

Vibrodriveability analysis on sandy soils

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Prologue

The actual development of offshore windfarms is increasing exponentially, and the actual projections shows that this trend will continue for at least 20 years. Mainly at the North Sea area but also Mexico, Brazil, Middle East, China and US. Making this offshore trend a global phenomenon.

The main type of offshore foundation are monopiles. They are currently driven into the soil by impact hammers. This kind of method can introduce high stresses into the pile, reducing its long-term life, provoke noise contaminations that environmental laws are aiming at, and, is a time-demanding installation.

Vibratory driving will be a major foundation installation technique, once it's mastered and fully understood. A cost-effective high-speed installation method with reduced noise contamination that ensures the long-term life of the pile.

On the negative side, the lack of field tests and reduced data base lead to an unpredicted behavior of vibrodriving, mainly refusal depth and penetration speed. This thesis will try to get one step further vibrodriving understanding.

Abstract

This thesis aims to put some light on this installation method. This phenomenon will be modelled using *Allwave-PDP*. A fully developed software from *Allnamics*, with proven reliability and 40 years of experience.

First chapter will describe the theoretical background on which vibratory analysis and simulations will be performed. Based on Stress wave theory and method of characteristics, measured stress waves will propagate through the pile interacting with the soil. Understanding this vibrated soil behavior will increase predictions accuracy.

Secondly, this thesis will target the unknowns, presenting, mainly, soil parameters. To define those parameters, formulas and correlations will be provided along with the influence that they could have within method of characteristics models.

So far, unpredicted response to vibrodriving and unstable vibrator performances leave vibratory driving analysis without a clear reference framework from which improve predictions. Based on previous chapter, a parametric analysis will be performed, along with a new methodology, to anticipate this irregular vibratory response.

Finally, this methodology will be tested with two real cases in order to confront it. As conclusion, this thesis will propose guidelines for vibrodriving and recommendations for further researches.

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To my Family and Guiomar for the motivation. Without their inspiration I would not have been able to do it.

“Life is the art of drawing sufficient conclusions from insufficient premises.”

Samuel Butler (1612-1680)

Key words

Monopile ; Offshore ; Vibrodriving ; Stress wave ; Cyclic loading ; Liquefaction ; Quake ; Damping ; Beta factor ; Frequency ; Static mass ; VDA ; VDP ; FPRS ; Amplitude Response

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List of parameters

Soil parameters

OCR	Pre-consolidation ratio	—
e_o	In situ void index	—
e_{cs}	Critical state void index	—
e_{min}	Minimal void index	—
e_{max}	Maximal void index	—
c	Soil cohesion	g
ϕ'	Friction angle	g
ϕ'_{cv}	Constant volume friction angle	g
ψ	State Parameter	—
D_r	Relative density	%
$\sigma_{v0'}$	Effective vertical pressure	kPa
σ_{vo}	Total vertical pressure	kPa
$\sigma_{ho'}$	Effective horizontal pressure	kPa
σ_{ho}	Total horizontal pressure	kPa
σ_{atm}	Atmospheric pressure	kPa
q_c	Cone resistance	MPa
f_s	Friction sleeve	MPa
R_f	Friction ratio	%
Q_{tn}	Normalized cone resistance	—
$Q_{tn,cs}$	Clean sand, normalized cone resistance	—
τ	Shear resistance	MPa
F_{Toe}	Toe resistance	kN
F_{shaft}	Shaft resistance	kN
$u_{q,toe}$	Toe quake	mm
$u_{q,shaft}$	Shaft quake	mm
F_{uy}	Soil yield stress	MPa
k	Soil yield stress	MPa/mm
C	Damping constant	Ns/m ³
β	Beta factor	—

Vibrator parameters

m_e	Eccentric moment	kg/m
f	Frequency	Hz
ω	Angular velocity	rad/s
F_{vib}	Maximal vibratory force	kN
M_{stat}	Static mass of the vibrator	kg
M_{dyn}	Dynamic mass of the vibrator	kg
M_{ec}	Mass of the exciter block	kg

M_{clamp}	Clamp mass	kg
T_{vib}	Vibrator period	s
T_{pile}	Pile period	s
$T_{pile,pen}$	Penetrated pile period	s
λ_r	Relative pile wavelength	—
$\lambda_{r,soil}$	Relative penetrated pile wavelength	—
a_{rel}	Relative acceleration	m/s^2
a, b, c	Optimization parameters	—

Pile Parameters

M_{pile}	Pile mass	kg
ρ_{steel}	Steel density	kg/m^3
L	Pile length	m
ϕ_{ext}	External pile diameter	m
ϕ_{int}	Internal pile diameter	m
χ_{ext}	External pile perimeter	m
χ_{int}	Internal pile perimeter	m
th	Pile width	m
A_{toe}	Pile toe area	m^2
F	Pile element force	kN
a	Pile element acceleration	m/s^2
v	Pile element velocity	m/s
u	Pile element displacement	mm
v_{pen}	Pile penetration speed	m/s
z_p	Pile penetration	m
ε	Deformation	mm
E	Young modulus	MPa
c_{steel}	Steel stress wave celerity	m/s

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I. Introduction

PILE DRIVING IS A DEEP FOUNDATION INSTALLATION TECHNIQUE THAT CONSIST OF TRANSFERRING FORCES FROM A HAMMER ON TOP OF THE PILE TILL THE SOIL SEABED, OVERCOMING ITS RESISTANCE AND ACHIEVING PENETRATION. THIS CAN BE DONE THROUGH IMPACT DRIVING, HIGH ENERGIES, OR THROUGH VIBRATORY DRIVING, HIGH FREQUENCIES.

1. General overview

a) Vibrodriving installation procedure

A vibrator is attached to the pile by a hydraulic clamp system, making a rigid connection between parts, shown Figure I-1. This dynamic system pile-vibrator is hoisted by a crane, and, vertically positioned to the required location.

Once the system is in a stable vertical position, the vibrator generates a sinusoidal force that is transmitted through clamp and pile to the soil. This cyclic process, loads the soil, reducing its resistance. The pile sinks into the soil by gravity forces that overcomes the degraded soil resistance.

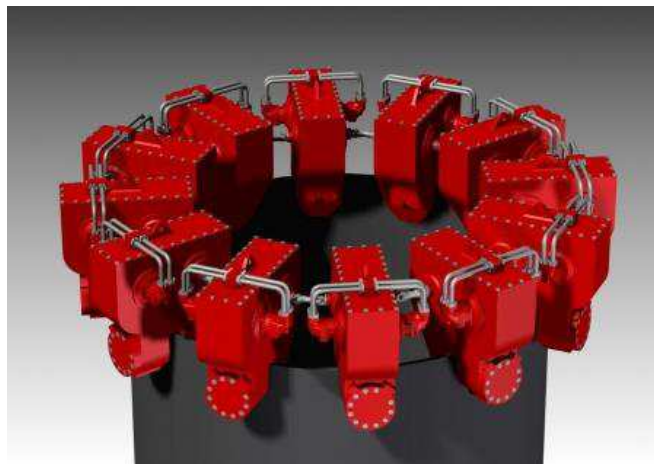


Figure I-1: Rigid clamp connection (Cape Vibro Hammers VLT)

Penetration continues until refusal depth is reached, where not enough forces reach pile bottom level (toe level) to overcome bottom soil resistance (toe resistance).

To avoid this refusal situation, some hammer operators vary hammer frequency trying to drive the pile deeper, sometimes successfully achieving it. That is the most interesting result on which this thesis is based on: being able to understand and predict the frequency influence on vibratory driving.



Figure I-2: Vibrodriving project test in Cuxhaven, (Allnamics BV, 2016)

Vibrators			Monopiles		
Frequencies	[Hz]	17-23	Diameters	[m]	4-8
Static mass	[ton]	42-150	Lengths	[m]	30-70
Dynamic mass	[ton]	119-235	Widths	[mm]	40-80
Eccentric moment	[kg.m]	640-1280	Weights	[ton]	100-1000

Table I-1: Offshore vibrators and monopiles common dimensions

b) Vibrodriving modelling basic assumptions

- An ideal vibrator is assumed, generating a purely vertical sinusoidal force at a steady frequency.
- Vibrator forces propagates through the pile and reach the soil following stress wave theory.
- One dimensional stress wave propagation is assumed, “forced”.
- Rigid body assumption is refused based on the rule of thumb proposed by (Viking & Holeyman, 2006) which is fulfilled for large offshore monopiles. Pile is therefore simulated in several discretized segments.

$$T_{vib}/4 = 1/4f_{vib} < T_n = 4L/c_{steel}$$

Equation I-1

c) Vibrodriving potential

Nowadays, the increase of energy demand is becoming an important issue, as countries are developing and global population is growing. Fossil fuels could run out in about 50 to 100 years, so energy transition will be crucial. As oil supply decreases, new offshore windfarms and oil platforms will be built. Future projections for offshore windfarms, only in the North Sea, ensure a continuous trend for the next 20 years at least. Competition will be crucial aiming at reducing costs.

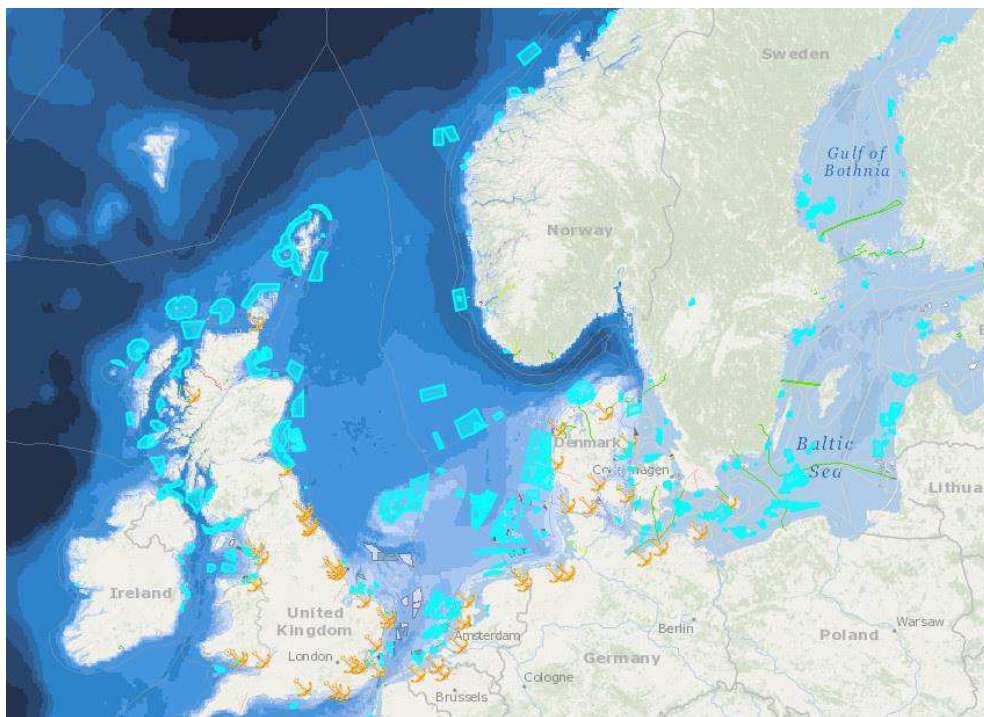


Figure I-3: Offshore Windfarm projections for North Sea, (www.4coffshore.com)

The main type of offshore windfarm foundation are monopiles, shown in Figure I-4, and represent approximately 25% of the construction cost as (Schaumann & Böker, 2005) state. Those costs can be optimized once vibrodriving predictions become more accurate.

<i>Issue</i>	<i>Description</i>
Efficient grid connection	To ensure a maximum amount of electricity delivered
Penetration depth/water level	As foundations go deeper, they become unpractical
Installation time	Expensive cost of construction vessels

Table I-2: Main offshore windfarm cost challenges

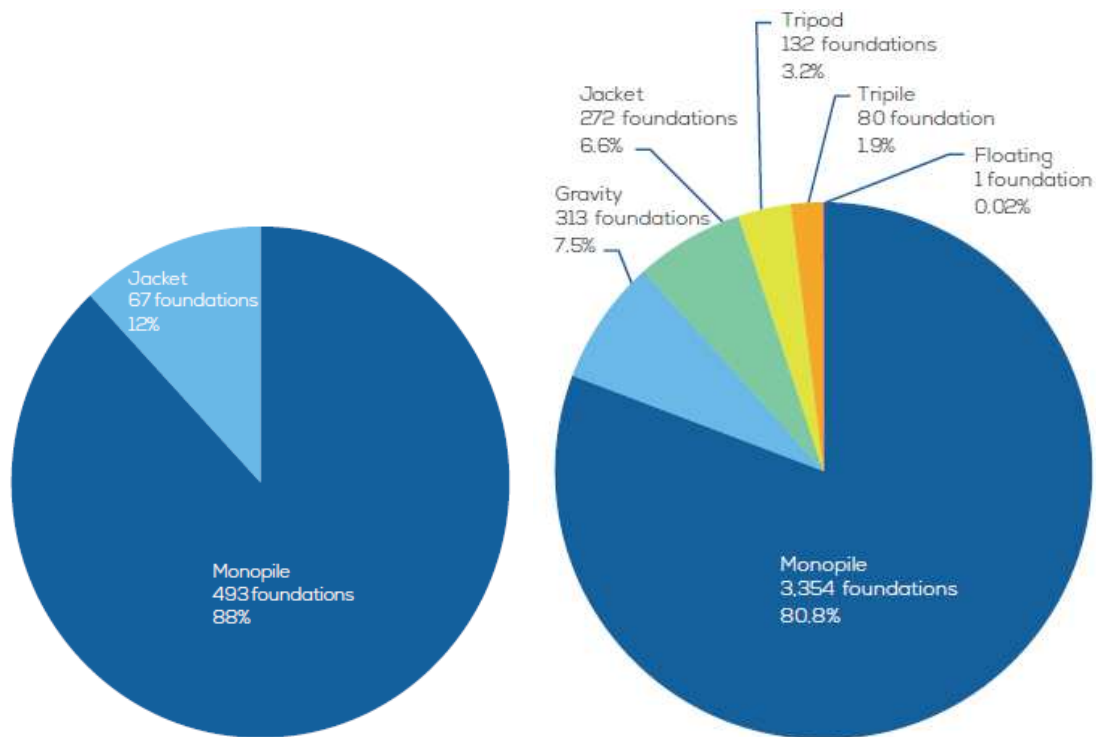


Figure I-4: Type of foundations built in 2016 (left), and cumulative foundations 2016 (right), (European Wind Energy Association, 2016)

Meanwhile, impact driving monopolizes monopiles installations. Widely used with a worldwide database, it is the preferred method and safest approach. However, vibratory driving bypasses impact driving inherent problems, summarized in Table I-3.

Advantages

Ensures long term life of the pile	=>	Doesn't induce high stresses into the pile
Lower noise contamination	=>	Minor vibrations during less time
High speed installation	=>	Cost-effective method
Pile repositioning	=>	Enables to correct pile inclination deviations
Faster handling procedure	=>	Serial installation

Disadvantages

Lack of field and laboratory tests	=>	Non-confronted theories
Non-disclosure agreements	=>	Difficulties to access the available data
Small data base	=>	Difficulties to establish patterns
Scatter of predictions	=>	Variable results
Method not yet understood	=>	Still to be developed

Table I-3: Vibrodriving potential regards impact driving

2. Thesis structure

d) Aim of the thesis

Improving vibratory driving predictions will be the aim of this thesis. The main vibrodriving unknowns are penetration speed of the pile (to assess driving times) and refusal depth (to assess whether the pile reaches the required bearing capacity).

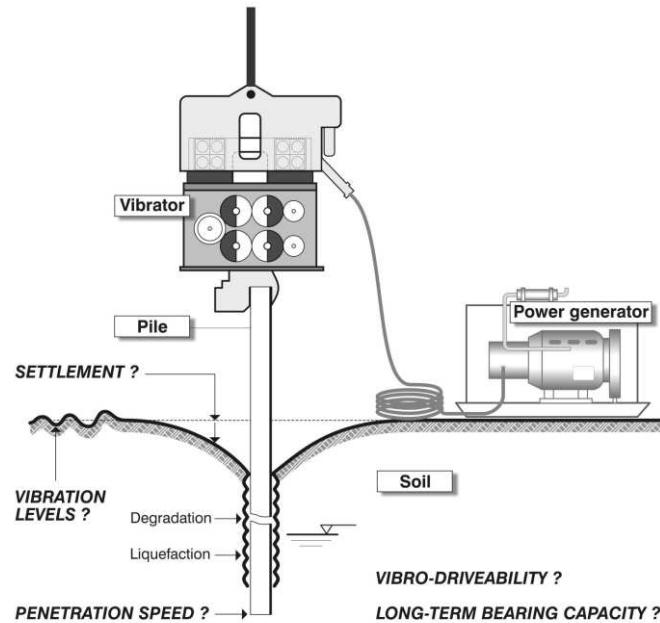


Figure I-5: Vibrodriving issues, (Holeyman, LeGrand, & Van Rompaey, 1996)

To improve predictions, the procedure hereunder is used. Future prediction models will be updated using new soil parameters. This will enable to quantify soil dynamic behavior empirically, increasing our current vibrodriving understanding.

