10^{-5} < \gamma_c < 10^{-3}$), the increasing relative motion between the grains, see Figure 2.9, can give rise to energy losses due to friction, abrasion and degradation of aggregates. The losses lead to wave attenuation, known as internal damping, which depends on the soil characteristics and on the cyclic strain amplitude $\gamma_c$, (Head & Jardine, 1992). The phenomenon is mostly rate independent and can be quantified by an equivalent viscous damping ratio $\xi$, (Whenham, 2011):

$$\xi = \frac{\Delta W_c}{2\pi \gamma_c \tau_c}$$  

(2.14)  

Where $\xi = \text{damping ratio}$  

[-]
$\Delta W_C$ = energy lost at each cycle  
$\gamma_C$ = cyclic strain  
$\tau_C$ = cyclic stress 

$[\text{J/m}^3]$  
[-]  
[Pa]  

Figure 2.9: Relative movement between grains depending on the shear strain level, from Bodare (1998).

For loading levels in the visco-elastic range, the shear modulus $G = G_{\text{max}}$ and damping ratio $\xi$ can be considered independent of the number of cycles. This behavior is called stable hysteresis and can be represented by the hyperbolic model in Figure 2.10. A stable hysteresis is a non-linear cycle independent model which assumes that any stress-strain curve of the soil is bounded by two straight lines which are tangential to it at small strains ($G_{\text{max}}$) and at large strains ($\tau_{f,\text{max}}$) respectively, (Whenham, 2011).

Figure 2.10: Shear stress-strain relationship in the hyperbolic model under a cyclic shear strain $\gamma_c$, modified after Whenham (2011), originally from Ishihara (1996).
Vibrations beyond the plastic limit: soil degradation

When subjected to repetitive large strain loading ($\gamma > 10^3$), the soil experiences permanent degradation due to fatigue of the soil skeleton in cohesive soils (Vucetic & Dobry, 1991) and effective stress reduction in granular soils, (Holeyman, 2002). The shear modulus becomes dependent on the number of cycles as presented in the degraded hysteresis of Figure 2.11 and in the experimental data of Figure 2.12. As the soil loses its stiffness, there is more and more movement between particles, leading also to increased frictional damping.

![Degraded hysteresis under constant shear strain loading. Image modified after Holeyman (2002), originally from Vucetic (1993).](image)

Figure 2.11: Degraded hysteresis under constant shear strain loading, modified after Holeyman (2002), originally from Vucetic (1993).

![Effect of cyclic stiffness degradation and strain level on $G/G_{\text{max}}$ for different plasticity indexes. Image from Vucetic & Dobry (1991).](image)

Figure 2.12: Effect of cyclic stiffness degradation and strain level on $G/G_{\text{max}}$ for different plasticity indexes, from Vucetic & Dobry (1991).
From Figure 2.12, it appears that soils with higher plasticity indexes are less prone to degradation. This has been observed by Vucetic & Dobry (1991) who suggested the following explanation: sands and non-plastic silts are more susceptible to plastic degradation due to particle rearrangement and loss of contact during vibration (see Table 2.6) while clay particles are more tightly joined to each other by electrical and chemical bonds which conserve an “elastic” character up to higher strain levels and make them less susceptible to cyclic degradation.

**Vibrations and granular soil fluidization**

**Under saturated conditions**, the irreversible tendency to achieve a denser packing when sheared back and forth is expressed by a build-up of pore pressure such that the effective stress is reduced to a value that may be close to zero, leading to the soil’s loss of shear strength and a phenomenon known as *liquefaction*, (Holeyman, 2002). The reader is referred to Denies (2010) for the distinction between the two types of liquefaction: flow liquefaction and cyclic mobility.

**Under dry conditions**, shear strength reduction in granular material has also been observed under high acceleration levels as suggested by Barkan (1962). As the shear strength of cohesionless soils is strongly related to the shear resistance at the points of contact between the soil particles, see Figure 2.9, loss of contact between the particles can explain the loss of shear strength, also called *vibro-fluidization*.

Experimentally, Rodger and Littlejohn (1980), as cited in Whenham (2011), distinguished the three states presented below:

- elastic state \((a < 0.6g)\)
- trans-threshold state \((0.7g < a < 1.5g)\) where the shear strength reduction takes place
- fluidized response state \((a > 1.5g)\).

Viking (2002), explained the loss of shear strength by the “free fall” motion experienced by dry soil particles when the peak acceleration of an idealized granular soil volume located beside the sheet pile shaft exceeds a site-specific threshold value corresponding to the initial vertical confining stress \((\sigma_v)\), and the vertical confining stress within the soil drops to nearly zero. During the “free-fall”, the grains lose contact with their neighbors, see Figure 2.13, until they interact again in the second half of the loading cycle.
More recently, both experimental and numerical work on dry sand performed by Denies (2010) has evidenced vibro-fluidization. Experimentally, Denies determined two behaviors separated by an instable transition state \((a \approx g)\):

- densification behavior \((a < g)\)
- vibro-fluid behavior \((a > g)\)

In the experiments described by Denies (2010), samples of dry fine quartz sand with uniform grain size (Fontainebleau sand) were poured into cylindrical transparent polycarbonate containers which were then vibrated vertically at various acceleration levels. For accelerations above 1g, Denies observed a convective behavior of the grains along the container wall which he explained by friction between the grains and the container wall, see Figure 2.14. During vertical vibration, the particles are closely packed on the way up, but expand to a lower density on the way down. This leads to higher friction from the walls in phase a) than in phase b). The grains therefore experience an overall downward drag along the container walls, thus initiating the convective movement. At an acceleration of 2.4g, Denies observed a vibro-fluid behavior, where he suggested, based on his observations, that the loss of shear strength is associated to chaotic convective rolls in the sand mass as represented in Figure 2.15.

With the help of Discrete Element Modeling (DEM), Denies (2010), also observed that for an acceleration level of 1.02g, the contact network between the grains is degraded and the force chains are broken resulting in shear strength degradation.
Figure 2.14: Convective behavior of the grains along the container wall, a) during relative upward motion of the black grain, and b) during relative downward motion of the black grain, as represented by Denies (2010).

Figure 2.15: Chaotic convection observed at $a = 2.4g$, as reported by Denies (2010).
2.3 Vibratory driving of sheet piles

Vibratory driving consists in conveying a vertical oscillating motion to an element in order to drive it into the ground. Though the mechanisms of vibratory driving are not completely understood, it is believed that the constant oscillation of the element reduces the ground resistance and allows for penetration under relatively low surcharge forces, (Viking, 2002 and 2006), (Whenham, 2011).

The possibility of using vibratory equipment for extraction as well as for driving has long been appreciated by contractors, (Iwanowski & Berglars, 1986). Vibratory driving is widely used today for the installation of sheet piles, however major engineering issues still remain concerning prediction of vibro-drivability and estimation of bearing capacity, as well as forecasting of generated noise, soil vibrations and settlements, (Viking, 2006).

The following sections further describe the vibratory driving system which is composed of:

- Driving equipment
- Driven element
- Soil

2.3.1 Driving equipment

Components

The vibratory driving equipment, see Figure 2.16, consists of a vibrator which generates the vertical oscillations and is powered by an electric or hydraulic motor fed by the power transmission, i.e. electrical cables or hydraulic hoses, connected to the power source which is usually a diesel engine driving a generator or a hydraulic pump. The vibrator can be free hanging, i.e. hanging from a crane, or leader mounted, i.e. maneuvered by connection to a guide or to an excavator boom for the lighter models, (Viking, 1997), (Massarsch, 2000), (Whenham, 2011).

![Figure 2.16: Vibratory driving systems: (a) free hanging and (b) leader mounted, modified after Whenham (2011).](image-url)
The vibrator itself, see Figure 2.17, is composed of a fixed part, the *supressor housing*, and a mobile part, the *excitor block* on which a *griping claw* is mounted to clamp the vibrated element, (Massarsch, 2000), (Viking, 1997), (Whenham, 2011).

The *supressor housing* is the static part of the vibrator unit. Its mass is referred to as the *bias mass*, and contributes to the vertical force exerted on the pile.

The *excitor block* contains an electric or hydraulic motor activating the *eccentric weights* which work in counter-rotating pairs to generate the vertical vibratory motion.

*Elastomer pads* isolate the static suppressor housing from the vibrating excitor block.

The *griping claw* connects the pile to the vibrator and aims at transmitting the driving force axially while minimizing the damage to the pile.

**Figure 2.17**: Components of a vibrator unit.

**Parameters of vibratory drivers**

In a review of the works of Massarsch (2000) and Viking (1997 and 2002), the following parameters were judged most significant for vibratory driving: the eccentric moment $M_e$, the driving frequency $f$, the vertical resultant of the centrifugal forces $F_{V,TOT}$, the static surcharge force $F_0$, the driving force $F_d$ and the free hanging double displacement amplitude $S_0$.

The *eccentric moment* $M_{ei}$, usually expressed in [kg.m], is a characteristic of the eccentric weights. Their center of gravity is located at a certain distance $r_{ei}$ from their rotation axis, see Figure 2.18, conferring an eccentric moment to each eccentric weight:

\[
M_{ei} = m_{ei} \cdot r_{ei}
\]

Where $M_{ei}$ = eccentric moment [kg.m] $m_{ei}$ = eccentric mass [kg] $r_{ei}$ = eccentricity radius [m]

**Figure 2.18**: Mass and eccentricity radius, from Viking (2002).
The total eccentric moment $M_e$ of the vibrator is the sum of its individual eccentric moments if all the eccentric weights are in phase:

\begin{equation}
M_e = \sum M_{ei}
\end{equation}

Most modern-day vibrators can adjust the relative phase position of the eccentric weights during driving in order to vary the magnitude of the total eccentric moment, see Figure 2.19. The eccentric moment, and thereby the displacement amplitude, see Eq. (2.24), can be eliminated during start-up and shut-down when the vibrator sweeps through all the frequencies from zero to the driving frequency and back. This avoids exciting the relatively low resonance frequencies of the surrounding soil strata and is very advantageous in densely built urban areas, which are more vibration-sensitive.

![Figure 2.19: Example of a technique used for adjusting a vibrator's eccentric moment from 100% of $M_e$, left, to 0% of $M_e$, right, from ABI GmbH (Piling with Vibration).](image)

The driving frequency $f_d$ usually expressed in [Hz] is the number of revolutions per second of the eccentric weights. With $\omega$ [rad/s] the angular frequency of the eccentric weights, $f_d$ can be expressed:

\begin{equation}
f_d = \frac{\omega}{2\pi}
\end{equation}

The vertical component $F_v$ of a centrifugal force is a time-dependent harmonic force that describes a sinusoidal path in time which arises from the projection of a rotating centrifugal force $F_c$, see Figure 2.20.

![Figure 2.20: Centrifugal forces and their projection for two counter-rotating eccentric weights, modified after Viking (2002).](image)

For each eccentric weight:

\begin{equation}
F_v(t) = F_c \cdot \sin \theta = F_c \cdot \sin(\omega t)
\end{equation}

With:

\begin{equation}
F_c = M_{ei} \cdot \omega^2
\end{equation}

Where $F_c = \text{centrifugal force}$ [N], $\theta = \text{rotation angle}$ [$^\circ$], $\omega = \text{angular frequency}$ [rad/s]
\( M_{ei} = \text{eccentric moment} \quad [\text{kg} \cdot \text{m}] \)

It is apparent from Figure 2.20 that the horizontal components \( F_h(t) \) of a pair of counter-rotating eccentric weights cancel each other out at all times whereas the vertical components \( F_v(t) \) combine. The sum of the vertical force generated by the individual eccentric weights is the total vertical dynamic force generated by the vibrator \( F_{V,TOT}(t) \):

\[
(2.20) \quad F_{V,TOT}(t) = \sum F_v(t) = \sum M_{ei} \cdot \omega^2 \cdot \sin(\omega t)
\]

\[
(2.21) \quad F_{V,TOT}(t) = M_e \cdot \omega^2 \cdot \sin(\omega t)
\]

Remark: \( F_{V,TOT} \) increases with increasing \( M_e \) and increasing \( \omega \).

The static surcharge force \( F_0 \) is the load of the bias mass minus the suspension force \( F_s \) of the crane in a free hanging system or of the hydraulic system in a leader mounted system. It should be noted that a downward pressure \( (F_s < 0) \) can be applied to the vibrator in a leader mounted system but not in a free hanging system. \( F_0 \) is given by:

\[
(2.22) \quad F_0 = m_0 \cdot g - F_s
\]

Where
- \( m_0 = \text{bias mass} \quad [\text{kg}] \)
- \( g = \text{gravity} \quad [\text{m/s}^2] \)
- \( F_s = \text{suspension force} \quad [\text{N}] \)

The driving force \( F_d \) applied to the head of the pile is the sum of the dynamic force \( F_{V,TOT} \) and the static surcharge force \( F_0 \):

\[
(2.23) \quad F_d(t) = F_{V,TOT}(t) + F_0
\]

\( F_d \) is a downward directed harmonic force oscillating around its average value \( F_0 \) whose theoretical variations are depicted in Figure 2.21.

---

*Figure 2.21:* Theoretical driving force \( F_d \) vs. time, modified after Viking (2002).
The free hanging double displacement amplitude $S_0$ is the theoretical peak to peak displacement of the free hanging sheet pile (no contact with soil or a neighboring sheet pile). $S_0$ depends on the eccentric moment of the vibrator and on the dynamic mass $m_{dyn}$, i.e. the combined mass of all the oscillating parts. Assuming a rigid body behavior, the double amplitude is expressed:

$$(2.24) \quad S_0 = 2 \cdot s_0 = 2 \cdot \frac{M_e}{m_{dyn}}$$

Where $S_0 =$ double amplitude $[m]$
$s_0 =$ single amplitude $[m]$
$M_e =$ eccentric moment $[kgm]$
$m_{dyn} = m_{excitor\ block} + m_{clamp} + m_{sheet\ pile} = \text{dynamic mass} \ [kg]$

Remark: $S_0$ increases with increasing $M_e$ and decreasing $m_{dyn}$.

The actual displacement amplitude of the pile $S_p$ is always lower than $S_0$ due to soil resistance, interlock clutching and other losses.

Classification of vibratory drivers

Vibratory drivers can be classified (see Table 2.8) according to the parameters listed in the previous section. It should be noted that the categories are not necessarily mutually exclusive and there are, for example, standard frequency variable eccentricity vibro-drivers.

Table 2.8: Types of modern vibratory drivers, modified after Holeyman (2002) and Viking (2006).

<table>
<thead>
<tr>
<th>Type</th>
<th>Frequency [Hz]</th>
<th>Eccentric moment $M_e$ [kgm]</th>
<th>Max. driving force $F_d$ [kN]</th>
<th>Disp. amplitude $S_p$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard frequency</td>
<td>&lt; 30</td>
<td>up to 230</td>
<td>up to 4600</td>
<td>up to 30</td>
</tr>
<tr>
<td>High frequency</td>
<td>&gt; 30</td>
<td>6 – 45</td>
<td>400-2700</td>
<td>13 – 22</td>
</tr>
<tr>
<td>Variable eccentricity</td>
<td>30 – 40</td>
<td>0 to 10 – 54</td>
<td>600-3300</td>
<td>14 – 17</td>
</tr>
</tbody>
</table>

The standard frequency vibrators are still the most in use today because they have a variable frequency range which makes them efficient in most soils. Their high displacement amplitude also makes them advantageous in dense soils.

High frequency vibrators were developed to avoid transmission of harmful ground vibrations due to resonance but their lower driving force and displacement amplitude limit their use.

The variable eccentricity vibrators are described on page 21. This technology is very advantageous in densely built urban areas which are more vibration-sensitive.

2.3.2 Sheet piles

Sheet piles are steel profiles usually installed in tight construction areas to form a temporary retaining wall around an excavation. The wall is formed by driving the individual sheet piles in interlock and can be made watertight to protect the pit from flooding or to prevent lowering of the groundwater table. The following geometric, mechanical and dynamic properties of sheet
piles have been judged important for the generation of ground vibrations: sectional properties, (Holeyman, 2002), eccentric clamping and interlock resistance, (Viking, 2002), dynamic behavior (Whenham, 2011).

**Sectional properties of sheet piles**

Holeyman (2002) characterizes sheet piles by the following parameters:

\[
\begin{align*}
A_P &= \text{profile section} \quad [m^2] \\
L &= \text{profile length} \quad [m] \\
\Omega &= \text{profile circumference} \quad [m] \\
E &= \text{material Young modulus} \quad [\text{MPa}] \\
\rho &= \text{material volumic mass} \quad [\text{kg/m}^3]
\end{align*}
\]

Given an embedment length \( z \) [m], the area in contact with the soil can be calculated, for both the shaft and the cross-section. Moreover, the mass of the profile and the longitudinal wave velocity in the profile are given by:

\[
\begin{align*}
(2.25) \quad m &= \rho AL \quad [\text{kg}] \\
(2.26) \quad c &= \sqrt{\frac{E}{\rho}} \quad [\text{m/s}]
\end{align*}
\]

**Eccentric clamping**

The sheet pile’s transversal and flexural motions are often overlooked, (Whenham, 2011) however, they can generate high levels of horizontal vibrations, (Lidén, 2012). Eccentric clamping is a common cause of pile flexural motion due to the introduction of a bending moment at the sheet pile head as described in Viking (2002).

![Figure 2.22: Effect of eccentric clamping, from Lidén (2012), originally from Viking et al. (2000).](image-url)
Interlock resistance
The interlock (or clutch) resistance $R_C$ is the shear force transmitted from one sheet pile to the next through the interlock, (Whenham, 2011). It is mainly caused by friction of soil particles in the interlock but also by steel on steel friction, especially if the sheet piles are not correctly aligned or are in bad condition after multiple uses, (Viking, 2006). Field studies by Legrand et al. (1993) mentioned in Viking (2002) have shown that the interlock resistance can cause a two to five times increase in the ground vibration magnitude. However, it is difficult to evaluate on site and is often overlooked.

