KTH Architecture and the Built Environment

Ground vibrations due to pile and sheet pile driving – influencing factors, predictions and measurements

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Licentiate thesis

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PREFACE

The work presented in this thesis has been carried out between September 2009 and March 2013 at NCC Engineering and the Division of Soil and Rock Mechanics, Department of Civil and Architectural Engineering at the Royal Institute of Technology. The work was supervised by Professor Staffan Hintze with assistance from Dr Kenneth Viking.

I would like to express my gratitude to the Development Fund of the Swedish Construction Industry, NCC Construction Sweden and the Royal Institute of Technology for the financial support given to this research project.

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Stockholm, February 2013

Fanny Deckner

SUMMARY

Ground vibrations due to pile driving are part of a complex process. Vibration is generated from the pile driver to the pile. As the pile interacts with the surrounding soil, vibrations are transferred at the pile-soil interface. The vibration propagates through the ground and interacts with structures, both above ground and underground. The vibration continues into the structure where it may disturb occupants and/or damage the structure.

In this thesis the study of the vibration transfer process due to pile driving is limited to the vibration source and the wave propagation in the soil. Vibration transmission to adjacent buildings and structures is not studied. However, impact of vibrations on buildings is briefly discussed in the literature study.

It is important to accurately predict the magnitude of ground vibrations that result from pile driving in urban areas, both over- and underestimated vibration levels lead to increased costs. A lot of research has been performed within this field of knowledge, but a reliable and acknowledged prediction model for vibrations induced by pile or sheet pile driving is still needed.

The objective of the research project is to increase the knowledge and understanding in the field of ground vibrations due to impact and vibratory driving of piles and sheet piles. This research project also aims to develop a reliable prediction model that can be used by practising engineers to estimate vibration due to pile driving. This licentiate thesis presents the first part of the research project and aims to increase the knowledge and understanding of the subject and to form a basis for continued research work.

The most important findings and conclusions from this study are:

- The main factors influencing vibrations due to pile and sheet pile driving are; (1) the vibrations transferred from the pile to the soil, (2) the geotechnical conditions at the site and (3) the distance from the source.
- The vibrations transmitted from the pile to the soil depend on the vibrations transferred to the pile from the hammer, the pile-soil interaction and the wave propagation and attenuation in the plastic/elasto-plastic zone closest to the pile.
- There is today no prediction model that fulfils the criteria of the "perfect" prediction model; reliable but yet easy to apply.

Future research should study the transfer of vibrations at the pile-soil interface, including the generation of a plastic/elasto-plastic zone in the area closest to the pile and how that affects the transfer of vibrations from the pile to the soil.

Keywords: ground vibration, pile, sheet pile, prediction

SAMMANFATTNING

Markvibrationer på grund av pålning är del av en komplex process. Vibrationer genereras från pålmaskinen till pålen. När pålen kommer i kontakt med den omgivande jorden överförs vibrationer mellan påle och jord. Vibrationerna fortplantar sig som vågor genom marken och träffar byggnader och andra konstruktioner, både ovan och under jord. Vibrationerna fortsätter in i byggnaden där de kan orsaka störningar eller skador.

I denna avhandling begränsas studien av vibrationsöverföringsprocessen till vibrationskällan och vågutbredningen i jord. Vibrationsöverföringen till intilliggande byggnader eller konstruktioner har inte studerats. Påverkan av vibrationer på byggnader diskuteras dock kort i litteraturstudien.

Det är viktigt att på ett tillförlitligt sätt kunna förutsäga markvibrationerna på grund av pålning i stadsmiljö, både över- och underskattade vibrationsnivåer leder till ökade kostnader. Forskning har tidigare utförts inom detta område, men en tillförlitlig och allmänt accepterad prognosmodell för vibrationer på grund av pålning eller spontning saknas fortfarande.

Syftet med forskningsprojektet är att öka kunskapen och förståelsen för markvibrationer som uppkommer vid installation genom slagning eller vibrering av pålar och spont. Forskningsprojektet syftar också till att utveckla en tillförlitlig prognosmodell som kan användas av yrkesverksamma ingenjörer för att uppskatta vibrationsnivåer orsakade av pålning. Denna licentiatavhandling presenterar den första delen av forskningsprojektet och syftar till att öka kunskapen och förståelsen inom ämnesområdet samt att skapa en plattform för det fortsatta forskningsarbetet.

De viktigaste resultaten och slutsatserna från denna studie är:

- De huvudsakliga faktorer som påverkar vibrationer orsakade av pålning är; (1) de vibrationer som överförs från källan till jorden, (2) de geotekniska förhållandena på platsen och (3) avståndet från vibrationskällan (pålen).
- Vibrationerna som överförs från pålen till jorden beror på de vibrationer som överförs från pålmaskinen till pålen, påle-jord interaktionen samt vågutbredning och dämpning i den plastiska/elasto-plastiska zonen som bildas närmast pålen.
- Det finns idag ingen prognosmodell som uppfyller kriterierna för den ”perfekta” prognosmodellen; tillförlitlig men ändå lätt att tillämpa.

Framtida forskning bör undersöka överföringen av vibrationer mellan påle och jord, innefattande uppkomsten av en plastisk/elasto-plastisk zon närmast pålen och hur det påverkar vibrationsöverföringen från påle till jord.

Nyckelord: markvibrationer, påle, spont, prediktion

List of Notations

Key symbols used in the text are listed below.

Greek Symbols

Symbol	Represents	Unit
α	Absorption coefficient	m^{-1}
β	Coefficient depending on probability of exceedance	-
γ	Shear strain	-
γ_c	Cyclic shear strain	-
γ_t	Threshold shear strain	-
θ_{crit}	Critical angle	rad
λ	Wavelength	m
λ_R	Wavelength of R-wave	m
λ_L	Wavelength of Love wave	m
ξ	Hysteretic damping	-
π	Pi	-
ρ	Material density	kg/m^3
σ	Stress	kPa
τ	Shear stress	kPa
τ_c	Shear stress mobilised at γ_c	kPa
υ	Poisson's ratio	-
ϕ	Diameter	m
φ	Phase angle	rad
ω	Angular frequency	rad/s

Roman Symbols

Symbol	Represents	Unit
A	Amplitude	m
A_{max}	Maximum displacement amplitude	m
A_p	Cross sectional area of the pile	m^2
a	Acceleration	m/s^2
c	Wave propagation velocity	m/s
c_B	Wave propagation velocity in the pile	m/s
c_H	Stress wave velocity in hammer	m/s
c_p	Wave propagation velocity of P-wave	m/s
c_R	Wave propagation velocity of R-wave	m/s
c_s	Wave propagation velocity of S-wave	m/s
D	Material damping	$(Hz\cdot s)^{-1}$
d	Depth	m
E	Elasticity modulus	MPa
e	Eccentricity	m

e_v	Void ratio	-
F	Force	kN
F_c	Centrifugal force	kN
F_d	Driving force	kN
Fi	Impact force	kN
F_v	Dynamic driving force	kN
F_0	Static overload	kN
f	Frequency	s^{-1} or Hz
f_d	Driving frequency	Hz
f_n	Natural frequency	Hz
G	Shear modulus	MPa
G_{max}	Initial shear modulus	MPa
G_s	Secant shear modulus	MPa
g	Acceleration of earth's gravity	m/s^2
$g(t,r)$	Propagation function or Green's function	-
H	Height of soil layer	m
h	Drop height	m
J_c	Damping factor	-
k	Empirically determined constant	$m^2/s\sqrt{J}$
L_H	Hammer length	m
L_p	Pile length	m
L_w	Stress wavelength	m
M	Deformation modulus	kPa
M_e	Static moment	kgm
M_H	Hammer mass	kg
m	Mass	kg
m_{dyn}	Total vibrating mass	kg
N	Number of loops/stories	-
n	Value depending on wave type	-
P	Dynamic force	kN
PI	Plasticity index	-
PPV	Peak particle velocity	mm/s
R	Soil resistance to static probing	kN/m^2
R_s	Shaft resistance	kN
R_t	Toe resistance	kN
r	Distance from source	m
r_0	Reference distance	m
r_{crit}	Critical distance	m
S	Double displacement amplitude	m
S_p	Contact area between shaft and soil	m^2
s	Slope distance	m
$s(t)$	Source function	-
T	Period	s
t	Time	s
u	Displacement	mm
u_0	Initial vibration velocity	mm/s
V_0	Coefficient of variation	-

v	Particle velocity	mm/s
v_g	Ground vibration velocity	mm/s
v_H	Particle velocity of hammer	m/s
v_{H0}	Velocity of hammer at impact	m/s
v_p	Particle velocity of pile	m/s
v_{res}	Resultant velocity	mm/s
v_{SRSS}	Simulated resultant particle velocity	mm/s
v_x	Particle velocity in x-direction	mm/s
v_y	Particle velocity in y-direction	mm/s
v_z	Particle velocity in z-direction	mm/s
W	Power supply	kW
W_0	Input energy	J
W_s	Dissipated energy	J/m^3
$w(t,r)$	Ground vibration function	-
x	Empirically determined constant	-
Z	Impedance	kNs/m
Z_H	Hammer impedance	kNs/m
Z_p	Pile impedance	kNs/m
Z_s	Soil impedance	kNs/m
Z_{sp}	Soil impedance for P-waves	kNs/m
z	Displacement	mm
$\dot{z}$	Velocity	mm/s
$\ddot{z}$	Acceleration	mm^2/s
z_s	Specific impedance	kNs/m^3
z_{sp}	Specific impedance for P-waves	kNs/m^3
z_{ss}	Specific impedance for S-waves	kNs/m^3

LIST OF PUBLICATIONS

This licentiate thesis is based on the work presented in the following publications.

Appended papers:

Paper I Deckner, F., Viking, K. and Hintze, S. (2012). Ground vibrations due to pile and sheet pile driving – prediction models of today. In *Proceedings of the European Young Geotechnical Engineers Conference* (Wood, T. and Swahn, V. (eds)). Swedish Geotechnical Society, Gothenburg, Sweden, pp. 107-112. *Peer-reviewed conference paper*.

Deckner performed the analyses and wrote the paper. Viking and Hintze supervised the work and contributed valuable comments.

Paper II Deckner, F., Viking, K. and Hintze, S. (2013). Factors influencing vibrations due to pile driving. Submitted to *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* in December 2012. *Journal paper*.

Deckner performed the analyses and wrote the paper. Viking and Hintze supervised the work and contributed valuable comments.

Paper III Deckner, F., Lidén, M., Viking, K. and Hintze, S. (2013). Measured ground vibrations during vibratory sheet pile driving. To be submitted to *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* in March 2013. *Journal paper*.

Deckner and Viking planned and took part in the field test measurements. Deckner and Lidén performed the analyses. Deckner wrote the paper. Viking and Hintze supervised the work and contributed valuable comments.

Related publications:

Lidén, M. (2012). *Ground Vibrations due to Vibratory Sheet Pile Driving*. Division of Soil- and Rock Mechanics, Royal Institute of Technology, Stockholm, Sweden, Master of Science Thesis 12/06.
Deckner supervised the work.

Deckner, F., Hintze, S. och Viking, K. (2010). Miljöanpassad pål- och spontdrivning i tätbebyggt område - etapp 2. Bygg & teknik, Vol. 102, Nr. 1, pp. 12-20.

Deckner, F., Lidén, M., Hintze, S. och Viking, K. (2013). Markvibrationer vid spontning för Karlstad teater. Bygg & teknik, Vol. 105, Nr. 1, pp. 25-30.

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1 INTRODUCTION

1.1 BACKGROUND

Environmental impact is defined as any change to the environment, whether adverse or beneficial. The surroundings may include nearby buildings, humans or animals in the neighbourhood, soils in the vicinity, fresh water and more. Pile and sheet pile driving in densely populated areas mainly impacts the environment through vibrations, settlements and/or noise. This research project has been limited to the study of vibrations. Settlements are briefly touched upon as a side effect of vibrations.

Vibrations can arise from many different sources in a modern society, for instance traffic, machines, hammering, explosions, earthquakes and construction work (IVA, 1983) (Holmberg, 1984). This study focuses on vibrations from pile and sheet pile driving. Vibration due to pile driving is a complex process that involves many parameters that vary during the process. A vibration is generated by the pile driver. After an interaction between the pile and the soil, the vibration propagates through the ground and inevitably interacts with structures in urban areas, both above ground and underground. The vibration then continues into the structure where it may disturb occupants and/or damage the structure (Hintze, 1994).

One trend in construction today is to increase demands on quality, while reducing construction time and lowering environmental impact. In addition, construction work today is frequently located in urban areas, adjacent to existing structures and humans. Construction work inevitably influences its surroundings. It may affect nearby buildings, streets, in-ground pipes and more, as well as disturb special equipment and people. Construction-induced vibrations include vibrations from activities such as blasting, excavation, demolition, compaction and driving of piles and sheet piles. Today it is believed that vibrations from pile driving are the most common sources of construction vibrations (Athanasopoulos & Pelekis, 2000).

Due to the increased concern of environmental impact and because construction projects are more often located in urban areas close to existing structures, vibration assessment and prediction has become of immediate interest. It is important to accurately predict the magnitude of ground vibrations that result from pile driving at construction sites. This has been discussed in Athanasopoulos & Pelekis (2000), Hope & Hiller (2000) and Massarsch & Fellenius (2008) and others. The models and methods for prediction of vibrations due to pile driving are inadequate today. A significant amount of research has been performed in this field of knowledge, see chapter references, but a reliable and acknowledged prediction model for vibrations induced by pile driving is still needed.

An inability of reliably predict vibrations due to pile driving leads to increased costs (Hintze, 1994). If vibration levels are overestimated, this leads to selecting more expensive and time consuming construction methods than necessary. However, if vibrations levels are underestimated they result in damaged structures, disturbed occupants and suspensions to the construction work.

The actual cost of damages caused by vibrations due to pile driving is unknown. However, a recent article in the Swedish press (Karlsson, 2013) estimates that damages and delays in construction projects has led to costs of about 2.7 billion Euros in 2010 in Sweden alone. Of these, an estimated 1/3 or 0.9 billion Euros are due to geotechnical errors.

1.2 AIM AND OBJECTIVE

The objective of this research project is to increase the knowledge and understanding in the field of vibrations due to impact and vibratory driving of piles and sheet piles. This research project also aims to develop a reliable prediction model that can be used by practising engineers to estimate vibration due to pile and sheet pile driving. The prediction model should be reliable and adaptable for use by practising geotechnical engineers. Addressing this problem will hopefully result in less environmental impact from pile and sheet pile driving in the future, which will reduce foundation costs and ensure the continued use of piles and sheet piles in urban areas.

This licentiate thesis, which includes a literature study and a field study, is the first part of the research project and aims to increase the knowledge and understanding of the subject and to form a basis for the continued research work. It aims to identify factors that influence vibration levels and survey the existing prediction models, from which areas that need further research can be identified. The upcoming second part of the research program will focus on the development of a reliable prediction model for vibrations due to pile and sheet pile driving.

1.3 EXTENT AND LIMITATIONS

The research will be focused on the environmental impact from pile and sheet pile driving in the form of vibrations. The installation methods discussed are limited to impact and vibratory pile driving. The thesis discusses vibrations from pile and sheet pile driving, in the text the word pile will refer to both pile and sheet pile unless it is stated to apply to only one or the other.

The study of the vibration transfer process due to pile driving is limited to the vibration source and the wave propagation in the soil. Vibration transmission to adjacent buildings and structures is not studied. However, impact of vibrations on buildings is briefly discussed in the literature study.

1.4 METHOD AND OUTLINE

This research project is founded on prior research in the field of impact and vibratory driven piles and sheet piles, within which Dr Kenneth Viking earlier published a doctoral thesis named *Vibro-driveability – a field study of vibratory driven sheet piles in non-cohesive soils* (Viking, 2002a).

To achieve the objective, the research project is divided into four different phases:

Phase 1 – Literature study
An introduction to the field of research and the underlying theories, what is known and what further research needs to be done.

Phase 2 – Field study/Case study
Initial tests and measurements are performed either in a real project or at a test site. The results are evaluated and analysed, and presented in a paper as well as a master's thesis.

Phase 3 – Theory development and numerical calculations
Based on previous theories, new theory development and numerical calculations a model is developed for evaluation and prediction of the vibrations induced in a pile driving project.

Phase 4 – Verification and implementation of the model in-situ
The developed model is tested and revised if necessary using comparisons between the model and measurement results.

This licentiate thesis concerns the work done within phase 1 and 2 as mentioned above.

This thesis is written as a compilation thesis and consists of five chapters, which are briefly described below, and three appended peer-reviewed papers.

Chapter 1 is an introduction describing the background and objectives of this study.

Chapter 2 covers a summary of the literature study including major findings and conclusions from previous work.

Chapter 3 contains a short summary of the field test performed within the scope of this licentiate thesis.

Chapter 4 comprises a short summary of each of the appended papers.

Chapter 5 presents the major conclusions from this study along with suggestions for future research within the field of vibrations due to pile driving.

2 LITERATURE STUDY

2.1 INTRODUCTION

A literature study based on available literature on environmental impact due to pile driving has been conducted as part of this licentiate thesis. Limitations have been made to literature available in English and Swedish. A list of all references can be found at the end of the thesis.

A summary of the literature study is presented here. The chapter begins with a review of the basics of dynamics and geodynamics. An explanation of the mechanisms and functions of piles and sheet piles and the installation processes is next, followed by a review of the vibration transfer process for pile driving. The environmental impact of vibrations due to pile driving is studied more closely, with a focus on the effect on soil, buildings and structures, and humans. In addition, the currently used methods for and predicting vibrations from pile driving are presented.

2.2 BASIC DYNAMIC THEORY AND GEODYNAMICS

To fully understand the problem caused by vibrations due to pile driving, it is necessary to know and recognise the underlying theories regarding dynamics and geodynamics. In this section, basic dynamic theory as well as theories and concepts regarding geodynamics are explained.

2.2.1 Basics of dynamics for vibrating systems

This section introduces the most common dynamics terminology and a few basic definitions related to vibratory motion.

2.2.1.1 Basic parameters

In Table 2.1 and Figure 2.1 some important parameters when it comes to vibratory motion are listed and shown.

Table 2.1 Expression, definition and unit for some important parameters in dynamics (Richart et al., 1970) (Bodare, 1996) (Nordal, 2009).

Parameter	**Expression**	**Unit**	**Definition**
A		m	Amplitude – displacement amplitude from the mean position
T	$2\pi/\omega$	s	Period – time for repetition, time for a full cycle
ω	$2\pi/T$	rad/s	Angular frequency
f	$1/T, \omega/2\pi$	s^{-1} or Hz	Frequency
c	$f\lambda$	m/s	Wave propagation velocity
v	$2\pi fA$	m/s	Particle velocity
λ	c/f	m	Wavelength – distance between successive crests or troughs of a wave
φ		rad	Phase angle

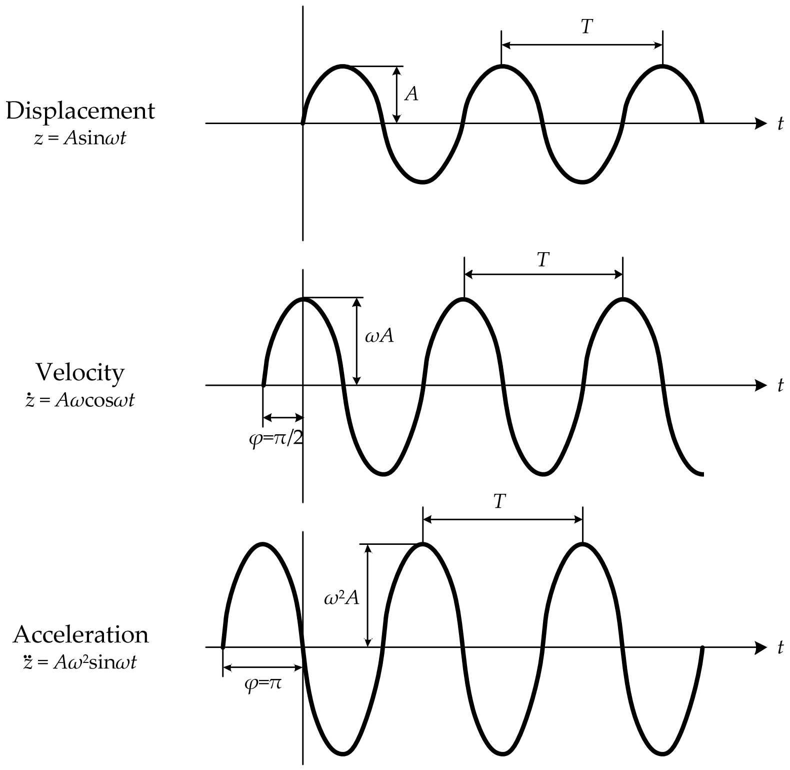

Figure 2.1 Parameters commonly used in dynamics, modified after Möller et al. (2000) and Holmberg et al. (1984).

2.2.1.2 Vibratory motion

A vibration is an oscillatory movement around a state of equilibrium, whereas a blow is a sudden change in the motion of a system. Any vibratory motion can be described using displacement, velocity or acceleration. There are different types of vibratory motion; the most common are described below.

Harmonic motion

The simplest form of vibratory motion is represented by sinusoidal or harmonic motion (Woods, 1997). Harmonic motion is a movement expressed by a harmonic function, see Figure 2.1, where the displacement, z, is a function of time, t. By differentiating the expression for the displacement, the velocity and acceleration are given. The velocity, $\dot{z}$, is the first derivative of z with respect to time, and the acceleration, $\ddot{z}$, is the second derivative. A harmonic motion can be expressed according to the following equations for vertical vibrations (Richart et al., 1970) (Kramer, 1996):

Eq. 2.1 $$z = A\sin(\omega t + \varphi) \quad \text{(m)}$$

Eq. 2.2 $$\dot{z} = \frac{dz}{dt} = A\omega\cos(\omega t + \varphi) \quad \text{(m/s)}$$

Eq. 2.3 $$\ddot{z} = \frac{d^2 z}{dt^2} = -A\omega^2 \sin(\omega t + \varphi) = -\omega^2 z \quad \text{(m/s}^2\text{)}$$

The most important features of harmonic motion are defined by three parameters; amplitude, angular frequency and phase angle. A is the single amplitude. Sometimes the double amplitude, also called the peak-to-peak displacement amplitude, is used, which is equal to $2A$ (Richart et al., 1970). The angular frequency, ω, describes the rate of oscillation in terms of radians per unit time. The phase angle, φ, describes the amount of time by which the peaks are shifted from those of a pure sinus function, see Figure 2.1 (Kramer, 1996). From the three equations above and from Figure 2.1 it can be seen that the velocity is phase shifted $\pi/2$ compared to the displacement (sine-cosine) and that the acceleration is phase shifted π compared to the displacement (sine respectively –sine) (Thurner, 1976).

Periodic motion

Periodic motion is a displacement-time pattern that repeats itself with a period T, see Figure 2.2a. Periodic vibrations are generated by many types of machines with a periodic working cycle, e.g. pumps, vibratory rollers, compressors and fans. In the case of pile driving, impact driving generates periodic vibrations of a transient type (Holmberg et. al., 1984).

Random motion

Random motion is a displacement-time relationship that never repeats itself, see Figure 2.2b.

Transient motion

Transient motion is an irregular, short-term motion that starts off at a high intensity and gradually subsides over a period of time, see Figure 2.2. An example of a transient vibration could be what a building experiences when impact pile driving is performed nearby (Holmberg et. al., 1984).

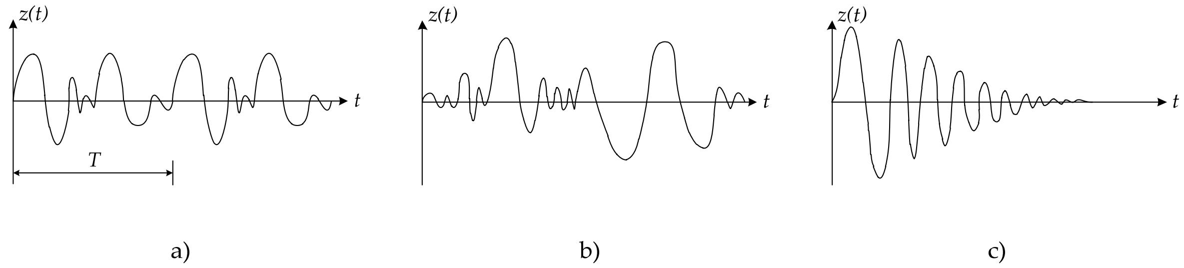

Figure 2.2 Examples of types of vibratory motion a) periodic motion, b) random motion and c) transient motion.

2.2.2 General wave propagation

Individual particles are excited by a force that transmits the motion to the adjacent particles. As the motion continues from particle to particle, it results in waves travelling through the material. Wave propagation is the transportation of energy through a medium without the transportation of any materials. As a wave passes through a medium, the particles in the material are excited around an equilibrium state and the particle is both deformed and moved, as well as receiving strain energy and kinetic energy. Wave propagation can be considered to have two separate motions; a wave travels through a medium with a wave propagation velocity, c, and the particles move with a particle velocity, v (Bodare, 1996).

Wave propagation velocity, c, refers to the speed at which a seismic wave travels through the ground while the particle velocity, v, refers to the speed at which an individual particle oscillates about an "at-rest" position. To characterise wave motion, the particle velocity is often used (Woods, 1997).

2.2.2.1 Resonance

During resonance the response of the system increases steadily, theoretically towards infinity. In practice, without damping something would break and result in failure. In reality, some damping always prevents the result from going to infinity (Nordal, 2009).

For a rod there are theoretically an infinite number of natural frequencies; however, for most practical problems the lowest frequencies are the most important (Richart et al., 1970).

2.2.2.2 Wave types

In an elastic half-space, there are different types of waves, see Figure 2.3. Some characteristics of the various wave types are described below.

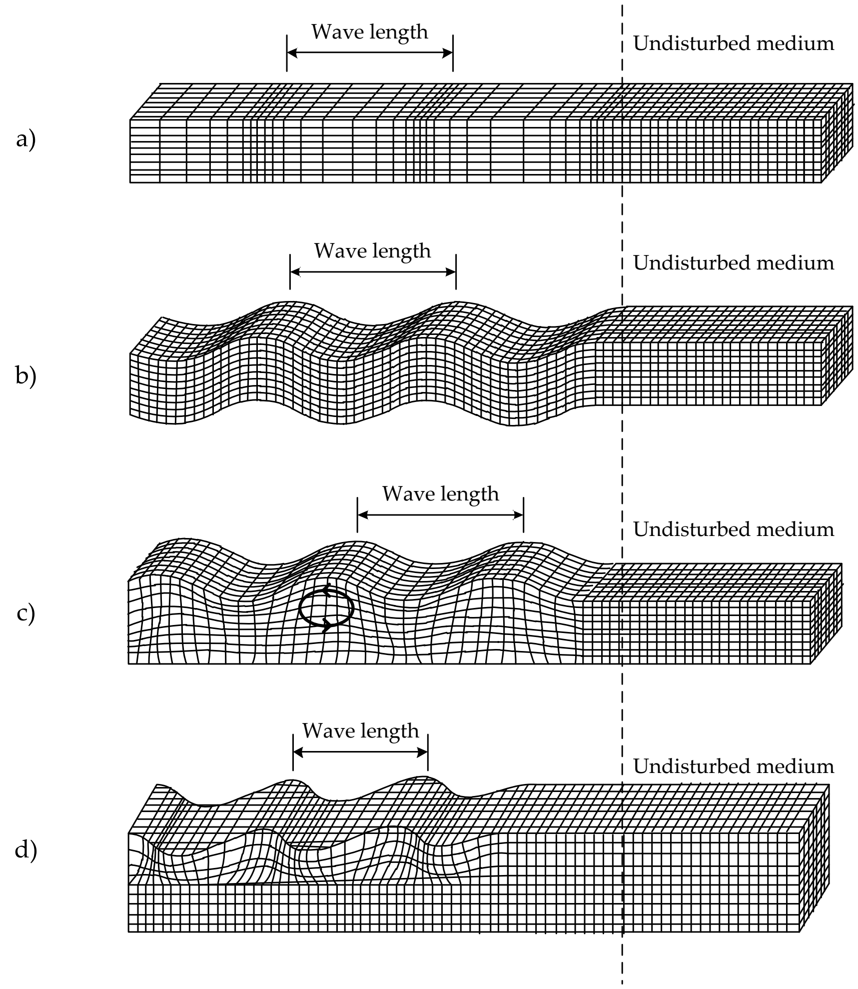

Figure 2.3 Displacement characteristics of different wave types, a) P-wave, b) S-wave, c) R-wave and d) Love-wave, modified after Woods (1997) and Kramer (1996).

a) P-wave	A push-pull motion in the direction of the wave
b) S-wave	Oscillation perpendicular to the propagation direction
c) R-wave	A sort of combination of P- and S-waves with ellipsoidal particle motion
d) L-wave	A snake-like movement

A more thorough description of the wave types follows.

<u>Body waves</u>

Body waves are named for the fact that they, unlike surface waves, travel inside a body or medium (Nordal, 2009). Body waves are generally divided into P-waves and S-waves. P- and S-waves exist one by one and are independent of each other in a full space. Davis (2010) mentioned another type of wave that can be present in saturated soil, called a Biot wave. This wave is a combination between a compression wave in a fluid and a compression wave in a soil.

P-waves

P-waves are also known as primary, compressional or longitudinal waves. P-waves are linked to a volume change in the medium as they involve successive compression and rarefaction (dilatational wave). Particle motion is parallel to the direction of wave propagation. P-waves can travel through both solids and fluids (Richart et al., 1970) (Kramer, 1996).

The P-wave (or primary wave) involves no shearing or rotation of the material as it passes through. P-waves are the fastest wave present in a solid material. In terms of the shear modulus and Poisson's ratio, the P-wave velocity can be written as (Kramer, 1996) (Möller et al., 2000):

Eq. 2.4 $$c_P = \sqrt{\frac{M}{\rho}} = \sqrt{\frac{G(2-2\upsilon)}{\rho(1-2\upsilon)}} = \sqrt{\frac{E(1-\upsilon)}{\rho(1-2\upsilon)(1+\upsilon)}}$$ (m/s)

Where M = deformation modulus or oedometer modulus (Pa)
G = shear modulus (Pa)
E = elasticity modulus (Pa)
ρ = material density (kg/m^3)
υ = Poisson's ratio (-)

S-waves

S-waves are also known as secondary, shear or transverse waves. An S-wave causes shearing deformations as it propagates through a medium. S-waves cannot travel through fluids due to the fact that fluids have no shearing stiffness (Kramer, 1996).

The S-wave involves no volume change and is an equivoluminal or distortional wave. The velocity of a shear wave can be calculated from (Richart et al., 1970) (Kramer, 1996) (Bodare, 1996) (Möller et al., 2000) (Massarsch, 2000a):

Eq. 2.5 $$c_S = \sqrt{\frac{G}{\rho}} = \sqrt{\frac{E}{2\rho(1-\upsilon)}}$$ (m/s)

Where G = shear modulus (MPa)
ρ = total density (kg/m^3)
E = elasticity modulus (MPa)
υ = Poisson's ratio (-)

S-waves are often divided into two perpendicular components, SH-waves and SV-waves. SH-waves are S-waves in which the particles oscillate in a horizontal plane. SV-waves are S-waves in which the particles oscillate in a vertical plane. Any given S-wave can be expressed as the vector sum of it's SH and SV components (Kramer, 1996).

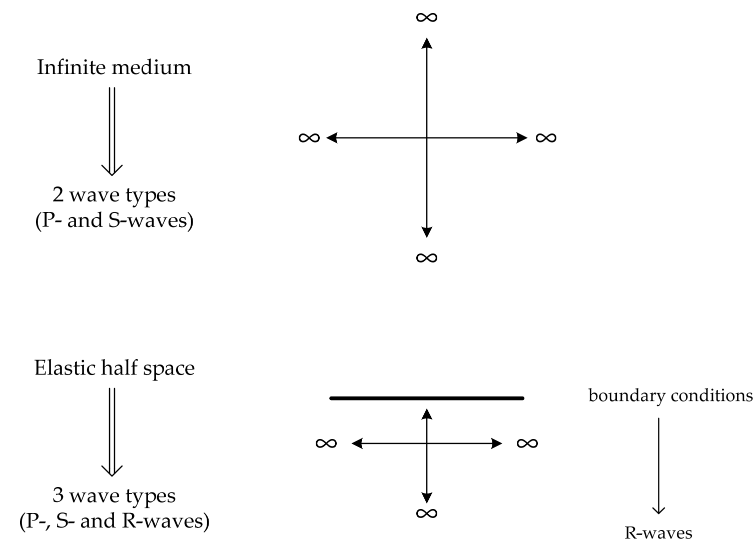

Figure 2.4 Wave types for different boundary conditions in elastic media, modified after Nordal (2009).