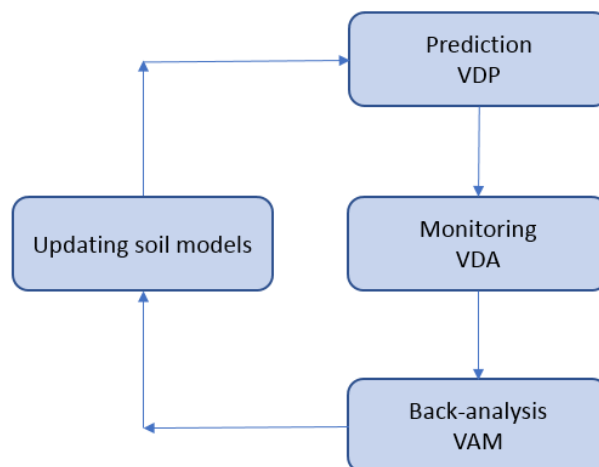


Figure I-6: Learning loop

e) Thesis Objectives

- ✓ Definition of vibrodriving main actors
- ✓ Definition of the frequency phenomena: Modes of vibration
- ✓ Understanding through simulations, soil parameters effects
- ✓ A methodology proposal to improve predictions
- ✓ A methodology proposal to match predictions with measured cases
- ✓ Suggestions for further research

f) Chapters description

Chapter II: Vibrodriving modelling

From general stress wave theory to particular soil and vibrator models.

Chapter III: Targeting the unknowns: Soil characterization

Characterizing vibratory soil behavior through main soil tests and parameters as well as their uncertainties.

Chapter IV: Implementations and tools for vibrodriving analysis

Definition and description of vibrodriving types of analysis, phenomena and new methodology.

Chapter V: Study cases analysis

Based on previous chapter, analysis confronting the new method with two study cases.

3. Glossary of main terms

Stress wave	Mechanical wave that propagates through a solid material. Wave velocity is determined by the material mechanical properties and it is constant for a homogenous material. ($c_{steel} = 5160 \text{ m/s}$)
Summation of effects (stress wave)	Specific interaction (add up of effects), of superposed stress waves among vibrator forces, mobilized soil resistances and soil reflections. This interaction defines the vibratory response at a defined pile level (location).
Toe resistance	Soil reaction acting on the transversal bottom area of the pile, superficially and equally distributed. Overcoming this resistance will allow penetration.
Shaft resistance	Soil reaction acting on the internal and external surface of the pile in contact with soil. Resistance that acts against pile movement, upwards and downwards.
Cyclic loading	Referred to soil behavior, this cyclic load will fatigue soil, degrading its resistance until a residual value.
Pore water pressure	Groundwater pressure exerted to soil particles. Two different types, hydrostatic and dynamic. Hydrostatic develops linearly with depth. Soil cyclic loading transfers to the groundwater dynamic pressure. This pressure is temporarily stored and will gradually dissipate ¹ completely.
Liquefaction	Occurs when a sandy saturated soil is cyclically loaded. Soil particles are prevented from contact between them, floating and rotating in a pressurized fluid. The soil then behaves as a fluid like state, which decreases soil resistance, even further than cyclic loading.
Cohesion	Interparticle force between grains. Prevents a soil to achieve liquefaction by maintaining contact between soil particles ² .

¹ The rate of pressure dissipation is determined by soil grain size.

² To not be confused with other contact force, friction, created by contact between grains, which is also present in sands.

	Clays	Silts	Sands
Upper bound Grain size	<0.063mm	0.063-	0.1-
Lower bound Grain size	/	0.1mm	2mm
Cohesion	Important	Medium	Null
Rate of water pressure dissipation	Long	Medium	Short

Table I-4: Characteristics of main soil types

Bearing capacity Load capacity of the soil applying forces to the pile.

Mode of vibration Referred to a vibrating system, phenomenon that amplifies/damps a response at a specific frequency and/or pile location.

Vibratory Driving Predictions (VDP) Prediction of the vibrodriving response, given a soil, pile and hammer data at a fixed frequency. Main output is the maximum achievable penetration, refusal depth.

Vibratory Driving Analysis (VDA) Monitoring vibratory pile installation through strain and acceleration sensors placed at the pile head. Enables to check hammer performance and stresses on the pile.

Vibratory Amplitude Matching (VAM) Post-analysis based on VDA measurements, accelerations and stresses. Generates a realistic soil model matching monitored signals with synthetic ones. Resulting soil model can be used to determine e.g. bearing capacity of the pile.

Frequency-Penetration Response Spectrum (FPRS) Prediction of a main variable possible amplitude responses at a specific pile location: Forces, accelerations, velocities and displacements, from now on (F,a,v,u), within a frequency-penetration plane.

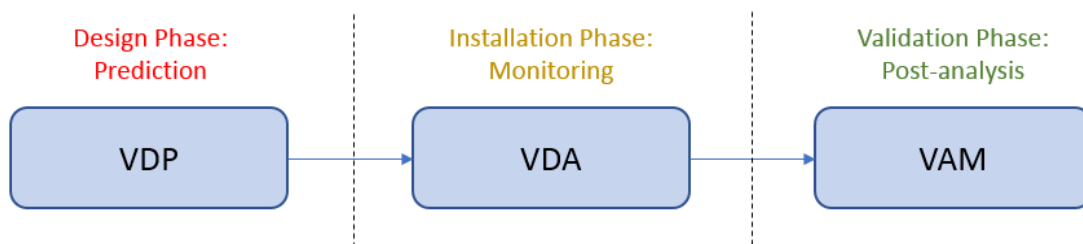


Figure I-7: Phases of a vibratory project

II. Vibrodriving modelling

HOW VIBRODRIVING IS MODELLED, FROM GENERAL STRESS WAVE THEORY TO PARTICULAR PILE, SOIL AND VIBRATOR MODELS.

1. Pile modelling: Theoretical background

IMPACT DRIVING HAS A LARGE DATABASE AND SEVERAL RESEARCHES THAT SUPPORT AND PROVE ACCURACY OF ITS MODELING. CREATED BY IMPACT, STRESS WAVES ARE MEASURED WITH SENSORS ATTACHED TO THE PILE. THOSE WAVES PROPAGATE FOLLOWING STRESS WAVE THEORY AND INTERACT WITH THE SOIL WITH THE METHOD OF CHARACTERISTICS SOLUTION.

VIBRATORY DRIVING FOLLOWS THE SAME THEORY. INPUT FORCE WILL BE, INSTEAD OF A HIGH ENERGY IMPACT, A CONTINUOUS SINUSOIDAL FORCE.

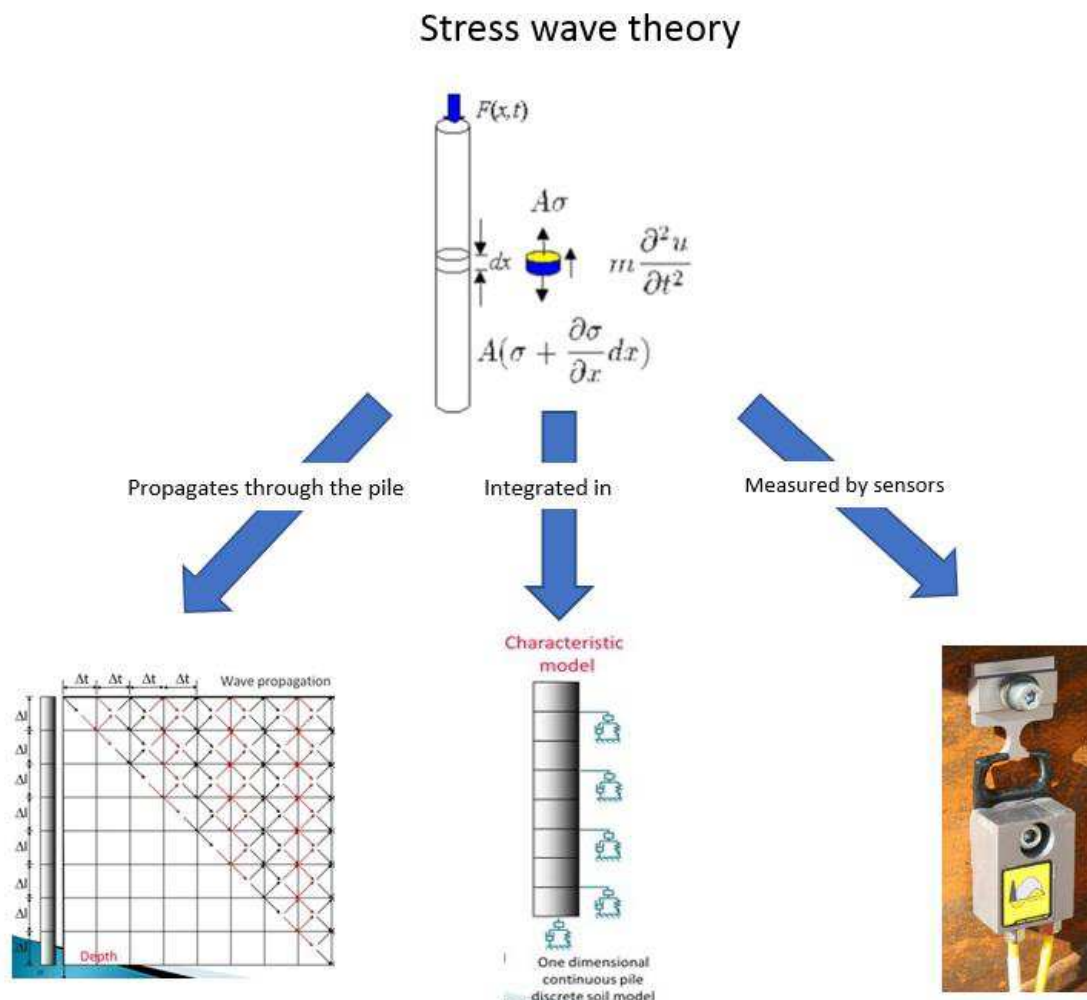


Figure II-1: Modelling vibrodriving through method of characteristics

a) Stress-wave theory

FROM AN INPUT FORCE, A STRESS WAVE WILL BE CREATED AND WILL PROPAGATE UNDISTURBED THROUGH A HOMOGENOUS MATERIAL.

There are some assumptions for this theory applied to piles:

- Discretizing the pile into several small segments enables to model how the stress waves propagates through the pile
- Undisturbed stress wave transmission within each pile segment
- Stress waves will propagate at constant speed related to pile mechanical properties, see *Equation II-8*
- As input forces are only vertical, and the pile has one dimension (length) dominating over two (width and diameter), stress waves are forced into one dimension, vertical one (length)

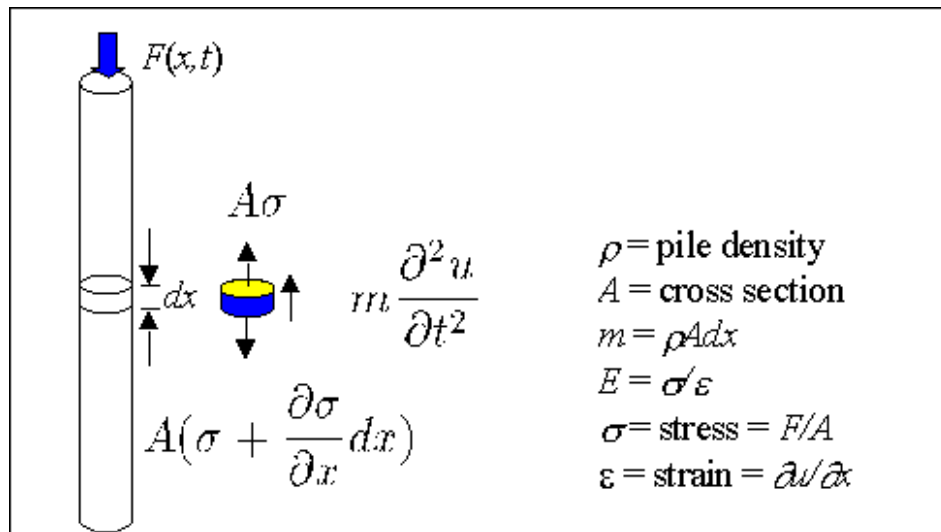


Figure II-2: Stress wave forces, (Van Delft, 2015)

From Figure II-2, we have the equilibrium of forces of an infinitesimal element caused by a stress wave:

$$A \left(\sigma + \frac{\delta\sigma}{\delta x} dx \right) = A\sigma + m \cdot a \quad \text{Equation II-1}$$

From Newton's law of motion, we have:

$$F = m \cdot a = \rho A dx \frac{\delta^2 u}{\delta t^2} \quad \text{Equation II-2}$$

From Hooke's law we have, in elastic regime³:

$$\sigma = E\varepsilon = E \frac{\delta u}{\delta x} \quad \text{Equation II-3}$$

Therefore, force can also be expressed as:

$$F = A\sigma = AE\varepsilon = AE \frac{\delta u}{\delta x} \quad \text{Equation II-4}$$

Replacing terms from Equation II-2 and Equation II-3 into Equation II-1:

$$EA \frac{\delta^2 u}{\delta x^2} dx = \rho A dx \frac{\delta^2 u}{\delta t^2} \quad \text{Equation II-5}$$

That can be rearranged as:

$$\frac{\delta^2 u}{\delta x^2} - \frac{\rho}{E} \frac{\delta^2 u}{\delta t^2} = 0 \quad \text{Equation II-6}$$

Or, the general form of stress wave equation:

$$\frac{\delta^2 u}{\delta x^2} - \frac{1}{c^2} \frac{\delta^2 u}{\delta t^2} = 0 \quad \text{Equation II-7}$$

With the stress wave propagation speed as:

$$c = \sqrt{\frac{E}{\rho}} \quad \text{Equation II-8}$$

b) Method of characteristics

IN 1860, A.J.C BARRE DE SAINT VENANT, APPLIED THE PRINCIPLES OF CONSERVATION OF MASS AND MOMENTUM TO THE WATER FLOW IN AN OPEN CHANNEL TO SOLVE THIS DIFFERENTIAL EQUATION. 70 YEARS LATER, THE SOLUTION WAS APPLIED TO PILE DRIVING.

The Method of Characteristics (MOC), applied by (Voitus van Hamme, Jansz, Bomer, & Arentsen, 1974), discretizes the solution into a finite number of stress waves following characteristics lines in the distance-time plane:

$$u = f^{\downarrow}(x - ct) + f^{\uparrow}(x + ct) \quad \text{Equation II-9}$$

The general solution, Equation II-9, considers that there are two waves travelling in opposite directions, upward and downward wave, shown in Figure II-3.

³ All vibrodriving stresses are below the yield stress of steel, therefore always acting in the elastic regime.

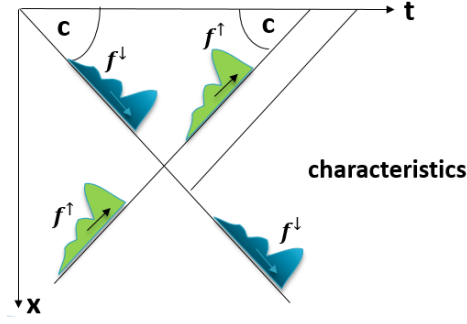


Figure II-3: Waves from method of characteristics, (Van Delft, 2015)

From Equation II-9, we can derive regarding time to obtain velocities:

$$v = \frac{\delta u}{\delta t} = \frac{\delta f^{\downarrow}}{\delta(x - ct)}(-c) + \frac{\delta f^{\uparrow}}{\delta(x + ct)}(+c) \quad \text{Equation II-10}$$

By convention we will have:

$$v^{\downarrow} = \frac{\delta f^{\downarrow}}{\delta(x - ct)}(-c) \quad ; \quad v^{\uparrow} = \frac{\delta f^{\uparrow}}{\delta(x + ct)}(+c) \quad \text{Equation II-11}$$

And also derive regarding displacement to obtain strain/stresses/forces:

$$F = -EA \frac{\delta u}{\delta x} = -EA \frac{\delta f^{\downarrow}}{\delta(x - ct)} - EA \frac{\delta f^{\uparrow}}{\delta(x + ct)} \quad \text{Equation II-12}$$

By convention, we will have:

$$F^{\downarrow} = -EA \frac{\delta f^{\downarrow}}{\delta(x - ct)} \quad ; \quad F^{\uparrow} = EA \frac{\delta f^{\uparrow}}{\delta(x + ct)} \quad \text{Equation II-13}$$

With the pile impedance (mechanical resistance to stress wave propagation) defined as:

$$Z = \frac{EA}{c} = A\sqrt{E\rho} \quad \text{Equation II-14}$$

We can regroup downward and upward forces:

$$F^{\downarrow} = Zv^{\downarrow} \quad ; \quad F^{\uparrow} = -Zv^{\uparrow} \quad \text{Equation II-15}$$

Therefore, by **measuring strains ($F = AE\varepsilon$) and accelerations ($v = \int a$) we can calculate how the stress waves propagate through the pile**. Those stress waves will propagate following characteristic lines in the distance-time plane shown in Figure II-4. **Pile is discretized into several continuous segments delimited by interfaces**. Each interface connects a pile segment to the previous/next one.