![Illustration of an interlock and varying interlock gap, modified after Viking (2002).](image)

**Figure 2.23:** Illustration of an interlock and varying interlock gap, modified after Viking (2002).

Dynamic behavior: rigid body motion
A common assumption in vibratory driving is that the sheet pile behaves as a rigid body, (Whenham, 2011). While in impact driving, the hammer impact causes a high frequency impulse to travel down the pile, in vibratory driving, the driving frequency is relatively low and the associated wave lengths are longer. For sufficiently long wave lengths, i.e. sufficiently low driving frequencies, the pile can be considered to move as a rigid body.

According to Massarsch (2000), the diagram in Figure 2.24 can be used to judge whether a pile behaves as a rigid body for a given frequency.

![Diagram for estimating whether a pile behaves as a rigid body for a given frequency, modified after Massarsch (2000).](image)

**Figure 2.24:** Diagram for estimating whether a pile behaves as a rigid body for a given frequency, modified after Massarsch (2000).
Viking (2002) cited the following “rule of thumb”, based on laboratory studies of O’Neil & Vipulanandan (1989), to determine whether a pile behaves like a rigid body for a given driving frequency:

\[
(2.27) \quad f_d \leq 0.1 \cdot f_n = 0.1 \cdot n \cdot \frac{c_p}{2L} = n \cdot \frac{c_p}{20L}
\]

Where
- \( f_d \) = driving frequency [Hz]
- \( f_n \) = natural longitudinal frequency of the pile [Hz]
- \( n \) = mode number [-]
- \( c_p \) = P-wave velocity [m/s]
- \( L \) = pile length [m]

Another rule suggested by Viking & Bodare (1998), mentioned by Viking (2002), is based on the idea that the time it takes for the driving force to change from zero to maximum (i.e. \( T_d/4 \)) should be greater than twice the time it takes a stress wave to travel back and forth along the pile (i.e. \( 4L/c_p \)):

\[
(2.28) \quad \frac{T_d}{4} \geq \frac{4L}{c_p} \quad \text{i.e.} \quad f_d \leq \frac{c_p}{16L}
\]

Where
- \( T_d \) = period associated to the driving frequency \( f_d \) [Hz]
- \( f_d \) = driving frequency [Hz]
- \( c_p \) = P-wave velocity [m/s]
- \( L \) = pile length [m]

### 2.3.3 Soil conditions

Vibratory driving was originally developed for sands and it is generally recognized that vibratory driving is particularly efficient in loose granular soils (Iwanowski & Berglars, 1986), (Dowding, 1996), (Massarsch, 2000). However, experimental results have shown that it is also possible to apply the method in moderately stiff saturated clays as well as unsaturated and dense sands even though the degradation of shear resistance is less pronounced in these soils, (Whenham, 2011). Moreover, the method is regularly used for cohesive soils in Sweden, with satisfactory results.

Soil conditions have been found to have an impact on the soil vibrations generated by vibratory driving. This aspect is developed in section 2.4.2.
2.4 Current understanding of sheet pile – soil interaction

The levels of groundborne vibration observed at any point depend on several features of the pile–soil interaction: the amount of transmitted energy, the shape this energy is transmitted in, i.e. the type of wave (P- or S-) and the primary location of the energy transfer (pile shaft and/or toe), which in turn depend on the soil type, saturation level, layering, etc, as well as vibratory equipment properties and operation.

Moreover, as a sheet pile is driven, the embedded shaft length increases and the properties of the soil encountered by the pile shaft and toe vary, (Hope & Hiller, 2000) leading to variability in the features listed above. This section describes the tools used today in order to describe the sheet pile – soil interaction.

2.4.1 Soil resonance

Vibratory drivers produce a steady-state motion, forcing the ground particles to vibrate in a certain mode, regardless of the ground characteristic frequency. Resonance can occur when the driving frequency coincides with one of the soil’s characteristic frequencies, (Hintze et al., 1997). The soil and the pile are thus oscillating together resulting in maximum ground vibrations and minimum penetration.

The phenomenon is called soil stratum resonance and is the excitation of a specific soil layer when the driving frequency coincides with its fundamental natural frequency which depends on the properties of the soil material and on the thickness of the layer, (Dowding, 1996):

\[
fd = \frac{cs}{4H}
\]

Where  
- \(fd\) = driving frequency \([Hz]\)
- \(cs\) = S-wave velocity \([m/s]\)
- \(H\) = layer thickness \([m]\)

Layers between 1-5 m thick therefore may produce potential resonance hazards for driving frequencies of 20-30 Hz in soils with shear wave velocities of 120-600 m/s. A good example of strong ground and structure vibrations due to resonance was reported by Erlingsson & Bodare (1996) at a rock concert in the Nya Ullevi Stadium in Gothenburg, Sweden. By jumping in time to the music at about 2.4 Hz, the audience excited vibrations of a 25 m thick clay layer.

2.4.2 Soil resistance

It has been suggested by several authors (D’Appolonia, 1971), (Hope & Hiller, 2000), (Massarsch & Fellenius, 2008), that soil resistance is the most important soil–related factor influencing piling-induced vibrations. In resistive soils, D’Appolonia (1971) suggested that since the set is low, a significant portion of the hammer (or vibrator) energy is transformed into groundborne vibrations. On the other hand, in easily penetrated soils, most of the hammer energy goes to the advance of the pile instead. As pointed out by Hope & Hiller (2000), the problem with this explanation is that it compares the work of a large force (high resistance) over a small distance and the work of a small force (low resistance) over a large distance.
Generally a distinction is made between the dynamic shaft resistance $R_S$, whose direction varies with the up- and downward motion of the sheet pile, and the dynamic toe resistance $R_T$, which varies between zero and a maximum value as depicted in Figure 2.25.

![Figure 2.25: Schematic representation of a) sheet pile penetrative motion, b) dynamic shaft resistance, c) dynamic toe resistance, from Deckner (2013), originally from Viking (2000).](image)

According to Massarsch (2000), the dynamic resistance along the shaft and the toe is mainly affected by four vibro-parameters, whose influence vary depending on the soil type (granular or cohesive) and firmness. The influence of these factors is summarized in Table 2.9.

<table>
<thead>
<tr>
<th></th>
<th><strong>Shaft resistance</strong></th>
<th><strong>Toe resistance</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil firmness</td>
<td>Low</td>
<td>Medium</td>
</tr>
<tr>
<td>Driving frequency $f_d$</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Sheet pile velocity $\dot{z}$</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Displacement amplitude $S_0$</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Number of cycles</td>
<td>++</td>
<td>-</td>
</tr>
</tbody>
</table>

**Table 2.9:** Influence of the main factors on shaft and toe resistance, according to Massarsch (2000).

<table>
<thead>
<tr>
<th></th>
<th><strong>Shaft resistance</strong></th>
<th><strong>Toe resistance</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil firmness</td>
<td>Low</td>
<td>Medium</td>
</tr>
<tr>
<td>Driving frequency $f_d$</td>
<td>+</td>
<td>++</td>
</tr>
<tr>
<td>Sheet pile velocity $\dot{z}$</td>
<td>+</td>
<td>++</td>
</tr>
<tr>
<td>Displacement amplitude $S_0$</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Number of cycles</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Where   + indicates an increase in shaft/toe resistance  
- indicates an reduction in shaft/toe resistance
Table 2.9 is mainly based on the contractive or dilative behavior of the vibrated soil. The increase or decrease in the shaft and toe resistance is based on assumed shear strength and pore water pressure variations caused by the expected soil densification or dilation.

As stated earlier, the soil’s resistance is highly dependent on the soil’s shear strength. The dynamic shaft and toe resistance are therefore also affected by the soil degradation mechanisms described on pages 15-18 and further developed in section 2.4.3.

The soil’s resistance, especially the dynamic toe resistance can also be affected by the force developed at the pile toe. As reported by Holeyman (2002) and Whenham (2011), Schmid (1967) and later Rodger and Littlejohn (1980) distinguished three regimes of toe resistance for granular soils:

- Sinusoidal resistance: the toe resistance varies as a sinusoidal function in phase with the exciting force. See Figure 2.26 A.
- Impact: the upward force in the pile exceeds the soil uplift resistance and the pile toe repeatedly separates from the soil and stamps it. The toe resistance increases with increasing amplitude of the exciting force. See Figure 2.26 B-C.
- Instability: the downward force in the pile exceeds the soil compressive resistance, i.e. leads to compressive failure. The toe resistance has reached its maximum and can no longer increases with the exciting force. See Figure 2.26 D-E.

![Graphs of force vs time for different regimes of toe resistance.](image)

**Figure 2.26:** Dynamic toe resistance in granular soils for increasing driving force, modified after Whenham (2011) originally from laboratory tests by Rodger & Littlejohn (1980).
2.4.3 Soil behavior in the near field
In the zone near the piling operations, described here as the near field, typically of the order of meters, the ground response is not well understood and the prediction methods are not well developed, (Head & Jardine, 1992).

Denies (2010) suggested the zoning presented in Figure 2.27. In the linear elastic zone, far field, the shear modulus of the soil remains unchanged in spite of the vibration transmission. In the nonlinear elastic zone, or near field, the shear modulus is progressively reduced until a residual value. Along the profile, a strongly remolded zone is often postulated where important convective movements take place, (D’Appolonia, 1971), (Denies, 2010). In that remolded zone it becomes very difficult to define a tangent modulus. Moreover, slippage can occur along the pile shaft, (Denies, 2010).

Sheet pile driving in clay
A conceptual model of clay disturbance during impact driving of displacement piles was suggested by Zeevaert (1950) as reported by D’Appolonia (1971) and presented in Figure 2.28. Because the driving is too rapid to allow for drainage, the clay is considered temporarily incompressible, and a volume of clay equal to the pile is assumed to be displaced, from the pile toe, sideways and upward into zone I which is thus a highly distorted and remolded zone. The soil in zone II, which may extend to several pile diameters, experiences smaller shear distortions but the lateral stress increases sufficiently for the clay to undergo undrained shear failure. In zone III, the lateral stress increases but not enough to cause shear failure.

Sheet pile driving in granular soils
Liquefaction has often been invoked as the explanation for shear strength reduction near sheet pile driving in granular soil, (Viking, 2002), but several authors have also made laboratory observations of shear strength reduction due to vibro-fluidization, see page 16. Viking (2002) made the hypothesis that the main factor for granular soils'
shear strength reduction near vibratory-driven sheet piles is soil vibro-fluidization and that liquefaction is merely an enhancing phenomenon in saturated conditions. According to Rodger & Littlejohn (1980) as reported by Denies (2010), the fluidized zone is located along the sides of the vibrated pile for profiles vibrated in loose soils and experiencing mostly shaft resistance. The fluidized zone is assumed to extend beneath the pile but the decrease of shear strength at the toe is not clearly demonstrated. However, for piles of larger point resistance, experiments by Rodger & Littlejohn (1980), cited by Denies (2010), highlight the fact that there is no fluidization beneath the pile end. Instead, the occurrence of an impact situation is assumed.

2.4.4 Conceptual models for vibration generation

**Toe- and shaft-generated vibrations**

Attewell & Farmer (1973), Head & Jardine (1992), Athanosopoulos & Pelekis (2000), Hope & Hiller (2000) all made a distinction between vibrations generated by the shaft and vibrations generated by the toe of the pile, see Figure 2.31.

Along the shaft of the sheet pile, resistive shear forces are believed to induce vertically polarized shear waves in the surrounding ground. For vibratory driving, the wave front is thought to propagate almost cylindrically from the pile shaft, (Attewell & Farmer, 1973), (Hope & Hiller, 2000), see Figure 2.31.

Massarsch (2000) presented the hypothesis that the vertical oscillation movement of the shaft also induces a horizontal movement in granular soils via the angle of internal friction. The horizontal vibration frequency should thus be double the pile’s. He also claimed that horizontal vibrations are relatively small in cohesive soils due to the low angle of internal friction.

At the toe of the sheet pile, the displacement (and occasional impacting) of the soil is supposed to generate both P- and S- waves which propagate from the pile tip with spherical wave fronts, (Athanosopoulos & Pelekis, 2000), see Figure 2.31.

Moreover, the principal source of vibration is thought to vary according to the soil encountered by the sheet pile, see Figure 2.29. During the penetration of a deep sand layer (low toe resistance), the main vibration source is the shaft, during penetration of a firm layer, most of the energy spreads from the toe, (Massarsch, 2000).

This complicates the definition of the distance to the source from a given observation point, see Figure 2.29. The horizontal distance is often used but this can cause a large error near the sheet pile. According to Hiller & Hope (1998) systematically using the toe as the sole source of vibration also causes errors. For example, when the shaft friction is dominant, the pile can act as a line source of vibration rather than a point source, which makes it difficult to define a distance to the vibration source.
The interaction of P- and S-waves in a half-space gives rise to surface R-waves. There are several rules of thumb to obtain the minimal distance $d_{\text{crit}}$ for R-wave generation, see Figure 2.30. Dowding (1996) presented one method proposed by Daemen et al. (1983) in the context of blasting and based on the P- and R-wave velocities:

\[
d_{\text{crit}} = \frac{c_R\cdot D}{\sqrt{c_P^2 - c_R^2}}
\]

Where $D = \text{depth of the source} \ [\text{m}]$
$c_R = \text{R-wave velocity} \ [\text{m/s}]$
$c_P = \text{P-wave velocity} \ [\text{m/s}]$

Another was suggested by Massarsch & Fellenius (2008). The closest distance $d_{\text{crit}}$ where R-waves can form for a given penetration depth $D$ is defined geometrically by:

\[
d_{\text{crit}} = D \cdot \tan(\theta_{\text{crit}})
\]

Where $\theta_{\text{crit}} = \arcsin\left(\frac{c_S}{c_P}\right)$

Massarsch & Fellenius (2008) suggest that $d_{\text{crit}}$ is approximately half the embedment depth for dry coarse-grained soil, grows shorter for saturated loose soils and is almost zero for clay.
Figure 2.31 summarizes the conceptual model for toe- and shaft-generated vibrations for vibratory sheet pile driving.

Figure 2.31: Generation of body and surface waves during vibratory driving, modified after Attewell & Farmer (1973).

**Impedance**

A theory using the acoustic impedances of the pile and the soil has been developed to quantify the vibratory energy transferred to the soil. The theory was developed for impact driving and can therefore not be fully transposed to vibratory driving but some elements can be of interest.

*Impedance* is a measure of a system’s resistance to an applied force, (Richart et al., 1970) and is defined as:

\[ Z = \rho \cdot c \cdot A_p \quad \text{with} \quad c = \sqrt{\frac{E}{\rho}} \]

Where
- \( Z \) = impedance of the material [N.s/m]
- \( \rho \) = material density [kg/m³]
- \( c \) = wave velocity in the material [m/s]
- \( A_p \) = cross section of the pile [m²]
- \( E \) = Young’s modulus of the material [Pa]
A material’s specific impedance can be defined as:

\[ z_s = \rho \cdot c \]  

Where \( z_s \) = material’s specific impedance \( [\text{N.s/m}^3] \)

According to Massarsch & Fellenius (2008), the dynamic shaft and toe resistance are related to pile velocity and to the specific soil impedance for S- and P-waves respectively. Assuming that the dynamic shaft and toe resistance govern the amount of vibratory energy transferred to the soil, Massarsch & Fellenius (2008) estimated the amount of transferable energy by using the ratio of pile and soil impedances.

The main conclusion from the theory is that the contrast in the soil and pile’s acoustic impedances limits the energy transmission. Since the piles are stiffer than the ground, this translates into:

- ground vibrations increase with decreasing pile impedance
- ground vibrations increase with increasing specific soil impedance.
2.5 Field tests – conventional methods and past experience

Vibrations caused by construction operations can be measured in real-time in order, for example, to provide the contractor with monitoring data or researchers with experimental data. The most common vibration measurement methods are presented and compared in this section along with analysis methods and data presentation habits. The results from a selection of previously published field studies are summarized in section 2.5.2.

2.5.1 Data acquisition, processing and presentation

Data acquisition is done in the field with the help of portable systems. Sensors convert the studied motion to an electrical signal which is transmitted through cables to an amplifying system and on to a tape, digital or paper recorder which can store the data until it is needed for analysis or presentation, (Dowding, 1996).

Sensors

Vibratory motions can be characterized by displacement, velocity or acceleration and there are specific sensors for each quantity. Vibrations caused by vibratory driving are generally measured with geophones, which are velocity sensors, and accelerometers, which are acceleration sensors. They are therefore the only types described here. Both types are designed on the basis of a single degree of freedom system where a mass on a spring moves in relation to a frame secured to the vibrating object (Richart et al., 1970). The inertia of the mass leads to a differential displacement between it and the frame.

A geophone's output signal is generated by a coil moving through a magnetic field, (Dowding, 1996) where the induced voltage is directly proportional to the relative velocity between the coil and the magnetic field (Richart et al. 1970). Geophones are designed to operate in frequencies above their natural frequency, typically 5 Hz, and their mass and size increase rapidly for lower natural frequencies, (Möller et al., 2000). Geophones are often used in ground vibration measurements (see section 2.5.2) and have a wide range of applications in dynamic soil testing, (Möller et al., 2000).

There are several types of accelerometers out of which the most common are the piezoelectric, piezoresistive and more recently developed variable capacitance accelerometers, (Dowding, 1996), (Endevco, 2012). Accelerometers are designed to operate under their natural frequency which is hardly ever less than 1000 Hz and usually considerably higher, (Richart et al. 1970). The piezoelectric accelerometers use the property of certain crystals to produce a voltage difference between their faces when subjected to the inertial force $m \cdot a$ of the seismic mass. They often have a very wide frequency range (a few Hz to 30 kHz) which makes them attractive for both shock and vibration measurements, (Dowding, 1996), (Endevco, 2012). The piezoresistive accelerometers use a solid-state silicon resistor whose electrical resistance changes in proportion to the mechanical stress caused by the seismic mass. They usually have a frequency range down to 0 Hz but their low sensitivity makes them difficult to use for vibration measurement, (Dowding, 1996), (Endevco, 2012).
The variable capacitance accelerometer is among the newer accelerometer technologies. It uses the capacitance variation which is caused by the movement of a suspended electrode (the proof mass) between two fixed electrodes (secured to the frame), see Figure 2.32 (a). Each sensor has many capacitor sets in order to make the cumulated capacitance difference detectable, see Figure 2.32 (b). They are characterized by a high sensitivity, measurement down to 0 Hz and excellent temperature stability. They are perfectly adapted for measurement of low frequency vibrations and steady state accelerations, (Endevco, 2012).