Surface waves

The ground is usually conceptualised as a semi-infinite body with a planar-free surface (an elastic half-space). The stress-free surface of an elastic half-space imposes special boundary conditions that result in waves other than body waves, namely surface waves. Surface waves are the result of interaction between body waves and the surface, see Figure 2.4. Surface waves travel along the surface with amplitudes that decrease roughly exponentially with depth (Kramer, 1996).

There are a number of different types of surface waves; the two most common are discussed below (R-waves and Love waves). Bodare (1996) also mentioned Stonely waves that can arise in the interface between two elastic materials; however, these waves have not been shown to be of importance in geodynamics and are not treated any further in this thesis.

R-waves

The most common type of surface waves are Rayleigh waves (R-waves). R-waves are a product of interaction of P- and SV-waves with the surface (Kramer, 1996). R-waves can be seen as combinations of P- and S-waves. Their motion near the surface is in the form of a retrograde ellipse, see Figure 2.3, while at the surface of water waves, the particle motion is instead that of a prograde ellipse. R-waves involve both vertical and horizontal particle motion (Kramer, 1996). At a depth of around $0.2\lambda_R$ the motion changes direction to rotate in a prograde direction (Bodare, 1996), see Figure 2.5.

The depth to which an R-wave causes significant displacement increases with wavelength. As such, R-waves with long wave length (low frequency) can produce particle motion at greater depths than R-waves with short wavelengths (high frequency) (Bodare, 1996) (Kramer, 1996).

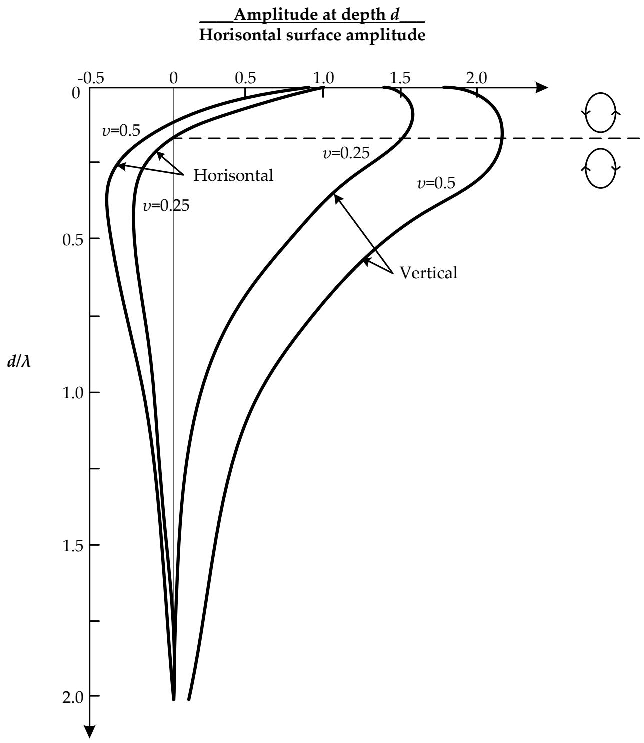

Figure 2.5 Horizontal and vertical vibration amplitude of the Rayleigh wave as a function of depth, Poisson's ratio and wavelength modified after Richart et al. (1970).

Figure 2.5 shows the Rayleigh wave's horizontal and vertical amplitude as a function of depth, d, Poisson's ratio, υ, and the wavelength, λ. From Figure 2.5 it is noticed that the vertical amplitude is greater than the horizontal amplitude and also that the vertical amplitude decreases rapidly with depth.

The velocity of the R-wave can be estimated according to the following equation (Holmberg et al., 1984) (Bodare, 1996):

Eq. 2.6 $$c_R \approx \frac{c_s(0.87 + 1.12\upsilon)}{1 + \upsilon} \qquad \text{(m/s)}$$

Where c_S = shear wave velocity (m/s)
υ = Poisson's ratio (-)

By inserting υ=1/3 in Eq. 2.6 $c_R \approx 0.93 c_S$, hence, the R-wave velocity is often approximated with the S-wave velocity.

R-waves are non-dispersive in a homogenous half-space, meaning that the propagating velocity is independent of vibration frequency (Richart et al., 1970). In a layered elastic half space the R-waves are dispersive and the propagation velocity depends on frequency (Jongmans & Demanet, 1993) (Whenham, 2011).

Love waves

Another type of surface wave is the Love wave, resulting from the interaction of SH-waves with a soft surface layer. Love waves are horizontally polarised shear waves and have no vertical component of particle motion (Kramer, 1996) (Athanasopoulos et al., 2000) (Whenham, 2011). Love waves only exist when there is a layer of low velocity overlaying a layer of higher velocity. In a homogenous half-space no Love-waves are produced (Auersch, 1995) (Athanasopoulos et al., 2000) (Whenham, 2011).

Essentially, Love waves consist of SH-waves that are reflected within the surface layer. The displacement amplitude of the Love wave varies sinusoidally with depth and decays exponentially with depth (Kramer, 1996) (Niederwanger, 1999). Love waves travel with a velocity that is between the shear wave velocity of the superficial layer and the shear wave velocity of the next lower layer (Richart et al., 1970).

The propagation velocity of Love waves are between the R-wave velocity and the S-wave velocity. The velocity of the Love wave varies with frequency between an upper and lower limit, hence they are dispersive (Martin, 1980) (Kramer, 1996). The wave propagation velocity for Love waves is dependent upon the wavelength, λ_L, and the frequency.

2.2.2.3 Waves in a layered body

According to Kramer (1996) a wave front is defined as a surface of equal time travel, see Figure 2.6.

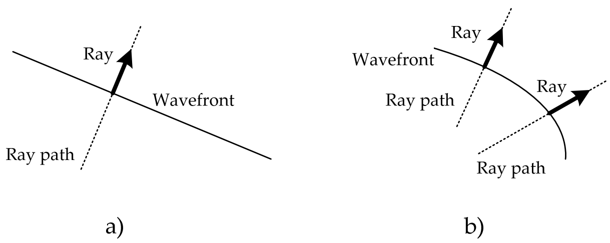

Figure 2.6 Ray path, ray and wave front for a) plane wave and b) curved wave front, modified after Kramer (1996).

A body wave travelling in an elastic medium that encounters a boundary with another elastic medium will partly be reflected back into the first medium and partly be transmitted into the second medium (Richart et al., 1970). In Figure 2.7 the different types of waves produced by incident P-, SV- and SH-waves are illustrated. P- and SV-waves approaching an interface involve particle motion perpendicular to the interface plane; hence they produce both reflected and refracted P- and SV-waves. For an incident SH-wave, no particle motion perpendicular to the interface occurs. As a result, only SH-waves are reflected and refracted and no P-waves or SV-waves are produced. Both the direction and amplitude of the incident wave affect the directions and relative amplitudes of the waves produced at the interface (Richart et al., 1970) (Kramer, 1996) (Bodare, 1996).

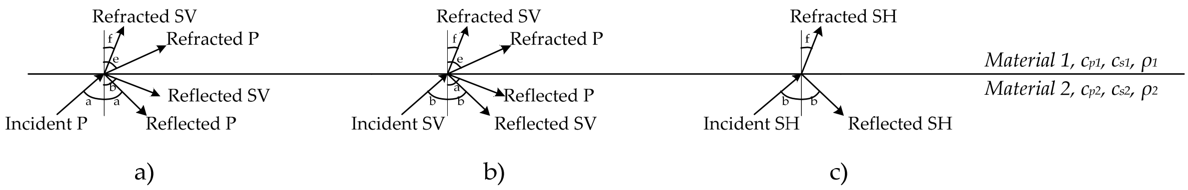

Figure 2.7 Reflected and refracted rays resulting from an incident a) P-wave, b) SV-wave and c) SH-wave, modified after Richart et al. (1970) and Kramer (1996).

For both P-and S-waves the angle of incidence is equal to the angle of reflection, while the angle of refraction is dependent on the angle of incidence and the ratio of the wave velocities of the materials on each side of the interface (Kramer, 1996). Snell's law can give exit angles for all waves (Richart et al., 1970):

Eq. 2.7
$$\frac{\sin a}{c_{p1}} = \frac{\sin b}{c_{s1}} = \frac{\sin e}{c_{p2}} = \frac{\sin f}{c_{s2}}$$

A half-space of multiple layers results in a complex array of waves as waves are reflected and refracted at each interface (Richart et al., 1970).

Waves cannot collide. If two or more waves exist within the same area these are added to each other, a phenomenon called interference. If the waves have the same frequency and reaches maximum at the same time (they are in phase), interference results in amplification. If the other wave instead is out of phase by half a wavelength, they will weaken each other. The combination of refraction, reflection and interference of waves means that in layered materials, amplification and weakening may occur that is very hard to theoretically foresee (Möller et al., 2000). The heterogeneities in the ground and the creation of new waves along with the reflection and refraction of ray paths cause the ground vibrations to reach a vulnerable object by many different paths (Kramer, 1996).

2.2.3 Vibration attenuation and damping

In an ideal linear elastic material, stress waves travel infinitely, without amplitude change. However, in real materials this type of behaviour is not possible; stress waves attenuate with distance. The attenuation is caused by two sources; the geometry of the wave propagation (geometric damping) and the material or materials through which the waves travel (material damping) (Kramer, 1996) (Massarsch, 2004).

2.2.3.1 Geometric damping

Geometric damping reduces the amplitude of the vibrations as distance from the source increases, due to the fact that the same energy is spread over an increasingly larger surface or volume. From the theory of energy conservation, the wave attenuation due to geometric damping can be described with the following expression (Woods, 1997) (Nordal, 2009):

Eq. 2.8 $$A_2 = A_1\left(\frac{r_1}{r_2}\right)^n$$ (m)

Where A_2 = amplitude of motion at distance r_2 from the source (m)
A_1 = amplitude of motion at distance r_1 from the source (m)
n = ½ for R-waves (-)
1 for body waves (-)
2 for body waves at the surface (-)

The value of n depends on wave type. Since surface waves propagate as expanding rings, the energy per unit area of the wave decays inversely proportional to the distance from the source and surface waves experience a lower geometric damping than body waves (Rockhill et al., 2003) (Kramer, 1996).

2.2.3.2 Material damping

Material damping is the loss of energy due to internal energy dissipation in the material as the soil particles are moved by the propagating wave. Wave energy is transformed to friction heat, and as the energy is converted and "lost" the amplitude of the wave decreases (Attewell & Farmer, 1973) (Heckman & Hagerty, 1978) (Holmberg et. al., 1984) (Kramer, 1996). The big difference between material damping and geometric damping is that in material damping, elastic energy is actually dissipated by viscous, hysteretic, or other mechanisms (Kramer, 1996).

Material damping can be described by the following exponential function (Dowding, 1996):

Eq. 2.9 $$A_2 = A_1 e^{-\alpha(r_2 - r_1)}$$

Where A_2 = amplitude of motion at distance r_2 from the source (m)
A_1 = amplitude of motion at distance r_1 from the source (m)

α = absorption coefficient (m^{-1})

The absorption coefficient, α, can be estimated according to (Athanasopoulos et al., 2000) (Massarsch & Fellenius, 2008):

Eq. 2.10 $$\alpha = \frac{2\pi D f}{c}$$ (m^{-1})

Where D = material damping $(Hz\ s)^{-1}$
f = vibration frequency (Hz)
c = wave propagation velocity (m/s)

The wave propagation velocity is usually either expressed by the surface wave velocity, c_R, or the shear wave velocity, c_s. According to Bodare (1996) Eq. 2.10 is valid under the condition that $D \ll 1$ applies.

From equation Eq. 2.10 it can be seen that the absorption coefficient, α, decreases by decreasing vibration frequency and increasing wave propagation velocity. Hence, a wave with low frequency is damped less than a wave with high frequency (Martin, 1980) (Holmberg et al., 1984) (Athanasopoulos & Pelekis, 2000) (Auersch & Said, 2010).

It is clear that the absorption coefficient, α, varies with the characteristics of the material, the wave type and the frequency. Generally, softer materials have greater values of α than harder materials; thus clay generally exhibits greater damping than, for example, sand (Holmberg et al., 1984) (Woods, 1997) (Athanasopoulos et al., 2000) (Möller et al., 2000). Through their measurements, Clough & Chameau (1980) showed that softer soils damped out vibrations faster than denser soils. Auersch & Said (2010) report strongest damping for a peaty soil.

Table 2.2 shows different values of α for different types of materials and frequencies. The coefficient is also dependent on the material's settlement characteristics. The values of α is important for correct estimation of the vibration attenuation, though reaching a satisfying value of α is difficult; however, tables such as Table 2.2 can be used to give an approximate value (Whenham, 2011).

Table 2.2 Attenuation coefficient according to classification of rock and soil materials (Dowding, 1996) (Woods, 1997).

Class	Attenuation coefficient, α (m^{-1})			Description of material
	5 Hz	40 Hz	50 Hz	
I	0.01 - 0.033	0.08 – 0.26	0.1 – 0.3	Weak or soft soil
II	0.0033 - 0.01	0.026 – 0.08	0.03 – 0.1	Competent soil
III	0.00033 - 0.0033	0.0026 – 0.026	0.003 – 0.03	Hard soil
IV	< 0.00033	< 0.0026	< 0.003	Hard, competent rock

Amick & Gendreau (2000) stated that the magnitude of the material damping depends on vibration amplitude, soil type, moisture content and temperature, for example. It has been seen that wet sand damps vibrations less than dry sand, since the pore water in the wet sand helps to carry compression waves that are then not subjected to friction damping. Amick & Gendreau (2000) also claimed that according to Barkan (1962), frozen soil attenuates vibrations less than thawed soil.

The material damping is also dependent upon the deformation size, see Figure 2.8 (IVA, 1979 and 1983). As the strain level increases and the soil element loses stiffness, an increase in damping is seen. The damping ability is connected to the energy dissipated in the soil (by friction, heat or plastic yielding) (Bodare, 1996) (Kim & Lee, 2000) (Whenham, 2011). It has been show that the plasticity index of the soil affects the damping for saturated soils, see Figure 2.8 (Bodare, 1996). Highly plastic soils have lower damping ratios than low plasticity soils (Whenham, 2011).

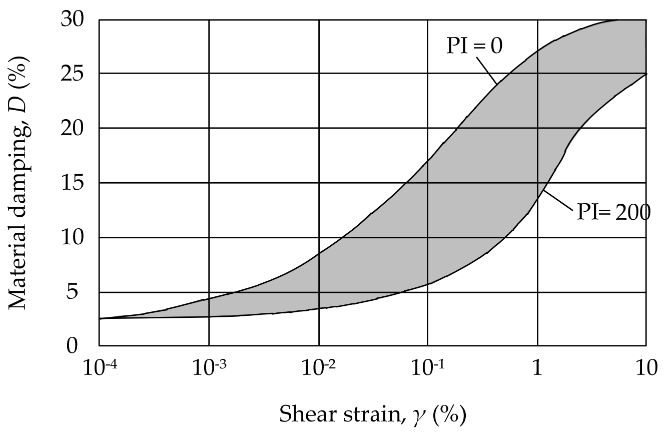

Figure 2.8 Relationship between material damping, shear strain and plasticity index (PI), modified after IVA (1979) and Whenham (2011, after Vucetic & Dobry, 1991).

2.2.3.3 Estimation of total damping for a propagating wave

Lamb (1904) presented a simple theory for the attenuation of ground waves propagating along the ground surface. The attenuation of a cylindrical Rayleigh wave in a homogenous elastic half-space is presented as:

Eq. 2.11 $$A \approx r^{-0.5} \quad \text{(m)}$$

Where A = wave amplitude (m)
r = distance from the source (m)

For the attenuation of surface waves generated by earthquakes, Galitzin (1912) developed a relationship for the attenuation between two points at distances r_1 and r_2 from the source:

Eq. 2.12 $$A_2 = A_1 \sqrt{\frac{r_1}{r_2}} e^{-\alpha(r_2 - r_1)} \quad \text{(m)}$$

Where A_1 and A_2 = vibration amplitude at distance r_1 respectively r_2 from the source (m)

α = attenuation coefficient (m^{-1})

After Lamb's (1904) and Galitzin's (1912) fundamental work the attenuation model has been studied further and developed over the years. However, the base for the geometric attenuation is still the same more than 100 years later, and the total attenuation of waves propagating in soil is approximated by:

Eq. 2.13 $$A_2 = A_1 \left(\frac{r_1}{r_2} \right)^n e^{-\alpha(r_2 - r_1)}$$ (m)

Where A_1 = vibration amplitude at distance r_1 from the source (m)
A_2 = vibration amplitude at distance r_2 from the source (m)
α = absorption coefficient (m^{-1})
n = ½ for surface waves (-)
1 for body waves (-)
2 for body waves along the surface (-)

This equation is only valid under homogenous conditions and when the depth to the rock surface is great (Möller et al., 2000). Athanasopoulos et al. (2000) concluded that Eq. 2.13 is satisfactory for describing the attenuation of Rayleigh waves with distance as long as correct values for the coefficients are used.

2.2.4 Dynamic properties

The soil's behaviour when subjected to dynamic loading is governed by its' dynamic properties (Kramer, 1996). Some of the most important properties are described in this section (except for material damping, which is described in the previous section).

2.2.4.1 Shear Modulus

The shear modulus, *G*, is a measure of the stiffness a material shows at shearing. The shear modulus in soil varies with the strain and has its largest values, G_{max}, at shear strains smaller than 10^{-5} (0.001 %), see Figure 2.9. For larger strains the soil behaviour becomes elasto-plastic and the shear modulus decreases as the inner damping increases. At shear strains of about 10^{-3} and larger, both the shear modulus and the damping is affected by the number of cycles and the frequency (Erlingsson & Bodare (1992 and 1996) (Möller et al., 2000) (Whenham, 2011). Just as for material damping, it has been shown that the shear modulus also depends on the plasticity index, *PI*, of the soil, see Figure 2.9 (Bodare, 1996).

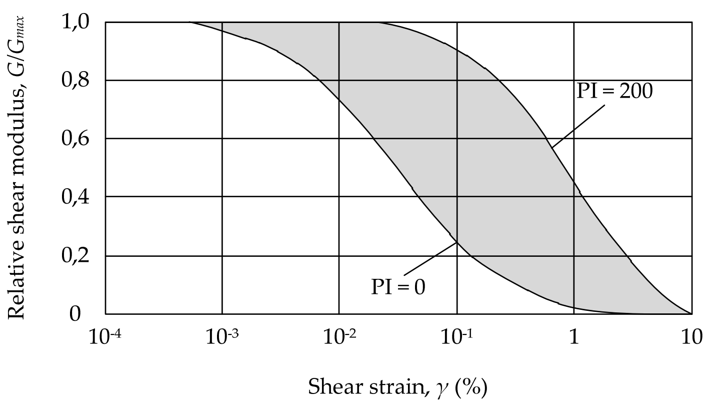

Figure 2.9 Relationship between shear modulus, shear strain and plasticity index (PI), modified after IVA (1979) and Whenham (2011, originally from Vucetic & Dobry, 1991).

The shear modulus, *G*, is related to the elasticity modulus, *E*, and the compression modulus, *M*, accordingly (Dowding, 1996) (Nordal, 2009):

Eq. 2.14 $G = c_s^2 \rho$ (MPa)

Eq. 2.15 $E = 2G(1+\upsilon)$ (MPa)

Eq. 2.16 $M = c_p^2 \rho$ (MPa)

Where c_s = shear wave velocity
ρ = density
υ = Poisson's ratio
c_p = compression wave velocity

Table 2.3 shows typical values of the shear modulus, *G*, for different soil and rock materials.

Table 2.3 Typical values of shear modulus, G, for some soil and rock materials (Head & Jardine, 1992).

Soil/Material type	Relative density	Shear modulus, G (MN/m^2)
Sand	Loose	15-110
	Medium	70-250
	Dense	230-1000
Clay	Soft	10-65
	Firm	55-190
	Stiff	160-450
Sandstone and shale	-	2600-20000
Unweathered igneous or metamorphic rock	-	8500-32000

2.2.4.2 Wave propagation velocity

It is important to emphasize the difference between the particle velocity, v, and the propagation velocity of the wave front, c. Waves move away from the source at a constant velocity, the propagation velocity. The propagation velocity depends on the characteristics of the transporting media and on the type of wave. The particle velocity is the velocity of displacement of a single individual particle as a wave passes (Heckman & Hagerty, 1978).

Table 2.4 gives typical values of the P-wave velocity, c_p, and the S-wave velocity, c_s, for different materials. The surface wave (R-wave) velocity, c_R, is only slightly lower than the shear wave velocity and the difference is usually considered negligible for practical purposes (Massarsch, 2004) (Massarsch & Fellenius, 2008).

Table 2.4 Typical values of wave velocities in different soils and materials, after Head & Jardine (1992).

Soil/Material type	**c_p (m/s)**	**c_s (m/s)**
Air	344	0
Ice	3000 – 3500	1500 – 1600
Water	1480 – 1520	0
Concrete	3400	2100
Steel	6000	3300
Granite	4500 – 5500	3000 – 3500
Sandstone, shale	2300 – 3800	1200 – 1600
Fractured rock	2000 – 2500	800 – 1400
Saturated moraine	1400 – 2000	300 – 600
Dry moraine	600 – 1500	300 - 750
Saturated sand/gravel	1400 – 1800	100 – 400
Dry sand/gravel	200 – 800	150 – 500
Clay below gw	1450 – 1900	80 – 500
Clay above gw	100 - 600	40 - 300
Organic soils	1480 – 1520	30 – 50

The body wave velocities depend on the stiffness and density of the material they travel through. Since geologic materials are stiffer in compression than in shear, P-waves travel at a higher velocity than S-waves (Kramer, 1996).

The propagation velocity is dependent on many factors, including temperature, effective stress, stratification void ratio and moisture content (Massarsch & Fellenius, 2008). Holmberg et al. (1984) and Woods (1997) stated that the velocity of stress waves in soil or rock depends on the unit weight and the moduli (Young's modulus and shear modulus) of the material.

The P-wave velocity depends on the degree of water saturation (groundwater conditions) in loose soils. Below the groundwater table, the P-wave velocity corresponds to that of water (~1450 m/s) (Massarsch & Fellenius, 2008). Since shear waves are unable to propagate in fluids and gases, the shear wave velocity does not change below the groundwater surface unless the density of the soil is changed (Massarsch & Fellenius, 2008) (Möller et. al., 2000). According to Richart et al. (1970) there seems to be no difference in shear wave velocity between dry, saturated and drained conditions. However, Massarsch & Fellenius (2008) stated that during pile driving the shear wave velocity can decrease due to excess pore water pressure and soil disturbance. The R-wave velocity is not affected by the groundwater level, however, it is generally said to be lower in moist soil (Head & Jardine, 1992).

The wave propagation velocity is also dependent upon Poisson's ratio, υ. Figure 2.10 shows the correlation between Poisson's ratio and the wave propagation velocity, as well as the relationship between the velocities of the different wave types. The P-wave velocity can be seen to increase rapidly as Poisson's ratio increases (Richart et al., 1970).

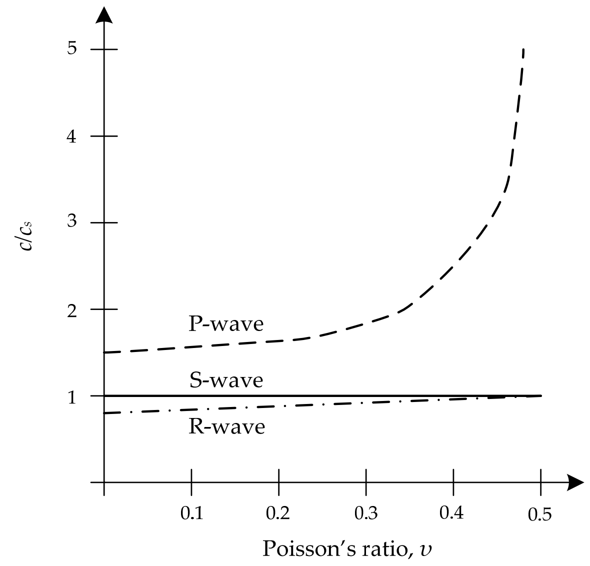

Figure 2.10 Relationship between the propagation speed, c, for different wave types, Poisson's ratio and the shear wave velocity, c_s, modified after Richart et al. (1970).

The shear wave velocity is strongly dependent on the void number and generally increases with depth (confining pressure), see Figure 2.11 (Richart et al., 1970) (Massarsch & Fellenius, 2008). In coarse-grained soils, the P-wave velocity is likely to increase below the pile toe due to compaction, while it may be reduced in fine-grained soils due to disturbance and pore water pressure increase (Massarsch & Fellenius, 2008).

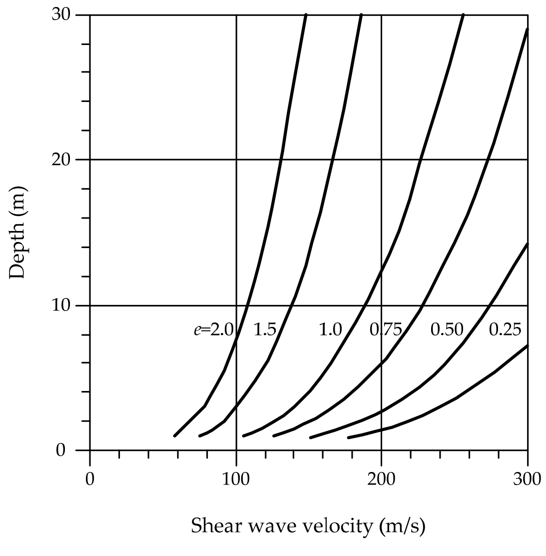

Figure 2.11 Correlation between shear wave velocity, void ratio (e) and depth for normally consolidated, saturated soil (Hintze et al., 1997, originally from Massarsch, 1984).

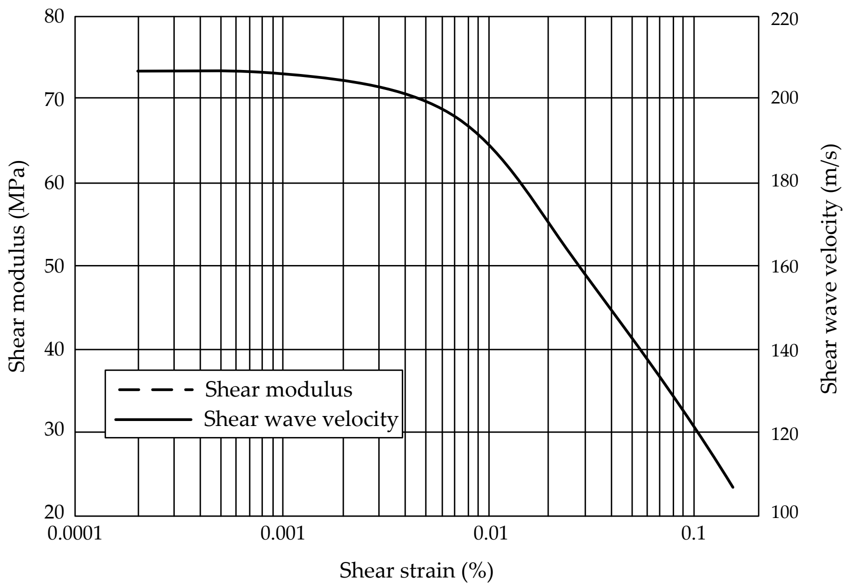

Figure 2.12 Result from a resonant column test on medium dense sand showing shear modulus and shear wave velocity with respect to shear strain, modified after Massarsch (2000a).

Since shear wave velocity is a function of the shear modulus, the shear wave velocity depends on the strain level. When the shear strain exceeds about 0.001% the shear wave velocity decreases considerably, see Figure 2.12 (Massarsch, 2000a) (Athanasopoulos et al., 2000) (Whenham, 2011). For shear strains of less than 0.001% the shear wave velocity is relatively constant and sometimes denotes low-amplitude shear wave velocity (Athanasopoulos et al., 2000).

2.2.4.3 Impedance

The ratio between force and velocity is called impedance. According to Massarsch & Fellenius (2008) impedance governs the transfer and propagation of vibrations in the pile, along the pile-soil interface and in the surrounding soil. Richart et al. (1970) stated that impedance is a measure of the opposition of a system to an applied force.

<u>Pile Impedance</u>

The pile impedance, Z_p, depends on the pile density, ρ, wave propagation speed in the pile, c_B, and the cross sectional area of the pile, A_p. The impedance can also be expressed as a function of the elasticity modulus, E (Bodare, 1996) (Massarsch, 2000b) (Massarsch & Fellenius, 2008):

Eq. 2.17 $$Z_p = \rho c_B A_p = \frac{EA_p}{c_B} = A_p\sqrt{E\rho}$$ (kNs/m)

When driving a pile, the force at the top of the pile must be greater than the penetration resistance of the pile. Impedance limits the amount of force that the pile is able to transmit from the pile head to the pile toe (Heckman & Hagerty, 1978), (Woods, 1997).

Common pile materials are concrete, steel and wood. The impedance of the piles will depend on the cross-section configuration of the piles. Generally timber piles have the lowest impedance due to the elasticity modulus of wood being lower than that of either concrete or steel, however, the cross-sectional area and its shape affect the impedance greatly (Woods, 1997).

In Table 2.5 typical values of acoustic impedance for different pile and ground materials are listed.

Table 2.5 Typical values of impedance and corresponding energy transmission coefficients (Hope & Hiller, 2000).

Material	**Mass density, ρ (kg/m^3)**	**P-wave speed, c_p (m/s)**	**Acoustic impedance, Z (MPa·s·m^{-1})**
Steel	7800	5700	44.5
Concrete	2400	5000	12.0
Sand (saturated)	2000	1500	3.0
Clay (stiff)	2300	2000	4.6
Sandstone	2400	2300	5.5

Soil Impedance

The soil impedance for P-waves, Z_{SP}, depends on the cross-section area of the contact between the pile toe and the underlying soil. It should not be mixed up with the specific soil impedance, z_P, which is a material property of the soil and does not involve the pile geometry. The soil impedance is strain dependent and needs to be adjusted for strain level during pile driving (Massarsch & Fellenius, 2008).

The soil impedance, Z_s, is given according to (Massarsch & Fellenius, 2008):

Eq. 2.18 $$Z_s = A_p c_p \rho_{soil}$$ (kNs/m)

Where A_p = cross-section area of the pile toe (m^2)
c_p = P-wave velocity in the soil (m/s)
ρ_{soil} = density of the soil (kg/m^3)

Specific impedance

Specific impedance specifies the relationship between the compressive stress and the particle velocity of a propagating wave and is a product of wave velocity and material density. Specific impedance is denoted by z (lower-case) and is defined by (Bodare, 1996) (Massarsch & Fellenius, 2008):

Eq. 2.19 $$z_s = \frac{E}{c} = \rho c = \sqrt{E\rho}$$ (kNs/m^3)

The specific impedances for P-waves and S-waves, respectively, are given by:

Eq. 2.20 $$z_{sp} = \sqrt{M\rho} = c_p \rho = \frac{M}{c_p}$$ (kNs/m³)

Eq. 2.21 $$z_{ss} = \sqrt{G\rho} = c_s \rho = \frac{G}{c_p}$$ (kNs/m³)

Where M = deformation modulus or oedometer modulus (MPa)
ρ = material density (kg/m³)
G = shear modulus (MPa)
c_s and c_p = S-wave and P-wave velocity, respectively (m/s)

2.3 INSTALLATION OF PILES AND SHEET PILES

There are a number of different installation methods for piles and sheet piles. Usually piles are driven by either impact or vibratory driving or a combination of both. Installation by drilling is also becoming more and more common. In this literature study installation by means of impact and vibratory drivers are studied further and mechanisms and theories behind the methods are described below.

2.3.1 Impact pile driving

2.3.1.1 Machines and components

A machine for impact driving consists of a base machine on to which a stabiliser is attached to hold the steering for the hammer. There are different types of hammers including drop hammers, diesel hammers, hydraulic hammers and pneumatic hammers.

Drop hammers consist of a weight that is lifted a certain height (drop height) and then released (dropped) onto the pile. The weight may be enclosed in a cylinder (Martin, 1980) (Hansbo, 1994). Drop hammers with weights of 3-4 tonnes are common in Sweden; hammers with weights of up to 8 tonnes exist (Stille & Hall, 1995).

The diesel hammer consists of a free piston in a cylinder. A small explosion is used to lift the piston. The piston is then usually allowed to fall free under gravity before hitting the pile cap (Martin, 1980).

Pneumatic hammers and hydraulic hammers work in principal the same as drop hammers, except that they have cylinders/pistons and hydraulic devices, respectively, to help lift the weight and even accelerate it downward as applicable (Martin, 1980) (Hansbo, 1994).

Impact hammers can be divided into light and heavy hammers. Heavy is when the weight of the drop hammer is larger than the total weight of the pile/sheet pile. Usually heavy hammers beat around 30-60 blows per minute while light hammers beat 300-1000 blows per minute (Holmberg et al., 1984).