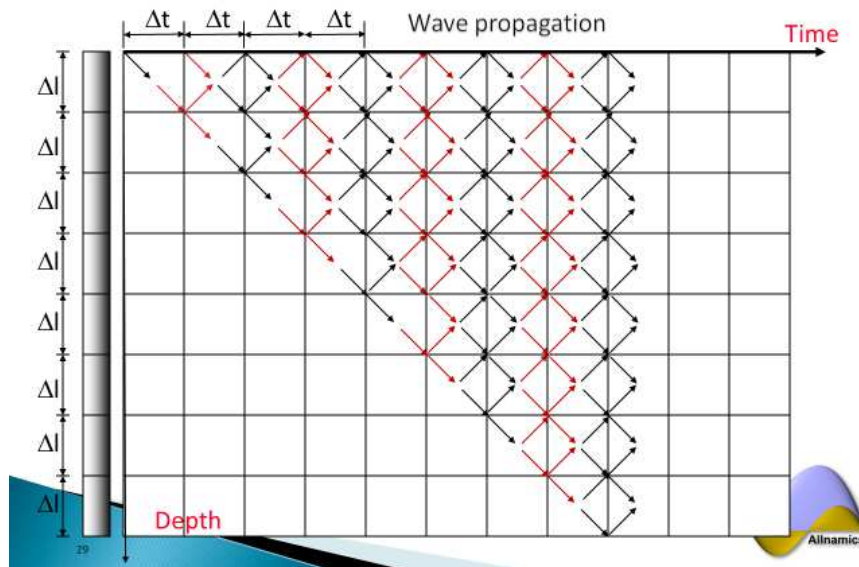


Figure II-4: Stress wave propagation, (Allnamics BV, 2016)

As stated in section II.a), stress waves travel at a constant speed and undisturbed within each pile segment. Therefore, the **only interaction with external forces (soil) will be modeled at the interfaces**. To ensure that all stress waves arrive at all interfaces at the same instant there is a relation that must be fulfilled, as it can be seen from Figure II-4:

$$\Delta l = \Delta t \cdot c$$

Equation II-16

Equation II-16 implies that stress waves arrive at all interfaces at the same instant, repeated each time step Δt defined by the element length Δl chosen. When stress waves arrive to an interface, some energy will be reflected and some refracted. Those calculations will depend on external forces caused by soil resistances.

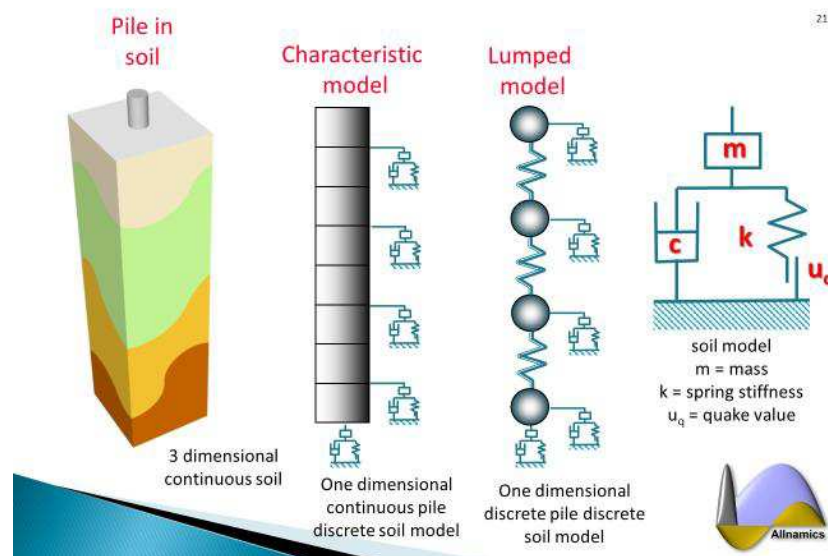


Figure II-5: Modelling method of characteristics, (Van Delft, 2015)

Model type	Type of force	Behavior	Depends on
Spring	Static	Elasto plastic	Displacement
Dashpot	Damping	Viscous	Velocity
Mass	Inertia	Inertial	Acceleration

Table II-1: Soil resistance models

As shown in Figure II-5, at each interface in contact with the soil a spring, a dashpot and a mass will be modelled to represent the soil interaction at that level. Each element will represent a force as the equation below shows:

$$F_{total} = F_{static} + F_{damping} + F_{inertia} \quad \text{Equation II-17}$$

Therefore, propagation of stress waves through the pile can be modelled. Combining previous equations with boundary conditions, all possible cases can be presented. Adding at each interface soil end reaction and soil friction, the model will be completed.

c) Stress wave boundary conditions

As shown in

Table II-2, for an impact force, a peak stress wave will propagate along the pile length, L , at a constant speed, c , defined in Equation II-8 and shown in Figure II-4.

Variable	Units	Pile top	Pile toe	Pile top
x	[m]	0	L	0
t	[s]	0	L/c	$2L/c$

Table II-2: Distance-time peak stress wave location

This generated stress wave, F_{hammer} , will propagate downwards, arriving at pile toe at L/c . It will be reflected, propagating upwards till pile top and so on. There are some interesting boundary conditions that enlighten how stress waves propagate through the pile, shown in Table II-3.

Condition	Model Description	Physical meaning
Free End $F(x = L, t) = 0$	No friction, no bottom resistance	Initial stages of pile driving, where soil has nearly no resistance
Fixed End $v(x = L, t) = 0$	No friction, full bottom resistance	Possible initial stage of pile driving with a rocky superficial soil layer
Soil Friction W	Friction interface example	Showing how stress waves interact with a friction layer

Table II-3: Main boundary conditions

All cases will begin with the initial conditions: $\begin{cases} x = 0 \\ t = 0 \end{cases} \begin{cases} F^\downarrow(x, t) = F_{hammer} \\ F^\uparrow(x, t) = 0 \end{cases}$

Free end reaction

At the end of the pile, toe level ($x = L$), in free end we have no resistance, $F = 0$.

$$F\left(L, \frac{L}{c}\right) = F^\downarrow\left(L, \frac{L}{c}\right) + F^\uparrow\left(L, \frac{L}{c}\right) = 0 \quad \text{Equation II-18}$$

$$F^\uparrow\left(L, \frac{L}{c}\right) = -F^\downarrow\left(L, \frac{L}{c}\right) = -F_{hammer} \quad \text{Equation II-19}$$

Therefore, from a compression wave travelling downwards, $F^\downarrow(+)$, a tension wave will be reflected travelling upwards, $F^\uparrow(-)$, in free end conditions.

Fixed end reaction

At pile toe level ($x = L$), in fixed end (bedrock) we have, $u(L, t) = 0$ and $v(L, t) = 0$.

$$v\left(L, \frac{L}{c}\right) = v^\downarrow\left(L, \frac{L}{c}\right) + v^\uparrow\left(L, \frac{L}{c}\right) = 0 \quad \text{Equation II-20}$$

$$v^\uparrow\left(L, \frac{L}{c}\right) = -v^\downarrow\left(L, \frac{L}{c}\right) = -F^\downarrow/Z = -F_{hammer}/Z \quad \text{Equation II-21}$$

$$F^\downarrow\left(L, \frac{L}{c}\right) = F_{hammer} \quad \text{Equation II-22}$$

$$F^\uparrow\left(L, \frac{L}{c}\right) = -Zv^\uparrow = F^\downarrow\left(L, \frac{L}{c}\right) = F_{hammer} \quad \text{Equation II-23}$$

$$F\left(L, \frac{L}{c}\right) = F^\downarrow\left(L, \frac{L}{c}\right) + F^\uparrow\left(L, \frac{L}{c}\right) = 2F_{hammer} \quad \text{Equation II-24}$$

Therefore, at toe interface, total force will be doubled. As a reflected wave, there will be a compression upward wave. **Notice that all possible toe reactions will be between free end and fixed end conditions.**

Soil friction

At each intermediate interface in contact with soil, friction will be modelled, W , with a force equilibrium as it follows, for two consecutive segments, 1 and 2;

$$F = F_1^\downarrow + F_1^\uparrow = F_2^\downarrow + F_2^\uparrow + W$$

Equation II-25

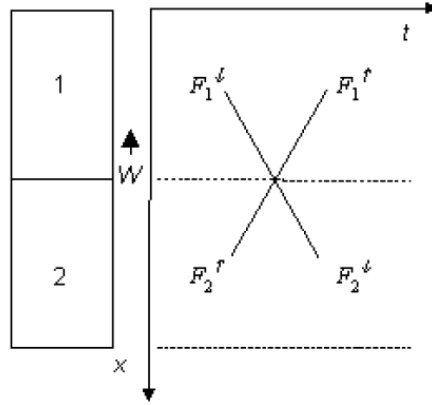


Figure II-6: Soil friction, (Allnamics BV, 2016)

At each interface, by applying continuity law, consecutive segments will have same velocity:

$$v = \frac{F_1^\downarrow}{Z} - \frac{F_1^\uparrow}{Z} = \frac{F_2^\downarrow}{Z} - \frac{F_2^\uparrow}{Z}$$

Equation II-26

Combining both equations, we can reformulate as:

$$F_2^\downarrow = F_1^\downarrow - 0,5 \cdot W$$

Equation II-27

$$F_1^\uparrow = F_2^\uparrow + 0,5 \cdot W$$

Equation II-28

At each interface, half of the friction will be subtracted within the transmitted downward force, $F_2^\downarrow$, Equation II-27, and half of the friction will be reflected as an upward wave, Equation II-28.

The model is finally completed, modelling how the stress waves propagate through the pile and interact with the soil⁴. These equations are implemented in *Allwave-PDP* software as an algorithm shown in Figure II-7, modelling forces, speeds, displacements and accelerations at each interface for each time step as shown in Figure II-4.

⁴ Furtherly explained in Soil Modeling Section.

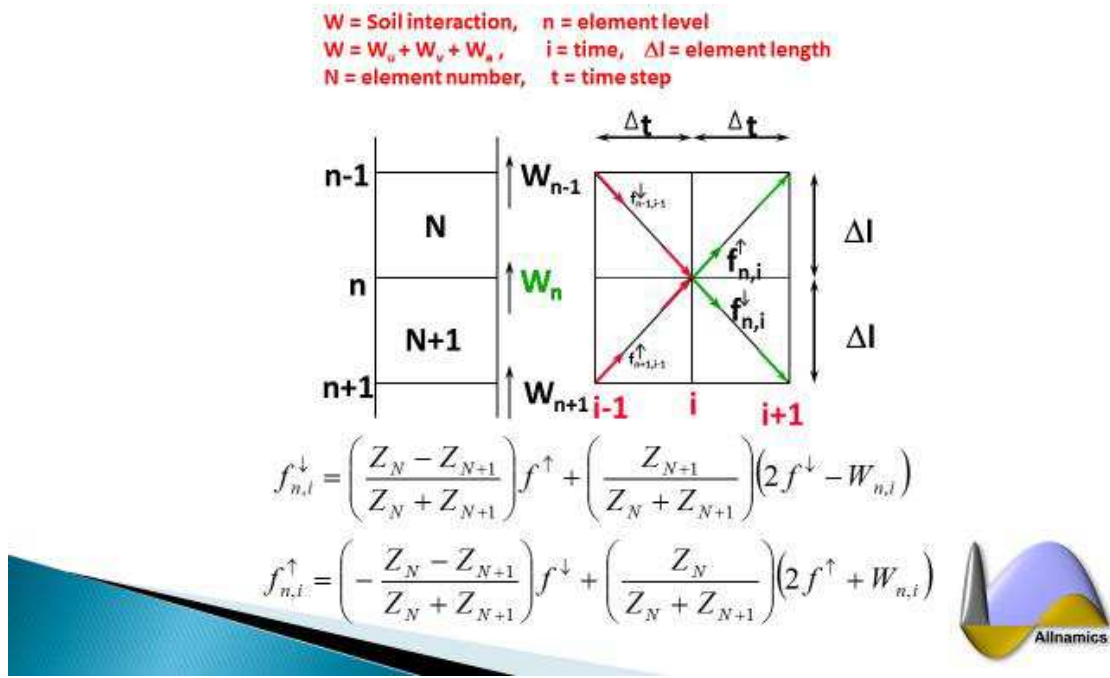


Figure II-7: Allwave-PDP Algorithm, (Allnamics BV, 2016)

d) Monitoring and Measurements

MEASUREMENTS WILL BE PERFORMED WITH TWO TYPES OF SENSORS, STRAIN GAUGES AND ACCELEROMETERS, BEING THE MAIN OUTPUT FROM VIBRATORY DRIVING MEASUREMENTS (VDA) THAT WILL BE ANALYZED.

Strain gauges

This type of sensor measures strains. The sensor is fixed to the pile by two bolts with a designed spacing. Therefore, compatibility of deformations will be in place between pile and sensor. A piezoresistive element placed between bolts, will vary its electric resistance when deformed. By adding a current, the sensor is able to send electric signals to a recorder, where they are digitized and calibrated. The monitoring software is able to translate them to strains⁵.

Accelerometers

Accelerometers are very similar to strain gauges. A small free oscillating mass is connected by piezoresistive elements to the sensor. When the accelerometer is excited, the free mass oscillates, deforming the piezoresistive connection. That deformation is again digitized, calibrated and converted to accelerations.

⁵ Reminder: Stresses and deformations are linearly correlated by the Young Modulus since vibratory stresses remain always in the elastic domain of the steel, Equation II-3.



Figure II-8: Strain gauge and accelerometer combined sensor (left), Riffgat monopile (right)

Pile main levels

Pile levels are defined from top till the bottom of the pile. The most interesting levels would be measuring level and toe level. Other pile levels can be analyzed for specific research purposes but are out of the scope of this thesis.

Level	Description
Tip / Head	Top part of the pile where hammer deliver forces
Measuring	1 pile diameter below top of the pile where sensors are placed
Middle	Middle part of the pile for analyzing and research purposes
Toe / Bottom	Bottom part of the pile, to see bottom resistance and stresses

Table II-4: Different pile analysis levels

Penetration speed

Required time for the pile to penetrate between two consecutive pile marks, as can be seen from Figure II-8. Usual set up provides transversal markings every 25 cm, and bigger ones every meter. An average penetration speed will be calculated between 25cm consecutive markings and established for the whole interval⁶. Usually speeds are given in mm/s for vibratory driving.

⁶ Further improvement could be necessary to precisely monitor penetration speed.

2. Soil modelling

SOIL IS MODELLED BY A SYSTEM OF SPRINGS, DASHPOTS AND CONCENTRATED MASSES, REPRESENTING STIFFNESS, DAMPING, AND MOVING SOIL⁷, RESPECTIVELY. THOSE SYSTEMS WILL BE IN PLACE ONLY AT EACH INTERFACE, WHERE ALL THE SURROUNDING FRICTION OR TOE RESISTANCE WILL BE CONCENTRATED.

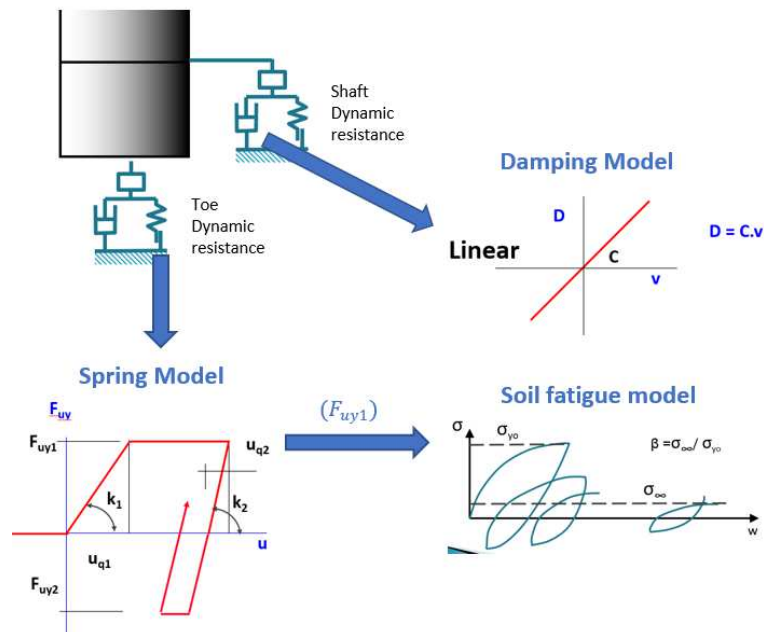


Figure II-9: Spring, Dampers and Soil fatigue models

a) Spring model: Quakes

SOIL STATIC RESISTANCE CAN BE MODELED ACCORDING TO AN ELASTO-PLASTIC SPRING DEFINED BY QUAKES AND YIELD STRESSES.

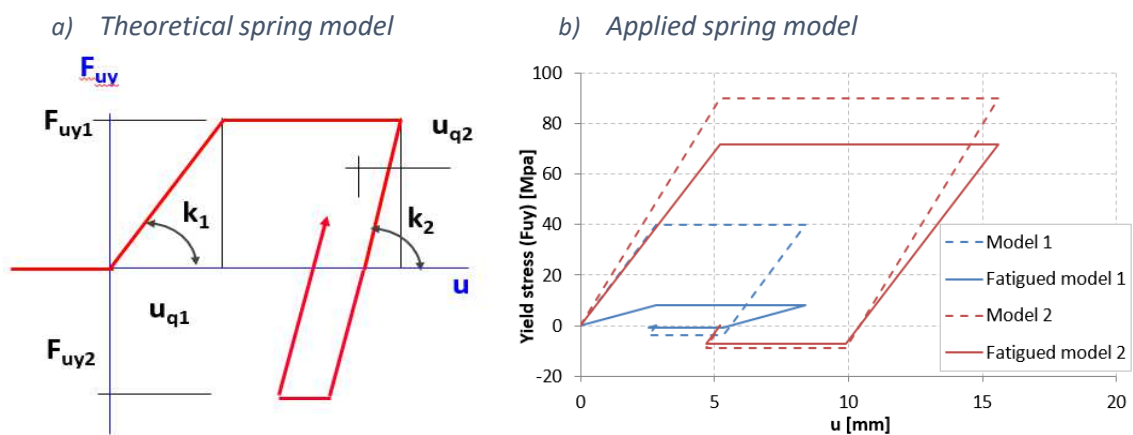


Figure II-10: Soil spring asymmetric elasto-plastic model, (Allnemics BV, 2016)

⁷ Moving soil will not be treated. It's hardly quantifiable and out of the scope of this thesis.

- F_{uy1} and F_{uy2} as yield stress [MPa]
- u_{q1} and u_{q2} as loading and unloading quakes [mm]
- k_1 and k_2 as spring stiffness F_{uy1}/u_{q1} and F_{uy2}/u_{q2}

As shown in Figure II-10, the elastic range is defined by a stiffness. The plastic range is explored when the displacement exceeds the quake, u_{q1} . Within that range, permanent soil deformation, i.e. pile penetration, is produced. F_{uy1} , or soil yield stress, will be defined on the basis of the widely accepted CPT test⁸ described in p.35, namely cone resistance q_c for unit toe resistance and friction sleeve f_s for unit shaft resistance.