The last 20 years have seen the accelerated development of *microelectromechanical systems* or *MEMS*. The technology is based on mechanical elements like cantilevers or membranes which are manufactured on scale closer to microelectronics than usual mechanics and is particularly adapted to the miniaturization of sensors, see Figure 2.33. The MEMS technology has profited from the development of silicon based micro-fabrication and is now applied to accelerometers, gyroscopes, pressure sensors and microphones to only cite a few, (Beeby et al. 2004), (Andrejašič, 2008).

Data processing
As the output of today’s sensors is most often in the form of current or voltage versus time, some transformations are necessary before the data is legible as vibratory information. The sensor’s calibration factor must be applied to the data and the eventual offset subtracted. Integration and differentiation can then be performed in order to obtain several derivatives of motion.
Data presentation: time and frequency domains
Depending on which information is most interesting to focus on, vibratory data (acceleration, velocity or displacement) can be presented in the time domain, as a time history, or in the frequency domain, as a frequency spectrum. As schematized in Figure 2.34, a time history is best to evaluate the amplitude of the vibratory motion and a frequency spectrum is the most practical to estimate the dominant frequencies, (Möller et al., 2000). Transition between the time and frequency domains is performed by the Fourier Transform, nowadays numerically implemented in the Fast Fourier Transform (FFT) algorithm used in most signal analysis software, (Hewlett-Packard, 1994).

Data presentation: components of motion
Particle velocity is often the derivative of motion chosen to characterize vibrations in the context of construction activities because it is proportional to the dynamic strain induced in the ground and can be linked to observed cracking, (Dowding, 1996), (Athanasopoulos & Pelekis, 2000). The peak particle velocity (PPV) can however have several definitions as listed by Head & Jardine (1992), Hiller & Hope (1998) and Athanasopoulos & Pelekis (2000):

(a) the peak component, i.e. the maximum single value of the three directional components;
(b) the peak vertical component;
(c) the peak instantaneous (true) vector sum of the three components;
(d) the maximum vector sum of the three components regardless of the time of occurrence, also called simulated resultant.

According to Dowding (1996), peak vibration levels should always be reported as the peak component or as the peak true vector sum. The maximum vector sum is considered too conservative by Dowding (1996) and Athanasopoulos & Pelekis (2000).

Particle acceleration can also be used to describe vibratory motions, see section 2.2.1. The development of miniaturized and reliable accelerometers has made them practical to use, especially for pile instrumentation or subsurface measurements, see section 2.5.2. The discussion about the definition of peak particle velocity is transposable to the definition of peak particle acceleration.

Particle displacement paths can be visualized by plotting particle displacement in three orthogonal directions, if a triaxial sensor has been used. For ground vibrations, particle displacement paths can help determine the type of wave causing the particle motion, (Athanasopoulos & Pelekis, 2000).
However, since most sensors measure acceleration or velocity, the signal needs to be integrated to obtain particle displacement, see page 39. It has been suggested that the particle motion could
be visualized without recourse to integration. Lidén (2012), for example, used particle velocity plotted in three orthogonal directions to present particle motion, see Figure 2.35.

![Figure 2.35](image)

**Figure 2.35:** Representing particle motion with the use of particle velocity, from Lidén (2012).

### 2.5.2 Previous field studies

This section presents the main results and conclusions from a selection of previously published field studies.

**Clough & Chameau (1980)**

The paper presents the results from a measurement program, started in 1978, monitoring the effects of vibratory sheet pile driving near the San Francisco Bay. It includes acceleration, velocity and settlement measurements. Only the acceleration measurements are described here.

**Sheet piles and driving equipment**

The sheet pile properties are summarized in Table 2.10.

**Table 2.10:** Sheet pile properties, summarized from Clough & Chameau (1980).

<table>
<thead>
<tr>
<th>Model</th>
<th>Length</th>
<th>Section modulus</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>-</td>
<td>10-15 m</td>
<td>742 cm³</td>
<td>63.4 kg/m</td>
</tr>
</tbody>
</table>

The vibratory driver was a free-hanging ICE Model 812 vibrator unit working at a fixed driving frequency of 18 Hz.

**Measurement equipment and procedure**

Accelerations were measured by a uniaxial accelerometer held by a vise clamp set firmly into the ground surface. The axis of the accelerometer was alternatively oriented “horizontally and transverse to the sheet pile” and “vertically”. By repeating the measurement at different locations, a distribution of peak acceleration with distance from the pile was established.

The output of the accelerometer was fed directly into a tape recorder; the tapes were processed using a Fourier Analyzer system.
Soil conditions
Three different soil profiles are distinguished here:
- MG1: Fill (~ 5-6 m) overlaying soft bay sediments, mainly silty clays, (~ 20 m)
- MG2: Fill (~ 4 m) overlaying firm natural sands
- E1 & E2: Gravel and sand fill (~ 9-11 m) overlaying bay mud (~ 6-12 m) lying on alternating layers of dense sand and firm clay.

Acceleration levels
Ground accelerations measured at the surface are presented in Table 2.11 for the different test sites and with the distinction made by Clough & Chameau between “normal driving” and “hard driving”. Hard driving occurred when the sheet pile encountered an obstacle and was “rattled” by stress waves reflecting from the pile toe.

Table 2.11: Ground acceleration levels at 3-5 m from the sheet pile, summarized from Clough & Chameau (1980).

<table>
<thead>
<tr>
<th>Test site</th>
<th>Direction</th>
<th>Normal driving</th>
<th>Hard driving</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>MG1 &amp; MG2</td>
<td>Horizontal</td>
<td>0.10g-0.15g</td>
<td>0.15g-0.3g</td>
<td>Peak accelerations were lower at MG1 than MG2 and the attenuation was higher at MG1.</td>
</tr>
<tr>
<td></td>
<td>Vertical</td>
<td>0.08g-0.13g</td>
<td>0.15g-0.2g</td>
<td></td>
</tr>
<tr>
<td>E1 &amp; E2</td>
<td>-</td>
<td>0.2g-0.3g</td>
<td>0.4g-0.5g</td>
<td>Horizontal and vertical accelerations had the same magnitude for E1 &amp; E2.</td>
</tr>
</tbody>
</table>

Comments and conclusions
- The soft clayey soils at MG1 reduced accelerations more rapidly than the denser, firmer soils at MG2, apparently due to their greater damping capacity.
- There was little observable difference in damping for horizontal and vertical vibrations.
- Higher absorption was observed for hard driving than for normal driving. This was thought to come from the higher strain levels induced by hard driving, which could lead to more material damping.
- Driving into rubble or rock (hard driving) led to ground acceleration levels twice as high as those observed under normal driving.

Athanasopoulos & Pelekis (2000)
The paper presents inner urban measurements of ground vibrations during the vibratory driving of sheet piles at nine different sites in Patras, Greece, reported in terms of particle velocities vs. distance from the source. Particle displacement paths were reconstructed by integration of the particle velocities.

Sheet piles and driving equipment
Three different type of sheet piles were used, see Table 2.12. The sheet pile lengths ranged from 7 to 10 m. Four different vibratory drivers were used, see Table 2.12.
Measurement equipment and procedure

The measurements were performed at various distances from the sheet pile, on paved surfaces and sidewalks, as well as on the ground floor and upper floors of adjacent buildings. Geophones were installed in three orthogonal directions at each measurement point so as to measure vertical, longitudinal and transversal particle velocities.

The geophones were connected to a portable PC Notebook equipped with a data acquisition card and loaded with software appropriate for digital signal acquiring and processing.

Soil conditions

The soil conditions were similar for the nine sites: alluvial deposits of gravel, sand, silt and clay down to a depth of 8 m, and below that, a layer of low plasticity clay was encountered in most of the sites.

Acceleration levels

Ground vibrations were reported by Athanasopoulos & Pelekis in the form of peak component particle velocity and are presented in Table 2.12. Corresponding accelerations are calculated in the present report with the help of the driving frequencies reported by Athanasopoulos & Pelekis.

Table 2.12: Driving conditions and particle velocity from Athanasopoulos & Pelekis (2000) as well as calculated acceleration levels.

<table>
<thead>
<tr>
<th>Test site</th>
<th>Sheet pile</th>
<th>Vibratory driver</th>
<th>Driving frequency f</th>
<th>Peak component particle velocity v</th>
<th>Acceleration $a = v \times 2\pi f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Larssen III (new)</td>
<td>MGF RBH 60M</td>
<td>13 Hz</td>
<td>7 mm/s at 0.2 m</td>
<td>0.06g</td>
</tr>
<tr>
<td>B</td>
<td>PU16</td>
<td>MS 5H4</td>
<td>23 Hz</td>
<td>5.3 mm/s at 1.5 m</td>
<td>0.08g</td>
</tr>
<tr>
<td>C</td>
<td>Larssen III (new)</td>
<td>MGF RBH 60M</td>
<td>20 Hz</td>
<td>15.0 mm/s at 1.6 m</td>
<td>0.19g</td>
</tr>
<tr>
<td>D</td>
<td>Larssen 22 (used)</td>
<td>ABI RE 10000/3</td>
<td>40 Hz</td>
<td>25.0 mm/s at 1.5 m</td>
<td>0.64g</td>
</tr>
<tr>
<td>E</td>
<td>Larssen III (new)</td>
<td>MGF RBH 60M</td>
<td>25 Hz</td>
<td>10.0 mm/s at 1.8 m</td>
<td>0.16g</td>
</tr>
<tr>
<td>F</td>
<td>Larssen III (new)</td>
<td>ICE 416</td>
<td>17 Hz</td>
<td>6.0 mm/s at 5.15 m</td>
<td>0.07g</td>
</tr>
<tr>
<td>G</td>
<td>Larssen III (new)</td>
<td>MGF RBH 60M</td>
<td>21 Hz</td>
<td>5.0 mm/s at 3.9 m</td>
<td>0.07g</td>
</tr>
<tr>
<td>H</td>
<td>Larssen III (new)</td>
<td>ICE 416</td>
<td>16 Hz</td>
<td>6.0 mm/s at 2.4 m</td>
<td>0.06g</td>
</tr>
<tr>
<td>I</td>
<td>Larssen III (new)</td>
<td>ABI RE 10000/3</td>
<td>24 Hz</td>
<td>35.0 mm/s at 1.0 m</td>
<td>0.54g</td>
</tr>
</tbody>
</table>
Particle displacement paths

Particle displacement paths for test site H are presented in Figure 2.36, projected on the longitudinal and transverse planes.

![Particle displacement paths](image)

**Figure 2.36**: Particle displacement paths at 2.4 m from the sheet pile, from Athanasopoulos & Pelekis (2000).

Comments and conclusions

- Velocities measured were generally lower than in other studies reported in their literature study, which they believed was due to favorable soil conditions in Patras.
- Particle displacement paths were predominantly of the vertically polarized Rayleigh type.

Viking (2002)

Viking (2002) presented a full-scale field test performed in 1999, in Vårby, a suburb of Stockholm, Sweden. The field test included measurement of ground vibrations and sheet pile acceleration during vibratory driving.

Sheet piles and driving equipment

The driving equipment consisted of a leader-mounted ABI vibrator unit (MRZV 800V) set to a driving frequency of 41 Hz.

Three brand new LX16 sheet piles were used as well as one PU16 sheet pile which was driven 6 times. All four sheet piles were 14 m long.

Measurement equipment and procedure

The PU16 sheet pile was instrumented with three accelerometers, two of which were mounted on the sheet pile vertically, one near the head and one near the toe. The third accelerometer was mounted horizontally at sheet pile mid-height in order to capture the longitudinal vibrations of the sheet pile.

The soil instrumentation consisted of three tri-axial geophones positioned on the ground surface, at three different radial distances from the sheet piles being driven. The geophones measured vertical, longitudinal and transversal particle velocities.
Soil conditions
Apart from the top 1.5-2.0 m layer of topsoil and clay, the soil conditions at the Vårby test site consisted of more than 40 m of glacial sand.

Acceleration levels: sheet pile
In the examples (A4, A5, B2, B3) presented by Viking, the vertical acceleration at the sheet pile toe is relatively constant around 17g for the sheet piles driven freely, see Figure 2.37. On the other hand, the vertical toe acceleration of the sheet piles driven in interlock are more variable and reach higher values, up to 30g.

![Figure 2.37: Vertical toe acceleration and lateral shaft acceleration vs. penetration depth a) with and b) without interlock, from Viking (2002).](image)

Acceleration levels: ground
One ground surface acceleration level is calculated by Viking (2002), using \( a = v \times 2\pi f \), at 1.2 m from a sheet pile driven in interlock and found to be 1.3g.

Comments and conclusions
- The ground vibrations displayed a steady-state response with a frequency essentially the same as driving frequency applied.
- Ground surface longitudinal vibrations were sometimes of the same magnitude or even higher than the vertical vibrations. This observation correlated with the lateral flexibility of the sheet pile, see Figure 2.37.
• Driving in interlock led to ground acceleration levels twice as high as those observed for sheet piles driven freely.
• The head and toe vertical accelerations were in phase. The acceleration at the toe was generally slightly higher than at the head, by about 2-3g during favorable driving. According to the author, this was due to inertial effects on the 0.6 m long sheet-pile mass below the toe accelerometer.
• Lateral acceleration at sheet pile mid-height varied between 50 and 100% of the vertical acceleration as long as the sensor was above ground level or the interlock. Passed that point, the lateral acceleration dropped to ~2.5g as a result of the surrounding soil, stiffening the lateral motion of the sheet pile.

Borel et al. (2002) and Arnould et al. (2006)
Within the context of the French National Project on Vibratory Driving, a full-scale field test was conducted in the harbor of Montoir, at the estuary of the Loire River, in 2001. The test consisted of the vibratory driving of closed-end steel tubes fitted with accelerometers and strain gauges, as well as measurement of ground vibrations during the driving.

Piles and driving equipment
Two closed-end steel tubes (Ø 339 mm, 14 mm thick) were installed for the test. The tubes were 32 m long but only driven to a depth of 18-19 m.
The vibratory driver was a free hanging ICE Model 815 vibrator unit with a fixed eccentric moment operating at a driving frequency of approx. 22-23 Hz.

Measurement equipment and procedure
Both piles were instrumented with accelerometers and strain gauges positioned at the pile head and toe. The sensors were connected to a central data acquisition system with a sampling frequency of 1200 Hz. The tubes’ penetration depth was measured optically.
Surface ground vibrations were measured by 3 three-axial geophones installed at 6 m, 12 m and 18 m from the driven pile. Their acquisition frequency was 1000 Hz.

Soil conditions
The subsoil consisted of 5 m fill (coarse sand and gravel), then alluvial deposits with lenses of sandy clay down to 22.5 m and below, mud clay with fine sand layers, known locally as “la jalle”, down to 36 m.

Acceleration levels
Only the behavior of the pile is reported here as the ground vibrations were measured too far from the source to be of specific interest in the context of this thesis.
The pile vibrations had the following characteristics:

\[ a) \quad b) \quad c) \]

**Figure 2.38:** a) Pile vertical acceleration at a penetration depth of approx. 11 m.  
b) Spectral analysis of the pile acceleration at a penetration depth of approx. 11 m.  
c) Pile displacement at a penetration depth of approx. 11 m.  
Modified after Borel et al. (2002).

"Flattening of the pile toe displacement"

**Comments and conclusions**

- The dominant frequency in the spectral analysis of the pile head acceleration corresponded to the driving frequency (22.5 Hz), however, it was 45 Hz for the pile toe which, according to the authors, translates the flattening of the pile toe displacement at the end of the downward movement visible in Figure 2.38 c).

**Ahlqvist & Enggren (2006)**

Within the context of Ahlqvist & Enggren’s MSc. thesis (SKANSA/KTH), two full-scale field tests were conducted in order to measure the ground vibrations generated from sheet pile driving. The same driving equipment and instrumentation were used in both sites. Sites A and B were located in Sweden, in the cities of Norrköping and Stockholm, respectively.

**Sheet piles and driving equipment**

The driving equipment consisted of a leader-mounted ABI vibrator unit (MRZV 800V) operating at a driving frequency of 40 Hz.
Four 6.2 m long LX16 sheet piles were driven at site A. Ten 9.2 m long LX16 sheet piles were driven at site B but only the results from the last six were presented.

**Measurement equipment and procedure**

Five geophones and three seismometers were used for the tests. They were installed on the ground surface in three orthogonal directions so as to measure vertical, longitudinal and transversal particle velocities. The sensors were distributed between fixed measurement points. A Sony digital tape recorder was used to store the signals from the geophones and seismometers.

**Soil conditions**

Site A: the soil consisted of 4-5 m of fill containing demolition material from previous buildings; under the fill there was 1-3 m of silt with sections of sand and gravel overlaying 4 m of clay. Site B: the soil consisted of 0-1.5 m of fill over 0-1 m of dry layered clay itself over 0-5 m till resting on bedrock.

**Frequency content**

Ahlqvist & Enggren registered an interesting frequency shift during the driving of sheet pile 1 at site A. The dominant frequency of the ground vibrations was 20 Hz, see Figure 2.39 a), while the driving frequency was 40 Hz. Compare with Figure 2.39 b) where the dominant frequency corresponds to the driving frequency (40 Hz).