2.3.1.2 Basic theory

The driving energy during impact pile driving comes from the hammer striking the pile head with downward impact velocity (Masoumi et al., 2007). The hammer energy is transferred by blow impulses via the pile cap to the pile head. Part of the blow energy is damped by the cap and some is lost in the contact area between the cap and the pile head. The rated energy varies between 5 up to 300 kJ per blow for the most commonly used impact hammers (Svinkin, 2005) (Svinkin, 2008). Of this energy it is estimated that only 30-50% is transferred into the pile (Svinkin, 2008).

Impact drivers can drive piles into any type of soil and in order for the pile to penetrate the soil, the static soil resistance must be overcome by the induced force in the hammer blow (Van Rompaey et al., 1995) (ArcelorMittal, 2008). In each blow the pile is accelerated out of rest, which means that the inertia and the shaft- and toe resistance must be overcome with each blow (Massarsch & Fellenius, 2008).

2.3.2 Vibratory pile driving

2.3.2.1 Machines and components

Vibratory driving is used throughout the world mainly for driving and extracting sheet piles. The reason why the technique is not commonly used for piles is believed to be because there is a lack of guidelines for driving to refusal and bearing capacity for piles driven with vibratory drivers (Viking, 2002b).

Vibratory drivers can be classified into three basic categories (Warrington, 1992):

1. **Low frequency machines** – Vibrator frequency between 5-10 Hz. Primarily used for pile types with a high mass and large toe resistance, e.g. concrete and large steel pipes.
2. **Medium frequency machines** – Vibrator frequency between 10-30 Hz. The majority of all vibratory pile drivers used today are of this type.
3. **High frequency machines** – Vibrator frequency greater than 30 Hz. This category is usually divided into two groups. First, machines in the 30-40 Hz range that are designed to minimize vibration of neighbouring structures. Second, resonant pile drivers that operate at frequencies of 90-120 Hz. The resonant pile driver induces resonant response in the pile, which facilitates driving.

The most common vibratory hammers consist of pairs of eccentrically mounted masses, see Figure 2.13. The masses are contained in a frame whose appreciable mass may be called the oscillator (or exciter block). The oscillator is isolated from the hammer support by a static mass (bias mass or suppressor housing). Between the oscillator and the static mass there is a very soft spring, generally consisting of elastomer pads. The static mass adds a static force to oscillator and pile. The pile is attached to the oscillator with a hydraulic clamp. The hammer is run by a power generator and a control panel is usually mounted on the power generator. The whole vibrator is mounted on a piling frame (Holeyman, 2002) (Rausche, 2002) (Viking, 2006) (Whenham, 2011) (Whenham & Holeyman, 2012). Vibratory driving systems can be either free hanging or leader mounted (Viking, 2006). A free hanging model is illustrated in Figure 2.13.

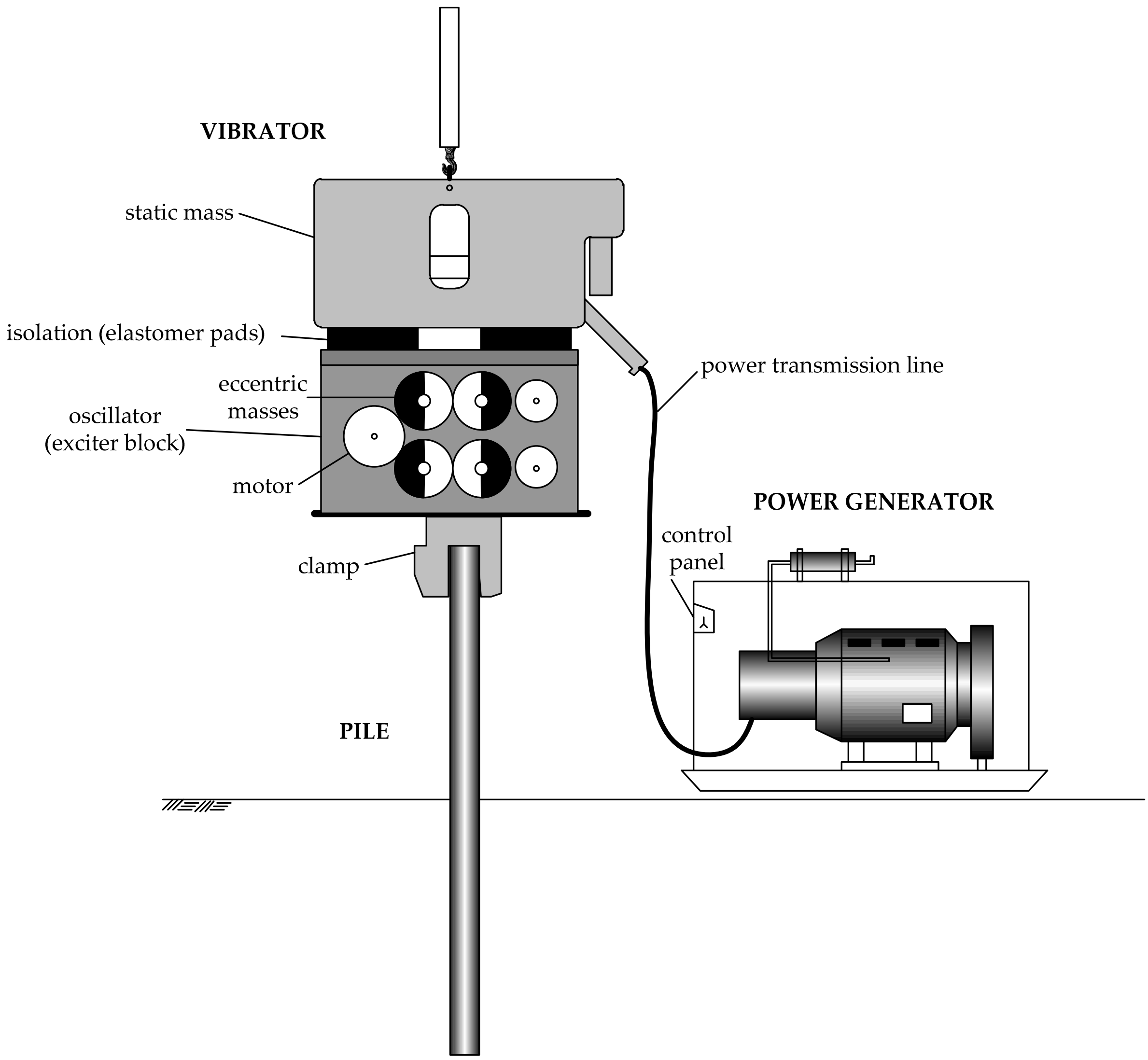

Figure 2.13 Equipment for vibratory driving of piles (free hanging model), modified after Massarsch (2000b) and Holeyman (2002).

On the market today there are two types of vibrators; hydraulic and electric. The difference is that the motor, housed in the vibrator, is powered by either a carrier mounted diesel-hydraulic or diesel-electric power pack. The hydraulic power pack is basically a diesel motor coupled to a hydraulic pump, which interacts with the vibrator via hydraulic hoses. Today hydraulic systems are most common. Hydraulic motors are smaller than electric motors and thus lighter, which is one of the reasons why hydraulic vibrators are more commonly used (Holeyman, 2002) (Whenham, 2011).

2.3.2.2 Basic theory

The oscillation of the vibrator is caused by the eccentric masses, which rotate with the same speed but in opposite directions, see Figure 2.14. The vibrator is then put in vertical vibration since the centrifugal force's horizontal components is diminished (Woods, 1997) (Massarsch, 2000) (Whenham, 2011).

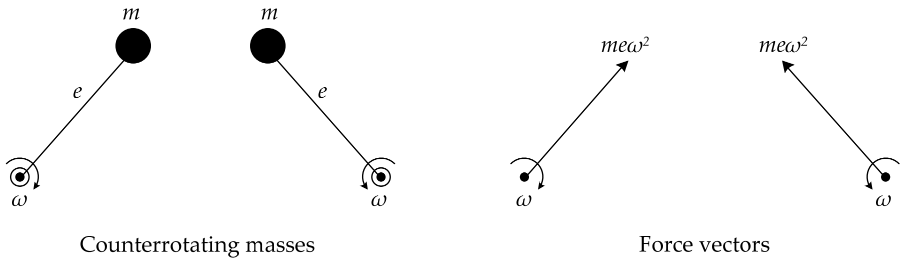

Figure 2.14 Counter rotating masses and the produced forces, modified after Richart et al. (1970).

Variable amplitude vibrators work according to the principle of two pairs of eccentrics that can move relative to one another. In this way the eccentrics can add themselves totally, add themselves partially or cancel each other out (Houzé, 1994). As the rotating eccentrics are kept in opposite positions the resulting moment is zero causing no amplitude of vibration, see Figure 2.15a. If the eccentrics are turned 60°, the resulting moment and amplitude of vibration reaches 50% of the maximum values, see Figure 2.15b. When one of the eccentrics in each pair is turned 180° they work in time with the other half creating maximum moment and amplitude, see Figure 2.15c (Houzé, 1994).

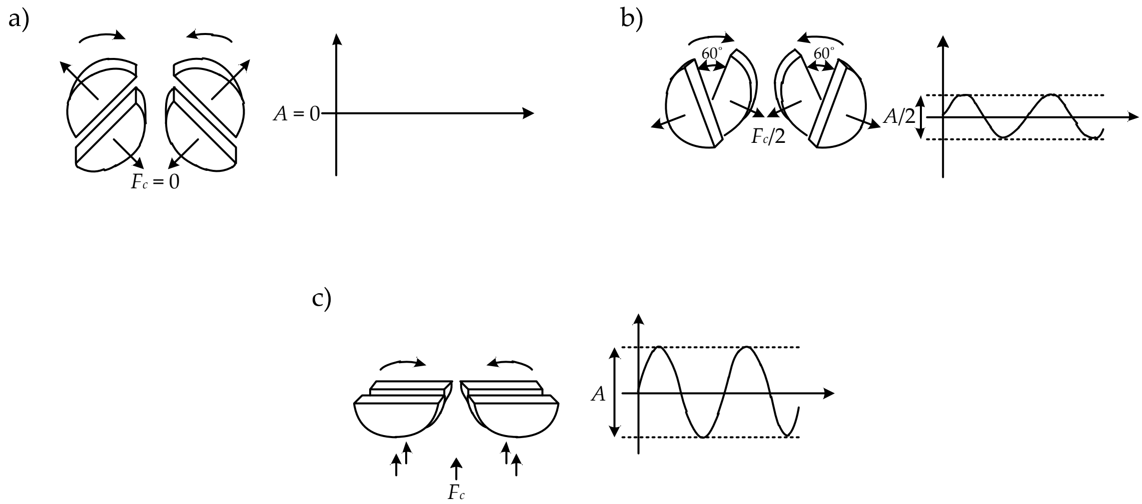

Figure 2.15 Variation of relative position of eccentrics in variable amplitude vibrators; a) eccentrics in opposite position, b) eccentrics turned 60° and c) eccentrics working in time, modified after Houzé (1994).

A vibratory driver drives the pile into the soil with two mechanical actions; a vibratory action and a stationary action. The vibratory action is produced by the counter-rotating masses and the stationary action by the weight of the pile and hammer (the static mass) (Holeyman, 2002). The vibration leads to pore pressure build up and eventually to liquefaction and a significant reduction of the static soil resistance, enabling the pile to penetrate the ground. If the soil conditions are suitable the pile/sheet pile is driven into the ground by its own weight and the weight of the vibrator (Houzé, 1994) (Van Rompaey et. al., 1995) (Niederwanger, 1999). Viking (2006) discussed that the loss in shear strength during vibratory driving is due to a drop in intergranular forces between the grains as the acceleration amplitude exceeds the initial overburden pressure.

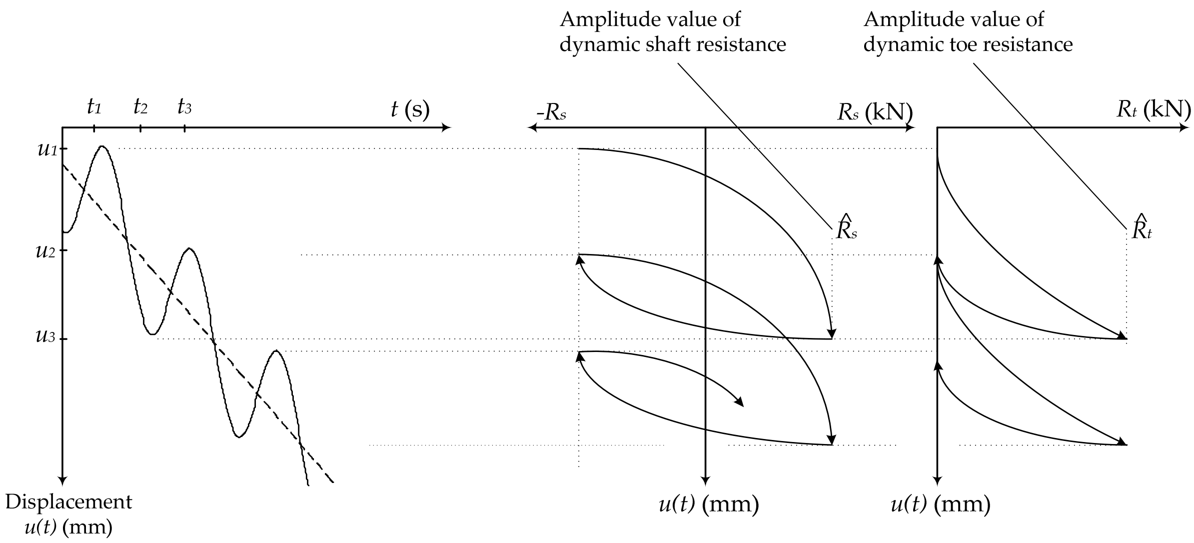

Figure 2.16 Schematic description of a) the penetrative motion, b) the shaft resistance and c) the toe resistance, modified after Viking (2000).

The penetration of a sheet pile during vibratory driving depends on the characteristics of the mechanical interaction and dynamic nature of the whole vibrator, sheet pile and soil system (Viking, 2002b). During vibratory driving the following forces act on the pile (Vanden Berghe & Holeyman, 2002) (Whenham, 2011):

- The vibrating force from the vibrator, F_v
- The static weight on top of the vibrator, F_0
- The friction resistance along the shaft, R_s
- The toe resistance, R_t
- The inertial force induced by the movement of the mass of the pile and the vibrator.

The driving force, F_d, consisting of the static overload, F_0, and dynamic driving force, F_v, varies with a sinus shape in time with the driving frequency. The penetration movement of the sheet pile, $u(t)$, is a downward sinus shaped displacement, correlated in time with the driving force, see Figure 2.16a. During penetration the dynamic shaft resistance, R_s, varies between positive and negative, in correlation with the upward and downward penetration motion, see Figure 2.16b. The dynamic toe resistance, R_t, on the other hand, varies between zero and maximum, also in correlation with the penetrative motion, reaching maximum at the lower end of the up- and downward motion, see Figure 2.16c (Viking, 2000) (Massarsch, 2000b) (Holeyman & Legrand, 1997).

2.4 VIBRATION TRANSFER PROCESS

Unless the entire chain of vibration transmission is considered, it is not possible to fully understand a ground vibration problem. In the following sections the most important aspects governing the propagation of driving energy from the pile driving equipment (the source) to the surrounding soil layers and further on to a potential damaged object will be discussed.

The vibration transfer process is here divided into three parts (same division is seen in e.g. Stille & Hall (1995) and Massarsch (2000a)), in turn divided into smaller parts, see Figure 2.17:

1. Vibration source
    a. Energy transfer between hammer and pile
    b. Vibration in piles
    c. Interaction between pile and soil
2. Wave propagation in soil
3. Damaged object
    a. Interaction between soil and structure
    b. Vibration transmission in structures

The damaged object including the important aspects of soil-structure interaction and vibrations transmission in structures is not studied in this licentiate thesis.

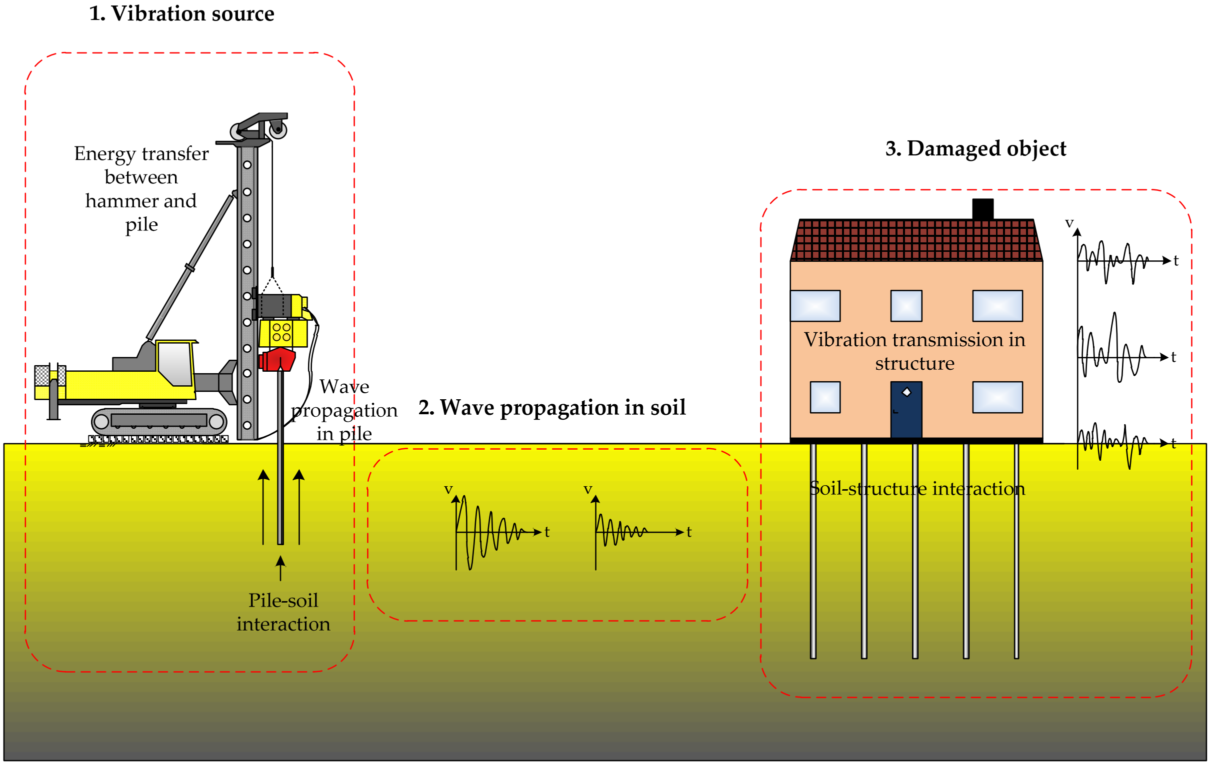

Figure 2.17 Schematic illustration of the vibration transfer during pile driving in urban areas, modified after Hintze et al. (1997).

2.4.1 Vibration source

2.4.1.1 Different Vibration Sources

Vibrations arise from a number of different activities. When it comes to man-made vibrations there are usually three different sources that are identified: operation of machinery, road and railway traffic, and construction activities (Athanasopoulos & Pelekis, 2000). Another important vibration source is natural vibrations such as earthquakes. Figure 2.18 show typical time sequences of vibrations caused by a) impact pile driving b) vibratory driving, c) blasting and d) earthquake.

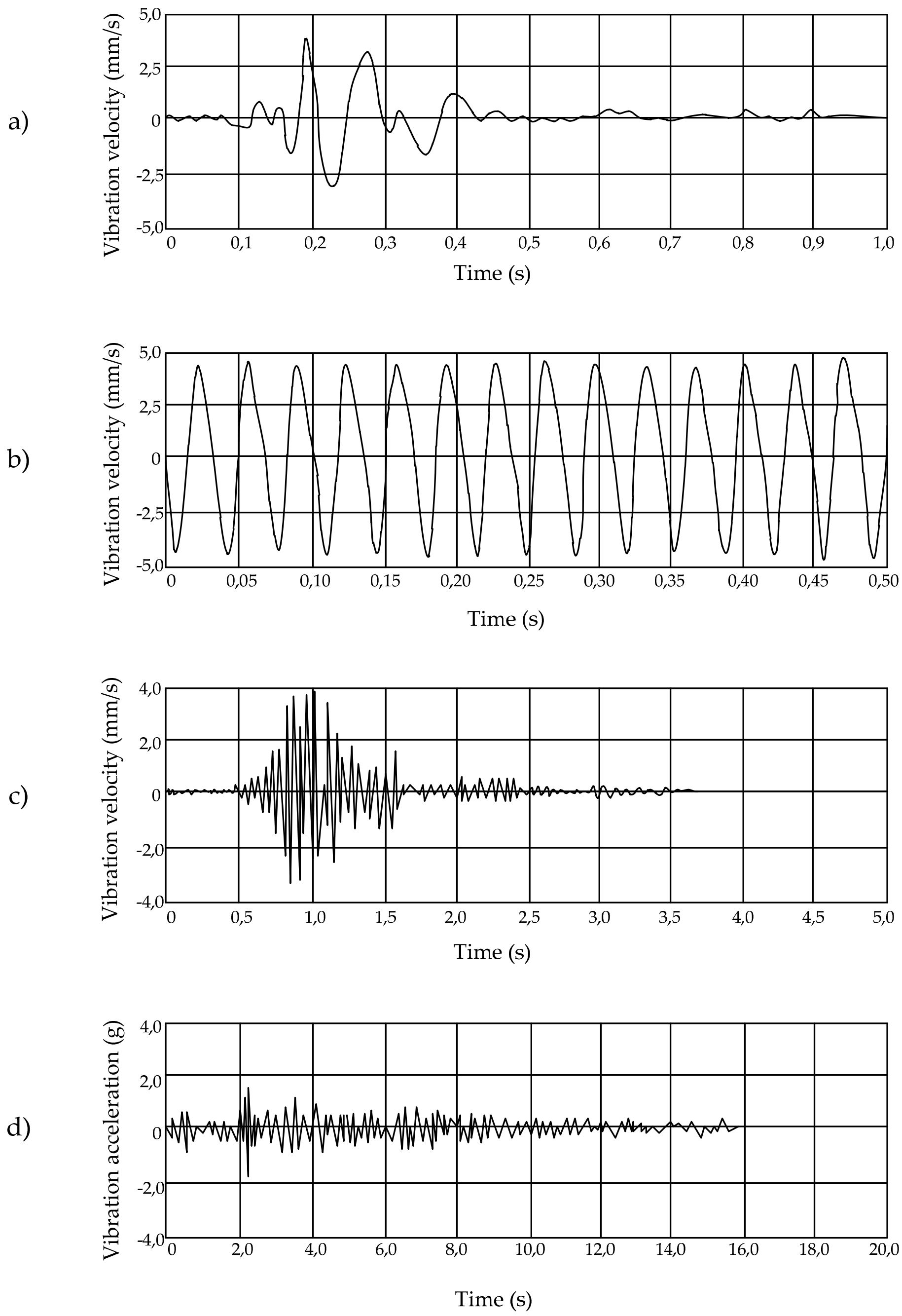

Figure 2.18 Typical vibration sequence from a) impact pile driving, b) vibratory pile driving, c) blasting and d) earthquake, modified after Möller et al. (2000) and Lidén (2012). Observe the different time-scales on the x-axis and note that d) is for vibration acceleration.

This thesis deals with vibrations due to pile driving by impact driving, which generates transient vibrations, and vibratory driving, which generates continuous vibrations.

One of the differences between impact driving and vibratory driving is the frequency of excitation. For vibratory pile driving, the frequency is relatively low and range from about 10 to 50 Hz, while for impact driving frequencies are higher, up to 300 Hz (Svinkin, 2004) (Thandavamoorthy, 2004) (Masoumi et al., 2007).

The vibrations generated by impact pile driving die out before the next blow, while the vibrations caused by vibratory pile driving are continuous during the time of driving (Wiss, 1967). According to Ziyazov et al. (1976) the duration of vibrations excited by one blow in impact driving does not exceed 1.5-3 periods and is not enough to infer resonance of buildings and structures. Also, the impact created by impact pile driving is not a single frequency, and only a few cycles of any given frequency occur, so resonance does not develop in the same way as for vibratory driving of piles (Woods, 1997).

Normally impact pile driving is considered to generate transient vibrations. However, for some impact drivers (e.g. double-acting air or diesel hammers) the strokes are so rapid that the vibrations do not fully die away between the blows. This could be considered as a less regular but continuous form of vibration and is sometimes called pseudo-steady-state vibration (Head & Jardine, 1992) (Svinkin, 2004). Sometimes impact pile driving is classified as intermittent vibration as it gives rise to transient vibrations with sufficient time between each blow for the amplitude to decay to an insignificant level.

2.4.1.2 Energy transfer between hammer and pile

Impact pile driving

During impact pile driving, a hammer hits the pile head. Energy is transferred into the pile by the impulse created when the hammer hits the top of the pile (Woods, 1997) (Massarsch & Fellenius, 2008). As the hammer hit the pile a compressional body wave is generated within the pile. The wave travels down the pile to the toe, where part of the wave energy is reflected within the pile and part is transmitted to the soil (Wiss, 1967) (D'Appolonia, 1971) (Head & Jardine, 1992).

Here follows a theoretical approach presented in Massarsch & Fellenius (2008) (also mentioned in Nordal (2009)) assuming no loss of energy. At impact the particle velocity of the pile head is zero, while the velocity of the hammer can be estimated from the drop height according to:

Eq. 2.22 $$v_{H0} = \sqrt{2gh} \qquad \text{(m/s)}$$

Where v_{H0} = velocity of hammer at impact (m/s)
g = acceleration of earth gravity (m/s^2)
h = drop height (m)

As the hammer hits the pile a stress wave is created simultaneously in the pile and in the hammer, see Figure 2.19. The hammer velocity slows down as the pile head accelerates. Since the forces need to be equal the following equation applies:

Eq. 2.23 $$Z_H v_H = Z_P v_P$$

Where $Z_{H,P}$ = impedance of hammer and pile respectively (kNs/m)
$v_{H,P}$ = particle velocity of hammer and pile respectively (m/s)

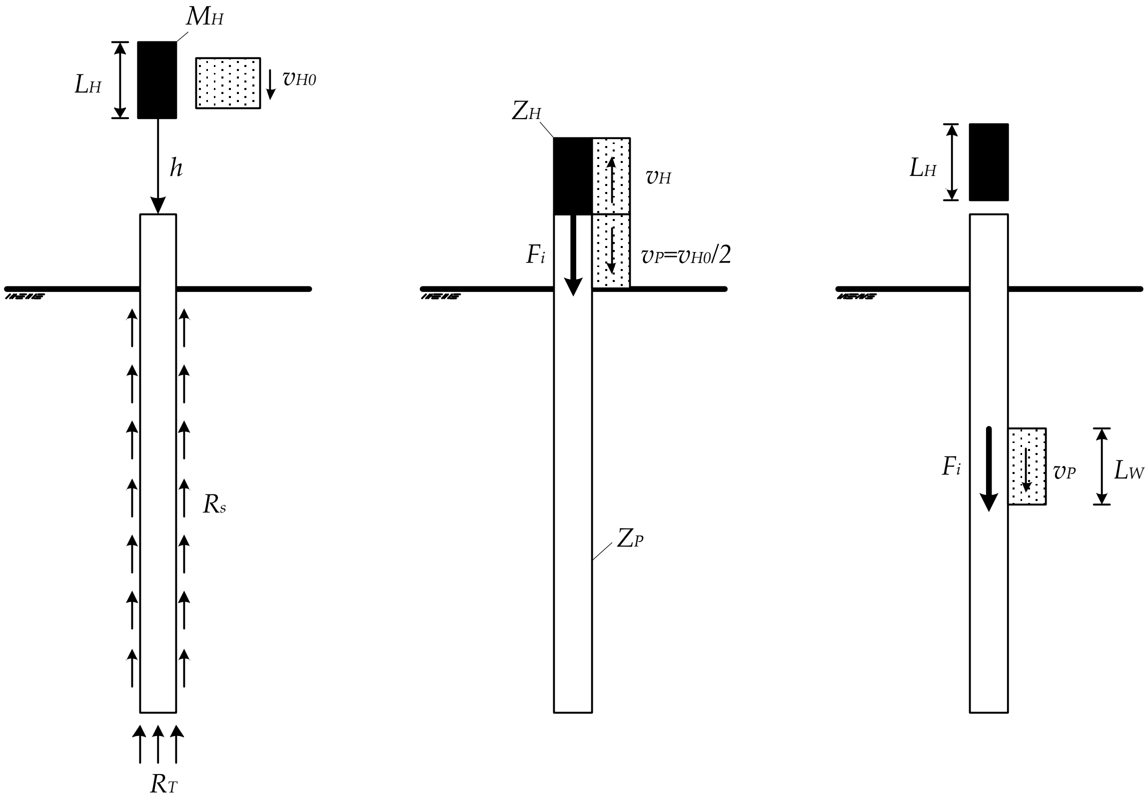

Figure 2.19 Stress wave in pile during impact driving, modified after Massarsch & Fellenius (2008).

At the contact surface the hammer velocity is decreasing while the pile head velocity is increasing, which gives:

Eq. 2.24 $$v_{H0} - v_H = v_P$$ (m/s)

Combining Eq. 2.23 and Eq. 2.24 give:

Eq. 2.25 $$v_P = \frac{v_{H0}}{1 + \frac{Z_P}{Z_H}}$$ (m/s)

As can be seen, the particle velocity in the pile is not affected by the hammer mass (m_H), but only by the hammer drop height and the impedance ratio of the hammer and the pile (Massarsch & Fellenius, 2008).

The duration of the impact of the hammer determines the length of the propagating stress wave. The time, t, during which the hammer and the pile head is in contact is the time it takes for the strain wave to travel from the top of the hammer to the bottom and back up to the top, i.e. $2L_H$, according to (Bodare, 1996) (Massarsch & Fellenius, 2008):

Eq. 2.26 $$t = \frac{2L_H}{c_H}$$ (s)

Where L_H = length of hammer (m)
c_H = velocity of stress wave in hammer (m/s)

Vibratory pile driving

The static moment (also known as torque), M_e, which is of great importance for vibratory driving, is the product of the rotating eccentric masses, m, and the distance to the rotational axle, e, according to (Woods, 1997) (Massarsch, 2000b) (see also Figure 2.14):

Eq. 2.27 $$M_e = \sum me$$ (kgm)

When the eccentrically supported masses rotate at an angular frequency it produces a centrifugal force, F_v. according to (Woods, 1997) (Massarsch, 2000b) (Rausche, 2002):

Eq. 2.28 $$F_V = M_e \omega^2$$ (kN)

Where M_e = static moment
ω = angular frequency

Only vertical components of the centrifugal force are transmitted to the pile since pairs of eccentric masses spin in opposite directions (Rausche, 2002). According to Richart et al. (1970), Whenham (2011) and Whenham & Holeyman (2012) the vertical component of the centrifugal force, $F_v(t)$, is a harmonic function describing a sinusoidal path in time:

Eq. 2.29 $$F_v(t) = M_e \omega^2 \sin(\omega t)$$ (kN)

Another important factor in vibratory pile driving is the displacement amplitude (double amplitude) generated by the rotating mass vibrator (Woods, 1997) (Massarsch, 2000b). The free-hanging double-displacement amplitude is a measure of a free-hanging vibrator and the upward and downward oscillating motion of the pile. The nominal double-displacement amplitude for a free-hanging driver-pile system, S, depends on the static moment, M, and the total vibrating mass, m_{dyn}, according to (Houzé, 1994) (Woods, 1997) (Viking, 2006) (Whenham, 2011):

Eq. 2.30 $$S = \frac{2M_e}{m_{dyn}}$$ (m)

Where m_{dyn} = total vibrating mass (vibrator + clamp + pile)

The real amplitude of the free hanging pile will always be smaller than the specified nominal amplitude since the dynamic mass is increased by that of the pile and there are losses due to soil resistance, for example (Holeyman, 2002) (Viking, 2006). Whenham & Holeyman (2012) actually show that the ratio between measured force in the pile and the nominal axial force is around 0.4-0.6. They observed that the force transferred to the pile is increased as penetration depth, i.e. soil resistance, increases.