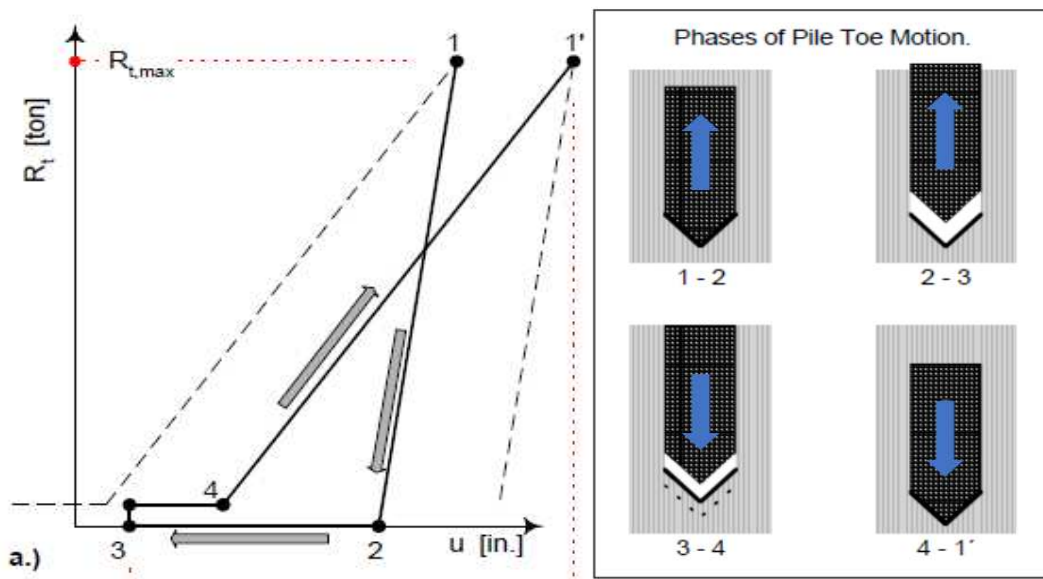


Figure II-11: Toe spring resistances, (Viking & Holeyman, 2006)

Well-illustrated in Figure II-11, by (Viking & Holeyman, 2006), enables to understand the phases that the pile motion exerts to toe spring⁹ revealing that a gap between pile and toe spring can be in place while the pile is being lifted.

Figure II-12 reveals how the shaft spring will act, constantly applying an upward or downward resistance against the movement of the pile.

It's interesting to point out that if at toe level, we do not achieve to mobilize higher displacement than quake value or force than the yield stress, we will have refusal.

⁸ In case any other type of soil investigation is available, some correlations can be found in Chapter III.

⁹ From c) Stress wave boundary conditions, the stiffness of the toe spring will be 0 for free end conditions and ∞ for fixed end conditions. As previously stated, all possible toe reflections will be simulated within those extreme boundary conditions.

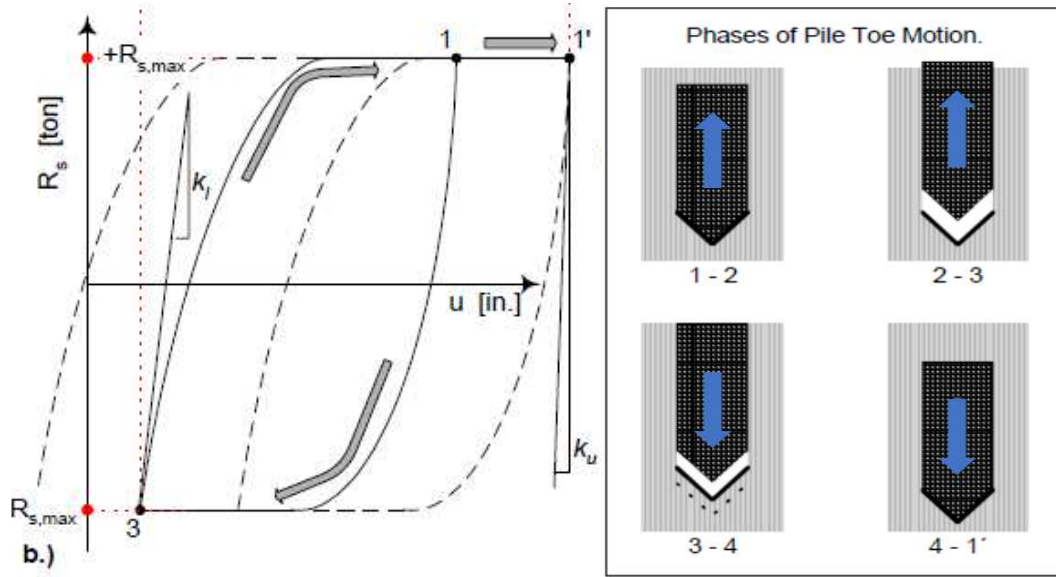


Figure II-12: Shaft spring resistances, (Viking & Holeyman, 2006)

Quakes

Quakes represent the elastic deformation threshold before the soil spring reaches plastic behavior. Determining quakes, though, is difficult because of several soil types and parameters. Smith model, (Smith, 1962) relates quakes values to young modulus, Poisson's ratio and yield stress of the soil and pile dimensions. Even though Smith damping and quakes can be estimated through theoretical equations, some deviations may appear. From (Allnamics BV, 2016), two formulas are proposed to determine quake values. For Toe quake as follows:

$$u_{q,toe} = \frac{F_{y1}}{4r_0 * \frac{G}{(1-\nu)A}} \quad \text{Equation II-29}$$

And for Shaft quake as:

$$u_{q,shaft} = 2 \cdot r_0 \cdot \frac{F_{y1}}{G} \quad \text{Equation II-30}$$

Those parameters are limited and correspond to impact driving experience. Vibratory driving applies cycles at a different rate with lower energies. Experience from Allnamics database shows that quakes of 2 mm are the most representative of different soil types, both for Toe and Shaft. (Hussein, Woener, Sharp, & Hwang, 2006) proposed values in the same range, between 2 to 5mm.

Simulations will use **as starting values a 2 mm quake**, both for shaft and toe springs. No difference has been pointed out between loading and unloading quakes. Both will remain equal as an assumption.

b) Damping model: Damping constant

DAMPING EFFECTS ARE RELATED TO THE ENERGY DISSIPATION IN THE FORM OF WAVE RADIATION OR TRANSFORMATION OF KINETIC ENERGY INTO OTHER FORMS (HEAT, ETC...) OF THE RESISTANCE OF A SOIL.

Featuring a parameter α , the velocity dependent resistance is defined by, according to (Heerema, 1979) for both models as:

$$D = C v^\alpha \quad [N/m^2] \quad \text{Equation II-30}$$

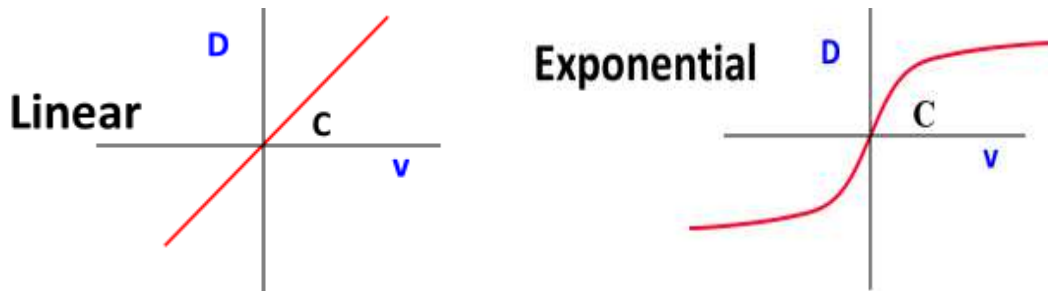


Figure II-13: Damper models, (Allnamics BV, 2016)

Like shaft spring resistance, presented in Figure II-12, it will act against upward and downward velocities. Two types of dampers are presented, linear or exponential corresponding to 2 main soil types: clays and sands.

For $\alpha = 1$, a linear (viscous) damping is generated while for $\alpha < 1$ a parabolic one is generated. Recommended α values are 0.2 for clays and 1 for sands, based on Allnamics' database and experience.

Damping constant

Defining damping constant, C , from Equation II-30, is arguably (with quakes) one of the most difficult parameter to define. Several formulas are proposed with a non-neglectable scatter within an extensive literature review by (McVay & Kuo, 1999). This user-dependent parameter affects impact and vibratory driving predictions. (Randolph & Simons, May 1986) proposed for circular cross sections the following damping constant formulas, that are used in the model, for toe damping constant:

$$C_{toe} = \frac{3.4 r_o^2 \sqrt{G\rho}}{1 - \nu} \quad [Ns/m^3] \quad \text{Equation II-31}$$

And for shaft damping constant:

$$C_{shaft} = 2\pi r_o \sqrt{G\rho} \quad [Ns/m^3] \quad \text{Equation II-32}$$

Damping constant indetermination is one of the sources of impact driving predictions scatter of results. Due to the fact that **vibratory driving modelling follows the same theory as impact driving, the same uncertainties and scatter of results are also imported.**

			Non-Cohesive	Cohesive	Limestone
GRL (1993)	Damping (s/ft)	Skin	0.05	0.20	-
		Toe	0.15	0.15	-
	Quake (in)	Skin	0.10	0.10	-
		Toe	D/120	D/120	-
EOD/ Database	Damping (s/ft)	Skin	0.050-0.421	0.051-0.316	0.105-0.272
		Toe	0.049-0.444	0.035-0.208	0.050-0.439
	Quake (in)	Skin	0.050-0.191	0.020-0.140	0.030-0.150
		Toe	D/45-D/533	D/35-D/200	D/44-D/300
BOR/ Database	Damping (s/ft)	Skin	0.084-0.490	0.074-0.736	0.04-0.535
		Toe	0.002-0.700	0.012-0.649	0.072-0.430
	Quake (in)	Skin	0.055-0.156	0.050-0.200	0.020-0.180
		Toe	D/51-D/333	D/64-D/375	D/39-D/600

Figure II-14: Quake and damping values (McVay & Kuo, 1999)

Choosing between damping formulas, as proposed from (McVay & Kuo, 1999), do not increase predictions accuracy from a geotechnical point of view. Notice that these formulas define a trend behavior more than an exact value. Local experience from nearby projects or using calibrated formulas from nearby areas are always more representative.

c) Soil fatigue model: Beta factor

SOIL RESISTANCE DEGRADATION CAN BE MODELLED ACCORDING TO THE BETA FACTOR FATIGUE MODEL.

Cyclic loading

Cyclic loading, or soil fatigue, stresses the soil periodically, reducing its resistance at each applied cycle. A loaded soil has a particular initial yield stress, $F_{initial}$. According to (Pecker, 2008), when several loading cycles are applied to a soil, a change in its microstructure, grain rearrangement, will occur. This hysteresis behavior, shown in Figure II-15 will decrease from initial yield stress to a residual one, $F_{residual}$, quantified by the Beta Factor.

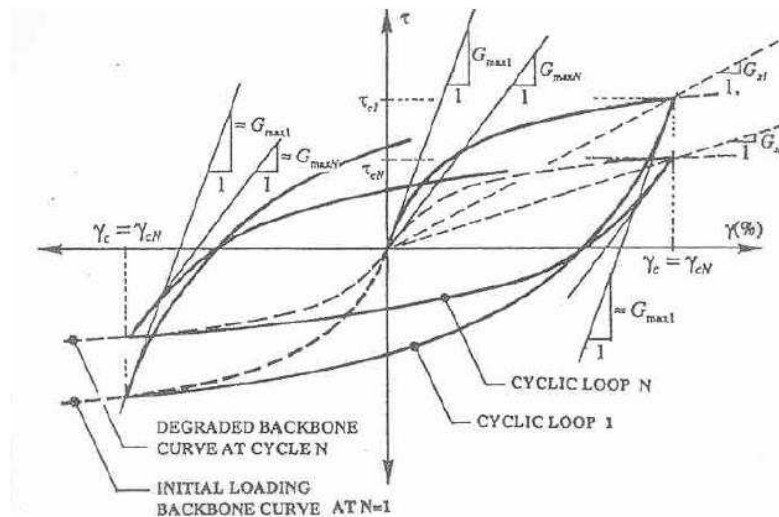


Figure II-15: Stiffness degradation with the number of cycles

Liquefaction

Liquefaction is a particular case of cyclic loading that degrades soil resistance further than any other phenomenon, occurring only in saturated sandy soils. Sandy soils possess just the right microstructure, grain size, to develop this phenomenon. At microscopic level two main forces govern soil particles interactions, contact forces and cohesion.

Contact forces

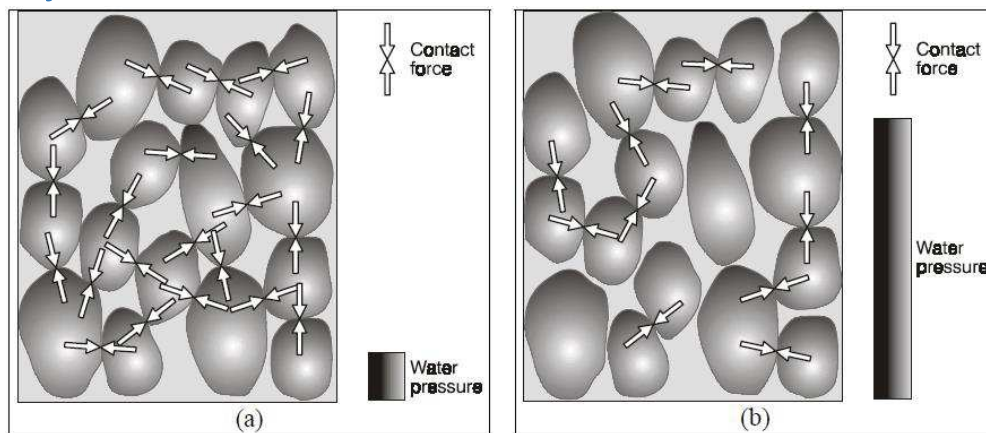


Figure II-16: Contact forces between soil grains, normal case (left) liquefaction case (right), (Berghe, 2001)

Contact forces, presented in Figure II-16, are a combination of water pressure and friction between soil particles at microscopic level. An increase of pore water pressure (induced for example by cyclic loading / vibrodriving) will absorb part of these contact forces, preventing particles to make contact between them. **This increase of pore water pressure enables particles to freely rotate endlessly, generating a fluid-like soil state**

at macroscopic level. The particular sand grain sizes, ranging from 0.1mm to 2mm, allow to develop and maintain, when cyclically loaded, water pressure, allowing therefore liquefaction.

Cohesion

Cohesion is an electrostatic force between particles created by molecular forces and water content. **It can prevent or difficult particles to lose contact, limiting the soil to reach a fluid state.** Cohesion is increasingly dominating over finer grain particles, mainly clays. Sand grain particles are large enough to avoid cohesion forces, allowing therefore liquefaction.

According to (Jefferies & Shuttle, 2011), simulating liquefaction presents two major problems:

- In constitutive models, it is difficult to quantify the rearrangement of particles caused by an increasing pore water pressure, as well as soil dynamic permeability. It is a highly detailed phenomenon where lots of assumptions or correlations have to be made.
- Simulating stress conditions when liquefaction are difficult to simulate with standard laboratory tests. Most of those tests simulate constant loading at a defined stress application rate.

Soil fatigue in dry sands

Soil liquefaction is only possible, in principle, in saturated sands. But as (Viking & Holeyman, 2006) states, dry sands can also artificially reduce its shear strength. They conclude that soil liquefaction is not the primary explanation of the shear strength reduction, allowing, in principle, to vibrodrive on dry sands as well, as Chapter V will show.

Beta factor

Beta factor is based on microscopic scale.

- Angular particles interact between them with higher friction. When cyclically loaded, higher beta factors than rounded particles will be in place.
- Cohesion will be directly correlated to beta factor. By maintaining contact between particles, higher beta factors will be in place, especially for clays.

Those microscopic phenomena, defined by particle shape and size parameters, will define the macroscopic response of a cyclically loaded soil. Therefore, defining Beta factor. Some proposed values can be seen in Table II-5. Further details can be found in Section III.2.

Type of Soil	Fatigue Factor β
Round Coarse Sand	0.10
Soft Loam/Marl, Soft Loess, Stiff Silt	0.12
Round Medium Sand, Round Gravel	0.15
Fine Angular Gravel, Angular Loam, Angular Loess	0.18
Round Fine Sand	0.20
Angular Sand, Coarse Gravel	0.25
Angular/Dry Fine Sand	0.35
Marl, Stiff/Very Stiff Clay	0.40

Table II-5: Beta factor values for different soil types, (Tara, Middendorp, & Verbeek, 2014)

After several loading cycles, soil resistance is reduced to a residual value, as shown in Figure II-17. The ratio between initial and residual resistance is the Beta factor introduced by (Jonker, 1987):

$$\beta = \frac{F_{residual}}{F_{initial}} [-] \quad \text{Equation II-33}$$

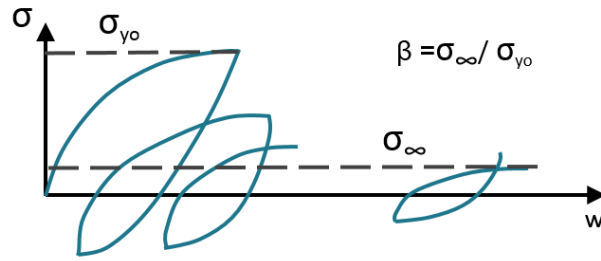


Figure II-17: Soil fatigue model

This factor is applied both on springs and dampers final resistances, as follows:

$$F_{uy1,final} = \beta_{toe/shaft} \cdot F_{uy1} [N/m^2] \quad \text{Equation II-34}$$

$$D_{final} = \beta_{toe/shaft} \cdot D_{initial} [N/m^2] \quad \text{Equation II-35}$$

From Equation II-17, we can finally have the maximum spring/static dashpot/damping forces, at toe shaft interfaces, as follows:

$$F_{static} = A_{toe/shaft} \cdot F_{uy1,final} [N] \quad \text{Equation II-36}$$

$$F_{damping} = A_{toe/shaft} \cdot D_{final} [N] \quad \text{Equation II-37}$$

Beta factor rate of application

This reduction of soil resistance is cyclically dependent, as shown in Figure II-18. Beta factor is usually set as a constant, considered to be fully applied due to vibrators high frequencies.