![Figure 2.39: Frequency content of the vertical vibrations measured at a) 15 m from the driven sheet pile (P1 at site A), b) 1 m from the driven sheet pile (P5 at site B), modified after Ahlqvist & Enggren (2006).](image)

**Acceleration levels**

Only particle velocities measured at site B are reported here because the measurement distances of site A were too large to be of specific interest in the context of this thesis. At site B, the nearest measurement point (mp1) was positioned in front of sheet pile 5, at a distance of 1 m.
Table 2.13: Peak vertical velocities from mp1 at site B, from Åhlqvist & Enggren (2006), and acceleration levels calculated for \( f = 40 \text{ Hz} \)

<table>
<thead>
<tr>
<th>Sheet pile</th>
<th>Dist. to mp1 [m]</th>
<th>Peak vertical velocity ( v ) [mm/s]</th>
<th>Acceleration ( a = v \times 2\pi f ) [g]</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>1</td>
<td>120 mm/s</td>
<td>3.02g</td>
</tr>
<tr>
<td>6</td>
<td>1.2</td>
<td>35 mm/s</td>
<td>0.88g</td>
</tr>
<tr>
<td>7</td>
<td>1.5</td>
<td>30 mm/s</td>
<td>0.75g</td>
</tr>
<tr>
<td>8</td>
<td>2</td>
<td>15 mm/s</td>
<td>0.38g</td>
</tr>
<tr>
<td>9</td>
<td>2.6</td>
<td>10 mm/s</td>
<td>0.25g</td>
</tr>
<tr>
<td>10</td>
<td>3.7</td>
<td>10 mm/s</td>
<td>0.25g</td>
</tr>
</tbody>
</table>

Figure 2.40: Peak vertical velocities from mp1 at site B, modified after Åhlqvist & Enggren (2006).

Comments and conclusions

- A frequency shift was observed between the driving frequency and the frequency of ground vibrations for one sheet pile in site A. No clear explanation was given by Åhlqvist & Enggren.
- Åhlqvist & Enggren concluded that maximum vibrations occurred when the penetration speed of the sheet piles was low.

Whenham (2011)

In 2007, a full-scale field test was conducted by Whenham and other researchers at the BBRI test site in Limelette, Belgium. A series of 11 driving and extraction tests were carried out using the same sheet pile but varying the driving frequencies, displacement amplitudes and clamping systems.

Sheet pile and driving equipment

The sheet pile used for the test was a double Z-shaped pile, whose properties are summarized in Table 2.14.

Table 2.14: Sheet pile properties, summarized from Whenham (2011).

<table>
<thead>
<tr>
<th>Model</th>
<th>Length [m]</th>
<th>Perimeter [m]</th>
<th>Section modulus ( [cm^3] )</th>
<th>Weight [kg/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>-</td>
<td>20</td>
<td>3.3</td>
<td>1880</td>
<td>156.6</td>
</tr>
</tbody>
</table>

Cross section
The driving equipment consisted of a free hanging ICE 36RF vibrator unit operating at driving frequencies from 20 to 38 Hz, single displacement amplitudes between 1.4 and 4.5 mm. Two clamping systems were also tested: a simple clamp (eccentric clamping) and a double clamp (neutral clamping).

**Measurement equipment and procedure**

The test sheet pile was fitted with five vertical accelerometers spread out from 1 m from the pile toe to 2 m from the pile head and five horizontal accelerometers, positioned in the upper half of the sheet pile (from 2 m from the pile head to 8.5 m from the pile toe), measuring in two perpendicular directions, as well as strain gauges. The soil instrumentation consisted of geophones and piezoelectric accelerometers installed on the ground surface at distances varying from 3 m up to 30 m from the sheet pile, as well as a dual seismic cone (SCPT equipment) which provided acceleration measurements at various depths. The sampling frequency was 4096 Hz for the sheet pile data, 2000 Hz for the surface accelerometers, 1000 Hz for the subsurface accelerometers and between 300 and 2000 Hz for the geophones.

**Soil conditions**

The subsoil consists of 1 m of fill over 2-5 m of loam, itself overlaying sand down to 60 m.

**Sheet pile accelerations**

The sheet pile accelerations are reported for two test series comparing the use of single and double clamps. The results are reported in Table 2.15.

<table>
<thead>
<tr>
<th></th>
<th>Single clamp</th>
<th>Double clamp</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical acceleration (V)</td>
<td>30-40g</td>
<td>15-30g</td>
</tr>
<tr>
<td>Longitudinal acceleration (P)</td>
<td>20-40g</td>
<td>10g</td>
</tr>
<tr>
<td>Transversal acceleration (H)</td>
<td>10g</td>
<td>5-10g</td>
</tr>
</tbody>
</table>

**Comments and conclusions**

- For all the series, the acceleration amplitudes were lower at the pile head than at the pile toe. A good concordance was obtained between the measurements and the analytical solution for the steady state vibrations of an elastic body resting on a spring and subjected to a vibratory force at the head.
- High lateral accelerations were measured on the sheet pile and it was suggested that the driving frequency repeatedly matched the pile's lateral resonant frequency. Whenham also illustrated the influence of the clamping device on the induced horizontal vibrations, see Table 2.15.
- Soil vibrations were higher during the driving phase than during the extracting phase if the extraction was done shortly after the driving. Extraction vibrations were drastically increased when a few hours passed between the driving and the extraction.
• From particle displacement paths, Whenham concluded that at a short distance from the pile (~ 5 m), the velocity had an elliptical shape similar to the R-wave motion. At a greater distance, the movement was predominantly radial, i.e. particle motion was mostly horizontal.

**Lidén (2012)**
Lidén’s MSc. thesis presents the vibration measurements performed in May 2010, during a trial sheet piling performed near the Karlstad Theater in Karlstad, Sweden.

**Sheet pile and driving equipment**
Four 12 m long brand new sheet piles of model PU12 were driven for the test. Vibration measurements were only performed for the last three (i.e. all in interlock).
The driving equipment consisted of a variable frequency vibrator of type Dieseko 2316VM driven hydraulically by a power pack Dieseko PVE480. The vibrator unit was a free hanging model but it had been, in this case, mounted to a multitask leader (Banut machine). It was operating at frequencies of 25-30 Hz.

**Measurement equipment and procedure**
Two triaxial geophones were placed on rods which were driven 0.5 m into the ground, at 3.4 m and 7.9 m from the sheet pile line. The fill had been excavated there, so that the geophones were placed on the sand layer. A third geophone, measuring only vertical vibrations, was placed about 15 m from the sheet piles.
The data collection was done in an Ava95 device which had four channels and could only record a sample for up to 70 s. Because of the limited sample length, only the beginning of each driving is recorded.
The sheet pile penetration depth was estimated from video recordings and chalk markings (10 cm spacing) on the profiles.

**Soil conditions**
The soil consisted of approximately 1 m of fill over an 8 m layer of loose fine grained river deposits, mainly sand with thin courses of mud and plant residues; the sand then transitioned into loose silt with courses of sand. A stiffer sand layer was located at a depth of 11-12 m. Beneath this stiffer layer, the soil consisted of clay down to 25 m below the surface. Under the clay there was a 0.5-1 m layer of moraine, resting on rock.

**Acceleration levels**
Ground vibrations were reported by Lidén in the form of particle velocity time histories and are summarized for sheet pile 3 in Table 2.16. Sheet pile 3 was chosen here because the driving frequency is reported by Lidén: 28 Hz.
Table 2.16: Peak particle velocity summarized from Lidén (2012) as well as calculated acceleration levels.

<table>
<thead>
<tr>
<th>Distance from the sheet pile line</th>
<th>Component</th>
<th>Peak velocity $v$ (mm/s)</th>
<th>Acceleration $\alpha = v \times 2\pi f$ (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.4 m</td>
<td>Vertical</td>
<td>5</td>
<td>0.09</td>
</tr>
<tr>
<td></td>
<td>Longitudinal</td>
<td>10</td>
<td>0.18</td>
</tr>
<tr>
<td></td>
<td>Transversal</td>
<td>15</td>
<td>0.26</td>
</tr>
<tr>
<td>7.9 m</td>
<td>Vertical</td>
<td>5</td>
<td>0.09</td>
</tr>
<tr>
<td></td>
<td>Longitudinal</td>
<td>3</td>
<td>0.05</td>
</tr>
<tr>
<td></td>
<td>Transversal</td>
<td>2</td>
<td>0.04</td>
</tr>
</tbody>
</table>

Comments and conclusions

- Horizontal vibrations were found to be larger than vertical vibrations in certain cases. Eccentric clamping and clutch friction are discussed as possible causes for the high horizontal vibrations measured. Lidén also suggested that the loose sand did not offer much resistance to the sheet pile’s bending, leading to high horizontal vibrations.
- Vertical vibrations were close to harmonic, strongly dominated by the driving frequency. Horizontal vibrations were periodic but far from harmonic due to higher frequency overtones. However, the dominating frequency was still the driving frequency.
- Soil conditions were found to affect the horizontal vibrations more than the vertical ones (reaching a stiffer layer for example only caused increase in the horizontal vibrations). Lateral bending of the sheet pile was suggested to explain this observation.
- Vibrations were found to increase with depth. Lidén explains this trend by the increasing contact surface between sheet pile and soil.
- Correlation of vibrations with CPT soundings was difficult to distinguish but Lidén partly attributed this to imprecisions in the penetration depth measurements.
2.6 Conclusions from the literature study

In order to clarify the role of the different elements involved in the generation of ground vibrations during vibratory driving of sheet piles, sections 2.6.1-2.6.3 divide these parameters into three categories according to which element they characterize (vibrator unit, sheet pile, soil). Section 2.6.4 offers a few recommendations regarding data presentation based on experience from previous field tests.

2.6.1 Vibratory driver – related factors

The vibratory driver – related factors which are most often mentioned as influential for the generation of ground vibrations are the following:

- **Driving frequency**: The driving frequency has been found to determine the ground vibrations’ frequency in most field tests, (Viking, 2002), (Lidén, 2012). The issues regarding driving frequency are mostly linked to resonance problems as it can match the soil stratum resonant frequency (linked to layer thickness and wave propagation velocity). This was the basis for the development of high frequency vibrators ($f_d > 30$ Hz) and variable frequency vibrators. Moreover, high driving frequencies seem to be favorable for shaft resistance in granular soil as they lead to (locally) higher accelerations and possible vibro-fluidization.

- **Displacement amplitude**: The displacement amplitude is mainly linked to vibration levels through slippage. For clays, high displacement amplitudes are necessary to overcome the pile – soil adhesion, allowing for sheet pile advance and reducing the vibrations transmitted to the ground. Displacement amplitude is itself dependent on the vibrator’s eccentric moment (positive correlation) and on the system’s dynamic mass (negative correlation).

- **Driving force**: The driving force, and its time variations, determines the amount of energy transferred into the pile. Depending on the soil type, a part of this energy goes to the advance of the sheet pile and the rest is absorbed by the soil in the form of permanent degradation and elastic wave propagation.

2.6.2 Sheet pile – related factors

The sheet pile has a strong part to play in the sheet pile – soil interaction through the following effects, mentioned in the literature study:

- **Rigid body behavior**: A common modeling assumption is that the sheet pile behaves as a rigid body, both longitudinally and laterally. However, several field tests have shown the contrary. Longitudinal deformations of piles have been observed by Borel et al. (2002), accompanied by high frequency overtones at the pile toe thought to translate the “flattening” of the pile toe displacement at the end of the downward movement. Also, Viking (2002) and Whenham (2011) both reported higher vertical accelerations at the pile toe than at the pile head, suggesting an elastic longitudinal deformation of the sheet pile. Lateral deformations of the sheet pile are treated separately in the next point.

- **Eccentric clamping and lateral flexibility**: It has been suggested by several authors (Viking, 2002), (Whenham, 2011), (Lidén, 2012), that gripping the sheet piles by the web (i.e. out
of the neutral axis) introduced bending moments in the sheet pile which lead to high longitudinal ground vibrations. Moreover, a sheet pile with a low section modulus is more prone to bending than a stiffer sheet pile. Field tests have provided support to the eccentric clamping hypothesis. Whenham (2011) observed sheet pile lateral vibrations which were two to four times larger with eccentric clamping (20-40g) than with neutral clamping (10g). Also, Viking (2002) and Lidén (2012) respectively attributed high lateral accelerations at sheet pile mid-height (50-100% of the vertical acc.) and high horizontal ground vibrations to the eccentric clamping of the driven sheet pile.

- **Clutch friction:** Field studies by Legrand et al. (1993), mentioned in Viking (2002), have found interlock resistance to cause a two- to fivefold increase in the ground vibrations. It is believed to be caused by friction of soil particles in the interlock but also steel on steel friction which occurs when the sheet piles are not correctly aligned or when they are deformed from multiple use. Viking (2002) himself, also illustrated the influence of clutch resistance on vibration generation: the vertical toe acceleration was relatively constant for the sheet pile driven freely (~ 17g) but was more variable for sheet piles driven in interlock, and reached higher values, up to 30g.

### 2.6.3 Soil – related factors

The soil is perhaps the most important and certainly the most variable component of the vibratory system. The main characteristics thought to influence vibration generation are the following:

- **Soil resistance:** It has been suggested by several authors that soil resistance is the most important soil – related factor influencing piling-induced vibrations, (D’Appolonia, 1971), (Hope & Hiller, 2000), (Massarsch & Fellenius, 2008). Along the shaft, resistive forces are believed to induce vertically polarized S-waves in the surrounding soil. At the pile toe, the displacement (and occasional impacting) of the soil is thought to generate both P- and S-waves propagating spherically from the toe, (Attewell & Farmer, 1973), (Hope & Hiller, 2000). The main source of vibration (shaft or toe) is believed to vary depending on the soil encountered. Loose soils tend to generate mostly shaft resistance whereas in firm layers, most of the energy spreads from the sheet pile toe, (Massarsch, 2000). In firm soils, an impact situation can develop at the toe, where the pile repeatedly separates from the soil and stamps it. Harder driving usually leads to higher ground vibrations, as observed by Clough & Chameau (1980). They reported that driving sheet piles into rubble or rock caused surface accelerations double those of normal driving at a distance of 3-5 m (from 0.1g for normal driving up to 0.2g in vertical direction and to 0.3g in horizontal longitudinal direction for hard driving) for soft bay sediments (silty clays) at 18 Hz. Moreover, Alqvist & Enggren (2006) concluded that maximum vibrations occurred when the penetration rate was low, which also can be linked to harder driving conditions.

- **Water content:** High water contents should increase the speed of P-waves but reduce the speed of S-waves (through loss of shear strength).
- **Plasticity and soil degradation:** Near the sheet pile, the zone where the ground response is not well understood has been coined “the near field”. Along the profile, a certain amount of slippage is likely to occur and a remolded zone is postulated where important convective movements take place and the soil’s shear strength is strongly reduced.

In cohesive soils, three zones have been postulated for impact driven displacement piles, a first zone which is highly distorted and remolded, typically about a half pile diameter, a second where the increased lateral stress is sufficient to cause undrained shear failure and a third where the stresses are not sufficient to cause failure. Moreover, the repetitive motion can cause fatigue of the soil skeleton which strongly reduces the shear strength and the ability of the soil to transmit vibrations in the near field. The extent of this field has not, to the author’s knowledge, been determined for sheet pile vibratory driving.

In granular soils, laboratory experiments by Denies (2010) have evidenced vibro-fluidization, i.e. a shear strength reduction accompanied by chaotic convective rolls caused by loss of contact between particles submitted to accelerations above 1g. In saturated conditions, liquefaction can also be observed and has long been used to explain the loss of shear strength in vibrated granular soils. However, Viking (2002) has suggested that it is mostly a phenomenon enhancing vibro-fluidization.

It also appears that the soil degradation is mostly temporary. Indeed, Whenham (2011) noticed (during sheet pile extraction) that the soil was in a disturbed state right after the vibratory driving but that the effect disappeared after a few hours.

### 2.6.4 Data presentation

Conclusions regarding the presentation of experimental data are briefly listed here as general recommendations which facilitate test result interpretation.

- **Components of motion:** Particle velocity is commonly used to characterize ground vibrations. However, particle acceleration can also be used but has, in the past, mostly been used to characterize pile motion. Particle motion can be visualized by plotting a component of motion (usually displacement but not necessarily) in three orthogonal directions.

- **Peak vibrations:** Peak vibrations should be reported as the peak component or as the peak true vector sum. In any case, it is important to mention which convention is being used.
3 Field study

3.1 Introduction

The present full-scale field study was planned and executed in the context of F. Deckner’s doctoral thesis. Chosen portions of the collected data were analyzed in this MSc. thesis, see Chapter 4, while the rest will serve in Deckner’s continued work. The study was performed on the construction site of a new orbital tramway in Solna, a suburb of Stockholm (Sweden). The tests were carried out during the driving of seven adjacent sheet piles, in collaboration with Hercules Grundläggning AB which owned the driving equipment and was responsible for the foundation work on site. The instrumentation and data acquisition was arranged by K. Allard and K. Lindgren, retired respectively from Geometrik i Stockholm AB and the KTH Wallenberg Laboratory.

The aim of the field study was to document sheet pile and soil vibrations during vibratory sheet pile driving with focus on:

- sheet pile – soil vibration transfer,
- influence of soil layers, and
- influence of clutch friction.

This chapter presents the different aspects of the field study: site conditions, (section 3.2), the tests’ chronology and layout, (section 3.3), as well as data collection and processing methods, (section 3.4). Finally, concluding remarks regarding these different aspects are made in section 3.5.

3.2 Site – related conditions

This section specifies the site – related conditions of the field study: its location and soil conditions as well as the specifications of the piling rig and sheet piles used on site.

3.2.1 Location

The construction site was located in Solna, a suburb of Stockholm (Sweden), at the northwest corner of the Frösundaleden – Solnavägen intersection. The studied sheet piles were to be part of the retaining wall protecting the west ramp of the tramway tunnel planned under Frösundaleden, see Figure 3.1. The test area was located in the vicinity of km 5 + 900 of the tramway tracks, see Figure A.1.
3.2.2 Soil conditions

The soil conditions presented in this section are estimated based on:

- a small topographic survey performed on May 27th 2013,
- soil investigation reports produced by WSP (2011),
- groundwater measurements performed by NCC and WSP from 2009 to 2013.