2.4.1.3 Vibration in piles

Impact pile driving

The impulse generated by driving gives a longitudinal stress wave in the pile that propagates from the pile head to the pile toe. The pile behaves as an elastic rod through which the longitudinal stress wave passes (Woods, 1997). The stress waves travels at a speed, c_B. The waves depend on the changes of cross-section of the pile and on the interaction of the pile with the surrounding medium at the pile boundaries (that is at the pile head, along the pile shaft and at the pile toe). When the downward wave reaches the pile toe it is reflected upwards and reversed (compression wave turns into a tension wave). On its way back up the pile, the wave again interacts with the shaft friction and reaches the pile head after time t which can be found according to:

Eq. 2.31 $$t = \frac{2L_p}{c_B} \quad \text{(s)}$$

Where L_p = pile length (m)
c_B = wave velocity in the pile (m/s)

The force, P, created by the impulse can be expressed as stress multiplied by area, according to (Woods, 1997) (Nordal, 2009):

Eq. 2.32 $$P = \sigma A_P = \rho c_B v_p A_p = Z_p v_p \quad \text{(kN)}$$

Where v_P = particle velocity at the top of the pile (m/s)
c_B = wave velocity in the pile (m/s)
A_P = cross sectional area of the pile (m^2)
ρ = density of the pile material (kg/m^3)
Z_P = pile impedance (kNs/m)

Vibratory pile driving

During vibratory driving, the whole system of vibrator and pile moves simultaneously up and down with the same displacement amplitude and acceleration (Viking, 2002a). This means that the vibrator-pile system can be assumed to be a rigid body and that the wave propagation in a vibratory driven pile/sheet pile can be neglected (Viking, 2002a). Massarsch (2000b) stated that a steel pile shorter than about 10 m oscillates as a stiff body. Viking (2006) presented a rule of thumb that should be fulfilled in order for the pile to behave as a rigid body. The rule of thumb is that one-fourth of the time period T for the chosen driving frequency, f_d, should be equal to or greater than the time, t, it takes for the stress wave to travel $4L_p$ of the pile:

Eq. 2.33 $$\frac{T}{4} = \frac{1}{4f_d} \geq t = \frac{4L_p}{c_B}$$

Whenham (2011) presented another rule of thumb for determining when the pile would behave as a rigid body:

Eq. 2.34 $$f_d \leq 0.1 f_n = \frac{c_B}{20 L_p}$$

Where f_d = driving frequency (Hz)
f_n = longitudinal natural frequency of a free slender bar (Hz)
c_p = longitudinal wave velocity in the pile (m/s)
L_p = length of the pile (m)

By comparing the above rule of thumb with results from measurements, Whenham (2011) reported that only 8 out of 72 piles/sheet piles fulfil the requirement of a rigid body. Viking (2002a) stated that results from field measurements show that the sheet pile head and toe display the same displacement amplitude and acceleration throughout the driving phase. Lee et al. (2012) concluded that the two sheet piles in their measurements behaved as rigid bodies during driving.

The dynamic force, P, that is transferred to the pile head and then further through the pile is given as a product of the impedance, Z_p, and the vibration velocity in the pile, v_p, just as for impact driven piles, see Eq. 2.32 (Massarsch, 2000b).

2.4.1.4 Pile-soil interaction

Energy transfer

The energy induced at the pile head is principally divided into energy used for penetration of the pile, energy reflected back up the pile and energy transmitted into the soil (Selby, 1991).

When the hammer hits the pile a body wave is created that travels along the pile. When the body wave reaches the pile/soil interface, part of the energy transmits to the soil and part of the energy is reflected (Attewell & Farmer, 1973). According to Attewell & Farmer (1973) the ratio between the energy transmitted to the soil and the energy of the wave reflected back into the pile is approximately 2 to 1 when the body wave passes through a steel-soil interface that is normal to the wave direction. Whenham (2011) reported that up to 50-60% of the energy transferred to the pile from the power pack is dissipated at the pile-soil interface.

The energy transmitted from the pile to the soil principally depends on the hammer and pile properties (Woods, 1997). According to Massarsch & Fellenius (2008) the length of the stress wave governs the transmission efficacy of vibrations from the pile shaft to the surrounding soil. They also show that the vibration transmission efficacy increases with decreasing pile impedance and increases with increasing soil density, hence, the pile material and pile impedance are important aspects of the vibration transmission. According to Whenham (2011), Westerberg et al. (1995) also stressed the importance of pile impedance along with the behaviour of the soil under dynamic loading when looking at pile-soil interaction.

Ground vibrations from impact pile driving have often been reported to be greater in stiff, dense, soils than in loose, soft soils. D'Appolonia (1971), Martin (1980) and also Head & Jardine (1992) explained this from the difference in resistance in different soils. He believed that the soil resistance rules how much energy is used to drive the pile down and how much energy is available to become ground vibrations. In resistive soil the set per blow is low and considerable energy is available for ground vibrations, while in low resistance soils the pile penetrates quickly and a small amount of energy becomes vibrations (Whyley & Sarsby, 1992) (Hiller & Hope, 1998) (Hope & Hiller, 2000). Attewell & Farmer (1973) and Nilsson (1989) on the other hand, explained this considering the partition of energy at the pile-soil interaction. A stiff ground generally has high impedance. Displacement piles are stiffer than the ground, hence, the energy transmission ratio at the pile-soil interface increases as the ground stiffness increases. Hope & Hiller's (2000) measurements cannot fully be explained with acoustic impedance effects. Hence, they suggested that perhaps both mechanisms occur; one part of the hammer energy that is governed by the transmission ratio transmits directly as ground vibrations to the soil from the pile toe. The remaining energy is available to drive the pile; however, some of this energy will cause elastic deformations in the ground.

The largest part of the energy is transmitted to the soil at the pile toe as long as the pile is not predominantly frictional, tapered or stepped. In these cases, more energy is transmitted from the shaft (Head & Jardine, 1992). During driving to refusal, all of the energy is transmitted to vibrations in the soil (Head & Jardine, 1992). The amount of energy that is transferred from the shaft and the toe respectively mainly depends on the soil layers' dynamic properties (Massarsch, 2000b).

<u>Wave generation</u>

Attewell & Farmer (1973), Head & Jardine (1992), Athanasopoulos & Pelekis (2000), Kim & Lee (2000) and Thandavamoorthy (2004) proposed the use of two sources of energy transfer for transmission of ground vibration from pile driving: the pile toe and the pile shaft, see Figure 2.20. At the pile toe, the displacement of soil generates both compressional P-waves and shear S-waves that propagate outward from the tip in a spherical wave form in all directions. The skin resistance of the pile leads to the generation of a conical wave front of vertically polarized body shear waves expanding from the shaft. The angle of the cone is quite shallow since the velocity of the driving impulse travelling down the pile at compression wave velocity is usually 10 times or even greater than the shear wave velocity in the soil. In practice, this means that the wave front emanating from the pile is assumed to be cylindrical, especially for vibratory driving (Woods, 1997).

As the P- and S-waves hit the ground surface some energy is converted into R-waves while some is reflected back into the ground. The R-waves propagate along the ground surface having both vertical and horizontal components of motion (Head & Jardine, 1992) (Athanasopoulos & Pelekis, 2000). The shear waves from the pile toe reach the ground surface at a distance that is approximately equal to the pile depth (Head & Jardine, 1992). Athanasopoulos & Pelekis (2000) present a figure showing the minimum distance from the source to where surface waves are developed due to reflection of body waves, see Figure 2.21. Amick & Gendreau (2000) stated that during pile driving when the source is below the ground surface, Rayleigh waves are formed at a horizontal distance of about a few meters from the pile.

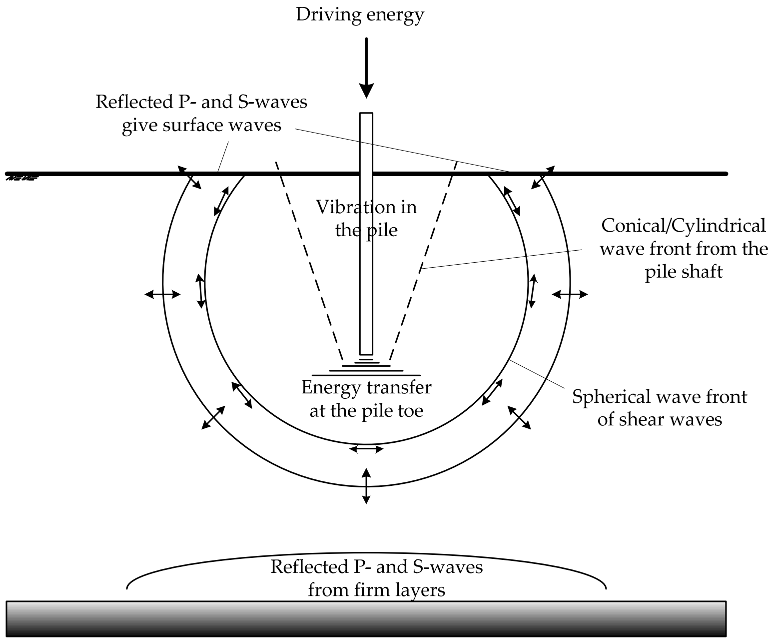

Figure 2.20 Schematic representation of different wave types that can be generated at pile driving, modified after Attewell et al. (1973) and Martin (1980).

The wave generation in Figure 2.20 is based on the assumption that only the elastic deformation of the soil is relevant to the transfer of vibrations. Reflections and refractions from underlying soil layers and the interaction of waves from the toe and the shaft will generate complicated arrays of particle motion (Head & Jardine, 1992).

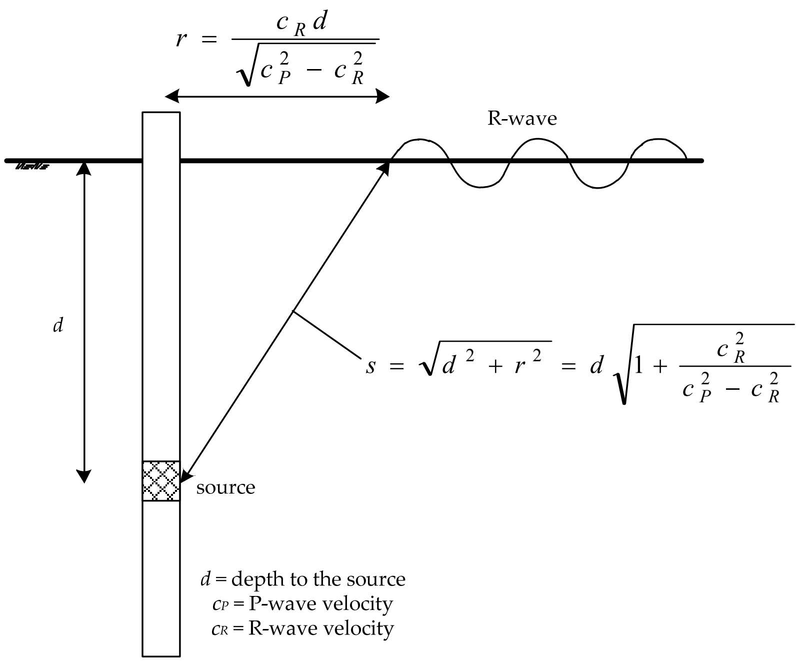

Figure 2.21 Determination of minimum distance from the source to the point on surfaces in which surface waves are generated, modified after Dowding (1996, originally from Daemon et al., 1983).

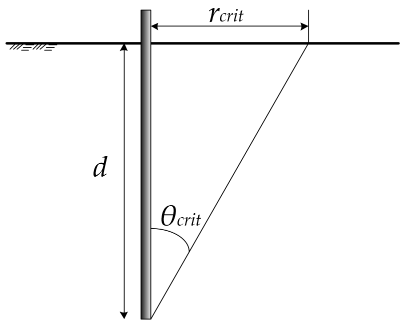

Figure 2.22 Critical distance and critical angle for reflection of surface waves during pile driving, modified after Massarsch & Fellenius (2008).

Massarsch & Fellenius (2008) introduced a distance called critical distance (see Figure 2.22), which is the distance from the pile to where a spherical wave (P-wave) emitted from the pile toe refracts as a surface wave when reaching the ground surface. The critical angle can be determined:

Eq. 2.35 $$\theta_{crit} = \arcsin\left(\frac{c_S}{c_P}\right)$$ (rad)

Where c_S = S-wave velocity (m/s)
c_P = P-wave velocity (m/s)

The critical distance, r_{crit}, from the pile, where wave refraction will occur at the ground surface, can now be determined from:

Eq. 2.36 $$r_{crit} = \tan\theta_{crit}\, d$$ (m)

Where d = pile penetration depth (m)

A table in Massarsch and Fellenius (2008) suggest that the critical distance from the pile is located at a distance approximately half the embedment depth of the pile in dry coarse-grained soil, while the critical distance in loose or soft soils below the groundwater level becomes much shorter and is in the case of clay almost zero.

Vibrations in the soil can also arise if the impact hammer or vibrator causes lateral deformations of the pile (Selby, 1991). According to Massarsch & Fellenius (2008) friction between pile shaft and granular soil can, during pile driving, give rise to a horizontal vibration component. This is important in the case of vibratory driving, while it is usually neglected when it comes to impact driving.

Field measurements during vibratory driving and soil compaction have shown that the vertical oscillation of the pile gives both vertical and horizontal vibrations in the surrounding soil. The horizontal vibration component arises from the friction between the pile shaft and the soil and can be in the range of 30-50% of the vertical vibration (Massarsch, 2000b).

According to Viking (2002b), lateral flexibility in the sheet pile can cause the generation of considerably higher ground vibrations. Laterally induced movement in a sheet pile is claimed by Viking to generally occur due to one or more of the following reasons:

- Sheet pile profiles are driven one at a time with vibrators equipped with a single clamping device holding the sheet pile in the web.
- Vertical alignment is neglected when interlocking a new profile with an already installed profile.
- Bad choice of vibrator equipment and bad equipment operation.
- Clutches in bad condition.

Strain level

According to Kim & Lee (2000) and Masoumi et al. (2006 and 2008) the energy from pile driving is high and causes plastic deformations in the near-field. Further from the pile it has been shown that the vibrations causes deformations within the elastic range. The large strain levels induced in the soil immediately adjacent to the driven pile cause the soil to behave non-linearly and degrade under cyclic loading (Denies & Holeyman, 2008) (Whenham, 2011). Consequently the soil stiffness (and thus wave velocity) decreases, and, especially along the shaft, the soil will be remoulded (Massarsch & Fellenius, 2008).

As discussed in section 2.2.4.1, the shear modulus decreases while material damping increases with cyclic strain (see Figure 2.8 and Figure 2.10). Aboul-ella (1990) confirmed this by stating that the high strains caused by pile driving in the soil adjacent to the piles reduce the dynamic shear modulus and increase the damping in that region. And Masoumi et al. (2009) have shown that a non-linear behaviour for the soil next to the pile leads to smaller levels of vibration.

The shear strain induced by the vibration can be estimated from the following relationship (Massarsch, 2000a):

Eq. 2.37 $$\gamma = \frac{v}{c_s} \qquad (-)$$

Where v = particle velocity (m/s)
c_s = shear wave velocity (m/s)

Svinkin (1996) refers to earlier studies (Svinkin, 1976) where in-situ measurements have been made by dropping a mass on the ground repetitiously in order to study the effects of the plastic deformations on the ground vibrations further from the vibration source. The results showed that despite a large plastic deformation at the point of impact, the ground surface vibrations at a distance of 43 and 57 m from the source did not change considerably as the plastic deformation grew larger.

During dynamic problems the strain levels vary within a large range. At small strain levels (typically $\gamma < 10^{-5}$), rock and soil behave as elastic materials. However, at moderate strain levels (10^{-4}-10^{-2}) most materials display elastic as well as plastic behaviour (Bodare, 1996). Table 2.6 shows soil behaviour for different strain levels.

Table 2.6 Soil behaviour for different strain levels, after Whenham (2011, after Ishihara, 1996).

Shear strain	**10^{-6}**	**10^{-5}**	**10^{-4}**	**10^{-3}**	**10^{-2}**	**10^{-1}**
	Small strain	Medium strain		Large strain		Failure strain
Elastic						
Elasto-plastic						
Failure						
Effect of load repetition						
Effect of loading rate						
Model	Linear elastic		Visco-elastic		Load history tracing type	
Method of response analysis	Linear		Equivalent linear		Step-by-step integration	

<u>Soil resistance</u>

When a stress wave propagates down the pile it encounters resistance. The resistance can be considered either in terms of penetration, via blow count, or driving resistance, via force. It is only the dynamic resistance that gives rise to vibrations emitted from the pile shaft or pile toe to surrounding soil (Massarsch & Fellenius, 2008). According to Waarts & Bielefeld (1994) the soil resistance is the reaction force of the soil on the pile.

The dynamic resistance that arises along the shaft and at the toe of the vibrating pile is mainly affected by four factors (Massarsch, 2000b):

- Vibration frequency (centrifugal force)
- Vibration velocity of the pile
- Displacement amplitude between pile and soil
- Number of vibration cycles

Shaft resistance

As the pile penetrate the soil the shaft resistance increase due to the increasing shaft area in contact with the soil (Whenham, 2011). According to Whenham (2011) the influence of the shaft resistance increases with the content of fine material (clay) in the soil. When driving in clay, pore water pressure build-up and soil remoulding reduces the shear resistance along the pile shaft considerably. On the other hand, when driving in granular soil the displacement of soil around the advancing pile might increase shaft friction (Hope & Hiller, 2000). Van Rompaey et al. (1995) also mentioned the problem of increasing soil resistance during vibratory driving due to compaction of the soil. This is a phenomenon that can be seen in most non-cohesive soils.

The shaft resistance acts in the opposite of motion regardless of whether the pile is moving up or down (Whenham, 2011), see Figure 2.16b.

The dynamic soil resistance at the pile shaft, R_s, can be given according to:

Eq. 2.38 $$R_s = z_s v_p S_p \qquad \text{(kN)}$$

Where z_s = specific soil impedance (kNs/m³)
v_p = particle velocity in the pile (mm/s)
S_p = contact area between shaft and soil (m²)

Toe resistance

The dynamic portion of the driving resistance at the pile toe, R_t, can be given from (Massarsch & Fellenius, 2008):

Eq. 2.39 $$R_t = J_C Z_P v_P \quad \text{(kN)}$$

Where J_C = damping factor (-)
Z_P = pile impedance (kNs/m)
v_P = particle velocity in the pile (m/s)

The toe resistance varies during driving and is zero at times when the pile is not moving downward (Whenham, 2011), see Figure 2.16c. As can be seen the curve does not display a linear relationship but a strain hardening loading and unloading curve (Viking, 2006).

Interlock resistance

In order to create a retaining wall, sheet piles are driven in lock. The friction between the two sheets as they are driven in lock gives rise to an interlock resistance. The condition of the locks and also the verticality of the driven piles affect the size of the interlock resistance. The magnitude of the interlock resistance affects the ground vibrations induced during driving (Whenham, 2011). The interlock friction is mainly due to soil particles in the locks; however, it is also to some extent due to steel to steel friction (Viking, 2006).

Viking (2006) claimed that results have shown that the ground vibrations generated during vibratory driving increased by 2-5 times when interlock friction was present. Lee et al. (2012) performed measurements using strain gauges mounted on two sheet piles (one without interlock friction and one driven in lock) during vibratory driving in sand. The results showed that the peak section forces were greater for the pile driven in lock. They also noticed that the interlock friction was not constant with penetration depth.

Viking (2002b) presented results, from measurement of ground vibration recorded during vibratory driving of sheet piles, showing that when considering the interlocking friction force between two sheet piles the induced ground vibrations are up to 2-5 times higher than when no friction force is considered.

<u>Soil stratification</u>

It is likely to believe that a stiff surface layer on top of a softer layer would indicate that vibrations problems occur during the beginning of driving. It is also expected to believe that vibration problems could occur during seating of the piles into a stiffer bearing layer at the end of driving (Hintze et al, 1997) (Woods, 1997).

Massarsch & Fellenius (2008) showed that during driving through a surface fill and underlying clay layer, a large part of the vibration energy is transmitted along the pile shaft and/or propagates as surface waves. However, when the pile reaches the dense glacial till at larger depth the vibrations measured agree best with those emitted as spherical waves from the pile toe.

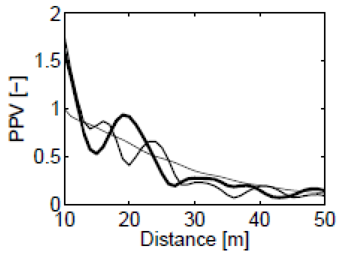

Figure 2.23 Experimental results of dimensionless peak particle velocity plotted against distance from the pile due to vibratory pile driving at a driving frequency of 25 Hz (Masoumi et al., 2006).

Masoumi et al. (2006) modelled the soil behaviour during vibratory driving in order to investigate the influences of the soil inhomogeneity. Their experimental results show that in inhomogeneous soil, diffracted body waves are reflected into the top layer and the shear wave front around the shaft is affected by the reflected and refracted waves.

Masoumi et al. (2006) also noticed that vibration amplitudes attenuate monotonically in homogenous soil. However, in a layered soil model or in a model with increasing stiffness with depth, the attenuation is oscillatory, see Figure 2.23. The oscillation is believed to be due to the interference of the reflected waves on the ground surface. Their results also showed that vibrations attenuate faster in a layered soil than in a homogeneous soil profile and indicated that the higher the operating frequency of the vibrator the more the vibrations are attenuated.

Based on experimental results, Masoumi et al. (2007) showed that when the penetration depth is smaller than the layer thickness, the layering has a relatively small effect on the ground vibrations generated by vibratory pile driving. However, when the penetration depth is greater than the layer thickness, the influence of the layering is large due to the reflection and refraction of waves.

The distance from the vibration source constantly changes during the driving of piles or sheet piles. During penetration into the ground, several vibration sources can exist at the same time, both from the pile toe and along the shaft (Massarsch & Fellenius, 2008).

During the installation of piles Massarsch (2004) identified three common situations that can cause excessive ground vibrations, see Figure 2.24:

a) Pile driving into a stiff surface layer. The energy source is situated at the ground surface and the vibrations will mainly propagate as surface waves.
b) The pile is driven into a medium dense or dense sand deposit. In this case the vibration energy will mainly be dissipated along the shaft of the pile. If the sand is very dense or if the pile hits any obstruction, vibrations may also be emitted in the form of compression waves from the pile base.
c) For example when driving to refusal of end-bearing piles, the pile is pressed hard against a hard material. In this case vibrations will propagate as body waves, mainly in the form of compression waves, towards the ground surface where they are transformed to surface waves.

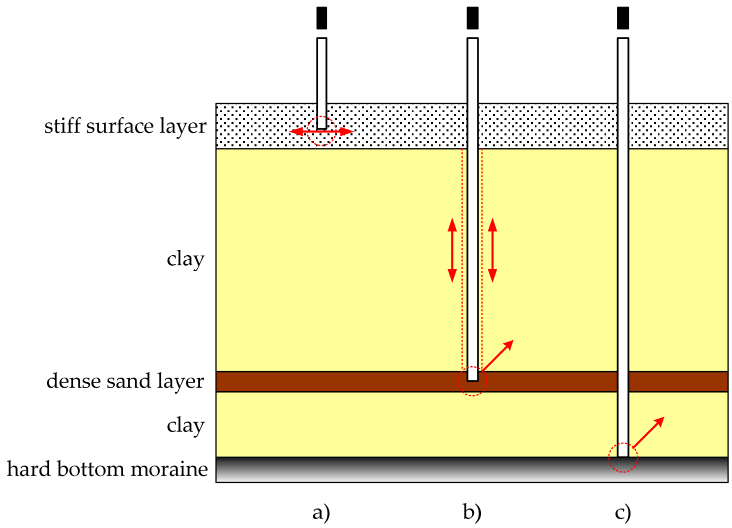

Figure 2.24 Typical sources of ground vibrations during pile driving, modified after Massarsch (2004).

It is difficult to know whether the pile shaft or the pile toe is the source of vibration or whether it is a combination. Hope & Hiller (2000) concluded that vibrations from impact pile driving depends more on the soil encountered at the pile toe than the soil conditions along the shaft.

During penetration through firm layers or at driving to refusal, the main part of the energy is transmitted from the toe in the shape of body waves (P- and S-waves). The interchange between the shaft friction and the toe resistance is complicated and the conditions change when the pile penetrates different soil layers. At the same time, vibrations can be produced along the shaft and from the toe and can contain a large frequency spectrum (Massarsch, 2000b).

2.4.2 Wave propagation in soil

2.4.2.1 Wave type

Miller & Pursey (1955), showed that the distribution of total input energy among the three elastic waves was 67% Rayleigh wave, 26% S-wave and 7% P-wave for a source located at the ground surface. However, Wolf (1994) presented evidence that the partition of energy carried by the different wave types actually is dependent on the vibration frequency. The earlier findings are applicable for very low frequencies so Wolf (1994) stated that for higher vibration frequencies seen in engineering practice, the largest part of the energy is actually carried by P-waves.

A common distinction is to divide ground vibrations due to a vibration source into near-field conditions and far-field conditions. Near-field conditions are considered to be where both body and surface waves are present and energy is dissipated due to plastic deformations in the soil. Further away from the source, in the far-field, vibrations mainly consist of surface waves and the behaviour of the soil is elastic (Massarsch, 2004) (Whenham, 2011). Masoumi & Degrande (2008) presented results from numerical modelling showing that in the near-field vertically polarized shear waves dominate while in the far-field the ground vibration is dominated by Rayleigh waves.

The near-field zone is not very well understood; neither its extent nor the wave propagation within the zone. Generally the near-field zone will be of the order of meters from the driven pile (Head & Jardine, 1992).

According to Gutowski & Dym (1976) the vertical vibration component is in almost all cases much larger than the horizontal radial and transverse components when the vibration source is pile driving. Head & Jardine (1992) and Svinkin (1996 and 2004) wrote that near a source inducing vertical vibrations, the ground vibrations are highly vertical; however, as the distance from the source increases, a horizontal component is rapidly generated. Athanasopoulos & Pelekis (2000) reported the opposite from their field study – as the distance from the source increases the motion becomes predominantly vertical as the horizontal component is reduced.

Since surface waves attenuate slower than body waves the vibration at large distances from the source is likely to be dominated by R- and L-waves (Martin, 1980). However, according to Head & Jardine (1992), it is not clear if true Rayleigh waves actually develop over the short distances that are generally dealt with when concerning problems of vibrations due to pile driving. They suspect that partly developed waves are instead generated at the ground surface. Attewell & Farmer (1973) also discuss that true Rayleigh waves are probably not developed until at a certain distance from the pile depending on source depth and wavelength. Wolf (1994) claimed that Rayleigh waves do not fully develop until a distance of half the Rayleigh wavelength from the source.

Dong et al. (2000) showed, using snapshots, that vibrations result in a complicated deformation pattern in the ground, due to the reflection and refraction of P- and S-waves as well as the surface waves. Sometimes a peak of vibration velocity is seen at a distance of about 10 m from the pile. This is caused by the overlap of surface waves from the pile movement at the surface with waves coming from the pile toe (Head & Jardine, 1992). Attewell et al., (1991) also reported that as a result of superposition of surface waves, caused by lateral movements of the pile, as well as waves from the pile toe, a maximum vibration level can be observed at a distance of approximately 10 m from the source.

2.4.2.2 Duration

Due to the dispersion of Rayleigh waves in an inhomogeneous soil, it is often observed that the soil response of the vibration from impact driving gets a longer duration as the distance from the soil increases (Svinkin, 2008). The dispersion gives rise to components of different frequencies travelling at different depths and thus with different velocities, resulting in the vibration records shown in Figure 2.25 (Auersch, 2010a) (Auersch & Said, 2010). According to Svinkin (2008), this is especially noticeable in saturated soils and in areas in which the soil is underlain by rock.

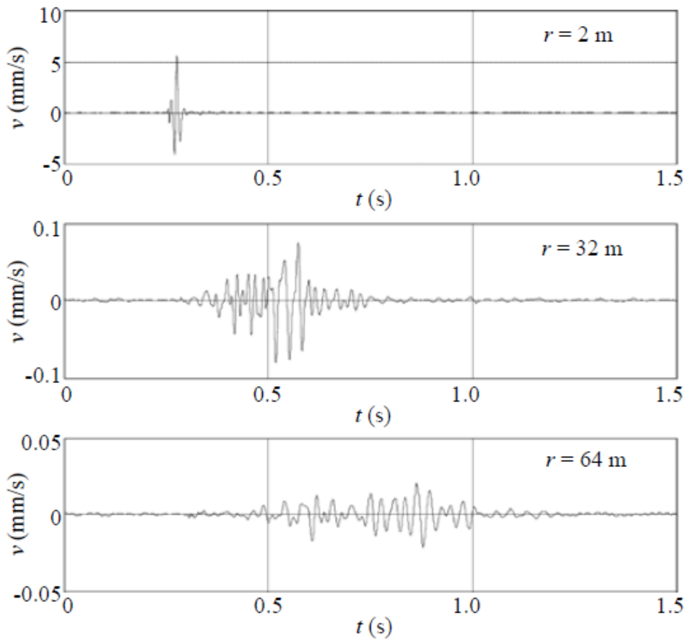

Figure 2.25 Time records of the impulse response, showing increasing duration by increasing distance from the source (Auersch & Said, 2010).

2.4.2.3 Influence of ground conditions

The ground conditions are highly important for the propagation of vibrations through the soil. As has been stated before, stiffer and denser soils transmit vibrations more readily than more compressible materials. Therefore the presence of any harder layers in the soil profile enables vibrations to transmit more easily, potentially resulting in higher vibration levels. It could also be that the pile driving itself alters the soil stiffness (Head & Jardine, 1992). Heckman & Hagerty (1978) also stated that hard objects or stiff layers in the ground may lead to the vibrations being transmitted over greater distances. However, Auersch & Said (2010) wrote that generally soft soils display larger vibration amplitudes in the near-field than stiffer soils.

As discussed in section 2.2.4.2 the groundwater table affects the wave propagation in soil. Wave propagation in soil partly takes place through the soil skeleton and partly through the liquid in the pores. P-waves can propagate through the liquid as well as through the particles. S-waves on the other hand can only propagate through the soil skeleton since water cannot transmit shear stress. R-waves propagate both in the soil skeleton and in the pore liquid (Hintze et al., 1997).

The results presented by Wiss (1967) showed a difference in vibration transmission between cohesive and non-cohesive soils and between wet and dry sands. For train vibrations it has been seen that low frequency oscillation (<10 Hz) is characteristic for cohesive soils, while oscillations with higher frequencies are characteristic for non-cohesive soils (Möller et al., 2000).

A highly plastic soil is linearly elastic to greater strains than other types of soils. As a result the damping factor is smaller and the problems of vibrations increase (for example the case of vibrations due to a rock concert in Gothenburg, Sweden) (Erlingsson & Bodare, 1992 and 1996) (Madheswaran et al., 2005).

2.4.2.4 Frequency content

From a vibration source, waves travel in all directions as fairly harmonic waves. Ground vibrations from vibratory pile driving generally coincide with or are close to the frequency of the vibrator. Vibratory driving generates steady-state vibrations, which force the soil particles to vibrate with the frequency of the driver, disregarding the natural frequency of the soil. Impact pile driving on the other hand excites the preferred frequencies of the ground and the vibration frequency is governed by the ground instead of the driver (Wiss, 1967) (D'Appolonia, 1971) (Head & Jardine, 1992) (Svinkin, 1996).

Damping properties in the soil naturally filter out high frequencies that might be induced by the driving process (Whenham, 2011). According to Martin (1980) soils can act as low-pass filters due to the frequency-dependent internal damping. For example, peaty and silty soils have preferred frequencies in the range of 5-10 Hz, while clays have preferred frequencies between 15-25 Hz. This is also proved by measurements performed during impact driving of sheet piles. Masoumi et al. (2007) observed that as the distance from the pile increases, the variety of frequencies in the ground vibrations decrease. This phenomenon is due to material damping reducing the variety of frequencies in the vibrations in the far-field.

Svinkin (1996) wrote that depending on the soil type, the frequency of the vertical ground vibrations may either increase or decrease with the distance from the vibration source. According to Ziyazov et al. (1976) the frequency of the ground vibration due to impact pile driving is independent of the distance to the source; instead it depends on the soil resistance to static probing, *R*. They proposed the following relationship for cohesive soils with R = 460-2400 kN/m^2:

Eq. 2.40 $$f = 0.00463R + 8 \quad \text{(Hz)}$$

The relationship is also illustrated in Figure 2.26.

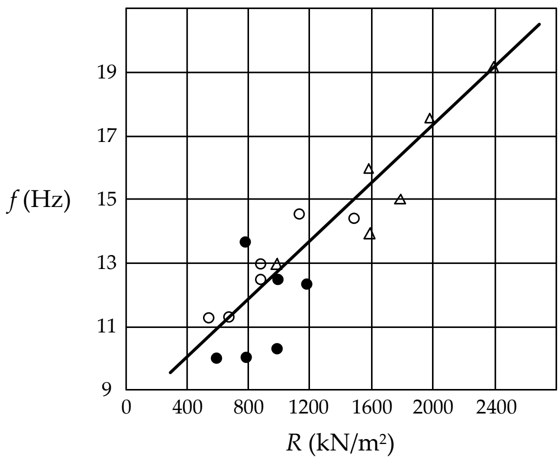

Figure 2.26 Variation of frequency of ground vibrations as a function of end resistance of static probing, R (Ziyazov et al., 1976).

Resonance

For a specific frequency and wave speed, the body height becomes the same as the wavelength, and a standing wave is produced. This happens at the body's natural frequency, hence when the vibration frequency and the natural frequency coincide resonance occurs (Möller et al., 2000). Soil and rocks do not have a natural frequency as such; however frequencies at which they transmit vibrations more readily can be observed. In Table 2.7 some general characteristic "natural" frequencies are given for different kinds of soils and rocks.