The rate of Beta factor application can be modelled following Figure II-18. Depending on the number of cycles, a percentage of Beta factor will be applied. The number of cycles will depend on time and on the vibrator applied frequency.

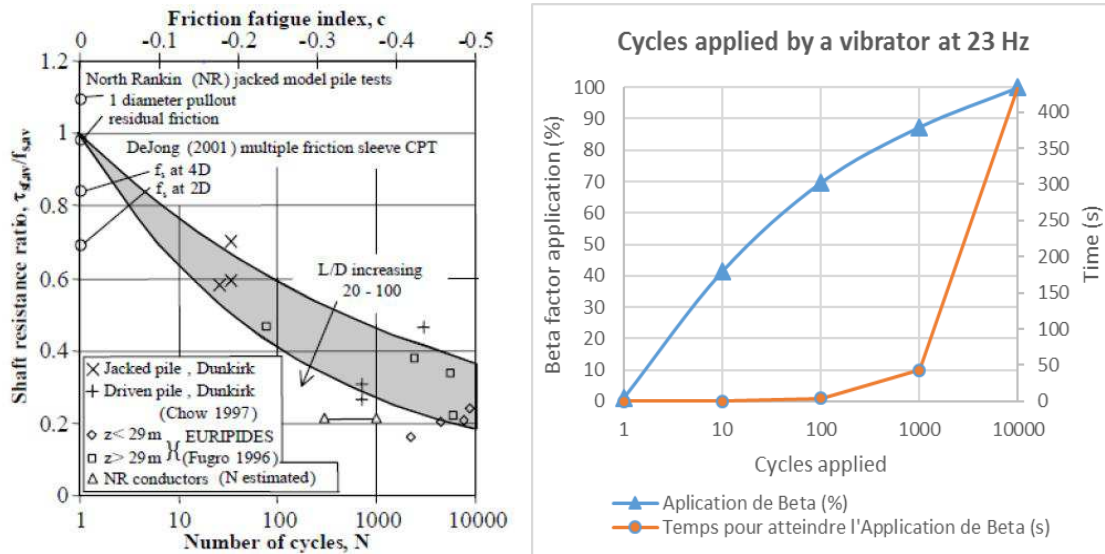


Figure II-18: Decrease of Shear strength regards to loading cycles applied (left) (Tara, Middendorp, & Verbeek, 2014), model extracted from graph (right)

Stage	Cycles applied	Beta factor application	Time
1	100	69,7%	4,3s
2	1000	87,4%	43s
3	10000	100%	430s

Table II-6: 23Hz Beta factor (%) application times

As seeing from Table II-6, 100 cycles are applied in few seconds, a minute for the second stage and around 7 minutes for fully application of Beta factor. The initial assumption that Beta factor is directly fully applied could be challenged.

Cycles are applied at different rates depending on the pile location. At toe, cycles are applied at normal rate whereas at shaft they are applied at double rate (upwards and downwards). Beta factor is therefore fully applied twice as fast at shaft. Most of shaft layers will be fully fatigued except the recently penetrated ones. Those new non-fully fatigued layers will have a minor influence regarding the total shaft resistance. Therefore, shaft fatigue can be considered as instantly fully applied.

On the other hand, an evolving toe beta factor could be in place according to Figure II-18, with major influence. Cycles applied at new vibrated toe layers could not fatigue fully soil resistance due to high penetration speeds¹⁰.

d) Clay problem

THE PRESENCE OF A CLAY LAYER COULD BE CRITICAL TO ENSURE VIBRODRIVEABILITY.

Cohesion

This continuous force will damp the effect of cyclic loading by maintaining contact between soil particles. It limits vibrodriving efficiency, preventing to reach complete soil resistance degradation. No liquefaction can be achieved in clayey soils.

Friction

Clay finer particles multiply exponentially their contact surfaces with the pile, presenting higher frictions than sands. This shaft resistance drains at each pile interface forces/displacements travelling along the pile, preventing that a sufficient amount reaches toe level. Pile toe is therefore unable to overcome the elasto-plastic threshold to achieve toe plastic behavior (penetration), inducing pile refusal.

Sticking clay

Occurs when forces cannot overcome frictional resistance of clay, it may “stick” to the pile. We can observe a drop of frequency since the clay will add weight to the pile, overloading the dynamic system, therefore the vibrator, and possibly inducing refusal. Its analysis is out of the scope of this thesis, since mainly sands are covered, but stated for future researches.

¹⁰ If high penetration speeds are predicted, soil could not be fully fatigued. Applying a reduced Beta factor will reduce penetration speeds. This highly sensitive aspect will be left out of the vibrodriving analysis, but it should be treated once predictions are refined.

3. Vibrator modelling

A POWER PACK PROVIDES A PRESSURIZED OIL FLOW, THROUGH HOSES, TO THE EXCENTRES. THOSE SYMMETRIC ECCENTRIC MASSES WILL COUNTERROTATE SYNCHRONIZED, PROVIDING A VERTICAL SINUSOIDAL FORCE. THIS FORCE WILL BE TRANSMITTED TO THE PILE BY A RIGID CLAMP CONNECTION. TO DAMP THIS OSCILLATORY LOAD TO THE CRANE, A SUPPRESSOR BOX WILL BE PLACED ON TOP OF THE DYNAMIC SYSTEM.

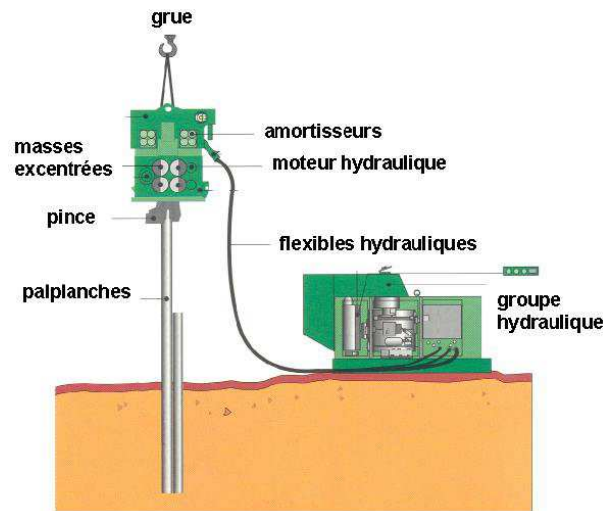


Figure II-19: Different parts of a vibrator, (Gonin, Borel, & Le Tirant, 2002)

a) Eccentric mass

DISC OF THE VIBRATOR WEIGHTED UNEVENLY THAT ROTATES, CREATING A CENTRIFUGAL FORCE.

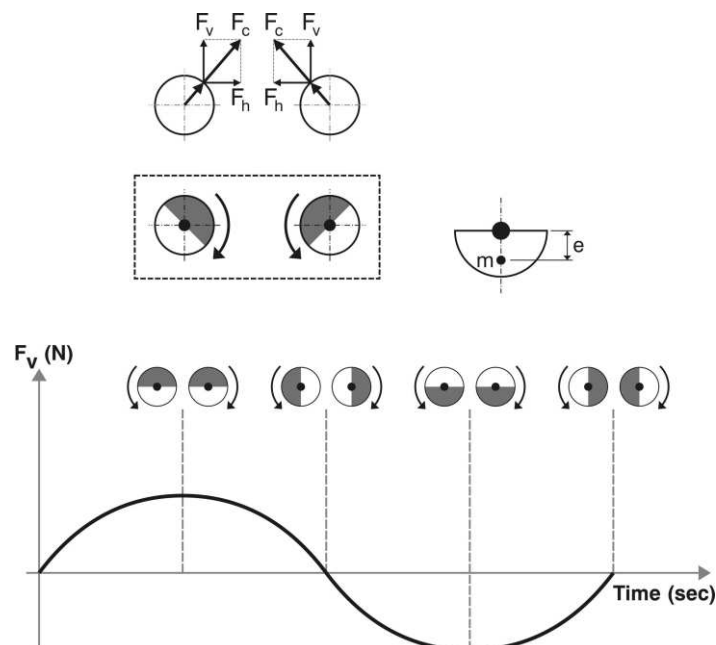


Figure II-20: Working principles of the eccentric masses, (Denies, 2010)

As eccentric masses counterrotate symmetrically, horizontal components are balanced resulting in a purely vertical sinusoidal force, shown in Figure II-20. The eccentric moment is set-up as a constant¹¹, defined as:

$$m_e = m_{uneven} \cdot e \quad \text{Equation II-38}$$

b) Frequency

The frequency of large vibrators ranges from 17 till 23 Hz. This has some mechanical reasons. For instance, higher frequencies could lead to stronger mechanical parts, overweighting the already large vibrator mass. The amplitude force generated by the vibrator will be, as follows:

$$F_{vib} = m_e \cdot \omega^2 = m_e \cdot (2\pi f)^2 \quad \text{Equation II-39}$$

Equation II-39, shows that a drop of frequency will reduce dramatically vibrator's amplitude force. Phenomenon that can be seen while harder vibrodriving is in place, reducing even further frequency, and so on. A re-lift operation is advised, so the vibrator regains its revolutions (only shaft resistances acting). Then, at maximal revolutions, a second attempt can be performed, lowering back the pile.

A hammer operator can modify the frequency according to the provided pressurized oil flow. When high vibrodriving resistances occurs, maximum oil flow is reached, and maximum frequency/capacity of the vibrator.

This frequency tuning will be our main driving parameter. Notice that this adjustment is far from being precise and it's performed when a refusal situation is encountered.

c) Dynamic mass

Dynamic mass is composed of the exciter block (or gearbox), clamp system and pile weight. Between those parts, a full rigid connection is in place, transferring perfectly the full force (no transfer losses are assumed). That mass will be a constant design parameter, defined as

$$M_{dyn} = M_{eb} + M_{clamps} + M_{pile} = M_{dyn,vib} + M_{pile} \quad \text{Equation II-40}$$

¹¹ Notice that some vibrators have extendable excentres, changing the eccentric moment generated. Those vibrohammers will not be covered in this thesis.

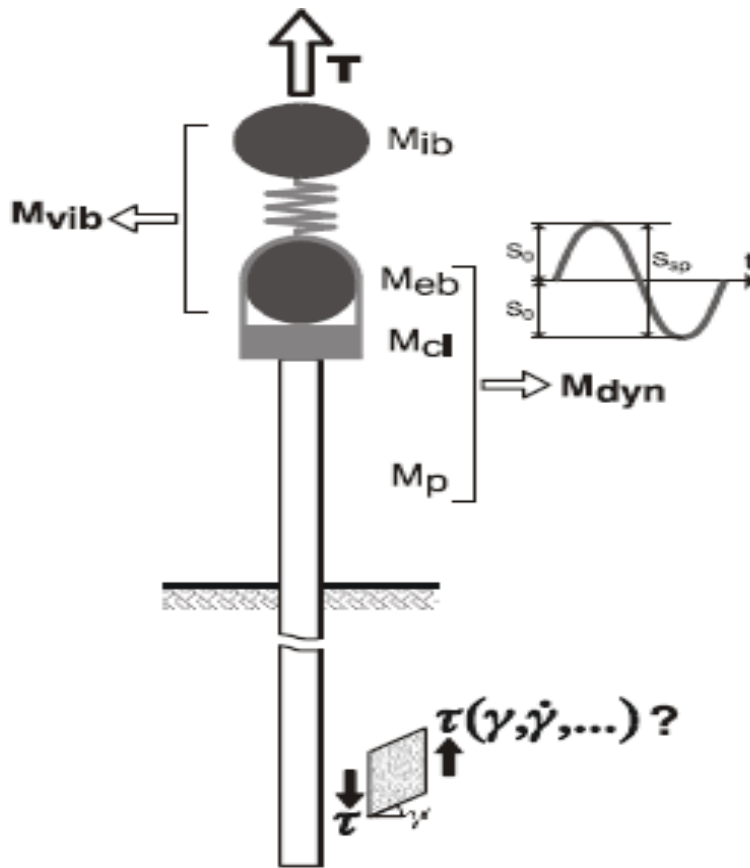


Figure II-21: Mechanical behavior of a vibrator, (Holeyman, LeGrand, & Van Rompaey, 1996)

d) Static mass problem

The suppressor box or isolator block, M_{ib} , shown in Figure II-21 and Figure II-22 represents the static mass of the vibrator. The main purpose of this part is to absorb vibrations generated by the gearbox. It allows crane cable to safely hoist the whole system with more stable/static/damped loads than vibratory ones. It is connected to the exciter block by a system of elastomers, represented by a spring in Figure II-21.

However, the net effect of the static mass of the vibrator is not perfectly modeled, nor accurately accounted for. This mass is usually considered as a dead weight on top of the vibrator element. It should be noted that the net effect of the static mass in the dynamic system varies depending on the hook pull, T , of the crane. Sometimes the limited hoisting speed of the crane can slow down a high penetration speed of the pile, pulling upwards. **This crane hook pull interaction inherent to vibrodriving, limits the accuracy of penetration speeds predictions.** Vibratory driving predictions will be able to assess whether or not penetration is possible rather than at which penetration speed.

Recording hook crane loads while vibrodriving is imperative, as stated by (Whenham & Holeyman, 2012). Next to that, the static part of the vibrator is connected to the vibrating part through elastomer pads (springs) of a non-specified stiffness from vibrohammer manufacturers. Therefore, the static part oscillates vertically when excited, generating second order inertia forces and dynamic loads.

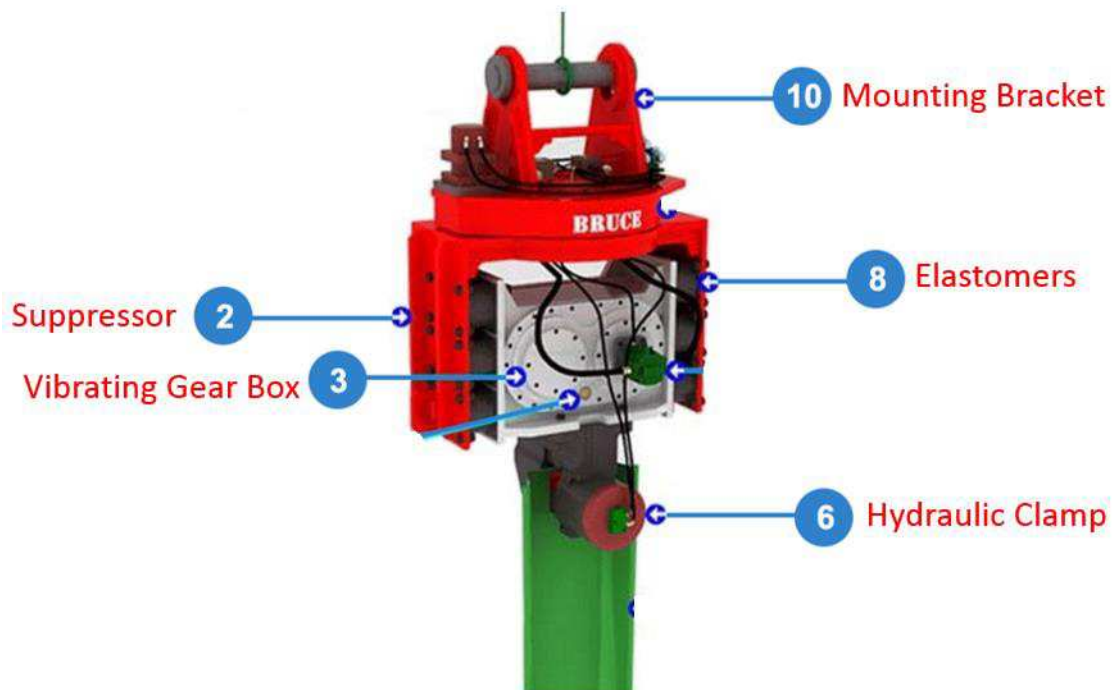


Figure II-22: Detailed scheme of a vibrator, (Bruce Piling Equipment, 2017)

Notice that no accurate devices are in place to measure how much weight is released at each time step. **Adding weight to the dynamic system could be effective sometimes, when some force is needed, and can provoke refusal, unbalancing a fragile force equilibrium.** In this thesis, 70% of the static mass will be modeled as a net static load. Further improvements will be needed in the future and a precise measurement of the hook load.

4. Conclusion

- ✓ Vibrator generates a sinusoidal force that propagates through the pile following one dimensional stress wave theory.
- ✓ Method of characteristics solution models the pile as a continuous element, discretized in several pile segments.
- ✓ Through each pile segment, stress waves are assumed to travel undisturbed.
- ✓ Between each pile segment, a spring, a dashpot and a mass will be modeled accounting for soil reaction at that level.
- ✓ Through strain gauges and accelerometers soil behavior can be monitored.