Topography

At the time of the field test, the ground in the test area was rather even, in average at level +7.70. However, according to WSP (2011), the ground level was between +6.5 and +7.5 before the construction of Frösundaleden in the 1960s.
Geotechnical conditions
The geotechnical conditions were estimated based on the information presented by WSP (2011), and in particular on the investigation points nearest to the test area, see Figure 3.2.

![Figure 3.2: Plan view of the field test area (in red), and the nearest investigation points, modified after WSP (2011). Section A-A refers to Figure 3.3.](image)

Based on the geotechnical investigation by WSP (2011), a soil profile was estimated along the characteristic section A-A in Figure 3.2. Figure 3.3 illustrates the site’s soil profile as well as the setup of the measurement and driving equipment.

The soil consists of fill, resting on clay, itself on frictional material and bedrock. Due to the large amount of previous construction in the area, the fill layer is rather thick, about 2-3 m in the area of the test. Underneath the fill material lays 7 to 10 m of clay. Estimated soil parameters are listed in Table 3.1. Below the clay is firm, frictional soil which was not further investigated by WSP (2011). It is assumed to be a 1.5-2 m layer of moraine on rock. The bedrock is estimated at levels -3 to -8, slanting towards the south.

**Table 3.1:** Soil properties of the characteristic profile, based on WSP (2011) and on-site observations.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Thickness [m]</th>
<th>$\rho$ [kg/m$^3$]</th>
<th>$\phi_k$ [°]</th>
<th>$\tau_{uk}$ [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill</td>
<td>2.7</td>
<td>1.8</td>
<td>34</td>
<td></td>
</tr>
<tr>
<td>Clay</td>
<td>7-13</td>
<td>1.8</td>
<td></td>
<td>15-20</td>
</tr>
<tr>
<td>Moraine</td>
<td>1.5-2</td>
<td>1.9</td>
<td>36</td>
<td></td>
</tr>
</tbody>
</table>

Groundwater
Groundwater measurements show levels between +4.0 and +5.8 over the past few years, with an average at level +4.6. According to WSP (2011), the groundwater table was sunk in the 1970s during the construction of the subway. It is assumed to have lain around level +7.0 before that.
Figure 3.3: Section A-A: situation sketch and soil profile, (see Figure 3.2 & Figure A.1 for plan location).
3.2.3 Driving equipment

The driving equipment used to drive sheet piles at this construction site was:

- a multi-task Liebherr piling and drilling rig (LRB 125 XL), see Figure 3.4, left,
- a leader mounted high frequency vibrator (Liebherr 1100 H), see Figure 3.4, right.

The equipment was owned and operated by Hercules Grundläggning AB.

The vibrator’s main properties are given in Table 3.2. See Appendix A3 for full specifications of both the piling rig and the vibrator.

*Table 3.2:* Driving equipment specifications, summarized from Liebherr (2010).

| Technical data – Vibrator 1100H | | |
|---------------------------------|-------------------|
| Eccentric moment               | $M_e$              |
| Max. frequency                 | $f_{\text{max}}$  |
| Max. centrifugal force         | $F_{V,\text{tot}}$|
| Max. amplitude                 | $s_0$              |
| Dynamic weight with clamp      | $m_{\text{dyn,vib}}$|
|                                 | 0 – 20 kg.m        |
|                                 | 38.4 Hz / 2300 rpm |
|                                 | 1160 kN            |
|                                 | 19 mm              |
|                                 | 2980 kg            |

*Figure 3.4:* Left: LRB 125 XL piling rig. Photo by K. Viking, 2013/05/28.
Right: Vibrator 1100 H. Photo by F. Deckner, 2012/06/29.
3.2.4 Sheet piles
The sheet piles driven during the field test were used Larssen 603 sheet piles. Figure 3.5 gives the dimensions for an L603 profile and Table 3.3 gives relevant section properties.

![Sheet pile profile Larssen 603, from Thyssen Krupp (2011).](image)

**Figure 3.5:** Sheet pile profile Larssen 603, from Thyssen Krupp (2011).

**Table 3.3:** Section properties for sheet piles L603, summarized from Thyssen Krupp (2011).

<table>
<thead>
<tr>
<th>Sectional properties – Sheet pile L603</th>
<th>( A_p )</th>
<th>83 cm(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cross sectional area</td>
<td>( \Omega )</td>
<td>180 cm</td>
</tr>
<tr>
<td>Circumference</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mass per m</td>
<td></td>
<td>64.8 kg/m</td>
</tr>
<tr>
<td>Second moment of inertia</td>
<td>( I_y )</td>
<td>3830 cm(^4)</td>
</tr>
<tr>
<td>Elastic section modulus</td>
<td>( W_y )</td>
<td>330 cm(^3)</td>
</tr>
</tbody>
</table>

In total seven sheet piles were driven, out of which five were 13.8 m long and two were 11 m long. Sheet piles SP1, SP2, SP3, SP4 and SP5 were the longer ones and were driven to the bedrock, approximately 11-11.5 m deep. Sheet piles SP6 and SP7 were shorter and did not reach down to the bedrock. They were driven to a depth of 10.5-11 m. The sheet piles’ location can be found in Figure 3.2 and in Figure A.1. The sheet piles were driven from west to east, starting with SP1.
3.3 Execution of the field study

The field tests were conducted over a three-day period, from May 27th 2013 to May 29th 2013. On May 27th, the soil instrumentation was installed and three of the sheet piles, (SP1, SP2, and SP4), were fitted with sensors. The measurements themselves were carried out on May 28th and 29th in the form of seven series which consisted in the driving of seven sheet piles.

Varying cable connections allowed for different measurement configurations for each series. Indeed, due to the limited amount of channels in the data acquisition system, only a portion of all the accelerometers’ output could be logged simultaneously. Table 3.4 summarizes the content of the different series.

Table 3.4: Content of the seven measurement series.

<table>
<thead>
<tr>
<th>Series #</th>
<th>Driven sheet pile</th>
<th>Clutch friction</th>
<th>Active measurement points</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>SP1</td>
<td>SP2</td>
</tr>
<tr>
<td>Day 2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>SP1</td>
<td>-1</td>
<td>x</td>
</tr>
<tr>
<td>2</td>
<td>SP2</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>SP3</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>4</td>
<td>SP4</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td>SP5</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>6</td>
<td>SP6</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>7</td>
<td>SP7</td>
<td>x</td>
<td>-</td>
</tr>
</tbody>
</table>

3.3.1 Day 1 – Installing the soil and sheet pile instrumentation

The installation of the soil and sheet pile instrumentation is summarized below. The experimental set-up is sketched in Figure 3.6.

Ground instrumentation

In order to install the ground accelerometers in the clay layer, see Figure 3.3, the fill layer was excavated in two locations (around MP1&MP2 and around MP3).

- Two holes were dug carefully by a mechanical excavator and the digging was interrupted as soon as the clay layer was exposed. This was at elevation +5 for both holes.
- The desired position of MP1, MP2 and MP3 were measured out in relationship to the planed position of the sheet piles.
- The sensors were inserted at the desired depth with the help of fiberglass rods, which were then pulled up, leaving the sensors in place. The orientation of the accelerometers was controlled and, when necessary, adjusted. See Figure 3.7 (top left and top center).
- A ø5 plastic tube was threaded over each cable and attached to a wooden plank set across the holes in order to keep the cables vertical during the refilling of the holes. A layer of fine material was shoveled in and lightly packed. The holes were then refilled carefully with the excavated fill material.

---

1 The convention used in the tables of this thesis is the following: x: Yes - : No
The 0 Hz output of the accelerometers was used to check the inclination of the accelerometers after installation. The angles listed in Table 3.5 represent the deviation from the sensor’s planned vertical position, i.e. its pitch and roll, see Appendix B2.4.

<table>
<thead>
<tr>
<th>Sensor</th>
<th>MP1-1</th>
<th>MP1-2</th>
<th>MP1-3</th>
<th>MP2-1</th>
<th>MP2-2</th>
<th>MP2-3</th>
<th>MP3-1</th>
<th>MP3-2</th>
<th>MP3-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axis</td>
<td>X</td>
<td>Y</td>
<td>X</td>
<td>Y</td>
<td>X</td>
<td>Y</td>
<td>X</td>
<td>Y</td>
<td>X</td>
</tr>
<tr>
<td>Angle [°]</td>
<td>-7</td>
<td>-2</td>
<td>+3</td>
<td>-2</td>
<td>-10</td>
<td>-2</td>
<td>-10</td>
<td>-2</td>
<td>+0</td>
</tr>
</tbody>
</table>

### Sheet pile instrumentation

Sheet piles SP1, SP2 and SP4 were fitted with three tri-axial accelerometers each.

- Protective iron shafts and steel supports were welded to the sheet piles. Three supports were positioned on each sheet pile’s center line, each destined to house an accelerometer, at levels 1 m, 6 m and 11 m from the toe. See Figure 3.7 (top right).
- The cables were threaded through the shafts and the accelerometers were slipped into their cases. Silicon was used as a moisture barrier and protection against small stones. It also held the accelerometers in place in their casings. See Figure 3.7 (bottom left).

The final set-up of the soil and sheet pile instrumentation is sketched in Figure 3.6.
3.3.2 Day 2 & 3 – Measurement series 1-7

The measurement series can be summarized by the following steps.

- **Marking and raising the sheet pile:** the sheet pile to be driven was marked every 0.1 m, then raised vertically and positioned where it was intended to be driven, hanging from the vibratory driver’s hydraulic clamp, barely touching the ground surface.

- **Connecting the cables to the two signal conditioning boxes:** since each series was planned with different cable connections, re-connecting cables to the signal conditioning boxes and documenting the connections was necessary before each driving.

- **Starting and synchronizing the acquisitions:** when the cable connections were ready, the two DAT recorders and the vibratory driver’s logging system were started. Then the go-ahead was given to the operator of the vibratory driver to begin pile installation.

- **Final recording:** when the installation of the sheet pile was complete, the vibratory driver was turned off, as well as its logging system and the DAT recorders. Notes were made concerning for example total driving time, penetration depth and general observations.

- **Repeating:** the procedure was then repeated with the next sheet pile in the line.

---

**Figure 3.7:**

- **Top left:** Ground sensor mounted on a fiber glass rod. Photo by C. Guillemet, 2013/05/27.
- **Top center:** Manual installation of the ground sensors. Photo by K. Viking, 2013/05/27.
- **Top right:** Welded steel case for SP2-2 and protective iron shafts. Photo by C. Guillemet, 2013/05/27.
- **Bottom left:** Installation of sheet pile sensor SP1-3. Photo by C. Guillemet, 2013/05/27.
- **Bottom right:** After installation of the seven sheet piles. Photo by C. Guillemet, 2013/07/29.
3.4 Data collection, acquisition and processing

The instrumentation system used to document the field tests consisted in five main parts:

- the vibratory driver instrumentation,
- a video camera,
- the sheet pile instrumentation,
- the ground instrumentation, and
- the data acquisition system.

The data collection and acquisition systems are illustrated in Figure 3.8. The separate elements are described in sections 3.4.1-3.4.4. Section 3.4.5 describes the data processing which transformed the digital raw-data into the results presented in section 4.2.

Figure 3.8: Illustration of the complete data collection and acquisition system used for the field tests.
3.4.1 Vibratory driver instrumentation

The piling rig was equipped with the Liebherr Process Data Recording (PDE®) system. The PDE® system records data both from the vibratory driver’s hydraulic control system and from external sensors, (Liebherr, 2011). The following information is logged at each trigger (which can be chosen either as a depth or time interval):

- penetration depth [cm]
- penetration speed [m/min]
- vibrator oil pressure \(^2\) [bar]
- static surcharge force \(^3\) [kN]
- vibrator frequency [Hz]
- amplitude [mm]
- time [hh:mm:ss]

The PDE® logs were saved on a memory card in .txt format.

For series 1-3, the trigger was chosen as a 10 cm depth interval. Due to the rapid sheet pile penetration rate, this amounted to 1 to 3 triggers per second during most of the driving. However, as the sheet piles reached the firmer frictional soil, the penetration rate grew more erratic and data collection was not correctly triggered. Therefore, the end of the driving (about 1 m), is absent from the vibratory driver’s log.

For series 4-7, the trigger was chosen as a 1 second time interval in order to avoid this.

3.4.2 Video recordings

As a back-up for the penetration depth and to keep a detailed record of the test chronology, the two entire test days were filmed with a video camera placed about 12 m from the sheet pile line. The sheet piles were marked with chalk every 0.1 m, enabling relatively precise depth readings.

3.4.3 Sheet pile and soil instrumentation

The sheet pile and soil instrumentation consisted of MEMS capacitive accelerometers, see section 2.5.1, of brand Analog Devices, models ADXL377Z and ADXL335Z respectively. The accelerometers’ specifications are available in Appendix B1. Appendix B2 describes the accelerometers’ testing and calibrations, performed in preparation of the field tests.

The ADXL377Z model was chosen for instrumentation of the sheet pile because it can measure large accelerations (up to 200g). These accelerometers were mounted in rectangular casings which slid into supports welded to the sheet piles, see Figure 3.9 (top center) and Figure 3.7 (top right and bottom left).

---

\(^2\) Since technical precisions regarding the vibratory driver’s logging equipment could not be obtained from the manufacturer, this information is unconfirmed.

\(^3\) Idem.
The ADXL335Z model was chosen for measurement of the ground vibrations because it has a high sensitivity (300 mV/g). These accelerometers were mounted in a tube-like casing set on a nail designed to hold the accelerometers straight during the measurements, see Figure 3.9 (top right and bottom).

### 3.4.4 Data acquisition system

The data acquisition system consisted of two acquisition chains working in parallel, see Figure 3.8 and Figure 3.10. Each chain handled 16 channels, totaling 32 channels for the complete acquisition system. For each series, the output from the sheet pile and soil instrumentation was re-distributed over the 32 channels, depending on the connections planned for that series. Synchronization between the two acquisition chains was done in the data processing phase, see section 3.4.5.

Each acquisition chain consisted of:
- a signal conditioning box,
- a DAT recorder, and
- a laptop computer.

**Signal conditioning boxes**
The signal conditioning boxes were assembled by Uno Persson from PC-Persson and consisted of:
- a power supply for the accelerometers and the amplifier,
- a terminal board,
- an amplifier (x 5),
- a low-pass filter set to 250 Hz, and
- output to the DAT recorder.

The two signal conditioning boxes were also equipped with a common self-test trigger button which simultaneously activated the self-test function in the accelerometers from both boxes. This function was used to synchronize the two acquisition chains, see section 3.4.5.

**DAT recorders**
The output from the signal conditioning boxes was recorded by two 16-channel SONY® PC216AX Digital Audio Tape (DAT) recorders, at a sampling frequency of 3000 Hz.

**PCscan**
A software package named SONY® PCscan II accompanied the DAT recorders. This software package enabled real time viewing of the recorded data and served as an interfacing tool between the DAT recorders and the computers to which the data was exported.

![Figure 3.10: Left: Signal conditioning boxes 1 and 2. Photo by C. Guillemet, 2013/05/29. Right: Complete acquisition system: 2 signal conditioning boxes, 2 DAT recorders, 2 laptop computers. Photo by K. Lindgren, 2013/04/18.](image)

### 3.4.5 Data processing
A minimum of data processing was necessary in order to extract analyzable results from the DAT recorder’s raw-data files, and to synchronize the three main sources of data, i.e. the vibratory driver PDE® logs and the two DAT recorders’ files. All the data processing was performed with the help of the MATLAB® software. The different steps are briefly described in this section.4

**Application of calibration factors**
The DAT recorders’ voltage output was divided by the accelerometers’ dynamic calibration factors and by the amplifiers’ amplification factor in order to transform the raw-data files into analyzable acceleration time histories. The transformation is shown in Eq. (3.1).

---

4 The data processing described here was only applied to series 1 and 2 because they were the only ones analyzed in the context of this thesis work.
Pile – Soil Interaction during Vibratory Sheet Pile Driving

(3.1) \[ a = \frac{U}{k_d \times 5} \]

Where

\[ a \] = particle acceleration \( [g] \)

\[ U \] = DAT recorder output \( [\text{mV}] \)

\[ k_d \] = dynamic calibration factor \( [\text{mV/g}] \)

\[ 5 \] = signal conditioning box amplification factor \( [-] \)

**Accelerometer axis transformation**

The accelerometers used for sheet pile and soil instrumentation have a built-in tri-axial coordinate system (XYZ), which depends on their mounting and installation position. In order to facilitate the result analysis, one axis set (VLT) was chosen for all the acceleration data, see Figure 3.11.

![Figure 3.11: Accelerometer axis transformation.](image)

**From acceleration time histories to peak particle accelerations**

After calibration and axis transformation, the recorded data was in the form of acceleration time histories, containing 3000 values per second. As it was sometimes more convenient to have the data in the form of peak particle accelerations, i.e. positive envelope curves of the acceleration time histories, the peak particle acceleration curves were extracted from the acceleration time histories, at a rate of 1 value per second, as illustrated in Figure 3.12.

![Figure 3.12: Transformation from a) acceleration time history to b) peak particle acceleration.](image)
Time synchronization
High precision time synchronization between the two DAT recorders’ output was achieved with the help of the accelerometer's self-test function. Indeed, before each driving, a common self-test trigger button was used to simultaneously activate the self-test function in the accelerometers from both boxes. The accelerometers' self-test response was then used as $t = 0$ s for the time histories of both acquisition chains. This process was repeated for each series.