Table 2.7 Typical natural frequencies for different types of soil from Wiss (1967), Head & Jardine (1992) and Niederwanger (1999).

Soil type	Typical "natural" frequency (Hz)
Very soft silt and clay	5-20
Peat	10-13
Clay	10-25
Sand and Gravel	30-40
Weak rock	30-80
Strong rock	> 50

There are multiple natural frequencies for all systems; however, usually only the lowest of these are of technical interest. The lowest natural frequency, f_n, can, for a homogenous soil layer, be estimated as (Erlingsson, 1999):

Eq. 2.41 $$f_n = \frac{c_s}{4H} \quad \text{(Hz)}$$

Where c_s = shear wave velocity (m/s)
H = height of soil layer (m)

According to Bodare (1996) a material does not in itself have a natural frequency. The natural frequency originates from interaction between the material and a cavity. The natural frequency is inversely proportional to the time it takes for an S-wave to travel a cavity radius. Therefore small cavities give high natural frequencies and large cavities give low natural frequencies.

2.4.2.5 Soil response during vibration

In Figure 2.27 a typical soil response to uniform cyclic loading is presented; a hysteresis loop (Holeyman & Legrand, 1994). A hysteresis loop is a diagram showing the cyclic shear strain, γ, and the shear stress, τ, in a closed curve. The size of the area within the curve represents the energy density lost in every cycle; the dimension is energy per volume unit (energy density) (Bodare, 1996).

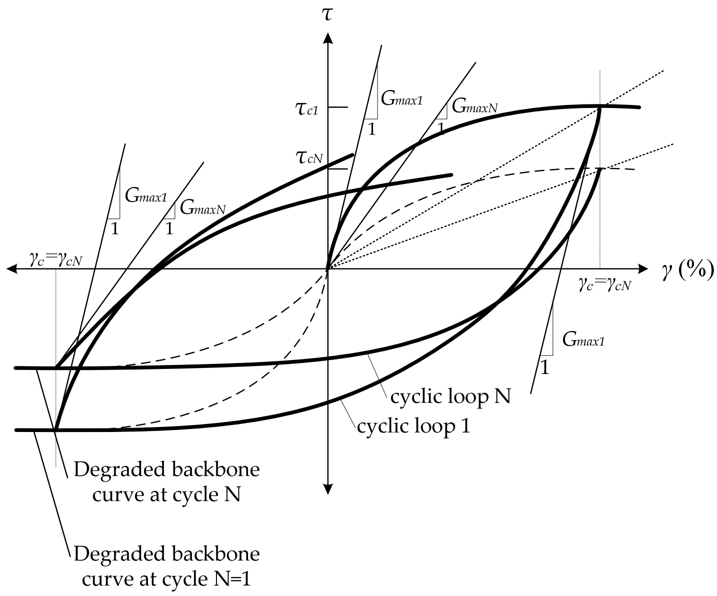

Figure 2.27 Soil behaviour under constant cyclic shear strain loading, modified after Holeyman (2002) (originally from Vucetic (1993)).

The hysteresis loop in Figure 2.27 represents what a typical soil subjected to symmetric cyclic loading with amplitude γ_c might exhibit. The inclination of the loop depends on the stiffness of the soil (Kramer, 1996) (Whenham, 2011). From the response the following fundamental parameters can be derived:

- G_{max} = initial (or tangent) shear modulus
- τ_c = shear stress mobilized at γ_c
- G_s = secant (or equivalent) shear modulus

G_s is strain dependent and needs to be described by specific laws within a given cycle. τ_{max} is the ultimate shear strength that is revealed at large strains. Both τ_{max} and G_{max} have been shown to decrease with the number of cycles (called cyclic degradation) (Holeyman, 2002). Cyclic degradation may lead to the soil losing its shear resistance almost completely, i.e. full liquefaction (Whenham, 2011).
The energy dissipated within a loop depends on the amplitude of the cyclic strain (Holeyman, 2002). From the energy lost during a given cycle, ΔW_s, the internal damping or the hysteretic damping can be represented by (Whenham, 2011):

Eq. 2.42 $$\xi = \frac{\Delta W_s}{2\pi \gamma_c \tau_c} \qquad (-)$$

The stress-strain relationship and the degradation law depend to a large extent on the soil type. Cohesive soils are less susceptible to cyclic degradation than non-cohesive soil as the particles are more tightly connected to each other. In non-cohesive soils the particles are able to rearrange and lose contact during vibrations (Whenham, 2011).

2.4.2.6 Reflection and refraction

In urban environments the soil and ground conditions are usually far from homogenous. The soil is overlain by hard layers (asphalt, paving stone etc.) and the ground is filled with pipes, tunnels and underground structures. This complicates the wave propagation considerably, resulting in multiple reflections and wave interference (Head & Jardine, 1992) (Whenham, 2011).

There are no simple theoretical solutions for wave propagation in layered soils (Waarts & Bielefeld, 1994). Reflected and refracted waves can have higher velocity than the incident wave. The direction and amplitude of reflected and refracted waves depend on the angle of incidence and the ratio of densities (velocities) of each material (Head & Jardine, 1992) (Massarsch & Fellenius, 2008). Two reflected and two refracted waves will be generated for each of the original waves every time a P- or S-wave encounters a boundary between soils with different properties. From Figure 2.28 this can be seen when two incident waves result in 8 new waves (Woods, 1997). This has earlier been described in section 2.2.2.3. As direct and reflected waves will have different path lengths a phase difference will be noticed between the direct and the reflected wave (Auersch, 2010c).

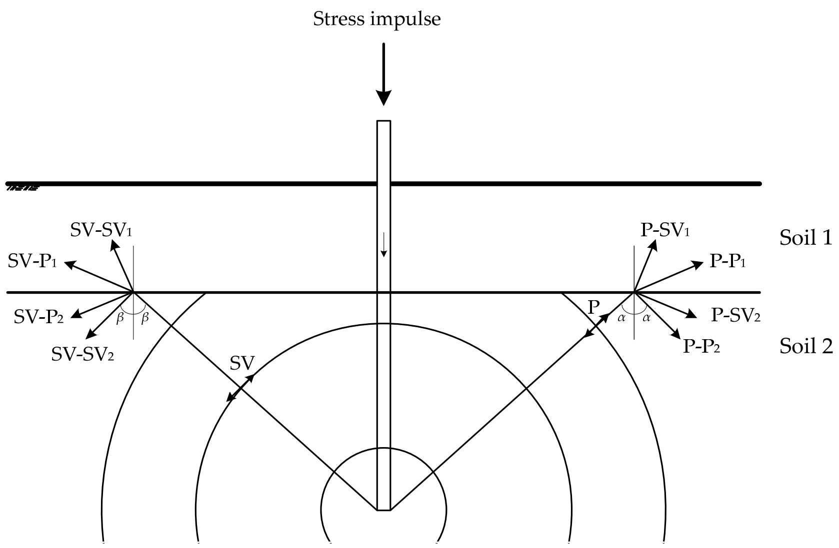

Figure 2.28 Partition of waves at soil layer boundary, modified after Woods (1997),

2.4.2.7 Vibration attenuation and damping

General vibration attenuation has earlier been described in section 2.2.3. In this section focus lies solely on attenuation of vibrations due to pile driving.

Attewell & Farmer (1973) stated that the attenuation due to material damping is small compared to the loss in geometrical damping, and according to Möller et al. (2000) material damping can be neglected when predicting vibrations from piling. Richart et al. (1970), on the other hand, stated that material damping is important even for small values of the material damping coefficient, α. In Figure 2.29 the importance of material damping, especially as the distance from the pile increase, is illustrated.

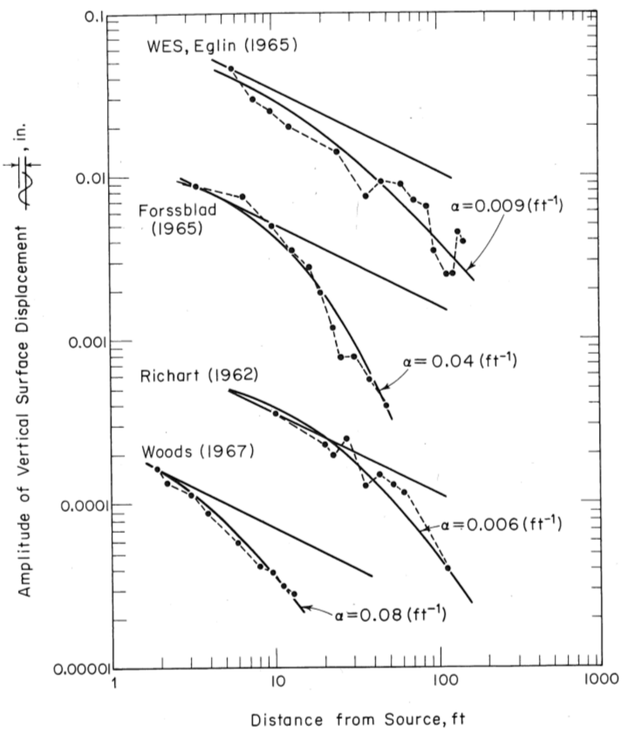

Figure 2.29 Attenuation of surface wave with distance from source of steady-state excitation (Richart et al., 1970).

Auersch (2010a) showed that the attenuation of ground vibrations depends on the damping of the soils, the frequency content of the source and geometry of the source (i.e. point load or line load). However, Athanasopoulos & Pelekis (2000) showed that the attenuation of ground vibrations did not differ considerably between five different cases with different soil conditions.

According to Auersch (2010b) a high material damping can be seen in near-surface soil because of the lack of confining pressure.

Vibrations from impact pile driving usually have a high frequency as they enter the ground at the pile-soil interface. High frequency vibrations are attenuated faster than low frequency vibrations. This implies that ground vibrations from impact pile driving are attenuated faster than ground vibrations from vibratory pile driving, which instead tend to give a standing wave and a fairly constant frequency spectrum over distance (Attewell et al., 1992b).

Gutowski & Dym (1976) argued that it is possible that ground damping is non-linear close to the vibration source where large vibration amplitudes are experienced. They state that there is more attenuation per wavelength in the non-linear zone. However, they also argue that it might be so that the damping is linear, but at large distances soil inhomogeneity results in reflection or scattering of waves to the surface.

One exception to the “normal” attenuation of vibrations in soils is brought up by Athanasopoulos & Pelekis (2000). They state that measurements in urban areas sometimes can give higher vibration levels for points slightly further from the source than another point closer to the source. This is due to the complex underground conditions in urban areas that often are far from homogenous and contain underground structures of various kinds, leading to complicated reflection patterns and interference of waves. Selby (1991) also presented results showing that the attenuation with distance from the pile is far from monotonic. In several cases, peak values are observed at a distance of about 10 m from the pile, as was also discussed in section 2.4.2.1.

The difficulty of using the wave attenuation relationship in Eq. 2.13 in practice has been to decide what value to use for α.

2.5 ENVIRONMENTAL IMPACT DUE TO VIBRATIONS FROM PILE DRIVING

Unavoidably, vibrations have an impact on the surrounding environment. This section presents the impact of vibrations generated by pile and sheet piles driving on soil, buildings and structures, and humans.

According to Woods (1997) there are three elements that must be present for potential development of problems due to pile driving:

1) Sensitive targets or receivers of vibration – can be any person or object that may be sensitive to vibrations.
2) Media through which the vibrations are transmitted
3) Source of vibrations

2.5.1 Impact on soil

Vibrations produced by pile driving have an impact on the soil through which they are transmitted (Heckman & Hagerty, 1978) (Madheswaran et. al., 2005). The effect of vibrations on soil and rock can be summarized as follows (Thurner, 1976) (Holmberg et al., 1984) (Hintze et al., 1997):

- Cohesive soil – Vibrations can reduce shear strength
- Non-cohesive soil – Vibrations can give settlements, liquefaction or even slides

Effects of vibration on non-cohesive soil are generally much more significant than the effects of the same vibration on cohesive soils.

A soil subjected to a continuous vibration will experience cyclic degradation as the number of cycles increase. For a soil subjected to transient vibrations, the response is more unknown. There is little information on the effects of repeated transient vibrations, with a peak stress occurring only once or twice for each blow with the remaining cycles being of lower magnitude (Wiss, 1981).

2.5.1.1 Settlement and heave in non-cohesive soil

The settlements from densification of the soil due to pile driving may in many cases have a much more detrimental effect upon adjacent structures than vibrations transmitted directly to the structure (Heckman & Hagerty, 1978). According to Massarsch (2000a) a high shear wave velocity and plasticity index decrease the risk for settlements and the size of the settlement is dependent upon the number of vibration cycles.

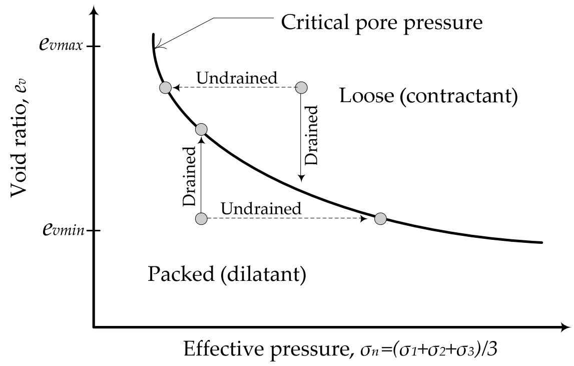

Figure 2.30 Compaction of soil concerning relative density and critical pore pressure, modified after Viking (2002a).

Under the strain of cyclic loading, loose sands tend to densify while dense sands dilate. According to Kramer (1996), Massarsch (2000a) and Viking (2002a) there exists a critical density or critical void ratio which the sand approaches when subjected to cyclic loading. Disregarding the material's initial relative density, loose or dense, the relative density closest to the pile/sheet pile will always change towards the critical relative density, see Figure 2.30.

According to Woods (1997) it is believed that vibrations from pile driving that cause settlement are likely to contain many cycles of low-amplitude shearing strains. Massarsch (2000a and 2004) agreed, stating that fundamental concepts and earlier published data show that the shear strain is the primary factor causing compaction of granular material and that compaction increases with shear strain amplitude.

The threshold strain, γ_t, is defined as the value of cyclic shear strain such that the cyclic shear strains less than γ_t will not cause any densification of dry granular soils or any pore pressure build-up in water-saturated soil (Massarsch, 2004). According to Massarsch (2000a and 2004) the risk for ground settlement or strength loss is very low if the shear strain level does not exceed 0.001%. Should the shear strain level caused by ground vibration exceed 0.1%, the risk for settlement or loss of strength is significant in cohesive soils.

Hintze et al. (1997) stated instead that compaction due to vibration does not happen until the acceleration exceeds a certain threshold value, or a critical acceleration. This basically means that a material that has been compacted beforehand to a certain void ratio, e_v, will not compact more until it experiences vibrations in which the acceleration is greater than what it has earlier experienced. Bement & Selby (1997) showed that loose granular saturated soils may compact during prolonged vibration if the particle acceleration exceeds 0.2-0.4g, but are limited to a depth of 10 m below the ground surface. They also showed that compaction is unlikely to occur at more than 5 m from the pile unless widespread liquefaction occurs.

Clough & Chameau (1980) showed that there is a correlation between strain (estimated as the amount of settlement at a point divided by the height of fill material beneath that point) and acceleration, the field data also show that as long as the accelerations are less than 0.1g the strains do not exceed 0.3%.

The size of the settlement depends on several factors, including soil type and stratification, groundwater conditions (degree of saturation), pile type and driving method (Massarsch, 2004). Hintze et al. (1997) stated that the size of settlements is mostly dependent on the magnitude of the vibration amplitude and the density of the soil. Bement & Selby (1995) showed from laboratory tests that a well graded soil with a high coefficient of uniformity experiences larger settlements from ground vibrations than more uniform soils. They also show that dry and saturated soils experience larger settlements than partially saturated soils. According to Heckman & Hagerty (1978), dry, loose to medium-dense sands, and saturated, loose to medium-dense sands are most susceptible to densification, while partially saturated or moist sands are less susceptible to densification. This can also be seen in discussions regarding optimal water content in soils for compaction.

It has been shown that already at a vibration measurement of 2.5-5 mm/s (on the ground surface) the pile driving gives a compaction of loose to medium-loose non-cohesive soil (Hintze et al., 1997). Clough & Chameau (1980) measured settlements during vibratory driving of sheet piles in mainly non-cohesive soil. Very close to the piles, the measured settlements were as high as 127 mm. However, settlement decreased rapidly with distance and was basically zero at 12 m from the piles. The difference in settlements between different locations was explained by the differences in soil density in the fills; lower densities coincided with larger settlements. Heckman & Hagerty (1978) proposed the use of Dutch Cone Penetration Tests and Standard Penetration Tests or equivalent for identifying zones in the ground with looser material that may densify during vibration.

2.5.1.2 Pore water pressure build-up

Pile driving can cause great pore water pressures (Hintze et al., 1997). The pore water pressure in less permeable soils does not have time to decrease before the next vibration giving a gradual increase of the pressure resulting in a decrease of the effective stress in the soil. The reduced effective stress in turns leads to reduced strength (D'Appolonia, 1971) (Nilsson, 1989) (Hintze et al., 1997) (Möller et al., 2000). When piles are driven into clay deposits containing layers of permeable material (saturated sand or silt) there is a risk that the excess pore water pressure will reduce the shear strength of the granular layers. This phenomenon has been observed to cause stability problems and slope failures (Massarsch, 2004).

Research done in Bangkok city by Muktabhant & Sasisuwun (1975), cited by Brenner & Chittikuladilok (1975), stated that excess pore pressure induced by pile driving was observed within a zone of about eight pile diameters.

Holeyman (2002) showed that the excess pore pressure from cyclic loading increases with shear strain and number of cycles.

Pile driving in loose, saturated sands or silts can generate high pore water pressures. The high pore water pressure can reduce the stability of slopes and excavations. Liquefaction is defined as a, often drastic, strength reduction of the soil, making it unable to support structures or remain stable. Liquefaction only occurs in saturated soils, and as such is most common near rivers, bays, and other bodies of water (Kramer, 1996).

2.5.2 Impact on buildings and structures

The correlation between vibrations and building damage is complicated for many reasons. One reason is that buildings are constructed in so many different ways, with different dimensions, building materials, construction methods, foundation types and executions (IVA, 1983).

Thandavamoorthy (2004) stated that a rule of thumb in pile driving practice is that structures within one pile length from the driven pile can be damaged due to vibrations.

2.5.2.1 Damage on buildings and structures

Building damage due to vibration can range from structural damage, such as major failures in the building structure, to architectural damage such as cracking of plaster (Martin, 1980).

Generally damage on buildings and structures can be divided into (see e.g. Brenner & Chittikuladilok (1975) and Head & Jardine (1992)):

- Architectural damage – damage to the appearance of surface finishes and fittings
- Serviceability damage – damage to the function of the building
- Structural damage – damage to structural parts of the building that potentially leads to failure or collapse
- Damage to building content

Structural damage to buildings often starts with the development of cracks in the structure. Other evidence of structural damage due to vibrations could be broken or cracked windows, building distortion due to settlement, or water leaking into a basement or out of a sewer or other conduit (Woods, 1997).

By many investigators, a peak particle velocity of 50 mm/s is considered a safe limit with respect to structures (Madheswaran et. al., 2005). The summary of vibration levels resulting in damage compiled by Head & Jardine (1992) indicated that most publications have placed the threshold for major damage between 50 and 100 mm/s. It is extremely unlikely that damage would occur at peak particle velocities measured at the foundation smaller than 2 mm/s. Particular care should be taken to old and historical buildings, as they are usually more sensitive to vibrations and also more costly to restore if damage were to occur.

Building contents, such as blinds and pictures, would begin to visibly move at 0.5 mm/s. Rattling of windows, crockery or loose objects would be audible and annoying at 0.9 mm/s (SA Government, 2007).

Many different types of equipment are highly sensitive to vibrations. For example computer systems and optical equipment (electron microscopes) function poorly and might even be damaged when subjected to vibrations (Head & Jardine, 1992). When assessing the risk for damage on sensitive equipment, Head & Jardine (1992) recommended that the manufacturer or operator is contacted for acceptable vibration levels. For very sensitive equipment (for example electron microscopes), vibration amplitudes as small as $24 \cdot 10^{-6}$ mm can be damaging (Woods, 1997). As a reference to vibrations generated by other activities, Table 2.8 is included.

Table 2.8 Example of vibrations in buildings under normal conditions (Stille & Hall, 1995 from New, 1990).

Vibration source	Resulting *PPV* (mm/s)		
	Modern steel frame office	**Modern masonry dwelling**	**Old dwelling (thick, lime mortar masonry)**
Normal walking	0.02-0.2	0.05-0.5	0.02-0.03
Foot stamping	0.2-0.5	0.3-3.0	0.15-0.7
Slamming doors	10-15	11-17	3-9
Percussive drilling	5-25	10-20	10-15

2.5.2.2 Damage mechanisms

Much of the building damages that results from construction work is said to be due to vibration. However, it is in fact caused by one of the mechanisms described below.

Settlement and heave

An indirect cause of building damage due to pile driving vibrations is settlement induced in especially non-cohesive soils. Settlement, particularly uneven settlement under a building, can cause extensive damage. Settlement is said to be seen at distances up to 10 pile diameters or even up to 10-15 m from the driven pile (Head & Jardine, 1992) (Hintze et al., 1997). According to Woods (1997), settlement damage to structures has been reported to occur up to 400 m from the pile driving site.

Ground distortion

Waves propagating along the ground surface result in an undulation of the ground surface down to a depth of approximately one wave length (Hintze et al., 1997) (Massarsch, 2004).

The wavelength, λ, is of great importance for the influence the ground distortion has on a building. The wavelength is in many cases somewhere between 10-60 m (Wiss, 1967) (Holmberg et. al., 1984). If the wavelength of the vibrations is considerably longer than the building length, the building is lifted and sunk down, see Figure 2.31a. If the building length and wavelength are approximately the same, the whole building can experience flexural (bending) stress, see Figure 2.31b. Should the building length greatly exceed the wavelength, the building's absolute location will not change, but parts of the building may experience flexural stress or parts may experience high acceleration forces, see Figure 2.31c (Thurner, 1976). If the foundation of the structure in Figure 2.31c is rigid, the wave in the ground will cause areas to experience negative reactions from the elastic soil base, which might lead to changes in the contact conditions between the structure and the elastic soil base (Svinkin, 2008).

According to Massarsch (2000a) there is a risk for building damage when the wavelength of the propagating wave is shorter than the building length. This could be the case for soft clays or silts below groundwater level, in which wave propagation velocity is low.

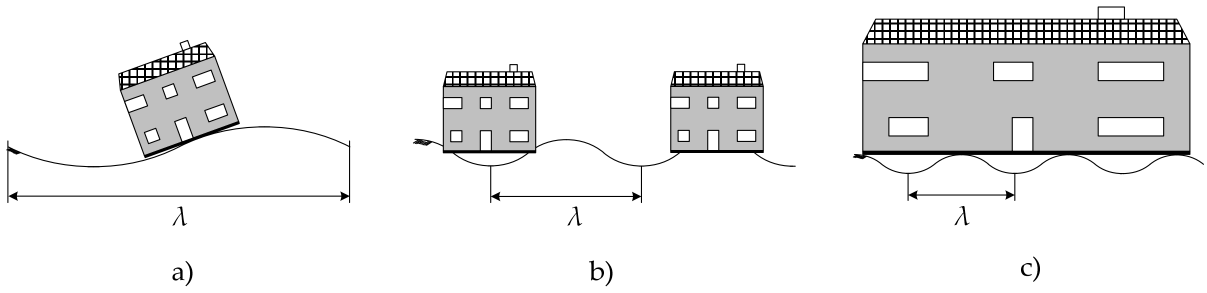

Figure 2.31 The wavelength's significance on the impact, modified after Thurner (1976).

<u>Direct vibration</u>

Direct vibration is when the damage is a direct result of the vibration velocity or the acceleration and the vibration frequency, see Figure 2.32 (Massarsch, 2004). For example dynamic amplification factors in the frame or in building parts can lead to structural or architectural damage (Hintze et. al., 1997).

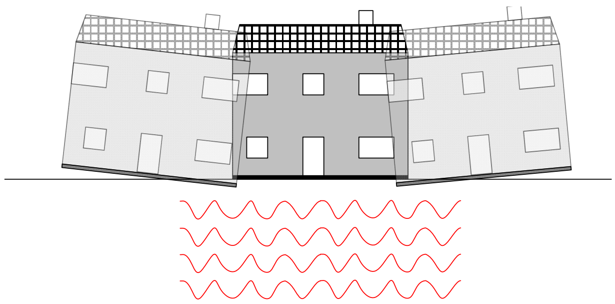

Figure 2.32 Direct vibrations, modified after Massarsch (2004).

According to Atlas Copco ABEM (1973), the Building Research Station in England (1970) showed from numerous investigations of ground and structural vibrations that there were no cases in which observed building damage was proven to be caused by the effects of vibration alone. Athanasopoulos & Pelekis (2000) and Svinkin (2005) said that the effect of direct vibrations on structures can be limited to a distance of one pile length from the driven pile. However, for structures susceptible to vibrations this distance can be considerably larger.

2.5.2.3 Damaging factors

From the findings in literature (Heckman & Hagerty (1978), Martin (1980), IVA (1983), Head & Jardine (1992), Stille & Hall (1995), Hintze et al. (1997), Svinkin (2005)), it is concluded that the following factors affect the vibration impact on a building:

- Frequency of vibration
- Magnitude of vibration
- Stiffness of building and building elements
- Damping characteristics of the building
- Type of construction
- Type of foundation
- Duration of vibration

- Wave form
- Condition of the building, e.g. initial static stress

It has been shown that damage to buildings can be correlated with the peak particle velocity (Martin, 1980). It is not the vibration velocities themselves that cause structural damage or human disturbance. When it comes to building damage, it is the resulting dynamic strains that are of concern (Rockhill et al., 2003).

When the effects of ground vibrations on buildings are considered, the range of frequencies in the ground vibration is of great importance. If the incoming vibration has a frequency at or near the structures' natural frequency, resonance occurs and the vibration at ground level is magnified in the structure. At resonance, the vibrations are amplified and damage is quite likely to occur (Wiss, 1967) (Heckman & Hagerty, 1978) (Stille & Hall, 1995) (Thandavamoorthy, 2004). Svinkin (2005) stated that resonant structural vibrations can be triggered at distances up to a few hundred meters from the driven pile. The natural frequency of a multi-storey building is often approximated by $f_n = 10/N$, where N is the number of stories (Head & Jardine, 1992) (Stille & Hall, 1995). Residential structures usually have a natural frequency between 4-10 Hz. As a result, the criteria for low-frequency vibrations have been set at a lower particle velocity than for high-frequency vibrations (Wiss, 1981).

Alpan & Meidav (1963) stated that acceleration alone is not a satisfactory criterion for susceptibility to damage. In earthquake areas it was observed that accelerations of 0.1g caused damage if associated with low frequencies, while accelerations of 1g or 2g were safe at high frequencies. From Figure 2.33 it can be seen that low-frequency vibrations require lower tolerances than high-frequency vibrations (Woods, 1997).

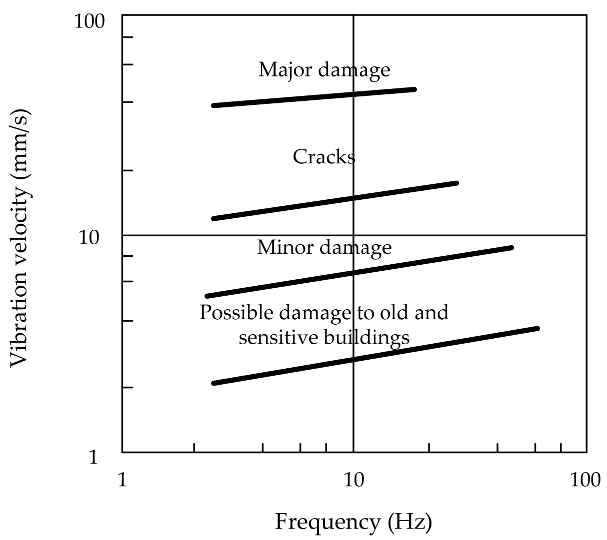

Figure 2.33 Relationship between peak particle velocity, frequency and possibility of damage is shown, modified after Möller et al. (2000).

Table 2.9 Excitation frequency in relation to natural frequency, vibration mode, dynamic stress and physical measure (Niederwanger, 1999).

Frequency of excitation (Hz)	0-5	5-10	10-60	>60
Building excited by	Machines			
	Earthquakes	Traffic, Vibratory hammers	Vibratory hammers, Blasting	Blasting
Natural frequency of	Whole building		Walls, Vertical vibrations of ceilings	Walls, Ceilings
	High rise structures	Low buildings		
Mode of vibration	Bending- and shear vibrations of the whole building	Combination of both	Bending- and strain vibrations of walls and ceiling	
Dynamic stress	Inertia forces	Combination of both	Stress caused by bending and strain	
Significant physical measure	Acceleration	Combination of both	Velocity of vibration	

Niederwanger (1999) presented an interesting table regarding excitation frequency and its relation to natural frequencies, vibration mode, dynamic stress and significant physical measure, see Table 2.9.

Kramer (1996) as well as Niederwanger (1999) stressed the factor of duration as an important parameter for building damage. A motion with short duration might not produce enough load reversals for damaging response to build up in a structure, even if the amplitude would be high. It could also be that a motion with moderate amplitude but long duration is able to produce enough load reversals to cause damage.

The importance of frequency in regard to resonance is much more accentuated when dealing with vibrations from vibratory driven piles than from impact driven piles. During impact driving the duration of the vibration is short (0.2-0.3 s) and resonance build-up of structural components is unlikely (Wiss, 1967) (Svinkin, 2005). According to Erlingsson & Bodare (1996) it takes about 15-20 load cycles to build up a steady state response of the ground. Wiss (1967) stated that the safe level of vibrations due to transient vibrations could be twice or even up to five times the safe level for steady-state vibrations. However, from the above reasoning it would mean that vibrations with a frequency over about 50 Hz during a period of 0.3 s would be able to give resonance.

2.5.3 Impact on humans

Human sensitivity to vibrations is a heritage from an era when people developed a perception system to warn them of landslides, flocks of animals and the like (Holmberg et. al., 1984). Hence, humans are very sensitive to ground vibrations, and even minor vibrations may attract complaints from people living or working in the vicinity of construction work (Stille & Hall, 1995) (Haegeman, 2002) (ArcelorMittal, 2008). Extremely large vibrations can be directly harmful to the human body; however, in practice such levels never occur during pile driving. Instead the problem is the disturbing effect and the expectation effect (Holmberg et. al., 1984).

2.5.3.1 Human perception of vibrations

Human perception is said to be the most difficult component to deal with. Measures of human perception and tolerance have been measured and results are applicable for the "average" individual. However, in a group of people it is the least tolerant individual who may control the situation (Woods, 1997).

Humans perceive vibrations at a very low level (Head & Jardine, 1992) (Hiller & Hope, 1998). According to Brenner & Chittikuladilok (1975) and Martin (1980) humans can perceive vibrations that are much lower, actually up to about 30 times lower, than the levels that usually cause architectural damage to buildings. As a result, the trigger for complaints and litigation is more often human perception and tolerance than actual physical damage to structures (Woods, 1997). In Table 2.10 the human perception of different vibration levels are shown.

Table 2.10 Approximate vibration level with corresponding human perception (Wiss, 1981) (Selby, 1991) (SA Government, 2007).

Approximate vibration level (mm/s)	Degree of human perception
0.10	Not felt
0.15	Threshold of perception
0.35	Barely noticeable
1.0	Noticeable
2.2	Easily noticeable
6.0	Strongly noticeable

2.5.3.2 Factors affecting human response

From literature (Head & Jardine (1992), Hintze et al. (1997), and Stille & Hall (1995) a list has been compiled of the factors that affect the human response of vibrations:

- Duration of vibration
- Current activity of a person (lying or standing, working or resting etc.)
- Accompanying noise
- Frequency of vibration
- Characteristics of vibration (transient or continuous)
- Vibration magnitude
- Physical and mental condition as well as personal attitude
- Time of day

Another critical factor when it comes to impact on humans is that when people feel vibrations they become concerned about building safety and begin to search for possible damage (Brenner & Chittikuladilok, 1975) (SA Government, 2007). Head & Jardine (1992) claimed that the reason that people complain about piling works is in many cases that they are temporary and intermittent, while representing a change to normal conditions with intense activity and sudden noise. The factor of personal attitudes can largely be helped by informing affected people about the nature and timing of the vibrations and assurances that the vibrations are monitored and under control (Head & Jardine, 1992). It has been noticed that the tolerated vibration level is higher if the cause for the vibration is known and no damage is to be expected (Stille & Hall, 1995).