Limits of the study

- Lack of vibratory tests for large monopiles forces to adapt models to a single study case. Open approach searches for behavior trends rather than exact results.
- As (Viking & Holeyman, 2006) stated, main soil testing techniques are developed to produce insight for static design purposes. Dynamic vibrodriving soil stresses are partially reproduced within main soil tests.

Problems to tackle

- Determining how main soil parameters affect vibrodriving
- Defining a tool to spot critical/optimal frequency-penetrations
- Defining a pile vibrodriving formula based on soil investigation
- Application of new tools to a real study case

Soil	Pile	Vibrator
Quakes	Buckling ¹²	Static mass ¹³
Damping	Inside friction factor ¹⁴	Energy delivered ¹⁵
Beta factor		

Table II-7: Main unknown parameters, green covered in the thesis, red, only stated

¹² Buckling will be neglected in vibratory driving due to the low stresses generated in the pile.

¹³ As stated in Section II.3.d), static mass will be simulated as a net static load equal to 70% of its dead weight. Future modelling improvements will be needed.

¹⁴ Inside friction factor defines the internal amount of friction within the pile. Due to no dissipation of internal soil excitation, it's resistance could be even further degraded. This thesis is focused on large offshore monopiles, unplugged scenarios, where excitation dissipation inside the big diameter pile is assumed. Further research could be interesting, especially for smaller diameter piles. Within our simulations full inside friction factor will be taken into account.

¹⁵ It will be assumed that the energy delivered by the vibrohammer is constant, although some power deviations on the vibrator (oil flow related) can actually occur, impossible to predict.

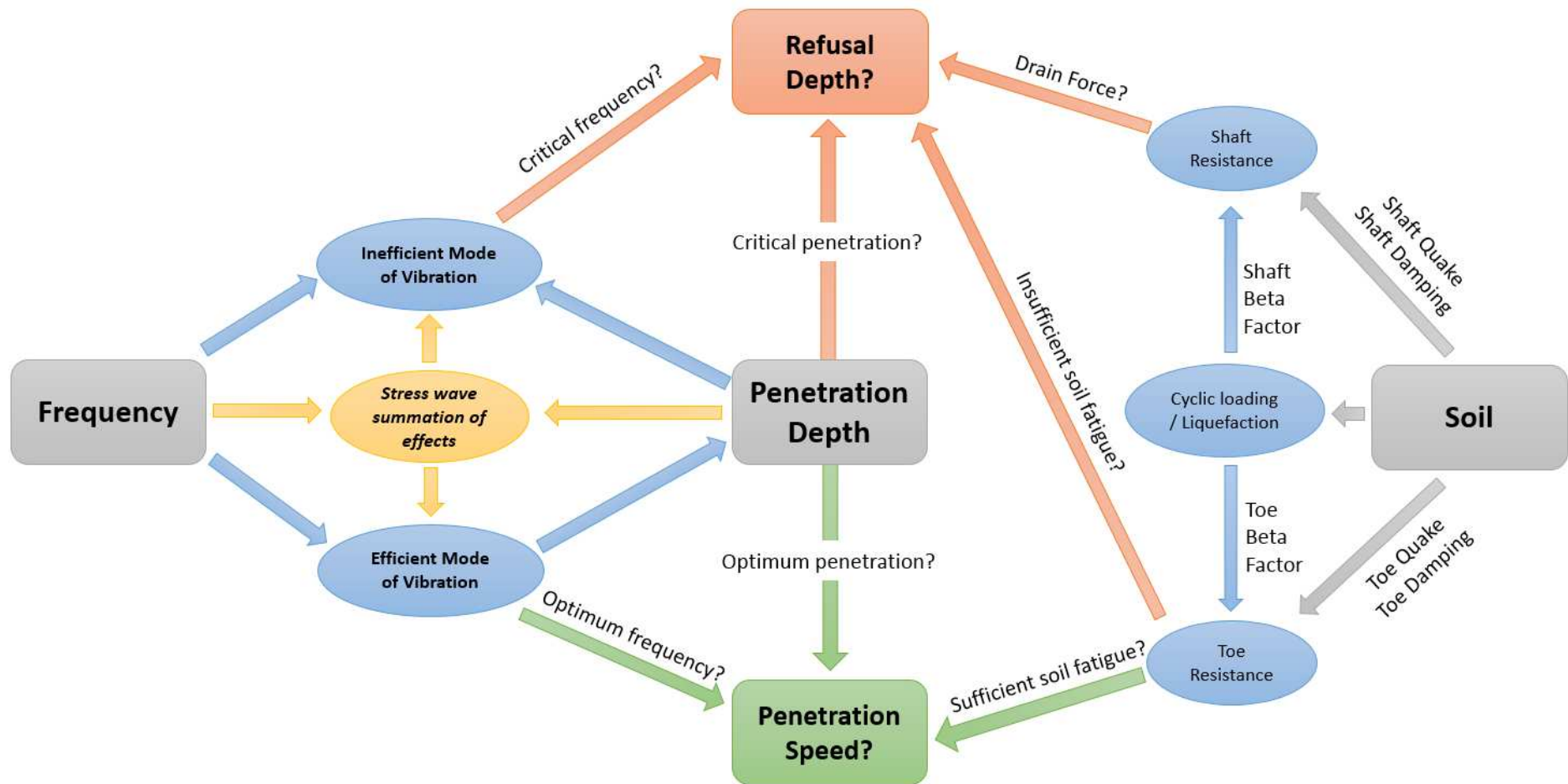


Figure II-23: Fundamental vibratory phenomena's interaction

III. Targeting the unknowns: Soil characterization

VIBRODRIVING MAIN UNKNOWNNS ARE SOIL DYNAMIC BEHAVIOR. THIS BEHAVIOR CAN BE DEFINED THROUGH SEVERAL PARAMETERS BASED ON TESTS AND CORRELATIONS. CHOOSING DIFFERENT SOIL TESTS OR CORRELATIONS CAN, OF COURSE, GENERATE DIFFERENT PREDICTIONS. THIS CHAPTER WILL SUMMARIZE MAIN SOIL TESTS AND SOIL PARAMETERS AS WELL AS THEIR LIMITATIONS TO UNDERSTAND HOW THE SOIL BEHAVES WHILE ITS VIBRATED.

1. Soil test: CPT

SOIL TESTS WILL MAINLY DEFINE TWO MAIN CHARACTERISTICS, SOIL TYPE AND SOIL RESISTANCE. THIS SECTION WILL PRESENT AN OVERVIEW OF THE EXISTING TESTS, RESULTS AND LIMITATIONS.

a) Methodology

CONE PENETRATION TESTS ARE WIDELY USED AND ACCEPTED ACROSS THE WORLD, HAVING A VERY LARGE DATA BASE. THIS IS THE MAIN TEST THAT IS PERFORMED IN GEOTECHNICAL INVESTIGATIONS. THEREFORE, A LARGE NUMBER OF CORRELATIONS HAVE BEEN MADE TO WITHDRAW SOIL PARAMETERS FROM THIS TEST.

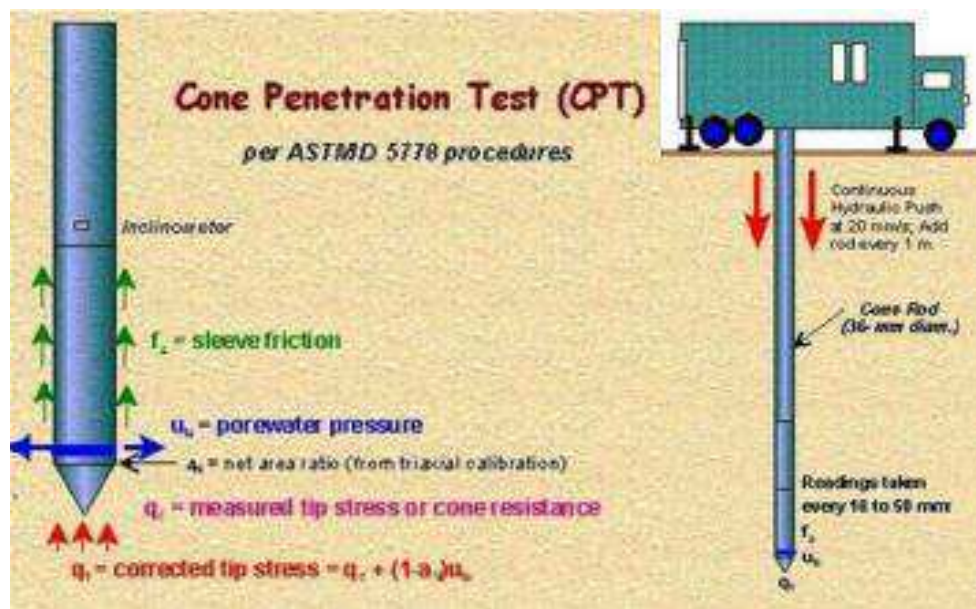


Figure III-1: CPT piezocone Tests, (Pinterest, 2017)

The CPT test consists of pushing an instrumented piezocone into the ground at a standard rate of 2cm/s by applying pressure on a connected steel rod, as shown in Figure III-1. The pressure applied to maintain the penetration rate is recorded as well as cone

resistance (red), friction sleeve resistance (green) and porewater pressure (blue), providing a continuous soil characterization. Therefore, we can withdraw from this test three key soil features:

- Toe yield resistance $\Rightarrow$ defined by the measured cone resistance q_c
- Shaft yield resistance $\Rightarrow$ defined by the measured friction sleeve f_s
- Soil type / Fatigue model $\Rightarrow$ defined by the measured friction ratio R_f

Cone resistance (q_c)

Resistance acting on the tip of the cone that determines toe resistance, with Q_c as the measured force acting on the tip of the cone and A_c as its area:

$$q_c = Q_c / A_c \text{ [MPa]} \quad \text{Equation III-1}$$

Corrected cone resistance depending on the pore water pressure measured:

$$q_t = q_c + u_2(1 - \alpha) \text{ [MPa]} \quad \text{Equation III-2}$$

Which will provide finally the toe yield resistance as follows:

$$F_{toe} = q_t \cdot A_{Pile,toe} \text{ [kN]} \quad \text{Equation III-3}$$

Notice that q_c will be stated afterwards since it's the most common notation even though it is referred to the corrected cone resistance.

Friction sleeve (f_s)

Resistance acting on the external cylindrical surface of the probe. Will be used to determine shaft resistance, although it is well known that sleeve friction has the weakest reliability of all the CPT measurements¹⁶ according to (Mayne, 2014). CPT test will directly provide f_s in kPa, therefore shaft resistance will be as follows:

$$F_{shaft} = f_s \cdot z_p \cdot (\chi_{int} + \chi_{ext}) \text{ [kN]} \quad \text{Equation III-4}$$

With z_p as pile penetration and χ_{int} and χ_{ext} as external and internal pile perimeter. Shaft friction isn't constant, so it will be discretized at each pile interface, concentrating all the surrounding averaged friction, see Figure II-9.

Friction ratio (R_f)

Expressed as a percentage of the ratio between friction sleeve and cone resistance (direct measurements),

$$R_f = (f_s / q_t) \cdot 100\% \quad \text{Equation III-5}$$

¹⁶ Undrained shear stress test is often used to check if friction sleeve values are appropriate or need correction.

Defines a continuous stratigraphy of the soil type. Usual friction ratios for sands are between 0.5 and 1%, sandy-silt-peat-clay mixtures from 1 to 3% and clays over 3%.

b) Test limitations

Soil Type	D_r	Ψ	K_o	OCR	S_t	s_u	ϕ'	E, G^*	M	G_0^*	k	c_h
Coarse-grained (sand)	2-3	2-3	5	5			2-3	2-3	2-3	2-3	3-4	3-4
Fine-grained (clay)			2	1	2	1-2	4	2-4	2-3	2-4	2-3	2-3

Figure III-2: Reliability of CPT correlated soil parameters, 1=Highest, 5=Lowest, (Robertson & Cabal, 2014)

As seen in Figure III-2, reliability of withdrawing soil parameters from CPT, is often not as good as expected. Despite this fact, it is still the best approach considering the costs of a larger geotechnical investigation that not always enables to improve predictions accuracy. **Local experience is always the best reference.**

Discontinuities

Sometimes soil investigation available to perform predictions is not even constant through the soil profile due to:

- Hard soils: At cone resistances over than 60MPa, the CPT piezocone can be damaged. Usually the test is stopped, the hard layer is drilled/sampled and a new redrive attempt is performed some meters below.
- Weak soils: At very weak soils, CPT can be stopped to extract a soil sample to analyze more precisely it's behavior since it could be critical from a structural design point of view.

Each CPT discontinuity, either for drilling or sampling, is interpolated from consecutive CPT measurements.

Set-up factor

Because of impact or vibratory driving, soil resistances can be diminished after the installation of a pile. **After pile driving, the soil can regain part of its initial resistance.** This well-known time dependent phenomenon is defined by the Set-up factor. After driving, soil particles are rearranged and pore water overpressures dissipated, generating an increase in friction. This is well known for impact driving, while, for vibratory driving is yet uncertain to determine. A very small amount of vibrodriving cases

had a CPT afterwards the installation to assess how much of soil strength is left. **This implies that resistance left over after some days of vibrodriving could be different than the assessed by VAM analysis.**

Vibro CPT test alternative:

According to (Viking & Holeyman, 2006) main soil tests, such as CPT, are developed to produce input for static design issues, where no dynamic parameters are currently tested and measured. (Bonita, 2000) proposed the basis of a vibro CPT that could reveal more accurately the real liquefaction potential of a given soil. Such test will be the only one, currently, to simulate real vibrodriving conditions, therefore generating more reliable outputs. Its design is still being performed but it seems a very promising testing alternative once duly calibrated.

c) Other tests

According to (Berghe, 2001), within vibrodriving, loading condition induced by the pile displacement along the shaft is comparable to a direct simple shear (DSS) test. Loading conditions at the pile toe are presumed to be similar to an undrained triaxial test (TXT).

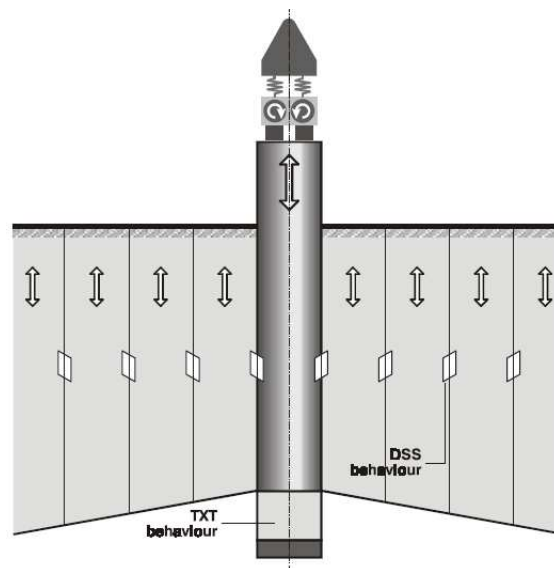


Figure III-3: Vibratory driving behavior, (Berghe, 2001)

Direct Simple Shear: DSS

As shown in Figure III-4, direct shear test consists of an apparatus containing a soil sample that is cyclic or monotonically loaded. Stresses are recorded and therefore as outputs stress-strain properties can be extracted, as well as the shear modulus. As (Berghe, 2001) stated, it reproduces the same vibrodriving loading conditions along the pile shaft. One of the limitations of this test is that pile-soil interface shearing conditions are not reproduced (unlike CPT friction sleeve) and can differ from soil-soil shear stress conditions.

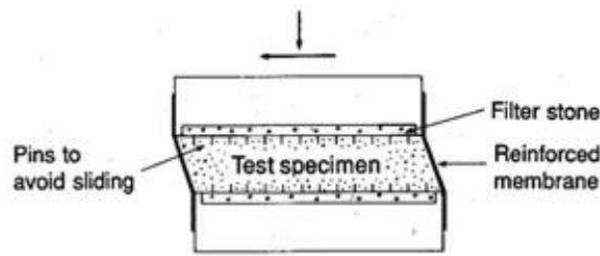


Figure III-4: Simple shear test set up, (Bjerrun & Landva, 1966)

Triaxial Test: TXT

Triaxial test consists of a controlled condition cell chamber containing a soil sample that is monotonically loaded. Stresses are generated axially and laterally to simulate the desired soil conditions. As outputs axial stress-strain properties can be extracted. As (Berghe, 2001) stated, it reproduces the same vibrodriving loading conditions of the pile toe.

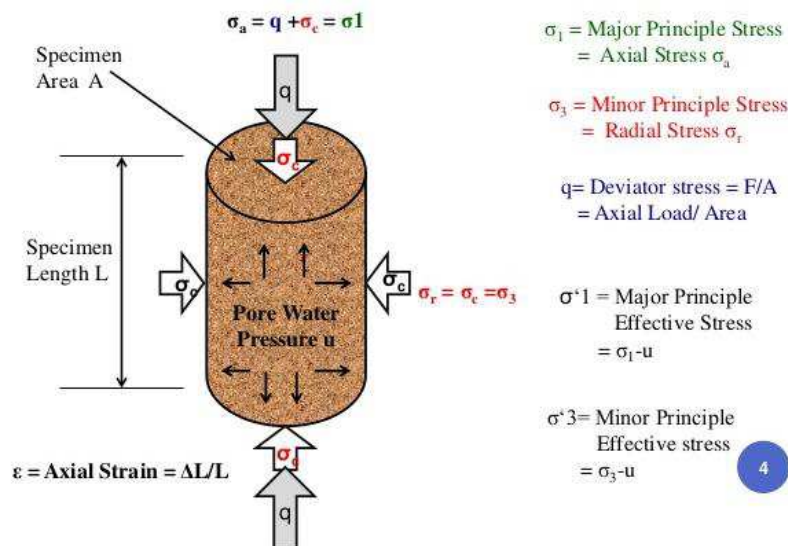


Figure III-5: Triaxial test loading conditions, (Parmar, 2016)

Sampling

Notice that a soil sample is needed to perform both DSS and TXT test. All sampling techniques (boring, coring,...), have a non-neglectable potential to disturb the site conditions of the sample. Therefore, analyzing a disturbed sample could lead to misinterpretation of the soil properties. CPT test measure the undisturbed soil conditions, and it's an overall acceptable soil test to define soil resistances. **Sampling cannot (hardly) be continuously performed along the soil profile, whereas CPT can.**

2. Particle parameters

PARTICLE PARAMETERS SECTION INTENDS TO EXTEND THE INSIGHT ON HOW THE MICROSTRUCTURE OF A GIVEN SOIL WILL AFFECT ITS MACROSCOPIC BEHAVIOR.

a) Size

AS STATED IN SECTION II.3, PARTICLE SIZE DETERMINE SOIL BEHAVIOR. WITHIN THE SCOPE OF THIS THESIS, LIQUEFACTION CAPACITY OF A SOIL WILL BE FOCUSED, SINCE IT'S DETERMINES THE DEGREE OF SOIL RESISTANCE DEGRADATION.