In a second phase, time synchronization of the DAT recorders’ files with the PDE® logs was treated manually. For series 1 and 2, the PDE® trigger had been chosen as a 10 cm depth interval, which amounted to 1 to 3 triggers per second depending on the penetration rate. The first step was then to reduce the PDE® logs’ frequency to 1 Hz. The transformed PDE® logs and the peak particle accelerations could then be matched, using the start of driving as a reference. The precision of this time synchronization is $\sim 1$ s.

Numerical integration
The data measured by the sheet pile and soil instrumentation was in the form of particle accelerations. However, the analysis of particle motion was done using particle velocities, see section 4.3.3. Velocity time histories were obtained from the acceleration time histories through numerical integration, using the trapezoid rule. This integration was done with the help of the cumtrapz function in MATLAB®.
3.5 Conclusions from the field study

The field study was globally a success, with regard to the fact that:

- all except two accelerometers functioned flawlessly;
- all the measurement series could be performed as planned or with minor modifications;
- extraordinary support was offered by the instrumentation and local construction teams.

There were however some limitations associated to the site conditions, in particular that:

- the work area was very tight which might have led to interference from existing objects such as buried pipelines, other sheet pile walls, etc.;
- the experimental site had undergone a lot of previous construction work which had led to an unusually deep fill layer, quite compact clay and large groundwater table fluctuations in the past fifty years.

The soil instrumentation and the use of the vibratory driver’s own logging system were the most critical part of the thesis with respect to experimental success or failure. The design and installation of the sensors was totally innovative, as was the use of the vibratory driver’s depth logging system in the context of an environmental impact research project. It is therefore interesting to assess their performance separately.

The installation of the ground sensors was very time consuming, in particular because of the unfavorable site conditions (thick fill layer, compact clay) and should probably be reviewed before future tests. However, having underground soil instrumentation was overall a great success, with regard to the fact that:

- the installation method succeeded in positioning the accelerometers vertically;
- none of the accelerometers were damaged by the driven sheet piles;
- the accelerometers tolerated their underground position, in particular the humidity;
- useful information was gathered from the ground accelerometers.

The use of the vibratory driver’s logging system has a mixed record:

- the method is overall more accurate and less labor-intensive than video depth recording;
- the logging system provides a wide range of data such as penetration depth, vibrator frequency, displacement amplitude, various oil pressures, etc..

However,

- synchronization with the other sensors was done manually whereas a cable connection might speed up data processing;
- technical information about the vibratory driver’s logging equipment could not be obtained from the manufacturer, so its sensor specifications are unavailable.
4 Results and Analysis

4.1 Introduction
The results obtained during series 1 and 2 (i.e. driving of SP1 and SP2 respectively) are presented in section 4.2.
The analysis presented in section 4.3 focuses on series 2 which, due to the sheet pile – sensor proximity, is the most appropriate to study sheet pile – soil interaction. However, series 1 is also used as a reference case because it was the only free driving (i.e. without clutch friction).
In section 4.4, the main conclusions drawn from the analysis are discussed in relation to the conclusions from the literature study.

4.2 Results

4.2.1 Vibratory driver – related results
Driving parameters associated to the vibrator unit can be found in Table 3.2. Several parameters were recorded by the vibratory driver logging system, see section 3.4.1, but only the following were used in the context of this thesis:

- vibrator frequency [Hz]
- penetration depth [cm]
- time [hh:mm:ss]

Driving frequency
The driving frequency was not varied during the field test; all the sheet piles were driven at a steady frequency of 35 Hz.

Penetration depth
For series 1 and 2, penetration depth was estimated mostly from the vibratory driver’s log but the video recordings were needed for the end of the driving (about 1 m) because of limitations in the vibratory driver’s logging system, see section 3.4.1. The accuracy of the depth data is about 0.1 m. SP1 and SP2 reached a depth of 11.5 m and 11.3 m respectively.

Time
The vibratory driver’s clock was read by the logging system at each trigger and stored on the memory card in the form hh:mm:ss. The maximum accuracy in the analyses involving data from

5 In order to collect background noise data, the acquisitions were always started a few minutes before the driving, which explains the time scales in the time histories presented here.
Driving of SP1 starts at t = 224s of series 1 and driving of SP2 starts at t = 107s of series 2.
the vibratory driver is thus 1s. Synchronization with the other measurement equipment is described in section 3.4.5.

Plotting the sheet pile penetration depth vs. time, see Figure 4.1, is a useful tool for the coming analysis because the depth can be correlated with the characteristic soil profile, see Figure 3.3 and Figures C.1-C.8.

![Figure 4.1: Toe penetration depth vs. time for a) SP1 and b) SP2.](image)

### 4.2.2 Sheet pile – related results

The data was logged in the form of acceleration time histories, at a rate of 3000 samples/s. Due to the limited amount of channels in the data acquisition system, only a portion of all the accelerometers’ output could be logged simultaneously. Table 4.1 shows the available data for the two series analysed here.

<table>
<thead>
<tr>
<th>Sensor Direction</th>
<th>SP1-1</th>
<th>SP1-2</th>
<th>SP1-3</th>
<th>SP2-1</th>
<th>SP2-2</th>
<th>SP2-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Series 1</td>
<td>V</td>
<td>L</td>
<td>T</td>
<td>V</td>
<td>L</td>
<td>T</td>
</tr>
<tr>
<td>Series 2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

**Acceleration levels**

The driving of SP1 and SP2 produced quite similar time histories, see Figure 4.2 and Figure 4.3, with the exception that driving was interrupted from t \( \sim \) 280s to t \( \sim \) 310s in series 1. Once the driving was resumed, only 0.1 m of additional penetration was achieved before progression was completely stopped by bedrock.

6 V: Vertical, L: Longitudinal, T: Transversal. See Figure 3.3.
Results and Analysis

Accelerometer orientation:
- Vertical (V)
- Longitudinal (L)
- Transversal (T)

**Position sketch**

**Figure 4.2:** Time histories of SP1 accelerations, series 1 (no interlock).
Figure 4.3: Time histories of SP2 accelerations, series 2 (interlock).
4.2.3 Soil – related results

The soil – related results are provided by the nine triaxial accelerometers installed in the ground, see Figure 3.3. The data was logged in the form of time histories, at a rate of 3000 samples/s. Due to the limited amount of channels in the data acquisition system, only a portion of all the accelerometers’ output could be logged simultaneously. Table 4.2 shows the available data for the two series analysed here.

Table 4.2: Available time histories from the ground accelerometers.

<table>
<thead>
<tr>
<th>Sensor</th>
<th>MP1-1</th>
<th>MP1-2</th>
<th>MP1-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Direction</td>
<td>V L T</td>
<td>V L T</td>
<td>V L T</td>
</tr>
<tr>
<td>Series 1</td>
<td>x x x</td>
<td>x x x</td>
<td>x x x</td>
</tr>
<tr>
<td>Series 2</td>
<td>x x x</td>
<td>x x x</td>
<td>x x x</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sensor</th>
<th>MP2-1</th>
<th>MP2-2</th>
<th>MP2-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Direction</td>
<td>V L T</td>
<td>V L T</td>
<td>V L T</td>
</tr>
<tr>
<td>Series 1</td>
<td>x x x</td>
<td>x x x</td>
<td>x x x</td>
</tr>
<tr>
<td>Series 2</td>
<td>x x x</td>
<td>x x x</td>
<td>x x x</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sensor</th>
<th>MP3-1</th>
<th>MP3-2</th>
<th>MP3-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Direction</td>
<td>V L T</td>
<td>V L T</td>
<td>V L T</td>
</tr>
<tr>
<td>Series 1</td>
<td>x x x</td>
<td>x x x</td>
<td>- x x</td>
</tr>
<tr>
<td>Series 2</td>
<td>x x x</td>
<td>x x x</td>
<td>- x x</td>
</tr>
</tbody>
</table>

**Acceleration levels**

Figure 4.4 and Figure 4.5 show the results from the ground accelerometers placed closest to the sheet pile line, i.e. from measurement point MP1. The results from the rest of the ground accelerometers are available in Appendix C.

The driving interruption during series 1 is visible in Figure 4.4. The trigger signals used for synchronization of the two DAT recorder, see sections 3.4.4 and 3.4.5, are visible in some of the figures in Appendix C.

---

7 V: Vertical, L: Longitudinal, T: Transversal. See Figure 3.3.
Figure 4.4: Time histories of soil particle acceleration at MP1, i.e. 0.5 m from the sheet pile line, series 1.
Results and Analysis

Accelerometer orientation:
- Vertical (V)
- Longitudinal (L)
- Transversal (T)

Position sketch:
- MP1
- MP2
- MP3

Figure 4.5: Time histories of soil particle acceleration at MP1, i.e. 0.5 m from the sheet pile line, series 2.
4.3 Analysis

The analysis focuses on the sheet pile acceleration levels, see section 4.3.1, as well as the ground acceleration levels measured near the driven sheet pile, see section 4.3.2. Section 4.3.3 attempts a description of the soil particle displacement paths through three-dimensional plotting.

4.3.1 Sheet pile vibrations

Sheet pile acceleration levels

The study of the sheet piles’ acceleration time histories, see Figure 4.2 and Figure 4.3, leads to the following observations.

Both series 1 and 2 seem to contain two different phases: the majority of the driving goes quite smoothly with relatively constant acceleration levels (t = 224-260 s and 107-165 s for series 1 and 2 respectively); however, the end of the driving causes increased acceleration levels for all the sensors.

The vertical acceleration is overall the largest for both series, ~ 20-30g during the smooth driving phase, and with accelerations around 40-60g at the end of driving. It is worth noting that the vertical acceleration of sensor SP1-3 even reached the upper boundary set for the data acquisition system (77.5g) and was truncated for t = 321-324 s.\(^8\)

The longitudinal acceleration ranges from 10g to 20g for both series 1 and 2 during the smooth driving. The higher placed sensors (SP1-1 & SP2-1) have higher longitudinal acceleration, ~ 15-20g, than the lower ones (SP1-3 & SP2-3), ~ 5-10g. This can perhaps be explained by the above ground vs. below ground position of sensors on levels 1 and 3 respectively. It also seems possible to visualize the moment when SP1-2 and SP2-2 go below ground (~ t = 255 s and 135 s respectively).

The transversal acceleration is always the lowest during smooth driving, with values below 5g. During the last part of the driving, the transversal accelerations increase significantly for all three sensors in series 1. The effect is less noticeable in series 2; perhaps due to the less extreme driving.

Influence of the soil layers

The sheet piles’ acceleration levels can also be plotted against penetration depth in order to illustrate the soil’s influence on vibration levels. In Figure 4.6 and Figure 4.7, the characteristic soil profile has also been represented in order to study its influence.

In the fill and the clay, i.e. the smooth driving phase, the vertical acceleration at the sheet pile toe is relatively constant, ~ 30g, for both series 1 and 2. The longitudinal component is high for the first 1.5 m, at 10-15g, and then drops to 5g in series 1 and to 7-10g in series 2. The high start-up

\(^8\) A maximum vertical acceleration of ~ 100 g at t = 322s was estimated from the shape of the truncated acceleration time history for sensor SP1-3 during series 1.
Results and Analysis

Figure 4.6: Peak toe accelerations vs. penetration depth for SP1, series 1.

Figure 4.7: Peak toe accelerations vs. penetration depth for SP2, series 2.
values are probably explained by the fact that the accelerometer is above ground for the first meter of driving. The higher longitudinal vibrations in series 2 could be caused by the presence of clutch friction. The transversal component also shows the same trend between series 1, at 3-4g, and series 2, at 6-7g.

When the toe enters the moraine, its vertical acceleration component increases and becomes more variable: 30-60g in series 1 and 20-40g in series 2. This corresponds to the harder driving visible in the time histories. It is linked to more difficult driving, a slower penetration rate, see Figure 4.1, and possibly the presence of stones leading to uneven driving. The longitudinal and transversal components were even more strongly affected, from ~ 5g up to 50-60g in series 1, and from ~ 5g up to 30g and 15g respectively in series 2. Driving against the bedrock causes the highest vertical accelerations: over 77.5g for SP1 and 50g for SP2. It also causes the highest longitudinal and transversal accelerations for SP2, ~ 30g.

**Characteristic motion**

Figure 4.8 shows a magnification of the particle acceleration time histories for SP1 and SP2 for smooth driving during series 1 and 2 respectively. The vibratory driver enduces a vertical harmonic motion in the sheet pile in series 1 but, in series 2, the motion seems to contain additional vibrations of higher frequency. This might be caused by the presence of clutch friction in series 2 which can introduce higher frequency overtones. The longitudinal and transversal components are periodic for both series 1 and 2 but quite far from harmonic due to many higher frequency vibrations. These might be caused by higher order bending and twisting modes of the sheet pile.

**Figure 4.8:** Magnification of acceleration time histories for a) SP1-1 during series 1 and b) SP2-1 during series 2.
Results and Analysis

Figure 4.9 compares the vertical acceleration time histories for the head and toe of SP1 during the sheet pile’s penetration through the moraine layer. The peak vertical acceleration at the pile head is \( \sim 35g \) while it is up to 60g at the sheet pile toe. Moreover, the irregularities in the downward portion of the toe acceleration pattern and their lower amplitude, slightly phase-lagged counterpart at the head, suggest an impact situation with stress waves travelling up the sheet pile. This can be caused by hard aggregates and stony blocks in the moraine.

![Figure 4.9: Head and toe vertical accelerations for SP1 in moraine. a) SP1-1, b) SP1-3.](image)

**Frequency content**

As discussed above, the sheet pile’s vibrations are not purely harmonic, especially near the toe. Figure 4.10 and Figure 4.11 present the spectra of the sheet pile toe acceleration, for series 1 (driven freely) and series 2 (driven in interlock), during three driving phases: smooth driving in clay, driving through moraine and driving against bedrock.

In clay, the driving frequency (35 Hz) is the dominant frequency for the sheet pile vertical vibrations in both series 1 and 2. In the longitudinal and transversal directions, the dominant frequency is 35 Hz for series 2 but is 70 Hz, i.e. double the driving frequency, for series 1.

In moraine, the two sheet piles seem to behave differently. Series 1 contains more powerful overtones while the spectra of series 1 have low amplitudes and are similar for clay and moraine. This is linked to a rougher driving in series 1 which could be caused by impact with some rock bits that SP2 did not encounter. It should be mentioned that when the sheet piles were extracted, we observed that SP1 was cleft by a stone but SP2 was intact, see Figure C.10.

At the bedrock, the overtones are very powerful in series 1. In the longitudinal direction, the dominant frequency is close to 200 Hz (!). These high frequency components could be caused by
Figure 4.10: Spectra of SP1 toe acc. components for \( t = 250.0-253.0 \) s, 272.0-275.0 s and 321.0-324.0 s, (series 1).

Figure 4.11: Spectra of SP2 toe acc. components for \( t = 130.0-133.0 \) s, 146.0-149.0 s and 170.0-173.0 s, (series 2).
higher order bending and twisting modes in the sheet pile. In series 2, there are also overtones but these are less than half the amplitude of the driving frequency, perhaps due to the interlock’s constraining effect in the longitudinal and transversal directions.

The effect of clutch friction in moraine and bedrock is difficult to analyze due to the other phenomena in play, like the multiplication of overtones in impact situations.

### 4.3.2 Sheet pile – soil vibration transfer

**Ground acceleration levels**
The study of the time histories for ground accelerations at MP1, i.e. 0.5 m from the sheet pile line, see Figure 4.4 and Figure 4.5, leads to the following observations.

The acceleration levels in the ground seem relatively constant for series 1, ~0.3-0.5g for MP1, while they seem more irregular and generally higher for series 2. This could be caused the clutch friction or simply by the increased proximity of the sheet pile.

The distinction between smooth driving and harder driving is not as clear as for the sheet pile vibrations. There is also much less of a difference between the three directions than for the sheet pile. The vertical acceleration is not always the largest; for example at MP1-2 and MP1-3 in series 1, the vertical acceleration is about 0.2g while the longitudinal acceleration is 0.5g.

The longitudinal acceleration is generally as high or higher than the vertical acceleration. In measurement point MP1-1, the longitudinal acceleration reaches 1.5g at \( t = 117 \) s in series 2, which was the highest acceleration measured in the ground.

The transversal acceleration levels are quite close to the others, ~0.3-0.5g in series 1 and for MP1-2 and MP1-3 in series 2. They are however exceptionally high for MP1-1 in series 2, up to 0.8g at the end of the driving. The corresponding SP2 accelerations are quite low (less than 5g) and the origin of these high transversal accelerations is not understood.

**Sheet pile – soil vibration transfer and elastic field attenuation**

In Figure 4.12 and Figure 4.13, the vertical and longitudinal accelerations measured at the sheet pile toe and in the ground are plotted in function of their depth and distance from the sheet pile line. The accelerations plotted are the peak values of each signal over the entire driving.

It becomes clear from the figure that only a small part of the sheet pile vibrations are transferred to the soil. It seems that between 90% and 99% of the sheet pile vibration magnitude is dispersed within the first 0.5 m.

The attenuation curves are very close together for the three sensor levels and it is difficult to see a trend concerning the influence of the sensor depth on the vibration damping. It seems that, in the present case, the damping is more or less depth-independent.
Figure 4.12: Peak acceleration levels at different depths and at increasing distance from the sheet pile line for series 1.
Figure 4.13: Peak acceleration levels at different depths and at increasing distance from the sheet pile line for series 2.
Shaft-generated vibrations
When comparing the time histories of the sheet pile accelerometers with the ones from the ground accelerometers it seems that the smooth driving / hard driving distinction is less visible in the ground than on the sheet pile. Table 4.3 compares average acceleration levels from SP1 and MP1 which were estimated from the SP1-(1-2-3) and MP1-(1-2-3) time histories of series 1. Series 1 is chosen because it has the most extreme acceleration levels.

Table 4.3: Average acceleration levels on SP1 and 0.5 m from the sheet pile line during penetration of different layers.