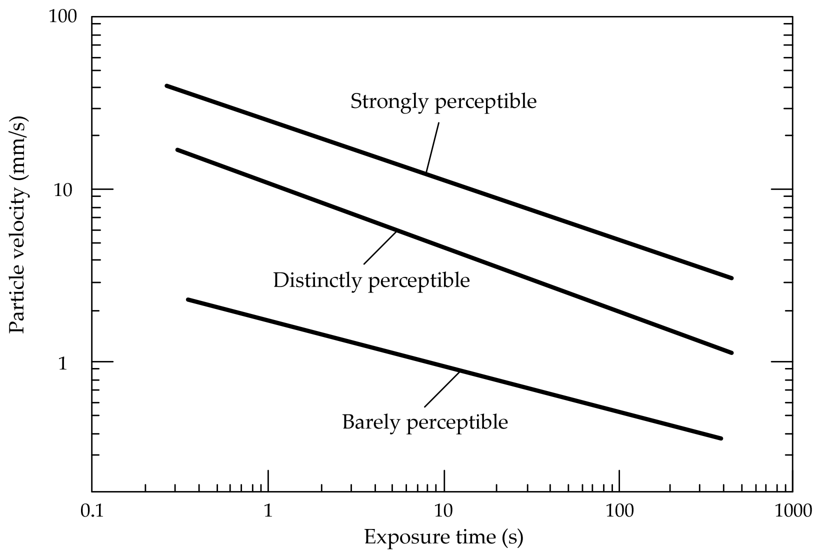

Figure 2.34 Human response to transient pulses of varying duration, modified after Woods (1997).

Experience has shown that for a person standing, vertical movements are more noticeable than horizontal movements and vice-versa for people lying down. Thus, the conclusion can be drawn that humans are more sensitive to vibrations in the body's longitudinal direction (Richart et al., 1970) (Möller et al, 2000).

In places normally without vibrations, people are more disturbed than in urban areas with a lot of background noise (Stille & Hall, 1995).

The vibration frequency affects how they are perceived; the human body's own vibration frequency can amplify the uneasiness of vibrations (Hintze et al., 1997) (ArcelorMittal, 2008). It has been seen that the human body is more sensitive to acceleration at low frequencies. This is particularly observable when frequencies are around 2-5 Hz, which is the resonance frequency of the human body (Brenner & Chittikuladilok, 1975). This is confirmed by the Swedish Engineering Society (IVA, 1983), who stated that vibrations within the frequency range of 1-20 Hz are usually especially disturbing for humans, while vibrations with frequencies greater than 20 or 30 Hz are normally considered a smaller problem.

Duration and time of exposure is another factor to consider, see Figure 2.34. According to the figure, the level of barely perceptible motion decreases from about 2.5 mm/s for 1 s of exposure to about 0.5 mm/s at 100 s of exposure (Woods, 1997).

It has been shown that the threshold of perception is practically the same for steady-state and transient vibrations. However, when it comes to causing annoyance for people, the level is considerably lower for steady-state vibrations than for transient vibrations (Brenner & Chittikuladilok, 1975). The vibration levels humans find disturbing from transient vibrations are 3-10 times higher than the disturbing vibration level from a continuous vibration, see Figure 2.35 (Wiss, 1967) (Stille & Hall, 1995).

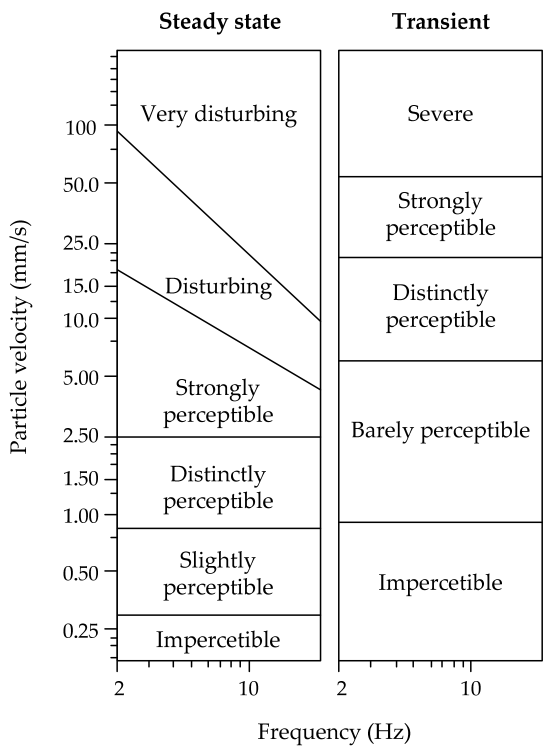

Figure 2.35 Comparison of human reactions to steady state and transient vibrations, modified after Stille & Hall (1995, originally from Reiher & Meister (1931) and Wiss & Parmlee (1974)).

2.6 MEASUREMENT OF VIBRATION

A crucial key for understanding and analysing the problem of vibrations induced by pile driving is to know the vibration levels actually caused by such activities. In this section vibration measurements and monitoring are discussed and methods are presented.

2.6.1 Measurement equipment

Usually vibration measurements are performed to determine magnitude or amplitude of motion (displacement, velocity, or acceleration) as a function of time. The measuring device selected must therefore be designed to measure one of these derivatives of motion. It is always preferable to measure the parameter to be controlled or evaluated directly in order to avoid errors in, for example, integration (Möller et al., 2000).

A few of the most common devices for measuring vibrations are described below.

2.6.1.1 Velocity transducers

A transducer is said to be any device or instrument that converts a physical phenomenon into an electrical signal (Richart et al., 1970). Velocity transducers used as measurement devices include geophones and seismographs.

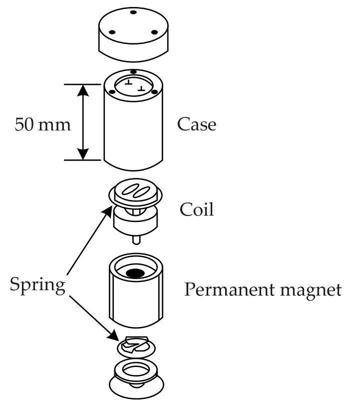

Figure 2.36 Principal components of a velocity transducer, modified after Richart et al. (1970).

Operation principles

The principal of operation for velocity transducers is that an electric coil moves in a magnetic field, see Figure 2.36. When the transducer is shaken the coil moves through a magnetic field produced by a permanent magnet. This mechanism produces a voltage that is proportional to the relative velocity between the coil and the magnet. Either the coil or the magnet may move. Velocity transducers with moving coils are insensitive to external magnetic fields, while those in which the magnet is the moving component are sensitive to external magnetic fields (Woods, 1997).

Since velocity transducers produce a voltage that is proportional to the particle velocity of the surface on which they are mounted, it is important that the mounting is done properly so that reliable representations of motion are made. A velocity transducer can be placed at considerable distance from the recorder since the voltage is not affected by cable length (Woods, 1997).

Velocity transducers can only measure in one direction. In order to measure both vertical and horizontal vibrations it is necessary to have different velocity transducers because of the internal mounting of a mass on springs (Möller et al., 2000). Usually three transducers are connected perpendicular to each other to enable measurements in three components of motion. Normally measurements are performed in the vertical and in two horizontal directions (longitudinal to the source and perpendicular to the source) (Möller et al., 2000).

According to Kim & Lee (2000) the response of velocity transducers becomes nonlinear at low frequencies; they also have a natural frequency since it is a single degree of freedom system. Due to this it is necessary to calibrate the exact voltage output for the geophone with frequency.

2.6.1.2 Acceleration transducers

Accelerometers are smaller than geophones, but have a larger frequency and dynamic range (Head & Jardine, 1992). It may be convenient or even necessary to use accelerometers for ground motions greater than 250 mm/s and frequencies higher than about 500 Hz (Woods, 1997).

The most common type of acceleration transducers are accelerometers.

Operation principles

There are several transduction principles for accelerometers. The most common type uses the piezo-electric properties of certain natural and artificial crystals. It works according to the principle that squeezing or shearing of one of these piezo crystals causes current to flow in a conductor attached to opposite sides of the crystal. The amount of generated current is proportional to the pressure or shear force. The function of the accelerometer is that the crystal is squeezed by a seismic mass, producing a force proportional to its acceleration (from $F=ma$) (Woods, 1997).

Accelerometers require supplemental (signal conditioning) instrumentation in addition to recording instrumentation, which is a disadvantage. Another disadvantage is the vulnerability of the cabling in field applications and the calibration of the transducer may also be dependent on cable length (Woods, 1997).

Usually accelerometers are constructed to operate in either shear or compression (Head & Jardine, 1992). Oil exploration geophysics led to the development of robust moving-coil accelerometers working in the frequency range of 2-200 Hz (Head & Jardine, 1992).

2.6.1.3 Displacement transducers

Displacement transducers are not as common as velocity or acceleration transducers for measurements of vibrations due to pile driving. For literature describing displacement transducers the reader is referred to e.g. Richart et al. (1970).

2.6.1.4 Component of motion to measure

There are three different quantities of vibration that can be measured: amplitude, velocity and acceleration. The quantity to measure is chosen according to what the measurements should show and the quantity that best describes the impact of the vibration on nearby objects. The maximum particle velocity is often used as a measure of vibration. Whyley & Sarsby (1992) and Athanasopoulos & Pelekis (2000) confirmed that in most studies of ground vibrations, the vibration intensity is described by the particle velocity.

One reason for the use of the vibration velocity is that the kinetic energy transferred into the soil during a vibration-generating activity is proportional to the square of the velocity (Niederwanger, 1999). Another reason is that the vibration velocity is a good indicator of the potential for damage since the induced dynamic stress in the building is proportional to the vibration velocity (Whyley & Sarsby, 1992) (Hiller & Hope, 1998).

Waves produced by earthquakes contain low frequencies. For these waves it is more common that acceleration is used as a damage measure. The vibration tolerance-level in vibration-sensitive equipment, such as computers or sophisticated laboratory equipment, is often expressed in acceleration (Atlas Copco ABEM, 1973).

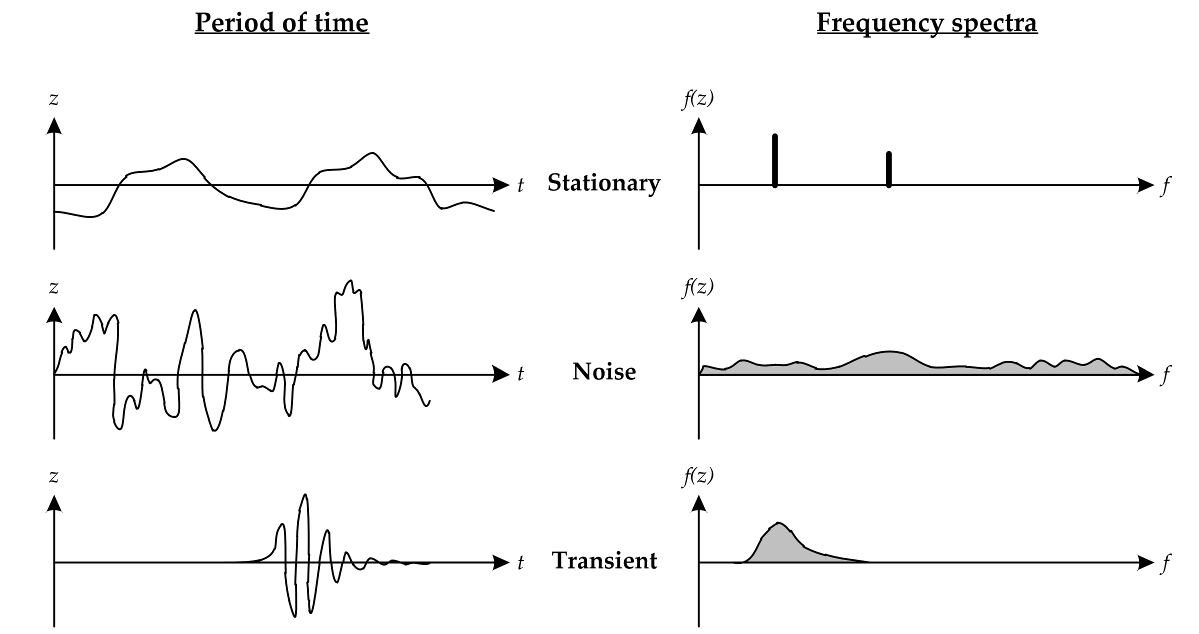

Figure 2.37 Examples of oscillation movements shown as both function of time and frequency.

2.6.2 Interpretation and presentation of results

Results from vibration measurements need to be interpreted and presented in order to be of any value.

2.6.2.1 Vibration Records

Vibration records can be obtained in many different ways. Most vibrations are recorded as voltage versus time. For each transducer, voltage is related to motion by the use of a calibration factor. Today, most recording is done in digital format in which the analogue voltage signal is converted to a digital format and then stored on a hard disk (Woods, 1997).

A vibratory motion is usually characterised either as a function of time or as a function of frequency, see Figure 2.37. In the time domain the maximum and minimum value can be interpreted. In the frequency domain the dominating frequency can be evaluated (Möller et al, 2000).

Complicated oscillation patterns are often shown as a function of frequency, a "frequency spectrum", in which the including frequencies are displayed (IVA, 1979). Different oscillation movements are shown in Figure 2.37, both as function of time and of frequency. The frequency spectrum is usually obtained by performing a fast Fourier transform (FFT) (Tamate et al., 1995).

2.6.2.2 Peak particle velocity (*PPV*)

Ground vibrations are, for engineering purposes, usually quantified in terms of peak particle velocity (*PPV*). *PPV* can be defined in several different ways, some of which are listed here (Head & Jardine, 1992) (Hiller & Hope, 1998) (Athansopoulos & Pelekis, 2000):

- $SRSS$ (simulated resultant) $v_{SRSS} = \sqrt{v_{x,max}^2 + v_{y,max}^2 + v_{z,max}^2}$

- Uni-directional peak $v_{\max(x,y\ or\ z)} = v_{x,max}\ or\ v_{y,max}\ or\ v_{z,max}$

- Vertical peak value $v_{z,max}$

- Instantaneous (true) resultant $v_{\max(t)} = \sqrt{v_{x(t)}^2 + v_{y(t)}^2 + v_{z(t)}^2}$

Vibrations from pile driving could be characterised by the simulated resultant PPV. The simulated resultant is also referred to as the $SRSS$ (square root of sum of squares) as it is the vector sum of the peak particle velocities in three mutually perpendicular directions, which may not occur simultaneously (Head & Jardine, 1992) (Rockhill et al., 2003). Sometimes the vibration intensity is described by the peak component of particle velocity or the peak value of the vertical component. The true vector sum or the instantaneous resultant is also used (Hiller & Hope, 1998) (Athansopoulos & Pelekis, 2000).

It is absolutely necessary to state which definition of the PPV is used since the nominal PPV can differ largely depending on the definition used. Head & Jardine (1992) and Rockhill et al. (2003) proposed the use of the simulated peak resultant ($SRSS$). However, according to Athanasopoulos & Pelekis (2000) $SRSS$ is no longer used as often, and is now considered as too conservative. They claimed that field data had indicated that compared to the true resultant, or the true vector sum as they call it, the peak component particle velocity was up to 25% lower while v_{SRSS} was 50% higher.

2.6.2.3 Particle displacement paths

Particle displacement paths can be plotted using ground vibration data in three orthogonal directions, see Figure 2.38. Particle displacement paths are obtained by combining the vertical and horizontal time histories of components of motion and can give information regarding the types of wave propagating away from the vibration source (Athanasopoulos & Pelekis, 2000) (Whenham, 2011).

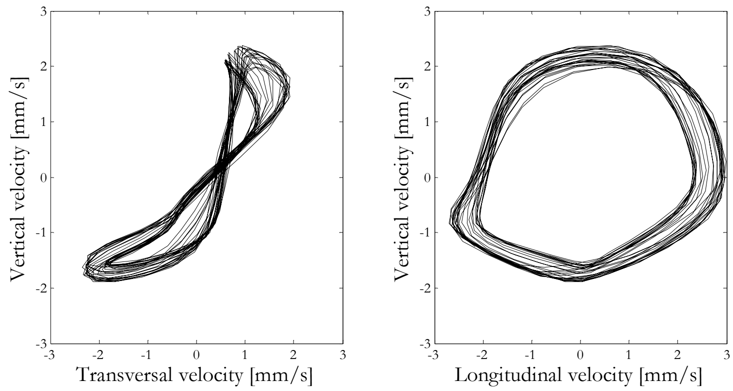

Figure 2.38 Examples of particle displacement paths (Lidén, 2012).

2.7 PREDICTION OF VIBRATIONS DUE TO PILE DRIVING

Head & Jardine (1992) stated that the best way to know if a piling operation will result in acceptable vibration levels is to perform trial piling at the site before the start of the real piling works. However, in many cases trial piling is not possible and a reliable prediction model is needed instead.

Prediction of the generated vibrations in a construction project can have important economic and technical consequences. Unnecessarily conservative assumptions lead to increasing costs and may also limit the choice of construction methods and/or delay the project. However, underestimating the environmental impact may lead to damaged structures, disturbed occupants and authorities may stop the construction work. Despite the fact that research on the subject has led to the development of new, mainly theoretical, prediction models there seems to be a general consensus in literature that as of today reliable methods for estimating the vibrations from pile driving are missing (see e.g. Hintze et al. (1997), Whyley & Sarsby (1992), Massarsch (2004), Jongmans (1996), Madheswaran et al. (2005), Waarts & Bielefeld (1994) and Davis (2010)).

This section presents the existing prediction models and methods for estimating vibrations due to pile driving. Furthermore, concepts and theory about factors influencing the predicted vibrations are included.

The existing prediction models can be divided into different categories depending on their approach. In literature different ways of categorising prediction models have been seen. For example, Whenham (2011) looked at empirical approaches, analytical approaches and numerical approaches. Davis (2010) divided prediction models into theoretical/ (semi-) analytical, in-situ testing/field measurements – Modular Prediction Approaches, empirical prediction/Direct Measurement Prediction Model (DMPM) and numerical models. In this study prediction models for prediction of vibrations from pile driving are divided into the following three categories:

- Empirical models – models based on empirical knowledge from previous measurements and experience
- Theoretical models – such as finite element models or analytical models
- Engineering models – sometimes also called mixed approach models, these are a mix of empirical models, theoretical models and engineering knowledge

The first step for prediction by means of measurement involves conducting vibration measurements at the site of interest during piling/sheet piling. The measurements are then evaluated and fit into an attenuation relationship. These types of prediction models are only applicable at the specific sites where measurement has been conducted. Prediction models based on measurements are not studied further in this literature study; only models that can be used without previous vibration measurements are presented.

2.7.1 Empirical models

Even if there are no generally accepted methods for predicting vibrations during pile driving, a lot of measurements and empirical knowledge exists. In most cases the calculation of ground vibrations is still based on rough empirical rules developed a long time ago (Hintze et al., 1997) (Massarsch & Fellenius, 2008).

Head & Jardine (1992), Hiller & Hope (1998) and Massarsch (2004) pointed out that empirical relations should only be applied in conditions similar to those for which they were developed. In particular empirical models may be unreliable close to the pile or in locations with intervening structures.

2.7.1.1 Attewell & Farmer model

Wiss (1967) discovered that the vibration magnitude due to pile driving varied by the amount of energy transmitted to the soil, the soil properties and the distance from the source and concluded that the particle velocity varied with the square root of the energy of the hammer. Since then many prediction models have taken the form of a power law (or energy-based prediction model) in which the ground vibration magnitude is assumed to be dependent on the hammer energy.

In 1973 Attewell & Farmer presented one of the first empirical prediction models, where they suggest that the vertical peak particle velocity, *v*, is given according to the general formula:

Eq. 2.43
$$v = k\left(\frac{\sqrt{W_0}}{r}\right)^x \qquad \text{(mm/s)}$$

Where k = empirically determined constant of proportionality (m²/s√J)
W_0 = input energy (hammer energy) (J)
r = radial distance between pile and monitoring point (m)
x = empirically determined index (-)

Attewell & Farmer (1973) concluded that losses due to material damping are small compared to losses due to geometrical damping. As a result, they suggested that material damping can be neglected for practical estimates of vibration from pile driving.

From their field measurements, Attewell & Farmer (1973) claimed that the results correlate quite well with the following relationship:

Eq. 2.44
$$v = \frac{\sqrt{W_0}}{r} \qquad \text{(mm/s)}$$

Which gives k = 1 and x = 1. However, Attewell & Farmer (1973) suggested that a constant of proportionality, *k*, of 1.5 should be used for practical conservative prediction of ground vibrations due to pile driving.

Table 2.11 Summary of values of parameters used in different prediction models, modified after Hope & Hiller (2000).

Literature	**Parameters**		**Velocity component**
	x	***k***	
Attewell & Farmer (1973)	1	1.5	Vertical *PPV*
Whyley & Sarsby (1992)	1	0.25 (soft or loose soil) 0.75 (stiff or medium dense soil) 1.5 (stiff or dense soil)	
Attewell et al. (1992a)	0.87	0.76	Vertical *PPV*
Hiller & Crabb (1998)		3 (stiff or medium dense soil)	
Head & Jardine (1992)	1 1.54*	1,5 (for r>0.5 m) 0.2*	*SRSS*
BSI (1992a)	1	0.75	
CEN (1998)	1	0.5 (soft cohesive soil) 0.75 (stiff cohesive soil) 1.0 (very stiff cohesive soil)	
ArcelorMittal (2008)	1	**Impact driving** 0.5 (soft cohesive soil, loose granular media, loose fill and organic soils) 0.75 (stiff cohesive soils, medium dense granular media, compact fill) 1.0 (very stiff cohesive soil, dense granular media, rock, fill with large obstructions) **Vibratory driving** 0.7 (all soil conditions)	

*At the base of the foundation

The general form in Eq. 2.43 has since been developed by various researchers. In 1981 Wiss proposed an equation equivalent to the above equation (Eq. 2.43) for the peak particle velocity. Wiss (1981) stated that x lies between 1.0 and 2.0 with a relatively common value of 1.5 and that the factor k depends on ground conditions and source type. According to Attewell et al. (1992a) a best-fit line from measurements of ground vibrations from both impact and vibratory pile driving give k = 0.76 and x = 0.87. Wiss (1967), Whyley & Sarsby (1992) and Hiller & Crabb (1998) showed that k varies with soil conditions. Nilsson (1989) stated that several field studies have shown that k does not exceed 0.75 for driving of piles and 1.5 for driving of sheet piles. According to Whyley & Sarsby (1992), k varied between 0.25-1.5 depending on soil type and x represents both geometrical and internal damping. According to ArcelorMittal (2008), k varied between 0.5 and 1.0 depending on soil type and driving method. Heckman & Hagerty (1978) proposed that k depends on the soil conditions and the impedance of the pile and varies between 0.2 and 1.5 with increasing k for decreasing impedance.

Table 2.11 gives a summary of values suggested for k and x in equation Eq. 2.43 found in literature. The data-fitting in the table are for all cases upper bound.

The actual source energy is difficult to estimate due to losses at the pile-soil interface, therefore Attewell et al. (1992a) and Whenham (2011) suggest that the notional energy quoted by the pile driver manufacturer is used for source energy, W_0. W_0 is expressed in joules (Newton meters) for impact pile driving and joules/cycle for vibratory pile driving. Head & Jardine (1992) give the following rules of thumbs for calculating the energy values in Joules for different driving system:

Drop hammers			
	Mass of hammer	=	*m* tonnes
	Drop height	=	*h* meters
	Energy per blow	=	9807**m***h* Joules
Diesel hammers			
	Rated energy per blow	=	*(R.E.)* kg-m
	Energy per blow	=	*(R.E.)**9.807 Joules
Vibratory drivers			
	Power supply	=	*W* kVA (= kilowatt = kiloJoules/s)
	Rated frequency	=	*f* Hertz
	Energy per cycle	=	1000*(*W*/*f*) Joules

<u>Whyley & Sarsby predictive plot</u>

Whyley & Sarsby (1992) presented a predictive plot, see Figure 2.39, based on Eq. 2.43 where $x = 1$ and k is 1.5 for line 1 (stiff or dense soil), 0.75 for line 2 (firm to stiff or medium dense soil) and 0.25 for line 3 (soft or loose soil). If ending up in zone (a) that means no problem, zone (b) indicates further investigation is needed and zone (c) is equivalent to redesign the work. The figure is applicable for impact driving and especially for sheet piles.

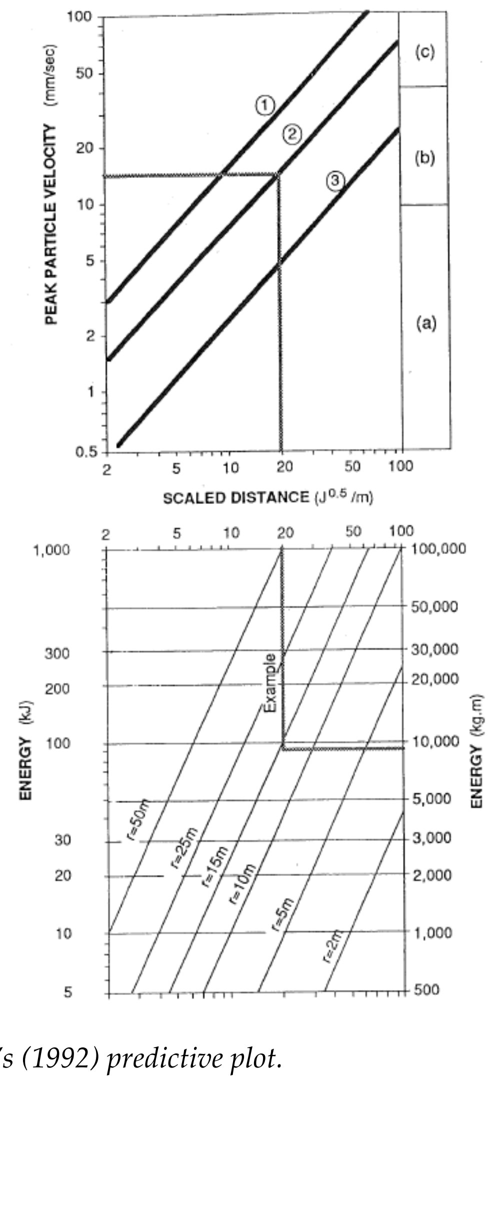

Figure 2.39 Whyley & Sarsby's (1992) predictive plot.

Svinkin empirical model

Svinkin (2008) presented a development of the energy-based relationship for the prediction of ground vibrations due to pile driving. Svinkin's model is based on determination of the vibration velocity on the pile head, and from that computes the ground vibrations. The following relationship is proposed for the ground vibration due to pile driving:

Eq. 2.45 $$v_g = v_p \frac{\sqrt{W_0}}{r} \quad \text{(mm/s)}$$

Where v_g = ground vibration (mm/s)

v_p = pile vibration at the pile head = $\sqrt{2 \frac{c_B}{Z_p L_p} W_0}$ (mm/s)

c = wave propagation velocity in the pile (m/s)

Z_p = pile impedance (kNs/m)

L_p = pile length (m)

W_0 = energy transferred to the pile (J)

r = distance from the pile to the point of interest (m)

Svinkin (2008) suggested that W_0 is determined as the rated energy times the efficiency. For vibratory driving W_0 is the maximum energy transferred to a vibratory driven pile per cycle of driving and determined from the maximum power times the period times the efficiency.

2.7.1.2 Handboek Damwanden model

Van Staalduinen presented an empirical prediction model that is based on 250 vibration measurements performed during the vibratory installation of sheet piles in the Netherlands. The model is presented in the Dutch "Sheet pile Handbook" (Handboek Damwanden, CUR publication 166). In the model, the Netherlands is divided into seven different parts that each represents a characteristic soil profile. From the results of the measurements a statistical analysis has been performed finally resulting in Table 2.12 and Table 2.13. Table 2.12 is valid for vibratory hammers with an eccentric force of up to 350 kN. For larger vibrators the vibration level should be adjusted according to:

Eq. 2.46 $$u_0 = u_{0,350} + 0.002(F - 350) \quad \text{(mm/s)}$$

Where F = eccentric force of the vibrator (kN)

Table 2.12 Data for vibration prediction for vibratory driven sheet piles (hammers up to 350 kN) (Handboek Damwanden).

Soil profile	u_0 (mm/s)		α (m)		V_0	
	Vert	Hor	Vert	Hor	Vert	Hor
1 (Amsterdam)	1.1	1.6	0	0	0.9	1.5
2 (Eindhoven)	1.9	2.6	0	0	1.1	0.8
3 (Groningen)	1.7	0.9	0	0	1.8	0.5
4 (Den Haag/Scheveningen)	1.9	2.6	0	0	1.1	0.8
5 (Maasvlakte)	-	-	-	-	-	-
6 (Rotterdam)	1.1	1.6	0	0	0.9	1.5
7 (Tiel)	1.1	1.6	0	0	0.9	1.5

Table 2.13 Data for vibration prediction for impact driven sheet piles (Handboek Damwanden).

Soil profile	u_0 **(mm/s)**		α **(m)**		V_0	
	Vert	**Hor**	**Vert**	**Hor**	**Vert**	**Hor**
1 (Amsterdam)	0.030	-	0.03	-	0.6	-
2 (Eindhoven)	-	-	-	-	-	-
3 (Groningen)	-	-	-	-	-	-
4 (Den Haag/Scheveningen)	-	-	-	-	-	-
5 (Maasvlakte)	0.040	-	0.00	-	0.6	-
6 (Rotterdam)	0.017	0.026	0.03	0.03	0.6	0.6
7 (Tiel)	-	-	-	-	-	-

The values from Table 2.12 and Table 2.13 are used in the following summarised equation in order to receive a predicted vibration, *u(r)*:

Eq. 2.47 $$u(r) = u_0 \sqrt{\frac{r_0}{r}} e^{-\alpha(r-r_0)} e^{0.7\beta V_0}$$ (mm/s)

Where β = according to Table 2.14 depends on the probability of exceedance
r_0 = reference distance set to 5 m

Table 2.14 Probability of exceedance and β-values (Handboek Damwanden).

Probability of exceedance	β**-value**
0.5	0.0
0.1	1.18
0.05	1.64
0.01	2.32
0.005	2.57
0.001	3.09

2.7.1.3 Attewell et al. model

Attewell et al. (1992a and 1992b) found that a quadratic regression curve was a better fit to field data from measurements of ground vibrations due to pile driving than the previously used linear regression curve (Attewell & Farmer, 1973). The developed model proposed the following equation for the prediction of vibration velocity due to pile driving:

Eq. 2.48 $$\log v = x_1 + x_2 \log\left(\frac{\sqrt{W_0}}{r}\right) + x_3 \log^2\left(\frac{\sqrt{W_0}}{r}\right)$$

Where v = vibration velocity (mm/s)
x_1, x_2 and x_3 = constants of proportionality (-), see Table 2.15 and Table 2.16
W_0 = input energy (J)
r = distance between source and point of interest (m)

Constants x_1, x_2 and x_3 are functions of the soil conditions at the site of pile driving (Attewell et al., 1992b). Proposed values of the constants of proportionality are given in Table 2.15 and Table 2.16 for impact respectively vibratory pile driving. In Attewell et al. (1992a) it is recognised that vibratory driving is in many ways different from impact driving, and for the estimation of ground vibrations from pile driving the two installation methods should be treated separately. Hence, the developed model makes a distinction between vibrations from impact driven and vibratory driven piles.

Table 2.15 Values of x_1, x_2 *and* x_3 *for **impact pile driving**, from Attewell et al. (1992b).*

Curve fit	x_1	x_2	x_3
Best-fit	-0.519	1.38	-0.234
Half a standard deviation	-0.296	1.38	-0.234
One standard deviation	-0.073	1.38	-0.234

Table 2.16 Values of x_1, x_2 *and* x_3 *for **vibratory pile driving**, from Attewell et al. (1992b).*

Curve fit	x_1	x_2	x_3
Best-fit	-0.464	1.64	-0.334
Half a standard deviation	-0.213	1.64	-0.334
One standard deviation	0.038	1.64	-0.334

Attewell et al. (1992b) proposed that the values for half a standard deviation should be used for normal construction work while one standard deviation should be used where high security against vibration is needed. For the best-fit line there is a risk of exceeding the estimated values of 50%, for half a standard deviation the risk is 31% and for one standard deviation the risk is reduced to 16% (Attewell et al., 1992b).

In Attewell et al. (1992b) tables for prediction of vibrations due to impact and vibratory driving are presented. The tables are based on Eq. 2.48 and values from Table 2.15 or Table 2.16 and aim towards helping practitioners that are not used to handling quadratic equations to make predictions on the construction site.

The horizontal distance along the ground between the source and the point of interest is usually taken as *r*. However, Attewell et al. (1992a) are well aware that this could lead to errors when a large amount of the vibration energy is transferred at the pile toe, especially at close range. Despite this they have chosen to put *r* as the horizontal distance between source and point of interest mainly for the sake of simplicity.