To achieve liquefaction a soil needs to be saturated and cyclically loaded. These loading cycles will build up some pore overpressure, preventing soil particles to make contact between them, changing the behavior of the soil to something similar to a fluid (cohesion less soils). Those two key factors, overpressure capacity and cohesion less are defined by the particle size of a given soil.

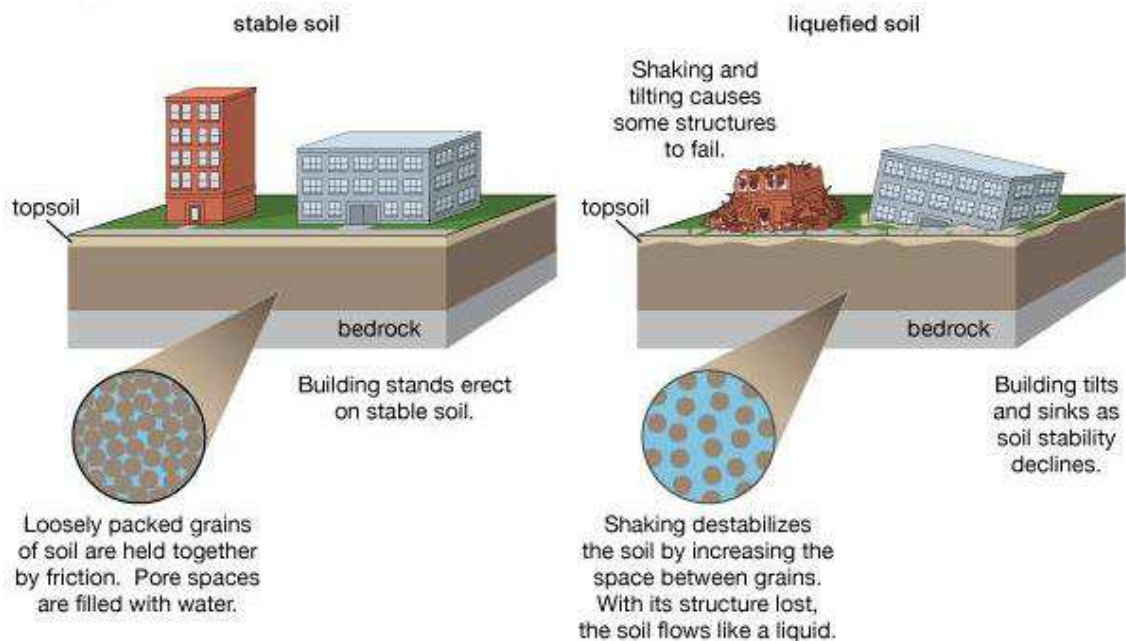


Figure III-6: Soil liquefaction, (Rafferty, 2012)

Overpressure capacity

The overpressure capacity is directly related to the permeability of a given soil. Large particle diameters ($>2\text{mm}$), are highly permeable (large voids between particles), where water evacuates instantly, and no overpressure can be built up. A decrease in particle sizes results in a decrease of permeability since fewer spaces between grains are in place. Therefore, sands ($0.1\text{-}2\text{mm}$) are able to build up some overpressure for a sufficient time to enable liquefaction.

Cohesion-less

It represents the interparticle forces between grains and can prevent a soil to achieve a fluid state (liquefaction) directly preventing grains to lose contact between them. For large diameter particle sizes ($>0.1\text{mm}$), like sands and gravels no cohesion is in place whereas lower diameters, like clays and silts, present cohesion.

Particle size testing: Sievering

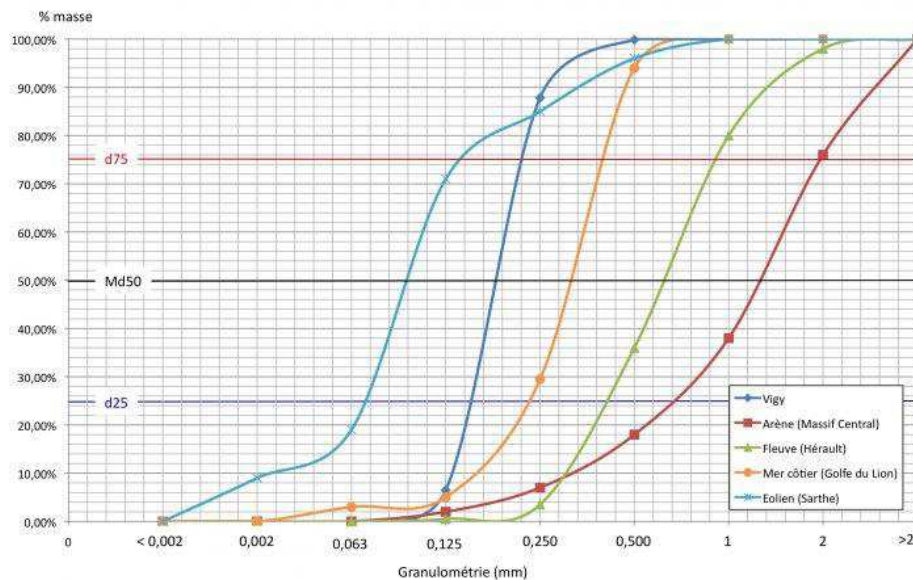


Figure III-7: Typical sand granulometry, fine (blue) to coarse (red), (Martin & Didier)

Sievering consists of passing a soil sample through different diameter square sieves. Between two consecutive sieves, certain amount of soil is retained, and weighted regarding the total soil sample. Therefore grain size distribution, granulometry, of a soil sample can be obtained, as shown in Figure III-7. Simple and fast test described ASTM E112-13.

Test limitations

Ideally, a single particle size determines a soil but actually a mixture of different grain sizes is observed. Within ASTM standard D2487-11, a unified soil classification system, USCS, determine the main soil type through percentages of different grain sizes families (sands, silts, clays,...).

Granulometry should be related to the liquefaction potential, quantified by the Beta factor. Increasing its value for higher clay/silt percentages or lowering it for pure sand distributions.

Currently, no Beta factor is related to particular grain distributions, only to main types. This analysis is out of the scope of this thesis but should be carefully analyzed once a bigger and more complete database is available.

b) Shape and angularity

PARTICLE SHAPE AND ANGULARITY ARE A SECOND MICROSTRUCTURAL ASPECT THAT CAN HAVE A MAJOR INFLUENCE AT THE LIQUEFACTION POTENTIAL OF A SOIL.

Indeed, to achieve liquefaction there is need pore water overpressure and cohesion less soil. These combined characteristics will reduce contact forces between grain particles that pore overpressure will absorb. Liquefaction potential will therefore depend on those contact forces, composed of friction and pressure.

Rounded shaped particles, perfectly suited for vibrodriving, will generate the lowest friction. Their smooth surface invites the particle to rotate endlessly, achieving a liquefied state. On the other hand, angular shaped particles are difficult/unable to liquefy due to its high friction.

Sieve analysis determines only the minimum square sieve aperture in which particles just pass¹⁷. This one-dimension description is insufficient to determine the shape and ratio of the other 2 dimensions, and so, insufficient to characterize the behavior of sand.

ASTM angularity test

Recovered from (Caltrans, 2007), Figure III-8 proposes a particle angularity and shape test that could reveal the liquefaction potential of soil particles. ASTM 2488-06 determines how to describe, for coarse sand or gravels sizes, those parameters.

Description	Criteria
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	Particles are similar to angular description, but have rounded edges
Subrounded	Particles have nearly plane sides, but have well-rounded corners and edges
Rounded	Particles have smoothly curved sides and no edges

Description	Criteria
Flat	Particles with width/thickness > 3
Elongated	Particles with length/width > 3
Flat and Elongated	Particles meet criteria for both flat and elongated

Figure III-8: Left, Descriptors for Particle Angularity, right, descriptors for Particle Shape, (Caltrans, 2007)

The main limitation of this test is its inaccuracy. Since particle descriptions are unprecise, subjective results could be in place. Relating them to numerical parameters that will have a huge influence in vibrodriving seems highly risked.

¹⁷ Defining only that the maximum particle dimension (x,y,z) is able to pass the square sieve.

Digital Image Processing method¹⁸

A new method is available to determine the other dimensions of the particle, or shape, Digital Image Processing, developed from (Vangla , Roy, Kavya, & Latha Gali, 2014). Using monochromatic light and a high definition camera, is possible to generate an image of the particles. With an algorithm written in MATLAB, real shape of the particles can be determined. This method tries to stablish how close the particle geometry is to a sphere or circle.

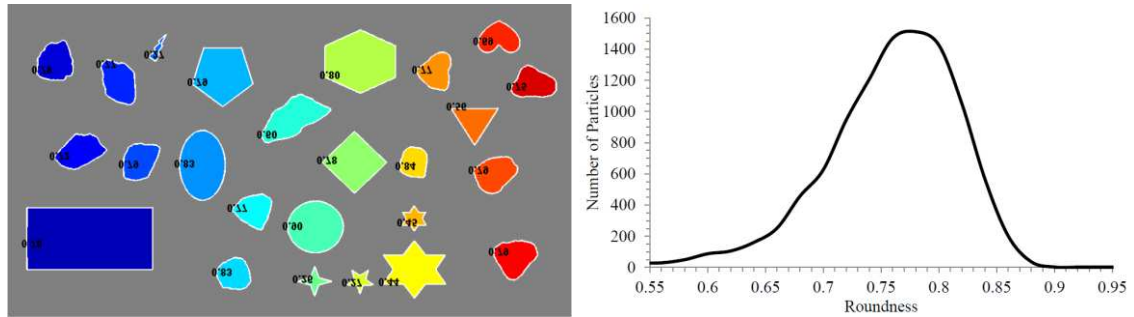


Figure III-9: Roundness values and histogram example, (Vangla , Roy, Kavya, & Latha Gali, 2014)

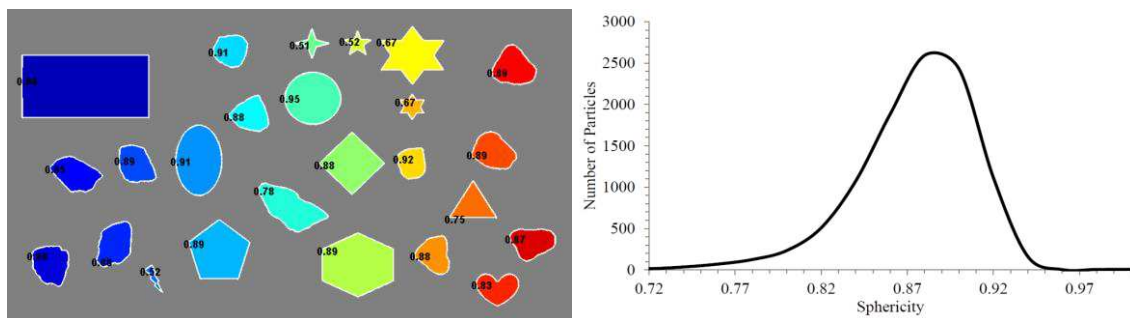


Figure III-10: Sphericity values and histogram example, (Vangla , Roy, Kavya, & Latha Gali, 2014)

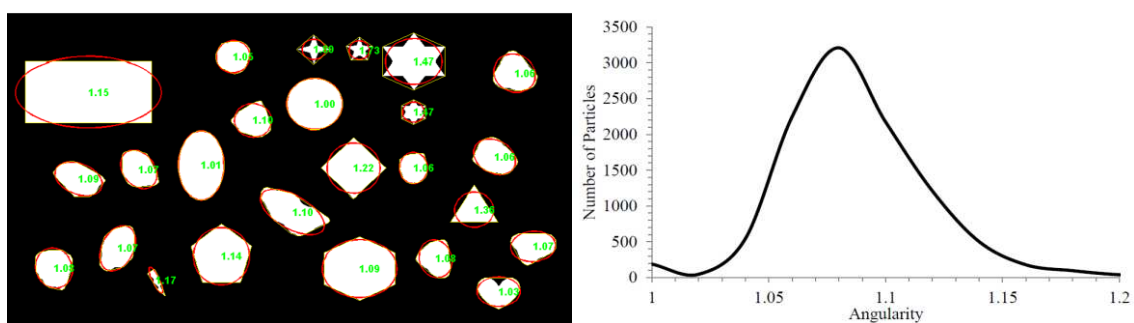


Figure III-11: Angularity values and histogram example, (Vangla , Roy, Kavya, & Latha Gali, 2014)

As shown, in Figure III-9, Figure III-10 and Figure III-11, are shown roundness, sphericity and angularity values and histograms. Combining those two an average, and more interesting, a mode value can be obtained.

¹⁸ Notice that this computer-based test will eliminate the subjectivity related to the ASTM test.

As explained, shape and size of grains will determine Beta factor value. Therefore, to obtain this factor a correlation with those two mode values should be done. **Low angularity, high sphericity or high roundness should be related to a high liquefaction potential**, and therefore to low beta values.

c) Carbonate content

THERE ARE TWO MAIN TYPES OF SANDS, QUARTZ-SILICA ONES AND CALCAREOUS ONES. QUARTZ-SILICA SANDS CAME FROM RIVERS THROUGH EROSION OF ROCKY MATERIALS. IT'S A COMMON TYPE OF SAND CHARACTERIZED BY ITS HIGH STRENGTH AND LOW COMPRESSIBILITY.

Calcareous sands are defined by a lower strength than silica, highly crushable and compressible. According to (Verastegui, 2014), Calcareous Sands came from organic debris in the ocean that have been accumulated through years and deposited in layers on the seabed.

Characteristics

The main component is calcium carbonate, a crushable organic compound. Because of its organic nature, this sand can be crushed through high stresses, creating new smaller particles, highly unstable, with unknown cone or friction resistance. According to (Verastegui, 2014), usually carbonate sands are characterized by angular particles. That could mean a higher shaft Beta factor (high friction).

Carbone content test

The ASTM method D 4373 – 96, determines the quantity of calcium carbonate (CaCO_3) of a given soil. The test uses hydrochloric acid with a dried soil sample in an enclosed reactor. A chemical reaction is unleashed between acid and carbonate calcium releasing CO_2 . This release of gas increases the pressure inside the enclosed reactor. Monitoring the pressure, the amount Carbonate content of a given soil can be determined.

Potential hazards

As a general estimation, carbonate soils represent 35% of the ocean seabed, a non-neglectable amount. Determination of carbonate content in sands could be interesting to predict strange soil behaviors. The real response of this kind of soil is hardly modellable. Therefore, it should be assessed as a risk.

3. Soil parameters: Defining soil behavior

SOIL PARAMETERS CAN BE DETERMINED BY SEVERAL CORRELATIONS OR TESTS. HERE ARE SUMMARIZED HOW TO CONVERT MAIN ONES TO CPT VALUES IN CASE NO OTHER SOIL INVESTIGATION IS AVAILABLE.

a) Behavior Index

BEHAVIOR INDEX IT'S A TERM DEVELOPED BY ROBERTSON IN 1990 THAT ESTIMATES SOIL BEHAVIOR TYPE (SBT) BY RELATING IT WITH ITS OWN CPT VALUES. CONE RESISTANCE AND FRICTION SLEEVE ARE USED TO STABLISH GENERAL BEHAVIOR OF THE SOIL AND ARE RELATED TO A SOIL TYPE. WORLWIDE USED, SUPPORTED BY A LARGE DATABASE.