<table>
<thead>
<tr>
<th>Penetrated layer</th>
<th>Vertical component</th>
<th>Longitudinal component</th>
<th>Transversal component</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SP1</td>
<td>0.5g</td>
<td>SP1</td>
</tr>
<tr>
<td>Clay</td>
<td>30g</td>
<td></td>
<td>40g</td>
</tr>
<tr>
<td>Moraine</td>
<td>50g</td>
<td>0.5g</td>
<td>50g</td>
</tr>
<tr>
<td>Bedrock</td>
<td>80g</td>
<td>0.5g</td>
<td>50g</td>
</tr>
</tbody>
</table>

Two observations can be made here. Firstly, while the sheet pile accelerations increase two to tenfold from smooth to hard driving, the ground accelerations are not even doubled. Secondly, for low sheet pile acceleration levels (below 10 g), a larger part of the vibration is transferred to the ground: 5-10% compared to 1% or less for higher acceleration levels. This indicates that the vibration transfer efficiency is higher for lower sheet pile acceleration amplitudes, meaning that a large increase in sheet pile acceleration only causes a slight increase in ground vibrations.

Toe-generated vibrations
In series 2, it seems that the approach of the sheet pile toe has a visible effect on the vibrations measured by the ground accelerometers. Figure 4.14 - Figure 4.16 show the accelerations amplitudes measured by sensors MP1-1, MP1-2 and MP1-3, i.e. at 0.5 m from the sheet pile line and respectively 2.7 m deep, 4.7 m deep and 6.2 m deep. In most cases, a local peak in the acceleration amplitude occurs when the sheet pile toe is near the accelerometer. For MP1-1 this is even the absolute maximum for all three directions.

By comparing with vibration levels from the rest of the driving, i.e. shaft-generated vibrations, it is possible to estimate the proportion of vibrations spreading from the sheet pile toe.
At MP1-1, (Figure 4.14), the influence of the sheet pile toe is very noticeable. It seems that 40% to 60% of the acceleration at MP1-1 between t = 115s and 120s is caused by the approach of the sheet pile toe. The effect is strongest in the longitudinal direction.
At MP1-2, (Figure 4.15), it seems that the proximity of the sheet pile toe is only responsible for 20-25% of the acceleration between t = 125s and 130s.
At MP1-3, (Figure 4.16), it is difficult to distinguish the effect of the approaching sheet pile toe from the rest of the vibrations.

These observations suggest that a significant proportion of the ground vibrations at 0.5 m from the sheet pile line are toe-generated. It also seems that the proportion of toe-generated vibrations decreases with increasing penetration depth (in the clay layer).
Results and Analysis

Figure 4.14: Influence of the approaching sheet pile toe on the peak particle accelerations at MP1-1, series 2.
Figure 4.15: Influence of the approaching sheet pile toe on the peak particle accelerations at MP1-2, series 2.
Results and Analysis

Figure 4.16: Influence of the approaching sheet pile toe on the peak particle accelerations at MP1-3, series 2.
Frequency content
Figure 4.17 and Figure 4.18 present the spectra of the ground acceleration at measurement point MP1-3, i.e. 0.5 m from the sheet pile line and 6.2 m deep, see Figure 3.3. The spectra are given for series 1 (no interlock) and series 2 (interlock), during three driving phases: smooth driving in clay, driving through moraine and driving against bedrock.

In clay, the dominant frequency is the driving frequency (35 Hz) for all three directions and for both series. There are practically no overtones in series 1 but a few in series 2, especially in the longitudinal direction. These might be caused by the presence of clutch friction in series 2.

In moraine, both series have more overtones than in clay. For series 2, the harder driving was not visible in the sheet pile’s spectra but seems to appear anyway in the ground vibrations. The driving frequency is still the dominant frequency for all directions.

Against bedrock, two interesting phenomena can be noted. Firstly, the vibrations seem to have a cut-off frequency around 100 Hz, above which very little of sheet piles’ vibrations were transferred. Secondly, lower frequency vibrations (18 Hz) are present, especially in the vertical direction. This is even the dominant frequency for the vertical acceleration in series 1 even though it is not the case for SP1.
Results and Analysis

Figure 4.17: Spectra of MP1-3 acc. components for $t = 250.0-253.0 \text{ s}$, $272.0-275.0 \text{ s}$ and $321.0-324.0 \text{ s}$, (series 1).

Figure 4.18: Spectra of MP1-3 acc. components for $t = 130.0-133.0 \text{ s}$, $146.0-149.0 \text{ s}$ and $170.0-173.0 \text{ s}$, (series 2).
4.3.3 Particle displacement path

The particle motion is analyzed here by plotting the orthogonal particle velocity components against each other. The velocities were obtained through numerical integration, using the trapezoid rule.

The particle velocity was chosen as a middle way between acceleration and displacement. It has the advantage of reducing the artificial amplification of high frequencies which occurs when plotting particle accelerations, while at the same time avoiding the accumulated error of a double-integration, necessary to obtain displacements.

The soil particle velocities are only plotted over a 1s time period in order to keep the figures clear.

Toe-generated vibrations

As illustrated on pages 84-87, the proximity of the sheet pile toe influences the accelerations measured by the ground sensors, especially for the accelerometers located higher up (MP1-1 & MP1-2). Figure 4.19 and Figure 4.20 represent the particle motion at MP1-1 and MP1-2 respectively when the sheet pile toe is slightly above MP1-1 (Figure 4.19) and slightly below MP1-2 (Figure 4.20).

The particle motions are relatively smooth ellipses. Both particle motions are quite strongly polarized but in varying directions. The particle motions are sketched at larger scale in Figure 4.21.

Figure 4.19: Particle motion at MP1-1 for $t = 117.0-118.0$ s, during series 2.

Figure 4.20: Particle motion at MP1-2 for $t = 128.0-129.0$ s, during series 2.
Shaft-generated vibrations
Once the sheet pile toe is sufficiently far from the sensor’s level, the vibrations measured can be considered mostly shaft-induced. During the smooth driving, the particle motion tends to be three-directional, with no strong polarization, see Figure 4.22. In the hard driving phase, see Figure 4.23, the particle motion becomes more erratic, translating the presence of higher frequency overtones. The motion mainly remains three-dimensional, with equally strong vertical, longitudinal and transversal components.

Figure 4.21: Principle sketch of the toe-generated particle motions observed at MP1-1 and MP1-2.

**Figure 4.22:** Particle motion at MP1-1 for \( t = 130.0-131.0 \) s, during series 2.

**Figure 4.23:** Particle motion at MP1-2 for \( t = 170.0-171.0 \) s, during series 2.
4.4 Discussion

The principal findings stemming from the analysis of the field test results are discussed here in relation to the conclusions drawn from the literature study.

4.4.1 Vibratory driver – related factors

The vibratory driver – related factors which were judged most influential from the literature study are recalled below. However, they were not consciously varied in this field study as the vibratory driver – related factors are not directly linked to the object of this work, i.e. sheet pile – soil interaction mechanisms.

- **Driving frequency**: The driving frequency (35 Hz) indeed had a strong influence on the ground vibrations as it became the ground’s dominant vibration frequency in all three directions for both series, for most of the driving. A brief exception is discussed at the end of section 4.4.3. Since the driving frequency was not varied, no conclusions can be made here regarding its influence on ground vibration levels.

- **Displacement amplitude**: The sheet piles’ single displacement amplitudes were relatively constant during the field test, ~ 7-8 mm in series 1 and ~ 6 mm in series 2. Moreover, the displacement amplitude was not logged in the last meter of driving due to inadequate settings, see section 3.4.1. It is difficult to draw any conclusions on the influence of the sheet pile’s displacement amplitude in these conditions.

- **Driving force**: The vibrator’s driving force was not studied in this field test and was therefore not discussed in the analysis.

4.4.2 Sheet pile – related factors

The sheet pile – related factors mentioned in the literature study and observed in the field study are mainly related to three issues:

- **Rigid body behavior and longitudinal flexibility**: Longitudinal deformations of piles have been observed by previous authors, (Arnould et al., 2006), (Viking, 2002), (Whenham, 2011), and associated with high frequency overtones at the pile toe, (Borel et al., 2002). A similar phenomenon was observed in series 1 where the acceleration at the sheet pile head was only 60% of the toe acceleration and the vibration pattern suggested an impact situation with higher frequency overtones and stress waves travelling up the sheet pile. This seems especially likely to occur during hard driving, in firm or blocky soils or against bedrock.

- **Eccentric clamping and lateral flexibility**: It has been suggested by several authors (Viking, 2002), (Whenham, 2011), (Lidén, 2012), that gripping the sheet piles by the web (i.e. out of the neutral axis) introduces bending moments in the sheet pile which lead to high longitudinal ground vibrations. In this field study, all the sheet piles were held eccentrically and high longitudinal ground vibrations were indeed observed. However, since none of the sheet piles were held neutrally, it is not possible to demonstrate the influence of eccentric clamping in this study.

- **Clutch friction**: Viking (2002) pointed out that interlock resistance can cause a significant increase in ground vibrations. Field studies by Legrand et al. (1993), mentioned in Viking (2002), reported a two- to fivefold increase in ground vibrations attributed to clutch
friction, and the field test performed by Viking himself (2002) registered a twofold increase in vertical sheet pile toe acceleration (from 17g to 30g). The main observations from the present field test concern both vibration amplitude and frequency.

During smooth driving, the longitudinal and transversal accelerations at the sheet pile toe were both doubled for the sheet pile driven in interlock, i.e. from 5g to 10g for the longitudinal acceleration and from 3g to 6g for the transversal acceleration. However, during hard driving, the longitudinal and transversal sheet pile vibrations were lower in series 2. It is reasonable to expect the clutch friction to have both a vibration enhancing effect, due to increased twisting of the sheet pile, and a vibration reducing effect, due to the interlock constraining the driven sheet pile. From the results presented here, it is difficult to establish which is dominant.

Driving in interlock could also have an influence on the sheet pile and ground vibrations’ frequency. Indeed, the vertical oscillations were perfectly harmonic for sheet pile 1 (driven freely) but contained additional vibrations of higher frequency for sheet pile 2 (driven in interlock). Similarly, the ground vibrations contained higher frequency overtones in series 2, especially in the longitudinal direction, which were not present in series 1.

These correlations are however difficult to make because of all the other phenomena in play such as the distinction smooth/hard driving and the increased sheet pile – sensor proximity in series 2. It should be noted that the field test’s sheet piles were used, but in excellent condition, and that the driver was very careful with the sheet piles’ alignment. These were factors mentioned by Viking (2006) as reducing the risk for high clutch friction.

4.4.3 Soil – related factors

Many authors have stated that soil resistance is the most important soil – related factor influencing piling-induced vibrations, (D’Appolonia, 1971), (Hope & Hiller, 2000), (Massarsch & Fellenius, 2008). Also, several field tests have reported higher ground vibrations linked to penetration of a stiffer soil layer. For example, Clough & Chameau (1980) observed that driving sheet piles into rubble or rock led to surface accelerations, both vertical and horizontal, double those of driving through soft bay sediments, at a distance of 3-5 m from the sheet pile line. The authors suggested that this “hard driving” occurred when the sheet pile encountered an obstacle and was “rattled” by stress waves reflecting from the pile toe. Lidén (2012) also reported an increase in ground vibrations, though mostly in the horizontal direction, when the sheet pile toe reached a stiffer sand layer. She attributed this observation to increased bending in the sheet pile caused by the increased toe resistance.

- **Influence of the soil resistance on sheet pile vibrations**: In the present field test, the actual sheet pile accelerations were measured in order to verify the influence of the soil layers on the sheet pile vibrations. It was very clear that both series 1 and 2 contained two different phases, which were named “smooth driving” and “hard driving”. The smooth phase corresponded to the driving through the fill and the clay and presented relatively constant accelerations in all three directions. The hard driving, corresponding to the sheet pile toe entering moraine and reaching bedrock, saw the sheet pile’s vertical acceleration increase
by ~ 100-150% for series 1 and grow more variable for both series 1 and 2. The longitudinal and transversal accelerations were even more strongly affected: both increased by ~ 1000% in series 1 and by ~ 600% and 300% respectively in series 2. The distinction between smooth and hard driving is also visible in the frequency content of the sheet piles’ motion. For smooth driving, the dominant frequency is 35 Hz (driving frequency) or 70 Hz (for the longitudinal and transversal directions in series 2). But in hard driving, there are very powerful high frequency overtones at the sheet pile toe. In the longitudinal direction, the dominant frequency is close to 200 Hz for series 1 (!). These high frequency components could be linked to higher order bending and twisting modes in the sheet pile. Moreover, the vertical acceleration at the sheet pile toe is approx. 70% higher than at the sheet pile head near the end of the driving. Also, the vertical acceleration pattern in the sheet pile recalls the “flattening of the pile toe” described by Borel et al. (2002). These observations suggest that the hard driving corresponds to an impact situation at the sheet pile toe caused by the encounter with hard elements like stony blocks in the moraine and further down, the bedrock.

• Influence of soil resistance on ground vibrations: The existing conceptual models for vibration generation are based on soil resistance along the sheet pile shaft and near the sheet pile toe. Along the shaft, resistive forces are believed to induce vertically polarized S-waves in the surrounding soil, (Attewell & Farmer, 1973), (Hope & Hiller, 2000). At the pile toe, the displacement (and occasional impacting) of the soil is thought to generated both P- and S-waves propagating spherically from the toe. The main source of vibration (shaft or toe) is believed to vary depending on the soil encountered, (Massarsch, 2000). Loose soils tend to generate mostly shaft resistance whereas in firm layers, an impact situation can develop, with higher toe resistance and more toe-generated vibrations.

In the present field study, the distinction between shaft- and toe-generated vibrations was investigated through the ground acceleration amplitude and through 3D plotting of the particle motion. Indeed, the analysis of the ground acceleration levels showed that the approach of the sheet pile toe had a visible effect on the vibrations measured by the ground accelerometers and thus that a significant proportion of the ground vibrations at 0.5 m from the sheet pile line was toe-generated. It also seemed that the proportion of toe-generated vibrations decreased with increasing penetration depth (in the clay layer), from 40-60% at 2.7 m to barely perceptible at 6.2 m. A hypothesis is that as the sheet pile progresses, the larger shaft – soil contact area leads to a larger shaft resistance, making the ground vibrations primarily shaft-generated, at least while the sheet pile toe is penetrating the relatively soft clay layer.

3D plots of the particle motion were made for different time frames: approach of the sheet pile toe, recent passage of the sheet pile toe, shaft-generated smooth driving vibrations, and hard driving vibrations. In the first two cases, the particle motions were strongly polarized smooth ellipses, whose orientation suggested that the toe was generating mostly spherically spreading S-waves, perhaps due to soil movements at the sheet pile toe. The third and fourth case, shaft-generated vibrations, presented three-directional motions with no strong polarization, instead of vertically polarized S-waves as
expected from the literature study. This confirms, as Lidén (2012) had suggested, that the focus of ground vibration studies should not only be the vertical vibrations.

- **Influence of plasticity and soil degradation on ground vibration levels:** A spatial model of cohesive soil degradation mentioned by D’Appolonia (1971) for impact driven displacement piles was described in the literature study. It contained three zones: a first zone which is highly distorted and remolded, typically about a pile diameter, a second where the lateral stress is sufficient to cause undrained shear failure and a third where the stresses are not sufficient to cause failure. Due to the geometry of a sheet pile, an adaptation of the model is necessary. Since the amount of laterally displaced clay is much smaller for a sheet pile than for a displacement pile, the increase of lateral stress seems negligible compared to the vertical stress caused by the pile penetration. A new model can be suggested where the shear failure in zone II is caused by high vertical strains rather than by an increase in lateral stress. Moreover, the repetitive motion is expected to cause fatigue of the soil skeleton, strongly reducing the shear strength and the ability of the soil to transmit vibrations in the near field. The extent of this field has not, to the author’s knowledge, been determined for sheet pile vibratory driving.

The comparison between the sheet piles’ and the ground’s acceleration showed that between 90% and 99% of the sheet pile vibration was dispersed before the first ground accelerometer, i.e. within the first 0.5 m, while the attenuation was much slower after that: only ~0.5% was lost in the next meter. This confirms that most of the vibration loss occurs in the near field, at less than 0.5 m from the sheet pile.

Moreover, while the sheet pile accelerations increased two- to tenfold from smooth to hard driving, the ground accelerations were not even doubled; indicating that the sheet pile – soil vibration transfer is more efficient for lower sheet pile accelerations. This observation supports the theories concerning soil degradation and fatigue presented in sections 2.2.4 and 2.4.3. It can also explain two other observations which were made in the analysis section: the fact that the vertical acceleration component was not necessarily the strongest at 0.5 m from the sheet pile, and the fact that the distinction between smooth and hard driving was not as clear in the ground as it was on the sheet pile.

- **Influence of plasticity and soil degradation on ground vibration frequencies:** During the hard driving, interesting observations could be made on the ground vibration content. Firstly, the vibrations seem to have a cut-off frequency around 100 Hz, above which almost none of sheet pile’s vibrations were transferred. Secondly, lower frequency vibrations (18 Hz) have high amplitudes, especially in the vertical direction, where this even becomes the dominant frequency for MP1-3. These observations suggest that the sheet pile – soil interface acted as a low-pass filter in this field study, which is perhaps linked to an increase in slippage at the sheet pile – soil interface for high frequency sheet pile vibrations.
5 Conclusions and proposals for further research

5.1 Introduction

The thesis work presented in this report aimed at collecting experimental data from vibratory sheet pile driving to be used in F. Deckner's doctoral thesis, as well as at initiating a reflection around the mechanisms involved in the sheet pile – soil interaction through the analysis of a selected portion of the collected data. The general conclusions drawn from this work are presented in section 5.2. Specific conclusions pertaining to the literature study and the field study can be found in sections 2.6 and 3.5 respectively.

As mentioned previously, only a small portion of the collected data was analyzed here. Further work is planned on the data collected in the context of this thesis. Moreover, some complementary proposals for further research are detailed in section 5.3.