2.7.2 Theoretical models

Theoretical models use a different approach for the prediction of vibrations due to pile driving than empirical models. Theoretical models are usually built up of numerical or analytical modelling in different computer programs. Davis (2010) has listed several numerical methods which can be used for prediction of ground vibrations, the most common that are present in existing prediction models are:

- Finite Difference Time-Domain Method (FDTD or FDM)
- Finite Element Method (FEM)
- Boundary Element Method (BEM)

FDM can take into account layering and anisotropy of the soil; however, there is uncertainty in the loss of the energy due to material damping. Another drawback of FDM is that it requires a high level of mathematical skills from the user (Davis, 2010).

FEM is popular for the modelling of problems in soil and rock materials; there are a number of commercial computer programs based on FEM (Plaxis being the most common among geotechnical engineers) with graphical user interfaces making it a popular tool in many situations. 2D FEM is not ideal for modelling pile or sheet pile driving as it generally is a point source and the soil conditions are usually complex giving diverging propagation paths in different directions (Davis, 2010).

BEM is somewhat more limited in its use than FEM and FDM due to its need for reformulation of the partial differential equations. To overcome the limitations with BEM the soil immediately next to the source can be modelled with FEM while the rest of the propagation path can be modelled using a coupled BEM model. For the modelling of ground vibration problems with infinite domains, BEM is considered to be better than FEM for efficiency, accuracy and user friendliness (Davis, 2010). Several of the existing prediction models mix different numerical methods in their prediction models.

Theoretical models often contain sub-models for the pile, the soil and sometimes also for objects susceptible to damage. The sub-models are modelled separately and thereafter connected to make the prediction. Connection between the sub-models is usually based on connectivity of vibration or force at nodes (Waarts & de Wit, 2004).

In this section some of the existing theoretical models are described in brief.

2.7.2.1 Waarts & Bielefeld model

Waarts & Bielefeld (1994) presented a model to predict vibrations from pile driving. The only necessary input data for the prediction is the type of pile and hammer and the results of a CPT-test.

The Waarts & Bielefeld model is actually divided into two different models. The model for the pile driving, described by the stress wave simulation program Tnowave, and the model for the wave transmission in soils described by the finite element package Diana. The Tnowave program is based on the one dimensional stress wave theory and it simulates the pile driving process for many combinations of pile driving hammers (both impact and vibratory hammers), pile types and soil conditions. From Tnowave the load applied to the soil is computed. The load, consisting of the force at the pile toe and the outside friction on the pile, is thereafter put into Diana. With Diana displacements, velocities and accelerations can be computed as a function of time in every point in the soil.

Waarts & Bielefeld (1994) chose not to model the compression wave in the FEM model. However, when comparing predicted values with measured values, a very obvious difference was the lack of compression waves in the predicted signals. The predicted peak accelerations were relatively close to the measured values. The predicted peak velocities, however, showed a difference compared to measured values. For predicted dominant frequencies and shear wave velocities, the correlation with the measured results was good.

Waarts & Bielefeld (1994) proposes a few improvements to their prediction model. For example, the pile-soil interaction can be improved taking degradation effects on the soil into account. Another improvement is to develop the determination of soil parameters from soil investigation results.

2.7.2.2 Holeyman model

The Holeyman (1993) model of calculating vertical shear waves propagating away from the pile is based on a radial discrete model. The cylindrical model is illustrated in Figure 2.40 and consists of disks or concentric rings with increasing distances the further from the pile they are located. The rings have their own, individual masses and transmit forces to their neighbouring rings. This method of soil modelling is meant to simulate geometric damping. Based on the stress-strain relationship the shear force-displacement relationship between successive rings is established (Holeyman & Legrand, 1997) (Holeyman, 2002) (Whenham, 2011). The model is said to be able to provide insight into vibration levels in the vicinity of the pile (Holeyman, 2002).

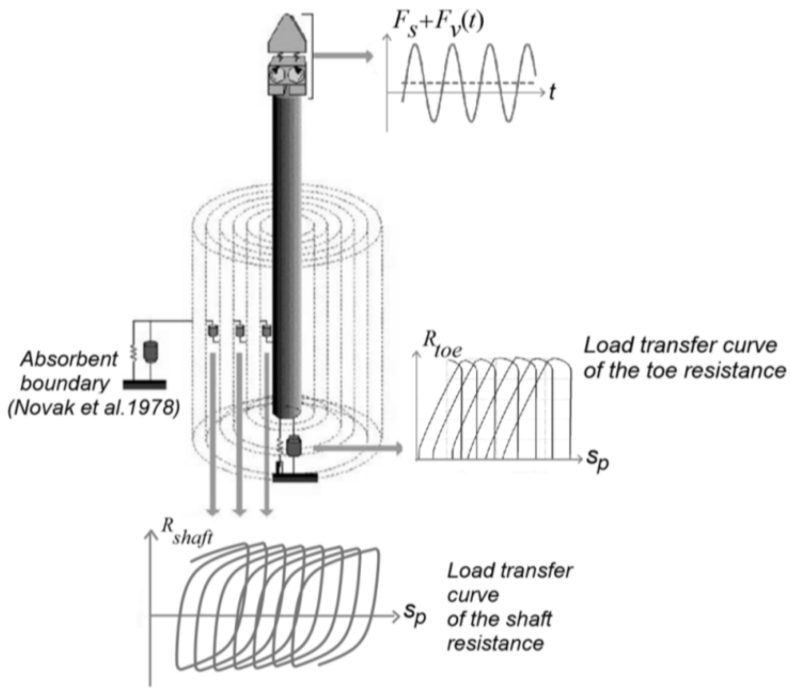

Figure 2.40 Vibratory driving model developed by Holeyman (1993) (Whenham, 2011 from Holeyman, 1993b).

2.7.2.3 EDT Toolbox (Direct stiffness method)

EDT Toolbox is presented by Whenham (2011) and has been developed at KULeuven in order to compute the response of a layered medium due to an external load. It has earlier been presented by Schevenels et al. (2009). The EDT Toolbox contains a Matlab function that calculates the Green's functions of the soil, based on the direct stiffness method. The method models the soil having linear behaviour. The load is modelled by inserting multiple external loads uniformly distributed between 0 and 2.25 m depth.

2.7.2.4 Finite Element Method (Plaxis)

Whenham (2011) have modelled vibratory pile driving in the commercial FEM software Plaxis. The problem set-up is an axisymmetric geometry extending 40 m in the radial direction and 25 m in the vertical direction, see Figure 2.41. The boundaries are chosen as absorbent boundaries at the bottom and right hand side, and have the function that compression waves that hit the boundary perpendicularly will be absorbed while shear waves will still give a small boundary effect.

The load is added as point loads distributed between 0 and 2.25 m at the centre of symmetry. This load model assumes that the force applied to the soil by the pile is equally distributed along the pile shaft.

The soil is considered to be linear elastic, and material damping is represented by a damping parameter proportional to the mass and stiffness of the system.

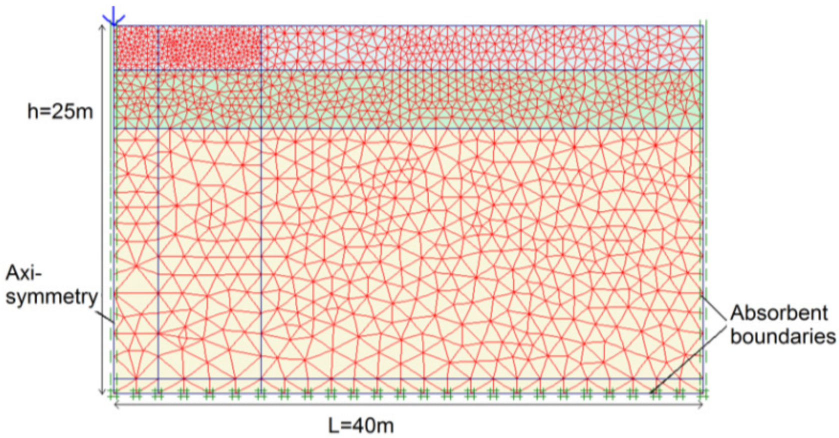

Figure 2.41 Set-up of Plaxis model (Whenham, 2011).

2.7.2.5 Masoumi et al. model

Masoumi et al. (2006, 2007, and 2008) presented a numerical prediction model made up of a coupled finite element-boundary element model in order to predict free field vibrations due to impact and vibratory pile driving. The pile is modelled as linear elastic material using the finite element technique and the soil is modelled as a horizontally layered elastic half-space using the boundary element technique. The pile-soil interaction is modelled using a subdomain formulation. As their focus is on vibrations in the far-field, Masoumi et al. (2006 and 2007) assumed a linear elastic constitutive behaviour of the soil as the deformations are believed to be relatively small. The damping is assumed to be independent of frequency and no separation is allowed between pile and soil. The soil is assumed to be horizontally layered.

To solve the system, the Structural Dynamics Toolbox in Matlab is first used to make the finite element model of the pile. Then the soil impedance and the modal responses of the soil are computed using the program MISS 6.3. From there on the soil tractions on the interface and then in the free field are computed.

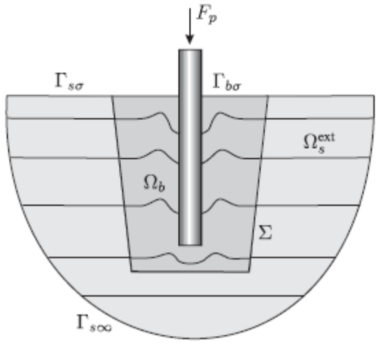

Figure 2.42 Geometry and outline of the problem (Masoumi et al., 2009).

Masoumi et al. (2008) noticed that their model seemed to overestimate ground vibration in the far-field. They believed this to be due to plastic strains induced in the soil in the vicinity of the pile leading to more material damping. Therefore, Masoumi et al. (2009) proposed the use of a model that includes a plastic zone in the vicinity of the pile. According to Masoumi et al. (2009) the model includes both the dynamic pile-soil interaction and the non-linear behaviour of the soil in the vicinity of the pile. The soil-structure system has been divided into two substructures: a bounded structure involving the pile and the section of the soil around the pile that may not behave linearly, and the unbounded linear elastic horizontally layered soil, see Figure 2.42. The pile and the soil closest to it are modelled using a time-domain finite element method and the soil is modelled as a horizontally layered elastic half-space using the boundary element technique in the frequency domain.

The simulation clearly showed the evolution of a plastic zone around the pile and below the pile toe for both impact and vibratory driving. Masoumi et al. (2009) also compared vibration levels received from their model with field measurements presented by Wiss (1981) showing good agreement between predictions and measurements.

2.7.2.6 Mahutka & Grabe (2006)

Mahutka & Grabe (2006) presented a model in which vibratory pile driving is modelled by non-linear dynamic finite element analysis with an explicit time integration scheme. The installation process of the vibratory pile driving is modelled using FEM and computations are done in the computer program Abaqus. The pile is modelled as a laterally supported rigid axisymmetric surface while the soil is discretised with axisymmetric continuum elements.

Mahutka & Grabe (2006) performed field tests to validate their model. They measured the acceleration at the pile as well as the vertical and horizontal velocities at four points located 1 m, 2 m, 4 m and 8 m from the vibratory driven pile. The measured results show a good agreement with the modelled results, see Figure 2.43.

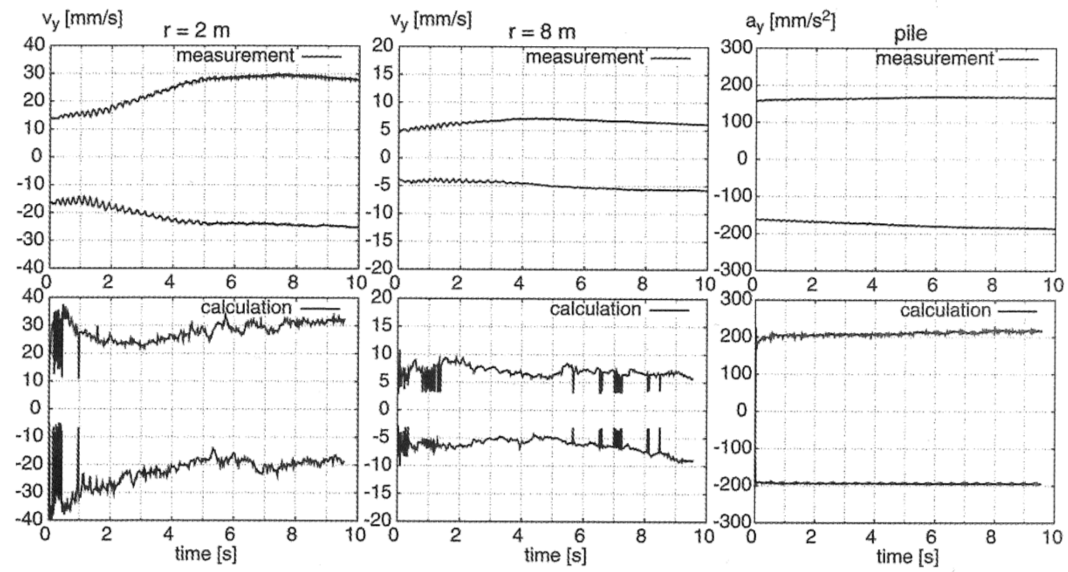

Figure 2.43 Measured and calculated vibration velocity at the ground surface and acceleration at the pile (Mahutka & Grabe, 2006).

2.7.2.7 Khoubani & Ahmadi (2012)

Khoubani & Ahmadi (2012) have created an axisymmetric finite-element model using Abaqus to predict vibrations in the form of *PPV* from impact driven piles in homogenous soil. In the model the entire penetration process, from the ground surface to the desired depth, is included. Plastic deformations in the soil next to the pile as well as a slip frictional contact between pile and soil are accounted for in the model.

The pile is driven down by the modelling of successive hammer impacts. Khoubani & Ahmadi (2012) have used one second as the time between each hammer blow in their model.

The *PPV* is computed at different distances from the centreline of the pile. The modelled results were compared with the results measured by Wiss (1981) and showed good agreement. They also compared their results with the numerical results presented by Masoumi et al. (2009). The results differs somewhat – Khoubani & Ahmadi (2012) reported higher values than Masoumi et al. (2009) for a distance of 5-9 m and vice versa for distances of 9-23 m.

Khoubani & Ahmadi (2012) also noticed that for all points more than 5 m from the pile the maximum *PPV* occurred at a penetration depth between 4.5-5.5 m.

From their sensitivity analysis Khoubani & Ahmadi (2012) concluded that the level of vibrations depended on the properties of the pile, hammer and soil. An increase in the impact force, the pile diameter or the soil-pile friction in their model resulted in an increase in *PPV*. If the elasticity modulus of the soil was increased, *PPV* decreased.

2.7.3 Engineering models

2.7.3.1 Massarsch & Fellenius model

Massarsch & Fellenius (2008) introduced a model for estimating vibrations from impact pile driving. The method includes the force applied to the pile head, the dynamic stresses in the pile and the dynamic resistance along the pile toe and pile shaft.

Massarsch & Fellenius (2008) suggested that the calculation of ground vibrations induced by pile driving with impact hammers be based on the following approach:

- Determine the dynamic pile hammer properties
- Determine the dynamic pile properties
- Estimate the peak particle velocity of the stress wave
- Assess the vibration transmission efficacy along the pile shaft and at the pile toe
- Calculate the propagation of spherical wave energy from the pile toe to the ground surface, taking into account wave reflection
- At the critical distance from the pile on the ground surface, calculate the vibration attenuation of surface waves
- Calculate the cylindrical waves from the pile shaft

Predicted values from the model were compared with measurement results from one driven pile in a case study presented by Nilsson (1989). Massarsch & Fellenius (2008) claimed that there was a good correlation between predicted and measured values. Massarsch & Fellenius (2008) pointed out that their prediction model does not take into account ground vibration amplification due to wave superposition when waves interact, such as from the pile toe and the pile shaft.

2.7.3.2 Jongmans' model

Jongmans (1996) presents a model that aims towards reconstructing the whole vibration signal generated during pile driving. The model contains two parts; the first part is based in the use of geophysical prospecting to represent the response of the site and the other part is an equivalent source function idealising energy transmission from pile toe to soil.

Jongmans' model is based on that the ground vibration at a distance *r* from the source can be given according to the following function:

Eq. 2.49 $$w(t,r) = s(t) * g(t,r)$$

Where $s(t)$ = source function
$g(t,r)$ = propagation function, also called Green's function

The two functions are determined separately. In order to determine the point load solution, Green's function ($g(t,r)$), it is suggested that each site is investigated by a standard seismic prospecting test. From the seismograms the soil conditions with different layers and dynamic properties are decided. The interpretation is based on the assumption that the ground is horizontally layered. By using a discrete wave number method, once the geometry and dynamic properties are known, Green's function is computed.

The source function simulates the input wave form and represents an equivalent linear source function. The function depends on pile type and driving method and varies with soil resistance at the pile toe. Jongmans' model assumes that vibrations are generated at the pile toe by a vertical force. The equivalent linear source function is determined from a vibration record close to the pile and the site's Green function. Jongmans suggests that a data-base of source functions for different driving methods and soil types could be set up.

Jongmans compared his model to results from a field test showing a good correlation regarding amplitude and wave forms of the vibrations.

2.7.3.3 Svinkin engineering model

Svinkin (1996) presented a prediction model based on the concept of the impulse response function. The impulse response function models behaviour of the soil. As, Svinkin (1996) puts it "the impulse response function is an output signal of the system based on a single instantaneous impulse input". In this prediction model, the output is a location of interest, the dynamic system is the soil and the input is the ground at the place for pile driving.

By setting up an experiment of applying known magnitudes of impact on the site of interest, for example by dropping a mass and recording the oscillation at impact, the impulse response function is determined. Once the impulse response function is known the dynamic loads for pile driving are computed by wave equation analysis. Finally Duhamel's integral (Smith & Downy, 1968) is used to find the predicted vibrations.

The prediction model is based on the assumption that the soil behaves as a linear material.

2.7.4 Uncertainties in prediction

From the study of Waarts & de Wit (2004) on the reliability of prediction models of vibrations due to pile driving and the report by Hintze et al. (1997) the following main sources of uncertainty in vibration predictions are identified:

- Correct determination or lack of information regarding input data such as soil conditions and hammer characteristics
- Simplifications and approximations in the modelling
- The effect of other factors such as time, control programs etc.

In their discussion about uncertainties in prediction of vibrations, Waarts & de Wit (2004) came to the conclusion that vibrations due to sheet pile vibratory driving are more difficult to predict than vibrations from pile driving. However, they conclude that overall the uncertainty in vibration prediction of today is quite large, even though the predictions are somewhat more reliable for theoretical models.

One of the main conclusions in the study of Waarts & de Wit (2004) are that the user of the prediction model has a massive influence on the outcome of the prediction. Another conclusion was that the uncertainty in vibration prediction generally is quite large; however, the use of sophisticated FEM-models reduced the uncertainty compared to expert judgement.

Hope & Hiller (2000) presented a review of the prediction models available at that time. They focus on vibrations from impact pile driving. In their review they find that the accuracy of the existing prediction models is limited. Most prediction models considerably overestimate the vibration magnitudes at distances less than 11 m from the pile. However, most prediction models are intentionally conservative.

Davis (2010) discussed the need for relative simplicity, the speed of calculation and the need for accuracy when it comes to prediction models. Pile driving projects are in almost all cases temporary and of a relatively short duration, as opposed to permanent vibration sources, such as e.g. railways or traffic. This calls for a difference in demands that are put on the prediction models. For pile driving the speed of calculation and relative simplicity (user-friendliness) are probably as important as accuracy, while for permanent sources accuracy is probably much more important than speed of calculation and a “simple” model.

According to Attewell et al. (1992a) it is quite reasonable that ground vibrations due to pile driving can be estimated by the use of empirical methods. They state that empirical methods are the most sensible and suitable for use on site. Athanasopoulos & Pelekis (2000) agreed, claiming that empirical models do not take strongly non-uniform soils nor the dynamic soil-structure interaction into account. However, they are easy to apply and thus valuable for piling practitioners

However, Massarsch & Fellenius (2008) showed that the energy-based, empirical approach widely used by engineers is too crude for reliable analysis of ground vibrations and can even be misleading. According to Massarsch & Fellenius (2008) the main limitation of empirical energy-based prediction models are the notion that the driving energy governs the ground vibrations, the exclusion of geotechnical conditions and the uncertainty in the input values. Svinkin (1996) and Hope & Hiller (2000) also draw the conclusion that prediction models not taking soil conditions into consideration are less accurate than prediction models taking soil conditions into account.

According to Selby (1991) the problem of ground vibrations caused by pile driving should not be approached analytically due to the complexity of the problem, resulting from the imprecision of the pile driving equipment and the inhomogeneity of the ground conditions, among other reasons. Therefore Selby (1991) recommends an empirical approach.

2.8 PREVIOUS FIELD STUDIES

This section presents main results and conclusions from field studies previously published.

2.8.1 Vibratory driven piles

2.8.1.1 Clough & Chameau (1980)

During measurements by Clough & Chameau (1980) the typical acceleration record from a vibratory driving showed a steady state response with a frequency that was practically equal to that of the vibrator. Measurements were performed at various distances from the sheet pile and a distribution of peak acceleration by distance was established. Within 3-5 m of the sheet piles the measured ground accelerations are in the range of 0.15g-0.30g for hard driving and 0.10g-0.15g for normal driving. The accelerations are shown to decrease rapidly with distance from the pile. At 12 m from the pile the ground accelerations were measured to about 0.08g and at 30 m to about 0.02g or less. At another test site, both the horizontal and vertical accelerations near the pile were in the range 0.4g-0.5g for hard driving, and around 0.2g-0.3g for normal driving. As at the other location, the acceleration magnitude decreased rapidly with distance.

By comparing field data from sheet pile driving with the common attenuation relationship (Eq. 2.13), Clough & Chameau (1980) showed a good correlation using α = 0.03/ft. From the comparison they draw the following conclusion regarding the absorption coefficient, α:
1. Soft clayey soils have higher values of α than denser, firmer soils, which indicate greater vibration attenuation with distance from the source.
2. Higher absorption coefficient is given for hard driving than for normal driving. This believes to be due to the fact that during hard driving higher strain levels are induced leading to more material damping.
3. There is not much difference in α-values for horizontal and vertical vibrations.

2.8.1.2 Athanasopoulos & Pelekis (2000)

Athanasopoulos & Pelekis (2000) performed ground vibration measurements during the driving of sheet piles at nine sites in Patras, Greece. The sheet piles were driven with vibratory hammers and measurements (geophones in three orthogonal directions) were conducted on the surface of pavements at varying distances from the installed sheet pile. Measurements were also done on ground floors in adjacent buildings, and at higher floors in most of the buildings.

Results from the measurements showed that in almost all cases the maximum value corresponded to the vertical component of vibration. When comparing the vibration intensities measured in their study to other studies reported in literature, they measured lower values, which they believe is due to favourable soil conditions in Patras.

From their measurements Athanasopoulos & Pelekis (2000) reconstructed the particle displacement paths at each measurement point. From the particle displacement paths Athanasopoulos & Pelekis (2000) observed that vertical vibrations of the Rayleigh type were the most common.

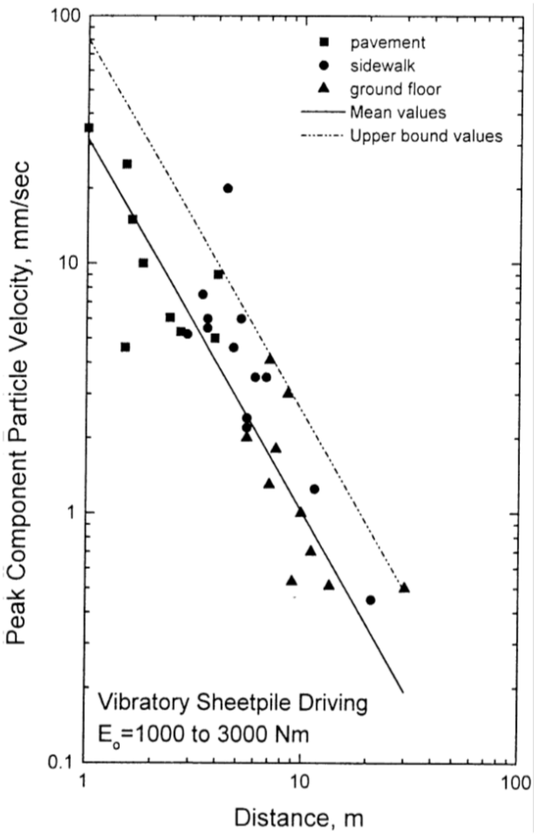

Figure 2.44 Results from ground vibration measurements (Athanasopoulos & Pelekis, 2000).

The results from all their measurements are plotted in Figure 2.44, with peak component velocity on the y-axis and distance on the x-axis.

The measurements of vibration levels in buildings showed an amplification of vibration levels at elevated floors, believed to be due to excitation of the building floors' natural frequencies. The vibration was amplified for each floor (up to 7 floors), however, the rate of increase of the amplification ratio decreased for each floor indicating that there might be a deamplification at higher floors in high-rise buildings.

2.8.1.3 Borel et al. (2002)

Borel et al. (2002) measured vibrations from vibratory driven piles in Montoir in France. The data were of a complex nature. Transverse vibrations were most important at a distance of 6 m from the pile, where they exceeded 15 mm/s, while the vertical velocities did not even reach 4 mm/s at that distance. However, at 12 and 18 m from the pile vertical velocities were predominant.

Borel et al. (2002) investigated resonance frequencies for the slender pile (ϕ339 mm) used in their field study and found that the resonance frequencies for lateral movement are much lower (1-17 Hz) than for vertical movement (50-150 Hz). This is believed to explain the high horizontal vibrations that were measured as it is possible that the driver's frequency repeatedly matched the resonant lateral pile frequency, and hence caused horizontal vibrations to be transmitted to the soil.

From measurements Borel et al. (2002) noticed that peak velocities were higher during driving through the upper sandy strata. When the pile reached the underlying sand and clay layers, the vibrations decreased by a factor of 2. At the end of driving vibration levels were seen to increase again as the driving speed decreased.

2.8.1.4 Ahlqvist & Enggren (2006a and 2006b)

Ahlqvist & Enggren (2006a and 2006b) performed field measurements at two different sites in Sweden during the installation of vibratory driven sheet piles.

The results from the measurements at test site A (Norrköping) showed that during the installation of one of the sheet piles the measured vibration frequency shifted between 20 and 40 Hz, with a four-fold increase in maximum particle velocity when the frequency shifted from 40 to 20 Hz. Ahlqvist & Enggren could not find any explanation for this phenomenon; the driving frequency from the vibrator was 40 Hz during the entire driving, even when the frequency of the system shifted to 20 Hz. The phenomenon was only seen during the driving of one of four sheet piles.

The measurements made by Ahlqvist & Enggren (2006a) showed that the vibrations from the sheet pile installation never reached the geophones and seismometers placed more than 40 m away from the sheet pile.

Ahlqvist & Enggren (2006a and 2006b) concluded that maximum vibration occurs when the penetration speed of the sheet pile is low. They also receive high vibrations when the driving frequency is low during the start-up and shut-down of the vibrator. They state that the best solution to avoid high vibrations in the surrounding environment is to give the vibratory driver operator real-time information about the vibrations during the sheet pile installation.

2.8.1.5 Whenham et al. (2009) and Whenham (2011)

On a test site in Limelette in Belgium, Whenham, along with other researchers, performed measurements on vibratory driven piles in 2007. Measurements of vibrations in the surrounding soil consisted of soil particle velocity at the surface and at depth using SCPT-equipment.

In Figure 2.45 the influence of the driving power on the vertical particle velocity is shown. From Figure 2.45 it can be seen that for penetration depths less than 6.1 m, the particle velocity increased as power increased, while it remained constant or even decreased for penetration depths greater than 6.1 m.

Whenham (2011) reported that for all the performed tests the acceleration and displacement amplitudes were lower at the pile head than at the pile toe.

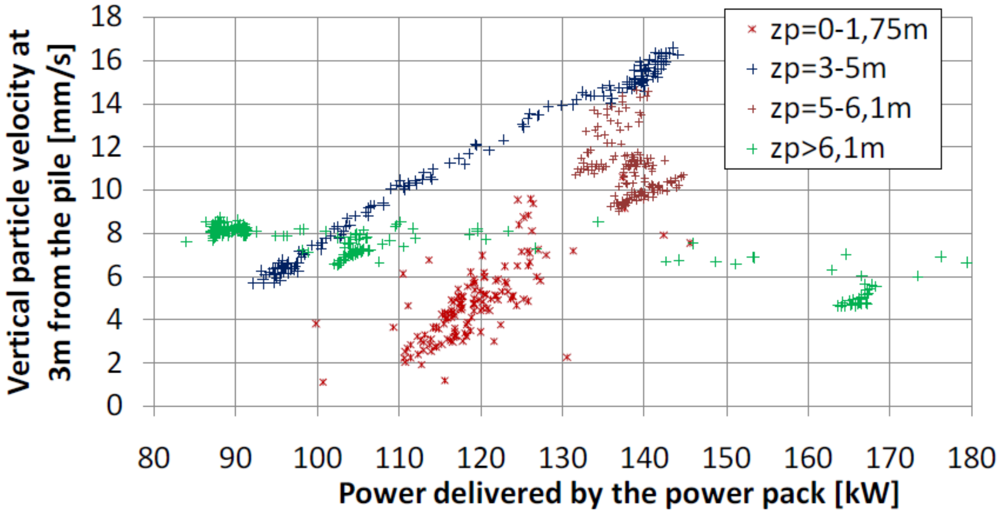

Figure 2.45 Vertical particle velocity as a function of power developed by the power pack (Whenham, 2011).

The results of the study indicated high horizontal vibrations, in most cases higher than the vertical vibrations. Whenham (2011) also showed results of the influence of the clamping device on the induced horizontal vibrations. For cases in which a single clamp was used (holding the sheet pile eccentrically) vibration levels were higher than for the cases in which the double clamps were used (holding the sheet pile in the neutral axis). The high horizontal vibrations observed in the project overall were believed to be explained by the driving frequency matching the natural frequency for lateral movement of the pile.

As the piles reached a more resistant soil layer at 4-5 m depth a sharp increase in the measured ground vibrations at the surface 3 m from the pile were seen. Attenuation curves with soil particle velocity plotted against distance to pile all showed a monotonic decrease with distance for all penetration depths.

From particle velocity paths Whenham (2011) concluded that at a short distance from the pile the velocity has an elliptical shape similar to the R-wave motion. At a greater distance the velocity path showed a predominantly radial movement. The results in the study also showed that the lower the driving frequency, the higher the vertical soil vibrations. The trend was less clear for the horizontal vibrations.

Whenham (2011) presented results from measurements of lateral vibration performed on the sheet pile during driving. As the accelerometer on the sheet pile reached the surface of the earth and started to penetrate the soil, a drop in the lateral acceleration amplitudes was seen. This is believed to be due to the confining pressure holding the sheet pile in place when penetrating into the soil.

2.8.2 Impact driven piles

2.8.2.1 Alpan & Meidav (1963)

Alpan & Meidav (1963) studied the vibrations outside and within buildings caused by driving piles in the vicinity. The results showed that the greatest particle velocity outside the building was attained immediately after each driving impact, while the greatest particle velocity inside the building was measured 350-400 milliseconds after the arrival of the impact waves. They also saw that the vibration record within the building could be separated into two different phases. The first phase started when the impulse arrived and lasted for about 0.2 s; the acceleration and frequency in the first phase were high with a relatively low kinetic energy. The second phase included lower frequencies with high particle velocities and lower accelerations.

2.8.2.2 Brenner & Chittikuladilok (1975)

In a paper by Brenner & Chittikuladilok (1975) vibrations from impact pile driving were evaluated for two sites in the Bangkok area. Measurements of vibration were performed on the ground surface, at three different depths below the ground surface and on adjacent buildings.

Some interesting results were found from the measurements; among others a sudden decrease in vibration was seen when the pile tip moved from the uppermost sand fill layer into the soft clay below. Then when the pile tip reached a fine sand layer further down in the ground the vibration increased and the maximum peak particle velocity was reached. An increase in vibration was also seen as the pile penetrated into a stiffer clay layer with greater penetration resistance. However, a distinct increase was only observed at surface points further away from the pile. Brenner and Chittikuladilok (1975) believed that a reason for this could be that at measurement points close to the pile ($r < 6$ m) the maximum vibration is not caused by waves originating from the pile tip but from waves having their source higher up along the pile in the soil profile. Then at greater distances, waves from the pile tip have become more dominant and give the largest amplitudes. It is also believed that they are probably composed of body waves radiated from the pile tip, and to some extent, by surface waves.

From their measurements Brenner & Chittikuladilok (1975) stated that the radial and tangential vibration component had values between 30-80% of the vertical component.

Brenner & Chittikuladilok (1975) also performed measurements of vibration levels at depths of 1.75 m, 3.25 m and 4.75 m below the surface. The results show that at a depth of 1.75 m the surface and subsurface vibrations were almost equal. At the other depths the subsurface vibration was always less than the surface vibrations. They explained this by the fact that surface wave amplitudes decrease rapidly with depth.

The results showed that the ground motion frequencies caused by the pile driving ranged between 12-25 Hz for the vertical component and between 28-33 Hz for the radial and tangential component. There were no correlations seen between frequency range and type of soil layer being penetrated.

Measurements within buildings showed that only about 30% of the ground surface vibrations were transferred to the walls of the building. For most measurements the radial and tangential component showed values less than 40% of the vertical vibrations. However, for a staircase in one building the tangential, but not the radial, component had the same order of magnitude as the vertical component. The measurements also showed that some vibration amplification with respect to the ground slab took place, though motion was still less than for the ground surface. The only structural member of the buildings which showed considerable amplification was the staircase. This amplification was probably due to resonance effects.

2.8.2.3 Brenner & Viranuvut (1977)

Brenner & Viranuvut (1977) performed vibrations measurements during pile driving at a site north of Bangkok, Thailand. Vibration measurements in the vertical direction were performed on the ground and on an adjacent building. Brenner & Viranuvut (1977) also performed Dutch cone soundings with the aim of finding a correlation between sounding results and recorded vibrations.

Their results showed that the vibration velocity appears to vary according to the cone resistance; however, the correlation is fairly weak. Brenner & Viranuvut (1977) pointed out that for layers of equal cone resistance the depth of the layer also influences the vibration level. No correlation was found with the other parameters studied (local friction, total friction and energy per meter of pile penetration).

2.8.2.4 Heckman & Hagerty (1978)

Heckman & Hagerty (1978) presented results from vibration measurements during impact pile driving of pipe piles. The results indicated that the peak particle velocity was directly related to blow count. They also showed that as the scaled energy decreased with increasing distance to the pile, the *PPV* decreased. However, as the piles were driven deeper an increase in blow count was not readily seen in measurements at the ground surface. This is explained to be due to the fact that a sand layer was present, damping the vibrations.

Heckman & Hagerty (1978) also studied results from impact driving of H-piles. The results showed that higher peak particle velocities were obtained from the driving of 305 mm H-piles than from the driving of 350 mm H-piles.

2.8.2.5 Ciesielski et al. (1980)

Ciesielski et al. (1980) presented results from ground vibration measurements during impact driving of casing tubes and vibratory driving of L-piles. Here results and conclusions from the impact driving are presented.

The results showed no correlation between the drop height of the hammer and the frequency of the measured vibrations. However, the results showed that there was a considerable change in the displacement amplitude, A_{max}, with increasing hammer drop height. The results showed an increase in A_{max} with decreasing penetration per blow. This is supported by the theory that the smaller the plastic penetration per blow of the hammer (with constant drop height), the greater the quantity of energy transferred into the soil and inducing ground vibrations. The penetration per blow in turn depends on the soil resistance, which gives the conclusion that the vibration is dependent upon the soil resistance (Ciesielski et al., 1980).

2.8.2.6 Martin (1980)

Martin (1980) used piezoelectric accelerometers for the measurement of vibrations induced by impact driving of sheet piles and steel case piles. The data showed that the main vibration component was sinusoidal. The conclusion from the results was that the measured vibration level depends on the presence of any subsurface layers and on the type of pile being driven. It could clearly be seen that when the pile entered a gravel layer at a case piling site, a much larger transverse vibration was seen, while the vertical and longitudinal components were basically unaffected by the change in soil type. Regarding the pile type, they concluded that case piles, with a circular cross-section, generated significant horizontal vibrations, whereas sheet piles produced predominantly vertical vibrations.

Martin (1980) also did measurements inside nearby buildings. The results showed that the ratio between the inside and outside vibration levels was around 0.5 in buildings with concrete floors, and around 1.6 in buildings with wooden floors.

2.8.2.7 Nilsson (1989)

Nilsson (1989) performed measurements of vibration in the ground due to impact driving of different pile types in Skövde in Sweden. Both concrete piles and steel piles were driven through a relatively hard upper layer (compacted sand) followed by 10-15 m of clay. The results showed that maximum vibration levels were measured when the piles passed through the hard upper layer.

Frequency analysis showed that the vibration signals have a frequency range of 0-60 Hz. The dominating frequency for the Rayleigh wave was between 8-15 Hz and the dominating frequency for the shear wave was between 30-50 Hz.

The field study showed that impact driving of concrete piles generates higher vibration values than impact driving of steel piles. The results also showed that the steel pile driving with a 40 kN hammer caused less vibration than the pile driven with the 15 kN's hammer. According to Nilsson (1989) this is due to the fact that the 40 kN hammer with low drop height transferred less energy into the pile.

2.8.2.8 Whyley & Sarsby (1992)

Whyley & Sarsby (1992) presented results from vibration measurements during impact driving of sheet piles and concrete piles at three different sites.

At site A, where sheet piles where driven, results showed generally lower values for vertical vibrations than for horizontal (radial and transversal). Both air and diesel hammers were used. However, there was no recognisable difference in the vibration level between the two hammers. The vibrations levels did not seem to differ with depth nor be any higher during final driving to refusal. This was also observed at site C, where concrete piles were driven.

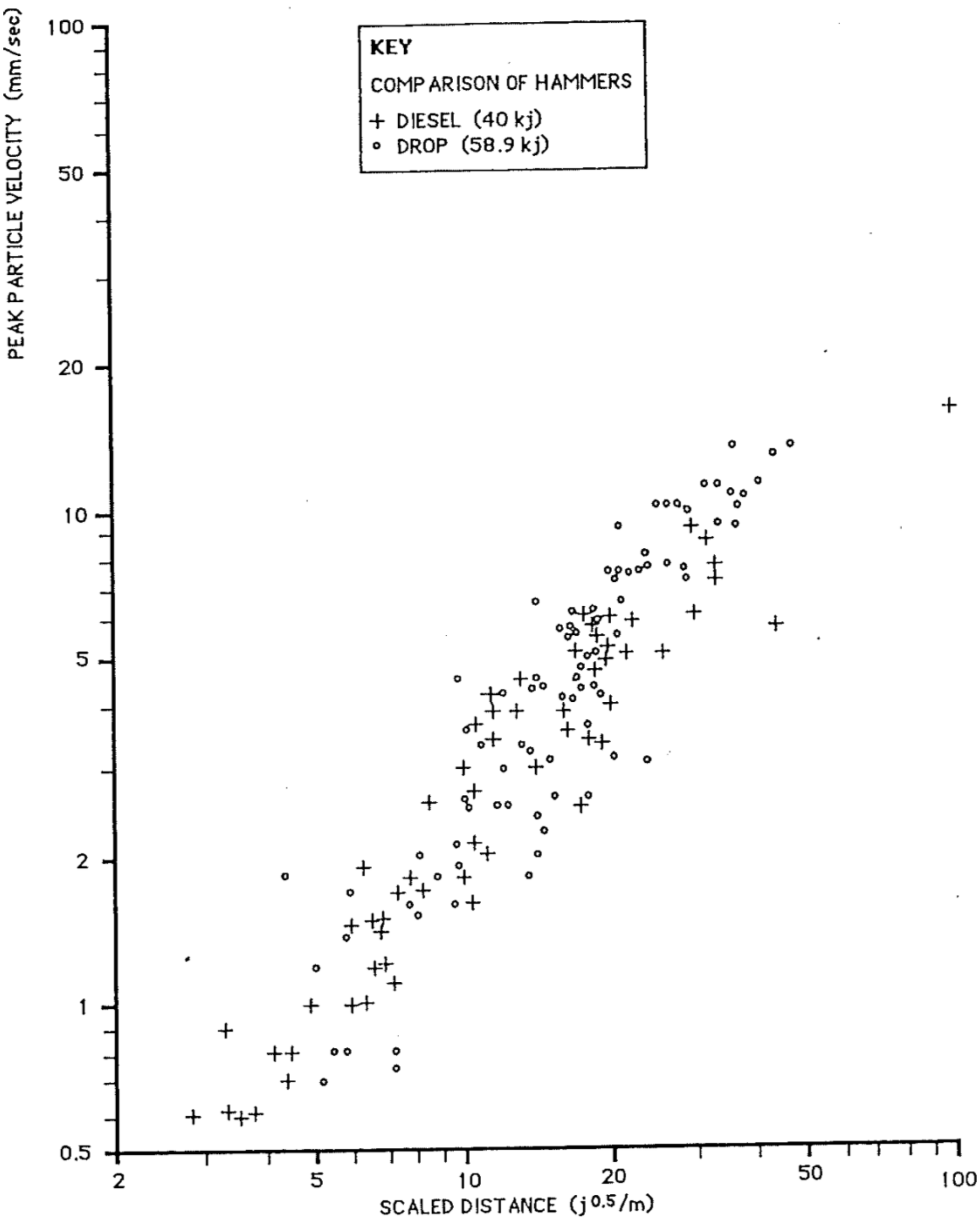

Figure 2.46 Data for diesel and drop hammers at site B (Whyley & Sarsby, 1992).

At site B, measurements where conducted on the ground as well as on and inside structures. Results indicated that the vibration levels on the structures were similar to the values outside on the ground. However, for some cases values on floors and ceilings were higher, probably as a result of long floor spans. A comparison was made between diesel and drop hammers, (see Figure 2.46), showing no difference between hammer types.

At site C, where concrete piles were impact driven, the horizontal components in the radial and transversal direction were higher than the vertical component.

2.8.2.9 Jongmans (1996)

Jongmans (1996) presented results from a field study in which ground vibrations from four different pile driving techniques were measured. All piles were impact driven with impact at either head or toe. Measurements were conducted using both vertical and tri-axial geophones at different distances from the pile.

Figure 2.47 shows results from the measurements both in the vertical and in the radial direction. The attenuation for the vertical component was linear, while the radial component showed a peak at a distance of about 10 m from the pile.

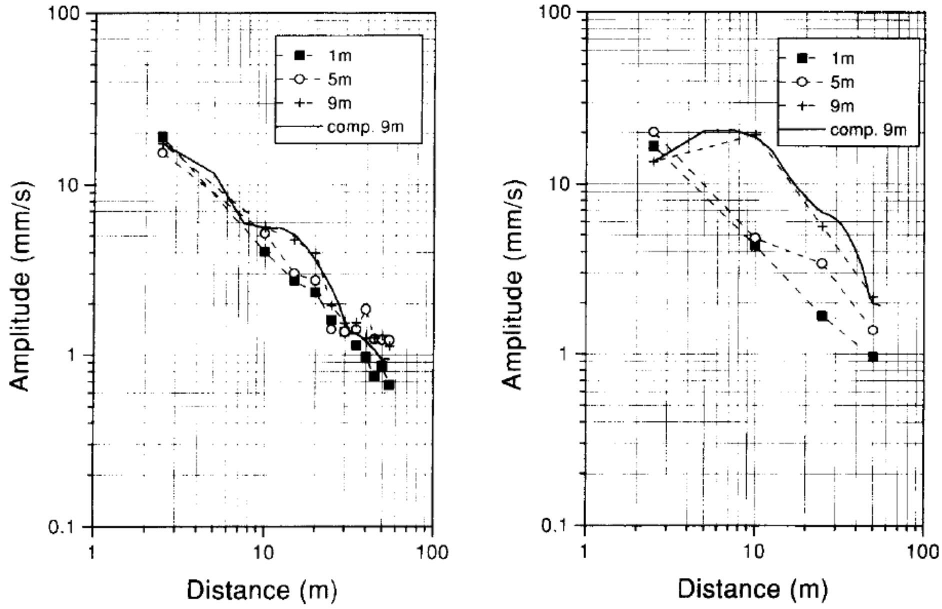

Figure 2.47 Measured particle velocity attenuation with distance for three pile toe depths. Left: vertical component and right: radial component (Jongmans, 1996).

2.8.2.10 Hope & Hiller (2000)

Hope & Hiller (2000) have analysed field measurements of ground vibrations from impact pile driving at several different sites in the United Kingdom. In some of their analysis they have included results presented by Uromeihy (1990).

Figure 2.48 and Figure 2.49 show that peak resultant velocity, v_{res}, was not linear to the horizontal distance, r, near the pile. Instead v_{res} showed a more linear behaviour when plotted against slope distance, s, near the pile. From the results in Figure 2.49 Hope & Hiller (2000) proposed the use of s rather than r over the whole range as s gives a good linearity throughout the range.

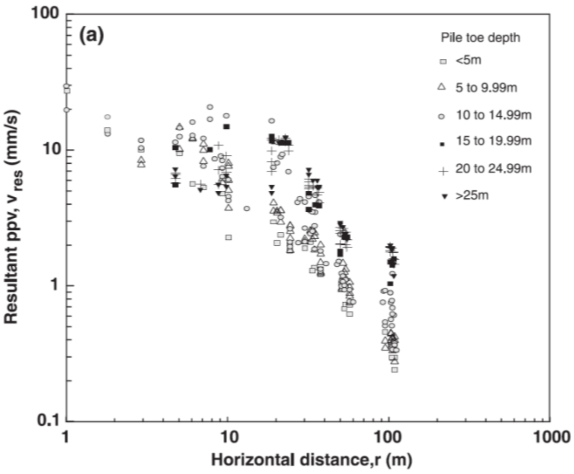

Figure 2.48 Ground vibrations from impact pile driving as peak resultant particle velocity against horizontal distance (Hope & Hiller, 2000).

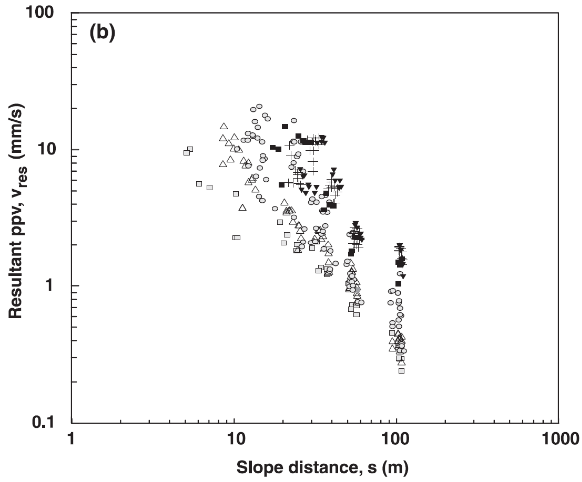

Figure 2.49 Ground vibrations from impact pile driving as peak resultant particle velocity against slope distance (Hope & Hiller, 2000).

Hope & Hiller (2000) also showed that there was no linear relationship between the resultant peak velocity and the potential energy of the hammer mechanism. This statement contrasts the assumptions made in most empirical prediction models. From their many measurements, Hope & Hiller (2000) concluded that empirical prediction models involving only distance and hammer energy have very little chance of reliably predicting ground vibrations from impact pile driving.

From their measurements, Hope & Hiller (2000) saw that *PPV* increased with the embedment depth of the pile. This could be due to numerous factors: the length of shaft in contact with soil increases with increasing embedment, the properties of the soil may vary along the shaft and the pile toe may hit soils with different properties.

2.8.2.11 Kim & Lee (2000)

Kim & Lee (2000) performed vibration measurements during impact pile driving of steel pipe piles in Pusan, South Korea. They measured both at the ground surface and at a depth of 15 m. Their results showed that the peak particle velocity at the surface decreases as the pile tip penetrated down in the ground. At a certain horizontal distance from the pile the magnitudes of the vertical motion measured at the ground surface were almost identical to the measured vibration at 15 m depth.

The measurements also showed that most of the energy was transmitted by vertical motion with frequencies below 10 Hz.

According to Kim & Lee (2000) the particle motions measured were mostly in the vertical direction and from that they conclude that vibrations due to impact pile driving can be characterized as vertical shear waves with conical wave fronts. Because of this, the vibration source can be classified as a point source generating body waves and the travel distance can be estimated as the horizontal distance from the source.

2.8.2.12 Hwang et al. (2001)

Hwang et al. (2001) studied the ground response during impact pile driving of concrete piles by measuring pore water pressure, ground deformation and vibrations during the driving of three concrete piles at a site in Taiwan. The vibrations were recorded during 10 s at each meter of penetration.

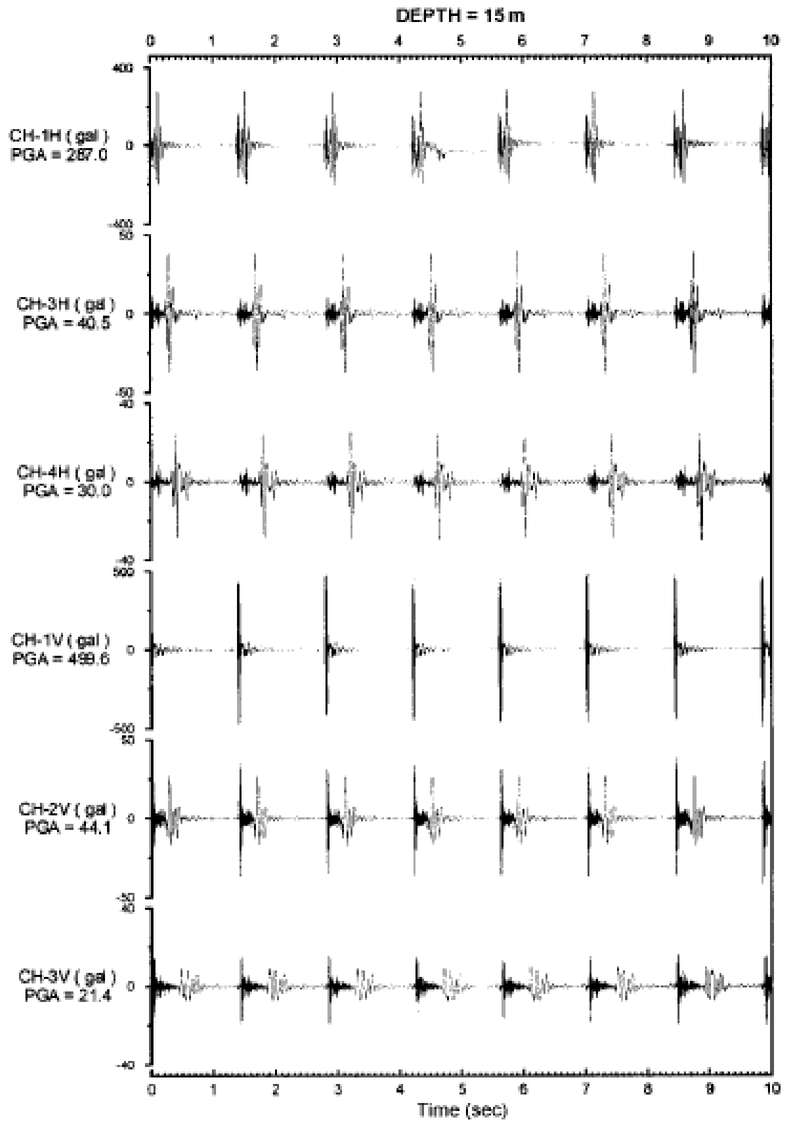

Figure 2.50 Acceleration time histories during driving of pile DP3 at a pile penetration depth of 15 m (Hwang et al., 2001).

Hwang et al. (2001) concluded that the vibration from the impact pile driving was of a high frequency and had a period of less than 0.5 s. However, it is noticed that the wave trace starts off at a high frequency, see Figure 2.50, after which the frequency becomes lower. This is explained as the high frequency being the body waves followed by the lower frequency surface waves. The pattern was more discernible at larger distances from the pile; at 5 m it was difficult to make a distinction.

The vertical vibrations were higher than the vibrations recorded in the radial horizontal direction. The results also indicated that the peak ground acceleration for the surface waves was larger than the peak ground acceleration for the body waves.

2.8.2.13 Thandavamoorthy (2004)

At a site in Chennai, India, Thandavamoorthy (2004) executed vibration measurements during impact pile driving of 600 mm diameter closed-ended steel casings in sand. Vibration measurements were performed on the ground and on an adjacent pile (driven earlier) using piezoelectric accelerometers. Measurements were performed at every 0.1 m of pile penetration for a duration of about 2 s.

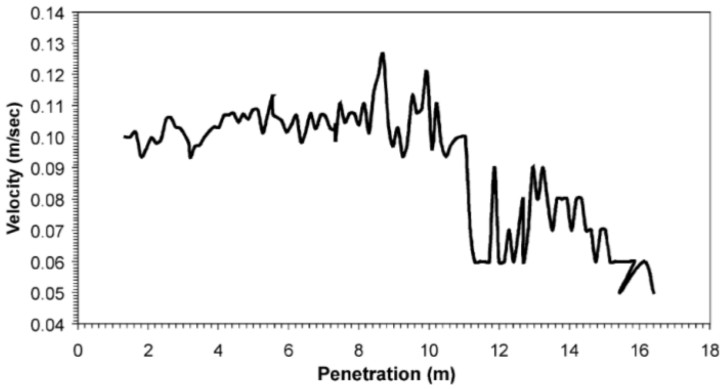

Figure 2.51 Vertical ground vibration velocity plotted against penetration depth 15 m from the driven pile (Thandavamoorthy, 2004).

Results indicated that as the soil got harder, the driving resistance increased and an increase in ground vibration was observed. At a distance of 15 m from the pile the maximum velocity was recorded as 126.2 mm/s, see Figure 2.51.

Measurements of vibration acceleration at the already installed pile at a distance of 6.25 m from the driven pile gave a maximum acceleration of 123.42 m/s^2, which is a value that could potentially harm the pile and reduce its bearing capacity. The frequency spectrum showed a frequency range of 0-500 Hz for measurements at the pile.

In Figure 2.52 vertical and horizontal accelerations are plotted against penetration depth at a distance of 3 m from the driven pile. The results indicated that the vertical vibrations were larger than horizontal vibrations. The frequency for the vertical vibrations in the ground ranged from 0 to 200 Hz and for the horizontal vibrations the range is 0 to 150 Hz.

From studying acceleration records at different depths of impact Thandavamoorthy (2004) concluded that body waves were predominant in the vertical acceleration while surface waves were predominant in the horizontal acceleration. Time histories for each blow showed a wave trace of high frequency in the beginning and a lower frequency for the following time. According to Thandavamoorthy (2004) the high frequency was primarily body waves and the lower frequency was the surface waves.

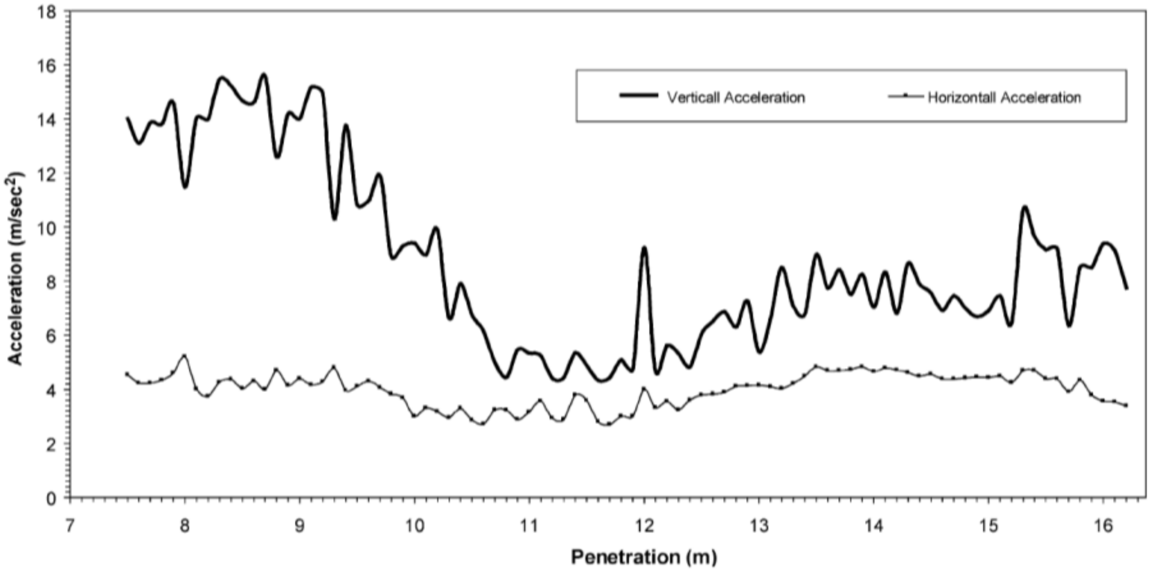

Figure 2.52 Vertical and horizontal vibration acceleration plotted against penetration depth 3 m from the driven pile (Thandavamoorthy, 2004).

3 FIELD STUDY – KARLSTAD THEATRE

A retatining structure was needed for a planned extension of Karlstad theatre, in the city of Karlstad in the western region of Sweden. The theatre building as well as several other buildings in the vicinity were constructed around the end of the 19th century and founded shallowly on dry rubble or raft foundations.

The theatre is located along the Klarälven river, which highly influence the geotechnical characteristics of the site. The upper parts of the soil mainly consist of loose, fine-grain river sediments, mostly sand. Below around 8 m from the ground surface, the sand transitions into loose silt followed by a stiffer sand layer. Below the sand/silt, a clay layer continues to a depth of about 25 m. The groundwater level corresponds to the water level in the river, approximately 3 m below the ground surface.

To investigate the possibility of using a vibratory driven sheet pile wall, a trial sheet piling was undertaken. The trial sheet piling was executed on May 4, 2010 and included the driving of four sheet piles. Measurement of ground vibrations were performed during the driving of the last three sheet piles. The piles were driven to a depth of about 11 m.

The ground vibrations during the sheet pile driving were measured using two tri-axial geophones and one uni-axial geophone. The tri-axial geophones were connected to a recorder, which were able to record an event at a sampling rate of 750 Hz during 70 s. The uni-axial geophone was connected to data acquisition equipment recording only maximum values. The measurement and recording equipment was supplied by Bergsäker AB.

The geophones were positioned 3.4 m, 7.9 m and 15 m respectively from the sheet pile line. In the two closest measurement points, velocity was measured in three directions (vertical, transversal and longitudinal) while only the vertical direction was measured in the third measurement point.

For presentation, analysis and discussion of the results of the field measurements the reader is referred to the work presented in Lidén (2012) and the third appended paper (Paper III).

4 SUMMARY OF APPENDED PAPERS

4.1 PAPER I

Ground vibrations due to pile and sheet pile driving – prediction models of today

Deckner, F., Viking, K. and Hintze, S. (2012). Ground vibrations due to pile and sheet pile driving – prediction models of today. In *Proceedings of the European Young Geotechnical Engineers Conference* (Wood T and Swahn V (eds)). Swedish Geotechnical Society, Gothenburg, Sweden, pp. 107-112. *Peer-reviewed conference paper*.

As part of construction work, pile and sheet pile driving unavoidably generates vibrations. Construction works today are often located in urban areas and along with society's increasing concern for environmental impact, the need for predicting vibrations before construction is of immediate interest. This study presents a review of the prediction models existing today. For prediction of ground vibrations from pile and sheet pile driving there are roughly three different types of models: empirical models, theoretical models and engineering models. A prediction model should be reliable in all cases in which it is meant to be used. It is also important that it is relatively easy to use and that the input data is easily obtained. This study concludes that, as of today, such a model is lacking. Today's models either lack in reliability or require great amounts of input data, knowledge and skills as well as time and money.

4.2 PAPER II

Factors influencing vibrations due to pile driving

Deckner, F., Viking, K. and Hintze, S. (2013). Factors influencing vibrations due to pile driving. Submitted to *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* in December 2012. *Journal paper.*

Vibrations due to pile and sheet pile driving are part of a complex process involving many elements and factors that influence both vibration magnitude and frequency. Better understanding and prediction of the vibrations generated will greatly benefit the civil engineering practice as well as the construction industry. An important component in understanding vibrations due to pile driving is to comprehend and recognise the factors that influence these vibrations. The objective of the present study is to identify factors that influence vibrations caused by pile driving. Furthermore, current models for prediction of vibrations are discussed and evaluated. Based on the literature study conducted, it is concluded that the most important factors are the geotechnical conditions, the energy generated at the source and the distance from the source. The identified factors should be included in order to create a reliable prediction model for vibrations caused by pile and sheet pile driving.

4.3 PAPER III

Measured ground vibrations during vibratory sheet pile driving

Deckner, F., Lidén, M., Viking, K. and Hintze, S. (2013). Measured ground vibrations during vibratory sheet pile driving. To be submitted to *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* in March 2013. *Journal paper.*

An extension was planned for the old theatre building in the city of Karlstad in Sweden. The theatre was constructed in 1893 and several of the surrounding buildings are of the same age. The theatre is located along the river bank of Klarälven and the dry rubble foundation is placed on top of a layer of loose sand. The old buildings along with the complex soil conditions made environmental impact a current issue. The research described in this paper was undertaken to provide data for the decision whether vibratory driven sheet piles could be an option for the retaining structure. Ground vibrations were measured during a trial sheet piling using geophones. Analysis of the results suggests that vibratory driven sheet piles would cause large settlements in the loose sand layer. It was also concluded that geotechnical conditions as well as distance from the source have large impact on the generated vibrations. A comparison between measured vibrations and predicted vibrations using empirical relations gave valuable insights for the development of future prediction models.

5 CONCLUSIONS AND FUTURE RESEARCH

The following section is a summary of the major findings and conclusions of this study. In addition, proposals for future research are included.

5.1 CONCLUSIONS

An important part of understanding and predicting vibrations due to pile driving is being aware of the factors that influence the magnitude, shape and frequency of vibrations. From the literature study and the field measurements, it is concluded that the main factors influencing vibrations due to pile and sheet pile driving are the vibrations transferred from the pile to the soil, the geotechnical conditions at the site and the distance from the source.

The vibrations transmitted from the pile to the soil are dependent on the vibrations transferred to the pile from the hammer, the pile-soil interaction and the wave propagation and attenuation in the plastic/elasto-plastic zone closest to the pile. The hammer-pile interaction and the vibration transmitted there-in is rather clearly understood and described in literature. Therefore further research needs to be focused on clarifying the actions at the pile-soil interface. The size of the plastic/elasto-plastic zone and the transmission of vibrations there-in and to the elastic part of the soil warrant further research.

It is clear that the geotechnical conditions affect the vibration magnitude, shape and the frequency content of the vibration generated by pile driving. However, the literature study has shown that it is still under debate and unclear which factors in the soil have the largest influence or if it is a combination of several soil parameters. To reach the final aim of the research program it is necessary to perform future studies showing which soil parameters are necessary to incorporate in a future prediction model of vibrations due to pile and sheet pile driving.

The distance between the source of vibration and the point of interest largely affects the vibration magnitude. The difficulty here lies in deciding the “correct” distance to use in, for example, a prediction model. The horizontal distance is easy to use; however it might be conservative especially in cases in which the distance to the source is short and the vibrations are transmitted at the pile toe. That argument validates the use of the slope distance over the horizontal distance; however, since the source at the pile can be both shaft and toe the slope distance could differ considerably during the penetration of the pile. Therefore, in the process of developing of a new prediction model it must be clarified which distance gives the most correct and reliably predicted vibrations.

There are other factors that have been mentioned in literature to affect the vibrations, foremost in the horizontal direction. That is for example the way the pile/sheet pile is held (eccentric clamping generates eccentric vibrations) and interlock friction. Interlock friction and eccentric clamping was also observed in the field study. The importance of such factors are difficult to quantify, however, the knowledge of their effect is highly important for future design of pile and sheet pile driving works and driving equipment. Therefore, further research on the effect of these factors is warranted in future field studies.

From the literature study it can be concluded that a prediction model to a large extent depends on the input data. Therefore, a reliable and well-functioning prediction model needs to be based on data that is accurate and descriptive for the problem, for example regarding driving equipment, pile, and soil. It is also necessary that the input data is relatively easy to obtain without great costs or time since predictions should be performed at a relatively early stage in the construction process.

The review of the current prediction models for vibrations due to pile and sheet pile driving give reason to conclude that as of today there is no prediction model that fulfil the criteria of the "perfect" prediction model; reliable but yet eays to apply. The current empirical models are too unreliable and tend to highly overestimate the vibration levels. The current theoretical models require great amounts of input data, great user-knowledge and are usually relatively time consuming. The current engineering models all lack validation to measured vibrations in order to be considered reliable.

5.2 FUTURE RESEARCH

This study has highlighted research areas that would enhance the understanding of vibrations due to pile driving as well as creating a platform for a new model for prediction of these vibrations. Areas that need to be studied further are:

- The transfer of vibrations at the pile-soil interface, including the generation of a plastic/elasto-plastic zone in the area closest to the pile and how that affects the transfer of vibrations from the pile to the soil.
- The influence of different factors and parameters of the geotechnical site conditions on the vibrations generated by piling.
- The influence of "work related" parameters on the generated vibrations, such as holding the sheet pile eccentrically and interlock friction.
- Which distance to use between source and point of interest?

Within this research program the focus of future research will lie on the pile-soil interaction and the vibration transfer between pile and soil. Attempts will also be made to further investigate which parameters of the geotechnical conditions that are the most important for the generated vibrations and how these best can be incorporated into a prediction model. The future studies within this research project will mainly consist of new field tests and the analysis of their results.

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