5.2 General conclusions

Only a small portion of the collected data was analyzed in this MSc. thesis but several interesting conclusions have been drawn from the two series analyzed. The data was analyzed in light of the literature study performed at the start of the thesis work, with particular focus on the influence of soil resistance and soil degradation on the sheet pile vibrations as well as on the sheet pile – soil vibration transfer. The main conclusions drawn from this study are presented below.

- The soil characteristics strongly influenced the sheet pile acceleration levels. Both analyzed series contained two distinct phases: “smooth driving” through the fill and clay, and “hard driving” through the moraine and against the bedrock. The sheet piles’ vertical accelerations were doubled from smooth to hard driving, while the longitudinal and transversal accelerations increased by as much as tenfold.

- The combined observations from previous field tests, (Clough et Chameau, 1980), (Borel et al., 2002), (Lidén, 2012), and the observation of the sheet pile acceleration patterns and frequency content led to the conclusion that the hard driving corresponds to an impact situation at the sheet pile toe caused by the encounter with hard elements like stony blocks in the moraine and further down, the bedrock.

- The approach of the sheet pile toe caused a visible increase in the ground vibrations measured at 0.5 m from the sheet pile line. The effect was strongest near the surface and decreased with increasing penetration depth. A hypothesis is that as the sheet pile
Pile – Soil Interaction during Vibratory Sheet Pile Driving

progresses, the larger shaft – soil contact area leads to a larger shaft resistance, making the ground vibrations primarily shaft-generated (in the clay layer).

- The toe-generated vibrations were strongly polarized smooth ellipses, whose orientation suggested that the toe was generating mostly spherically spreading S-waves, perhaps due to soil movements at the sheet pile toe.

- The shaft-generated vibrations were three-directional with no strong polarization, instead of vertically polarized S-waves as expected from the literature study. This confirms, as suggested by Lidén (2012), that the focus of ground vibration studies should not only be the vertical vibrations and that phenomena such as bending and twisting of the sheet pile should not be neglected.

- Between 90% and 99% of the sheet pile vibration was dispersed before the first ground accelerometer, i.e. within the first 0.5 m. This confirms that most of the vibration loss occurs in the near field, less than 0.5 m from the sheet pile.

- The sheet pile – soil vibration transfer was more efficient for lower sheet pile accelerations and for lower frequencies. A hypothesis is that this is due to increased slippage and soil degradation for higher sheet pile accelerations levels and frequencies.

- An adaptation of the spatial model of cohesive soil degradation presented in the literature study is suggested here, in light of the observations from this field study: a first zone which is highly distorted and remolded, perhaps a few millimeters or centimeters, a second where the shear stress is sufficient to cause undrained shear failure, and a third where the stresses are sufficient to cause plasticity without reaching failure. These three zones form the “near field”, which is less than 0.5 m in radius (based on the data from the present field test). Outside this zone, the soil is assumed to behave elastically.

5.3 Proposals for further research

This thesis work was planned as a part of the research program Vibrations due to pile and sheet pile driving in urban areas and in particular, as the beginning of a study on pile – soil interaction. Several important factors have been identified in this work but further research is necessary to achieve a better understanding of how they affect the ground vibration levels. Suggestions for future research are:

- Carrying out complementary field tests to study the effect of different soil types on both the sheet pile motion and the generated ground vibrations.

- Varying the clamping method to document the effect of eccentric clamping on sheet pile lateral motion and on the generated ground vibrations.

- Investigating the vibration transfer efficiency for various sheet pile acceleration levels and driving frequencies, as well as various sheet pile displacement amplitudes.
• Further investigating the near field, in particular its characteristic size, strain levels and damping coefficients. Given the success of the soil instrumentation used in this field study, sensors could be placed even closer to the sheet pile in future field tests.

• Numerical modeling of the near field, using experimental data for back-calibration.

• Evaluating the power dissipated at the pile – soil interface and the energy actually transferred to the far field. An energy transfer approach to the problem could facilitate the connection with previous studies on the vibrator – pile power transfer.
6 References


on vibratory pile driving and deep soil compaction, Louvain-La Neuve, Belgium, 9-10 September 2002, pp. 181-192.


Appendix A  Site – related conditions

A1. Construction site overview

Figure A.1: Set up of the field test in relation to the construction site layout. Section A-A refers to Figure 3.3.
A2. Extract from geotechnical investigations

Figure A.2: Section 5/890 (N-track). Extract from TVBN313-120-401-2510, (WSP, 2011).
A3. Driving equipment specifications

**LRB 125 XL Piling and drilling rig** – Technical data from Liebherr (2010).

<table>
<thead>
<tr>
<th>Technical data</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Leader length</td>
<td>15 m</td>
</tr>
<tr>
<td>Capacity hammer including cap plus pile</td>
<td>12 t</td>
</tr>
<tr>
<td>Max. hammer weight</td>
<td>6 t</td>
</tr>
<tr>
<td>Max. pile weight</td>
<td>6 t</td>
</tr>
<tr>
<td>Max. pull</td>
<td>200 kN</td>
</tr>
<tr>
<td>Max. torque</td>
<td>120 kNm</td>
</tr>
<tr>
<td>Working radius machine</td>
<td></td>
</tr>
<tr>
<td>Center of rotation — center pile</td>
<td>3.15 — 5.35 m</td>
</tr>
<tr>
<td>Stepless rig inclination adjustment</td>
<td></td>
</tr>
<tr>
<td>Lateral inclination</td>
<td>± 1:20</td>
</tr>
<tr>
<td>Forward inclination</td>
<td>1:6</td>
</tr>
<tr>
<td>Backward inclination</td>
<td>1:3</td>
</tr>
<tr>
<td>Vertical leader adjustment above</td>
<td></td>
</tr>
<tr>
<td>ground level (depending on radius)</td>
<td>5 m</td>
</tr>
<tr>
<td>Leader swing range</td>
<td>± 90°</td>
</tr>
</tbody>
</table>

**1100H Vibrator** – Technical data from Liebherr (2010).

<table>
<thead>
<tr>
<th>Technical data</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Static moment</td>
<td>0 – 20 kgm</td>
</tr>
<tr>
<td>Max. frequency</td>
<td>2300 rpm</td>
</tr>
<tr>
<td>Max. centrifugal force</td>
<td>1160 kN</td>
</tr>
<tr>
<td>Max. amplitude</td>
<td>19 mm</td>
</tr>
<tr>
<td>Total weight without clamp</td>
<td>3250 kg</td>
</tr>
<tr>
<td>Total weight with single clamp</td>
<td>4200 kg</td>
</tr>
<tr>
<td>Dynamic weight with clamp</td>
<td>2980 kg</td>
</tr>
</tbody>
</table>
Appendix B  Accelerometer specifications, testing and calibration

B1. Accelerometer specifications

Sheet pile and soil instrumentation specifications – Summarized from Analog (2012).

<table>
<thead>
<tr>
<th>Model</th>
<th>ADXL-377Z*</th>
<th>ADXL-335Z*</th>
</tr>
</thead>
<tbody>
<tr>
<td># of axes</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Acceleration range</td>
<td>+/- 200g</td>
<td>+/- 3g</td>
</tr>
<tr>
<td>Shock survival</td>
<td>+/- 10 000g</td>
<td>+/- 10 000g</td>
</tr>
<tr>
<td>Sensitivity</td>
<td>6.5 mV/g</td>
<td>300 mV/g</td>
</tr>
<tr>
<td>-3dB bandwidth</td>
<td>0 – 1.6 kHz</td>
<td>0 – 1.6 kHz</td>
</tr>
<tr>
<td>Output type</td>
<td>Analog</td>
<td>Analog</td>
</tr>
<tr>
<td>Noise density</td>
<td>2400 µg/√Hz</td>
<td>300 µg/√Hz</td>
</tr>
<tr>
<td>Voltage supply</td>
<td>3 – 3.6 V</td>
<td>1.8 – 3.6 V</td>
</tr>
<tr>
<td>Current supply</td>
<td>300 µA</td>
<td>350 µA</td>
</tr>
<tr>
<td>Temperature range</td>
<td>-40° – 85° C</td>
<td>-40° – 85° C</td>
</tr>
<tr>
<td>Package size</td>
<td>3 mm x 3 mm x 1.45 mm</td>
<td>4 mm x 4 mm x 1.45 mm</td>
</tr>
</tbody>
</table>

* Z stands for RoHS Compliant which is a EU norm for Restriction of Hazardous Substances, here lead (Pb).
B2. Accelerometer testing and calibration

B2.1 Laboratory testing – 2012/11/09

Objective
The purpose of the testing was to check that the accelerometers, once wired and in their casing, performed as expected regarding correlation between their output and the inputted vibration, (direction, frequency, and amplitude).

Set up
The testing was performed in the KTH Soil and Rock Mechanics laboratory. A vibration exciter of model Brüel & Kjær Type 4808 was used to generate controlled vibrations. A steel rod, connected to the vibration exciter, transmitted the vibrations to an experimental sand box, see Figure B.1 (left).

One accelerometer of each type (ADXL-377Z & ADXL-335Z) was mounted in its casing, see Figure 3.9, and connected to a signal conditioning box, see Figure B.1 (right), itself connected to a computer where the output signals could be visualized.

Test
By changing the orientation of the sensors in relation to the vibration rod, the vibrations were checked to occur on the correct channel.
By varying the frequency and amplitude of the vibrating device, the corresponding variations were checked qualitatively in the output signals.

Conclusions
The accelerometers performed as expected regarding sensing of the exciting vibration’s direction as well as variations of amplitude and frequency.
It was also noticed that when the signal conditioning box was connected to the electricity network, the 50 Hz network frequency was observed in the output signals. In order to avoid this, grounding of the conditioning box was necessary.

**B2.2 Field testing – 2013/11/16**

**Objective**
The purpose of the testing was to check that sufficient contact could be achieved between the accelerometers and the sheet pile or soil in field conditions.

**Set up**
The testing was performed near Rotebro station, on an NCC construction site where sheet piles had previously been installed, see Figure B.2 (top left). One accelerometer of each type (ADXL-377Z & ADXL-335Z) was mounted in its casing, see Figure 3.9, and connected to a Picoscope oscilloscope, itself connected to a computer where the output signals could be visualized. The sheet pile accelerometer was clamped to a sheet pile, see Figure B.2 (top right), while the ground accelerometer was installed in the ground, see Figure B.2 (top left). The ground accelerometer was placed about 0,5 m from the sheet pile and about 0,2 m deep. Because the ground was frozen, it was necessary to bore a hole for the accelerometer which was then installed with a fiberglass pole, see Figure B.2 (top center). The hole was then filled with loose sand material to improve the contact between the accelerometer and the ground.

![Figure B.2:](image)

*Top left: After installation of the two accelerometers.*
*Top center: Ground accelerometer mounted on a fiberglass rod for installation.*
*Top right: Clamped sheet pile accelerometer.*
*Bottom: Hammer used to generate vibrations in the sheet pile.*
*Photos by F. Deckner, 2013/01/16.*
Test
Vibrations were generated in the sheet pile with the help of a hammer, see Figure B.2 (bottom). Background vibrations were also measured in one series.

Conclusions
The accelerometers performed as expected and both background and hammer induced vibrations could be observed in the output signals of accelerometers SP and MP. The difficulties encountered while installing the ground accelerometer implied that extra time should be planned during the field study for the installation of the ground accelerometers.

B2.3 Dynamic calibration – 2013/04/18

Objective
The purpose of the calibration was to obtain the relationship between the acceleration experienced by the accelerometers and their outputs in mV, i.e. their calibration factors, within the range of expected frequencies.

Set up
Calibration was performed in Kent Lindgren’s lab in Älvsjö. Information regarding the calibration equipment is presented in Table B.1. A vibration exciter of model Brüel & Kjær Type 4808 was used to generate controlled vibrations, see Figure B.3 and Figure B.4. It was controlled by a function generator & analyzer whose output was amplified by a power amplifier. See Table B.1 for the calibration equipment’s references. A reference accelerometer was used to calibrate the new accelerometers. Both accelerometers were attached to the same steel plate for back-to-back calibration according to the set up in Figure B.3.

Figure B.3: Sketch of the back-to-back calibration system.
The reference accelerometer’s output was collected by the function analyzer and the signal viewed on an oscilloscope, see Figure B.4. The other accelerometer’s output was collected in one of the signal conditioning boxes prepared for the field tests, see Figure B.4 (left). It consisted of:

- a power supply for the accelerometers and the amplifier,
- a terminal board,
- an amplifier (x 5),
- an adjustable low-pass filter, and
- an output to the function analyzer.

The output collected by the function analyzer was compared to the signal from the reference accelerometer in order to obtain the correct calibration factor for the calibrated accelerometer.

**Table B.1:** Calibration equipment.

<table>
<thead>
<tr>
<th>Equipment</th>
<th>Brand and Model</th>
<th>Series number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vibration exciter</td>
<td>Brüel &amp; Kjær 4808</td>
<td>2633121</td>
</tr>
<tr>
<td>Function generator &amp; analyzer</td>
<td>Hewlett Packard 3662A</td>
<td></td>
</tr>
<tr>
<td>Amplifier</td>
<td>Ling TPO 300</td>
<td></td>
</tr>
<tr>
<td>Reference accelerometer</td>
<td>PCB 301A11</td>
<td>2429</td>
</tr>
<tr>
<td>Oscilloscope</td>
<td>Nicolet 410</td>
<td></td>
</tr>
</tbody>
</table>

**Figure B.4:** Left: Calibration equipment, on the table from left to right and top to bottom: signal conditioning box, voltmeter, printer, oscilloscope, function generator and amplifier.  
Center: Ground accelerometer mounted horizontally on the vibration exciter.  
Right: Sheet pile accelerometer mounted horizontally on the vibration exciter.  
*Photos by K. Viking, 2013/04/18.*
Calibration
The calibrations were carried out with a frequency sweep from 2 Hz to 252 Hz for both ground and sheet pile accelerometers.

The output obtained for each axis of each accelerometer was plotted over the swept frequency range and the precise values were noted for three specific frequencies: 10 – 20 – 30 Hz. The corresponding calibration factors $k_d$ were calculated by dividing the response at the chosen frequency by the amplification factor and adjusting the units:

\[
A.1 \quad k_d = \frac{Y_a}{5} \times 9.81
\]

Where $k_d = \text{dynamic calibration factor (or sensitivity)} \quad [\text{mV/g}]

$Y_a = \text{response at chosen frequency} \quad [\text{mV}/(\text{m/s}^2)]

$5 = \text{amplification factor} \quad [-]

Conclusions
The frequency responses obtained all complied with the accelerometers’ data sheet and were considered constant over the expected range of frequencies. The calibration factors used in the field study correspond to the 30 Hz outputs.

B2.4 Static calibration – 2013/04/24

Objective
The purpose of the calibration was to obtain the relationship between the pitch and roll angles\(^9\) of the ground accelerometers and their outputs in mV within the range of expected angles. Pitch and roll are the angles between the accelerometers X and Y axis respectively and their theoretic horizontal position (perpendicular to the earth’s gravitational pull), see Figure B.5.

\[ \begin{align*}
\theta &= \text{pitch} \\
\phi &= \text{tilt} \\
\psi &= \text{roll} \\
1g &= \text{earth's gravitational pull}
\end{align*} \]

\[ \text{Figure B.5: Schematic view of tilt, modified after Fisher (2010).} \]

\(^9\) If necessary, the accelerometer’s tilt, i.e. angle between z axis and earth’s gravitational pull, can be obtained based on measured pitch and roll by using the fact that the X, Y, Z axes are orthogonal.
Set up
Calibration was performed in Geometrik’s lab in Stockholm, on the calibration device used for the company’s inclinometers. The device consists of a hanging arm which can be held at an angle by a peg pushed through a graduated plate, see Figure B.6 (center). A rubber adaptor was used to securely mount the accelerometer on the device, see Figure B.6 (left). The output of the accelerometer was read on a handheld digital multimeter (APPA 106 RMS), see Figure B.6 (right).

![Figure B.6: Left: Rubber adaptor. Center: Tilt calibration device. Right: Terminal board and multimeter. Photos by C. Guillemet, 2013/04/24.]

Calibration
The calibration was carried out with for the X and Y axes of each ground accelerometer. The range of angles covered is -20 to +20 degrees.

Conclusions
Using the calibration factors obtained here, the ground accelerometers’ inclination can be calculated based on the measured static output. This can for example be done after installation in order to check that the accelerometers are correctly positioned, see Table 3.5. If they are at an angle, the vibration measurements can be corrected accordingly. The angle of the accelerometers can be checked again after the sheet pile driving to see if they were disturbed.
Appendix C  Additional material from the field study

C1. Complete time histories for series 1 & 2
Figure C.1: Time histories of SP1 accelerations (series 1) with SP1 penetration depth and localization of figures.
Figure C.2: Time histories of soil acceleration at MP1 (series 1), i.e. 0.5 m from the sheet pile line.
This signal was not logged.

**Figure C.3:** Time histories of soil acceleration at MP2 (series 1), i.e. 1.5 m from the sheet pile line.
Figure C.4: Time histories of soil acceleration at MP3 (series 1), i.e. 6 m from the sheet pile line.
Figure C.5: Time histories of SP2 accelerations (series 2) with SP2 penetration depth and localization of figures.
Figure C.6: Time histories of particle acceleration MP1, i.e. 0.5 m from the sheet pile line, series 2.
Figure C.7: Time histories of particle acceleration at MP2, i.e. 1.5 m from the sheet pile line, series 2.

This signal was not logged.
Figure C.8: Time histories of particle acceleration at MP3, i.e. 6 m from the sheet pile line, series 2.
C2. Additional photographs

**Figure C.9:** Installation of sensors MP1(-1-2-3) and MP2(-1-2-3). Note the peat layer (0.5 m) under the fill. Photo by K. Viking (2013/05/27).

**Figure C.10:** Sheet pile 1 (right) and sheet pile 2 (left) after the field test. Note the deformation and cleft in sheet pile 1. Photo by C. Guillemet (2013/05/29).