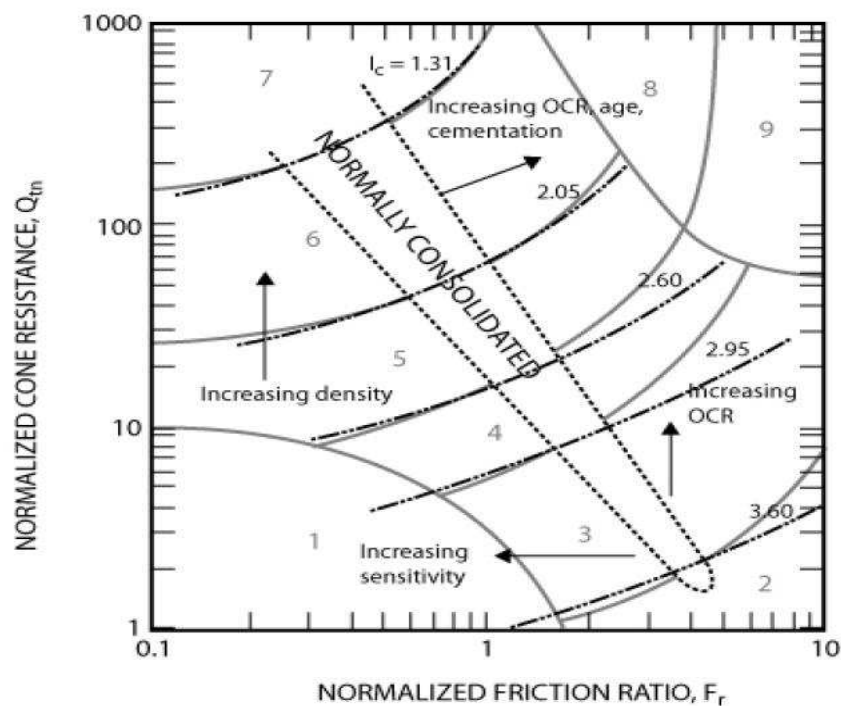


Figure III-12: SBTn Robertson Abacus, (P.K. Robertson, K.L. Cabal, 2015)

Zone	Soil Behavior Type	I_c
1	Sensitive, fine grained	N/A
2	Organic soils – clay	> 3.6
3	Clays – silty clay to clay	2.95 – 3.6
4	Silt mixtures – clayey silt to silty clay	2.60 – 2.95
5	Sand mixtures – silty sand to sandy silt	2.05 – 2.6
6	Sands – clean sand to silty sand	1.31 – 2.05
7	Gravelly sand to dense sand	< 1.31
8	Very stiff sand to clayey sand*	N/A
9	Very stiff, fine grained*	N/A

Table III-1: Main soil SBTn intervals, (P.K. Robertson, K.L. Cabal, 2015)

According to (P.K. Robertson, K.L. Cabal, 2015), the following formula is used:

$$Q_{tn} = \frac{q_c - \sigma_{vo}}{p_{atm}} * \left(\frac{p_{atm}}{\sigma_{vo'}} \right)^n \quad \text{Equation III-6}$$

With the stress exponent n as:

$$n = 0,381 * I_c + 0,05 * \left(\frac{\sigma_{vo'}}{p_{atm}} \right) - 0,15 \quad \text{Equation III-7}$$

And with the stress exponent as:

$$I_c = \sqrt{((3,47 - \log(Q_{tn}))^2 + (1,22 + \log(F_r))^2} \quad \text{Equation III-8}$$

With the Normalized Friction Ratio as:

$$F_r = \frac{f_s}{q_c - \sigma_{vo}} * 100 \quad \text{Equation III-9}$$

Notice that to calculate the index, we need to make some iterations to calculate the stress exponent and finally have the final Soil Behavior Type.

b) Friction Angle

DEFINES FRICTION BETWEEN PARTICLES. THE SOIL DESCRIBED COULD STAND STATICALLY WITH A MAXIMAL SLOPE EQUAL TO THIS ANGLE. FOR SANDS THIS ANGLE RANGE FROM 25° TILL 45°.

The general Mohr-Coulomb strength criterion is, as follows:

$$\tau_{max} = c' + \sigma_{vo}' \tan(\phi') \quad \text{Equation III-10}$$

Several correlations are able to determine this value from the CPT cone resistance. A small review between some of them reveals the scatter of possible results. Over formulating the soil can be unproductive¹⁹. I selected the following correlation from (Kulhawy & Mayne, 1990) which is widely accepted, where $Q_{t1} = Q_{tn}$:

$$\phi' = 17,6 + 11 * \log(Q_{tn}) \quad \text{Equation III-11}$$

Formula that can be extended as:

$$\phi' = 17,6 + 11 * \log \left(\frac{q_c - \sigma_{vo}}{p_{atm}} * \left(\frac{p_{atm}}{\sigma_{vo'}} \right)^n \right) \quad \text{Equation III-12}$$

Generating the cone resistance of a certain friction angle soil (for quartz silica sands):

¹⁹ Notice that several correlations are in place, and usually they are approximated to local experience. Changing between correlations only varies empirical fitting parameters without enlarging parameter precision. Deciding which correlation fits better to a study case is left to the engineer.

$$q_c = \left(\frac{10^{\frac{\phi' - 17,6}{11}}}{\left(\frac{p_{atm}}{\sigma_{vo'}} \right)^n} \right) * p_{atm} + \sigma_{vo} \quad \text{Equation III-13}$$

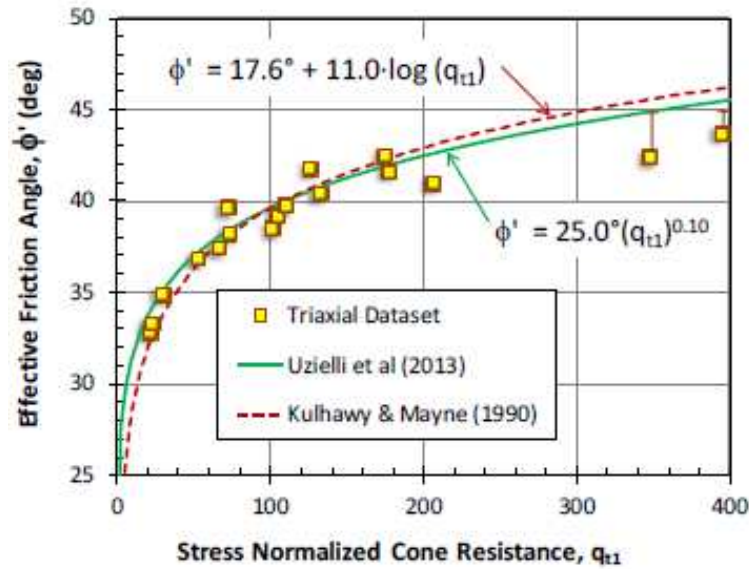


Figure III-13: Direct relationships for friction angle of Quartz-Silica sands, (Mayne, 2014)

c) Relative density

RELATIVE DENSITY IS A SOIL PARAMETER THAT DETERMINES THE COMPACITY, VOID RATIO, OF A GIVEN SOIL. DR=100% WOULD BE A SOIL AT MAXIMUM COMPACITY.

$$D_r = \frac{e_{max} - e_o}{e_{max} - e_{min}} \quad \text{Equation III-14}$$

Initial void index, e_o , could be determined by oedometric laboratory test. Whereas determination of e_{max} and e_{min} are far more complicate. The table below regroup some typical values for specific sands.

Type	e_{max}	e_{min}
Monterey Sand	0,85	0,56
Tin Shui Wai Sand	0,69	0,305
Toyoura Sand	1,03	0,61
Ottawa Sand	0,82	0,5

Table III-2: Maximal and minimal void ratios, (Jefferies & Been, 2006)

Despite the difficulty of establishing those minimum and maximum void ratios, several correlations between CPT and relative density have been made to bypass those uncertain parameters. For this thesis, (Jamiolkowski et al., 2001) formula will be used:

$$D_r = 100 * \left[0,268 * \ln \left(\frac{q_c / \sigma_{atm}}{\sqrt{\sigma'_{vo} / \sigma_{atm}}} \right) - 0,675 \right] \quad \text{Equation III-15}$$

Again, it can be rearranged as it follows to generate a soil profile:

$$q_c = \sigma_{atm} \sqrt{\sigma'_{vo} / \sigma_{atm}} * e^{\frac{D_r / 100 + 0.675}{0.268}} \quad \text{Equation III-16}$$

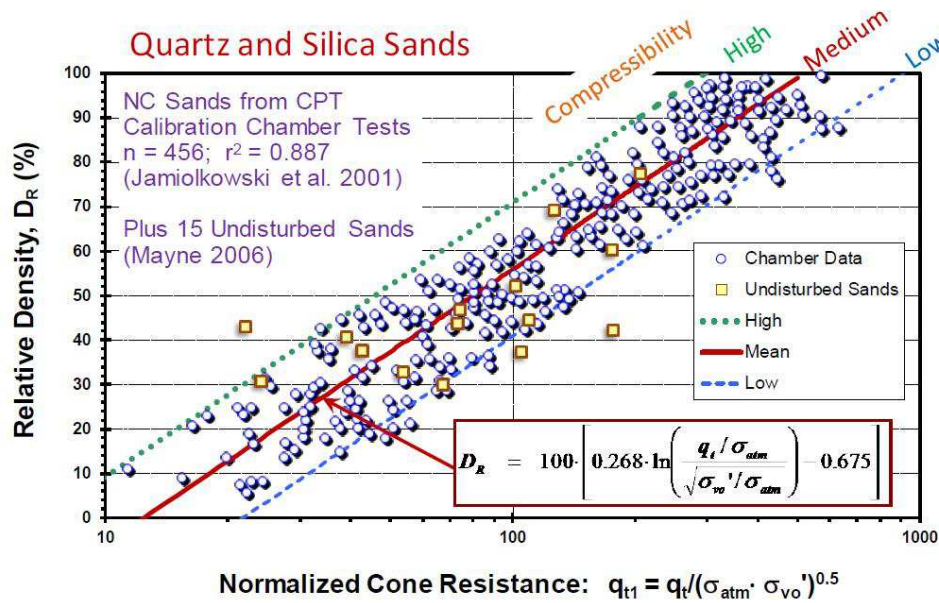


Figure III-14: Relative Density from CPT's on Quartz-Silica Sands, (Mayne, 2014)

Relative density could be altered by vibrodriving, due to soil rearrangement, varying if the soil is densified (contractive) or loosened (dilative). Those soil evolutions can be predicted or estimated with our next parameter, the State parameter.

d) State parameter

STATE PARAMETER CHARACTERIZE SOIL BEHAVIOR, DILATIVE OR CONTRACTIVE. IT'S RELATED TO SOIL PARAMETERS AS SHEAR MODULUS AND COMPRESSIBILITY AND CAN BE LINKED TO RELATIVE DENSITY.

$$\psi = e_o - e_{cs} \quad \text{Equation III-17}$$

Interesting parameter able to define how a given soil will evolve or at least in which expected direction. Critical state theory states that when a soil is stressed it evolves from

its current/initial void ratio, e_o , towards it's critical state void ratio, e_{cs} . As (Jefferies & Been, 2006) stated, there are two possibilities to reach that critical state:

Dilatancy, $e_o < e_{cs}$

A lower initial void ratio (higher compacity) than critical state one (lower compacity) is common for highly dense soils. When cyclically stressed, particles tend to critical state void ratio. Dilatancy therefore increases soil volume, rearranging particles into a bigger form of packing and finally becoming a loosened soil (regarding the initial state).

Contractive, $e_o > e_{cs}$

A higher initial void ratio (lower compacity) than critical state one (higher compacity) is common for loosened soils. Again, when cyclically stressed, particles tend to critical state void ratio. Contractive therefore reduces soil volume, rearranging particles into a smaller form of packing and finally becoming a dense soil (regarding the initial state).

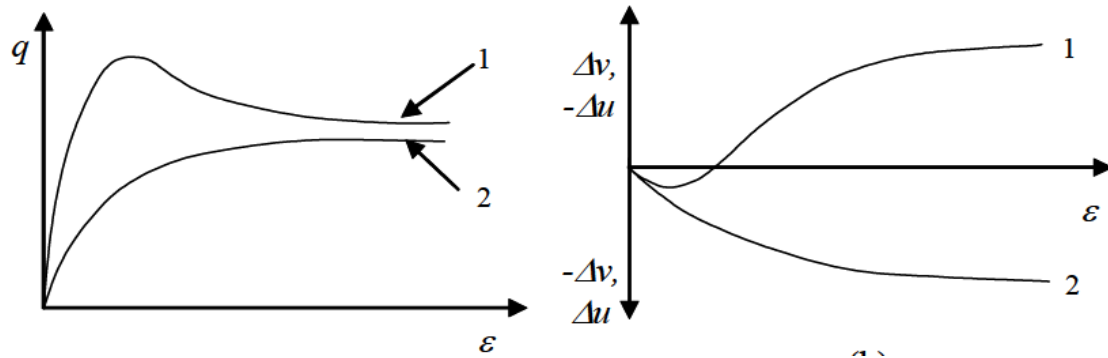


Figure III-15: Evolution of stressed Sand, (1) Dilative, (2) Contractive, (Kramer, 1996)

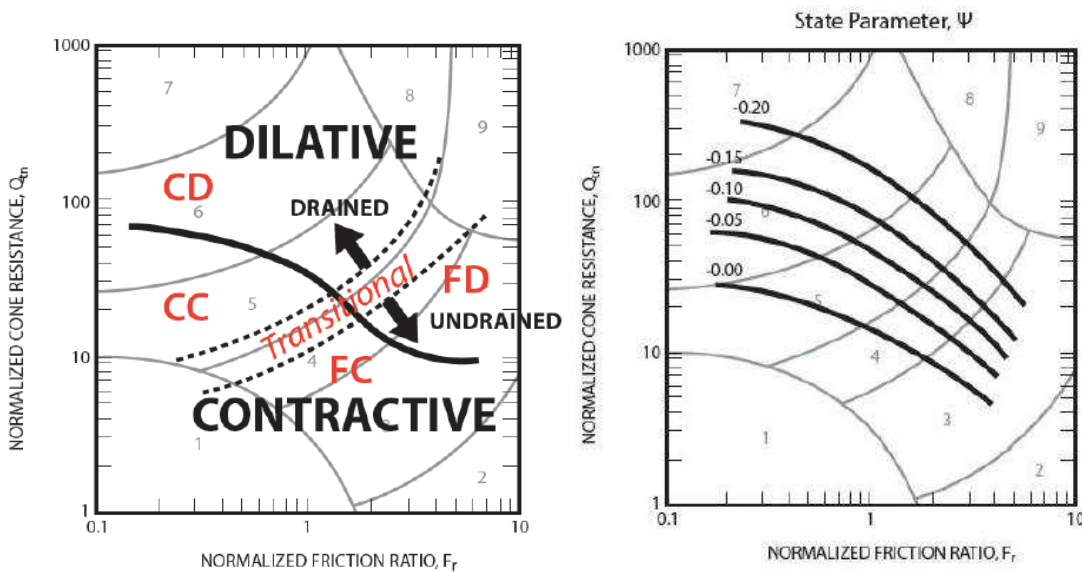


Figure III-16: SBTn Chart, different Soil (left), and, different state parameter (right), (P.K. Robertson, K.L. Cabal, 2015)

From (Jefferies & Shuttle, 2011) and shown in Figure III-16, a soil with a negative state parameter will be dilative and a positive one will represent a contractive soil. According to (P.K. Robertson, K.L. Cabal, 2015), dilative soils will have a state parameter below -0.05 and the above will behave as contractive. As usual we will relate it to CPT values as (Robertson & Cabal, 2014) proposed:

$$\psi = 0,56 - 0,33 * \log(Q_{tn,cs}) \quad \text{Equation III-18}$$

Again, this equation can be rearranged to be simulate a soil for a given state parameter:

$$q_c = \left(\frac{10^{\frac{\psi-0,56}{-0,33}}}{\left(\frac{p_{atm}}{\sigma_{vo'}}\right)^n} \right) * p_{atm} + \sigma_{vo} \quad \text{Equation III-19}$$

Both phenomena's can have a strong influence in vibrodriving. This hardening or loosening behavior can increase or reduce toe and/or shaft resistance. State parameter could lead to a better understanding of the cyclically loaded soil behavior, defining the soil resistance degradation, therefore the Beta factor value.

Notice that cone resistance values could evolve during or after vibrodriving. A slower penetration rate than predicted could be explained by a densification of the soil whereas a faster one, a loosening. Keep in mind that those variations can be explained by too many parameters. Only through an initial CPT, monitoring vibrodriving and a post CPT, reliable conclusions about the soil state could be performed.

4. Conclusion

- ✓ CPT test, worldwide accepted, is at the base of soil resistances determination.
- ✓ Particle parameters mainly size and shape ones, define microscopically the macroscopic soil fatigue potential (Beta factor).
- ✓ Most soil parameters can be related to CPT values and therefore used to model a soil profile.
- ✓ Friction Ratio and Soil Behavior Index can estimate soil type.
- ✓ Relative density but mainly state parameter could reveal the macroscopic soil fatigue potential as well.

Limits of the study

- CPT cannot be performed in very hard soils.
- Accurate particle shape tests are not currently used.
- Critical state void ratio, defining the state parameter is difficult to define. Due to usual sampling disturbance, initial void ratio could be also not perfectly accurate.

IV. Implementations and tools for vibrodriving analysis

VIBRATORY DRIVING MODELS REMAIN UNTESTED DUE TO THE LACK OF VIBRATORY FIELD TESTS. AS A CONSEQUENCE, NO LEARNING CURVE FROM EXPERIENCE CAN BE DEVELOPED YET, WHERE UNEXPECTED PREDICTION RESULTS ARE COMMON. MEANWHILE, MATCHING PREDICTIONS WITH MEASUREMENTS GENERATES AN EXCESSIVE AMOUNT OF TIME-CONSUMING SIMULATIONS TRYING TO REVEAL SOIL BEHAVIOR. TOOLS PRESENTED IN THIS CHAPTER INTEND TO FASTEN THIS PROCEDURE.

1. Types of vibrodriving analysis

ALLWAVE-PDP[®] WAVE EQUATION SOFTWARE WILL BE USED TO PERFORM PREDICTIONS BASED ON THE VIBRODRIVING MODEL PRESENTED IN CHAPTER II. ALLNAMICS-VDA[®] MONITORING SOFTWARE WILL BE USED TO PERFORM VIBRODRIVING MEASUREMENTS WITH ALLNAMICS[®] OWN SENSORS.

a) Software calculations: Single and multiple runs

Single runs

Allwave-PDP[®] predicts vibrodriveability response **at a selected pile penetration**, for a given soil, pile and vibrator. As main outputs, Forces, Accelerations, Velocities and Displacements (F, a, v, u) over time will be provided. As shown in Figure IV-1, deriving the mean displacement trend (red), penetration speed (v_p) will be calculated (black). Refusal is in place when displacement trend line is flat, ($v_p = 0$).

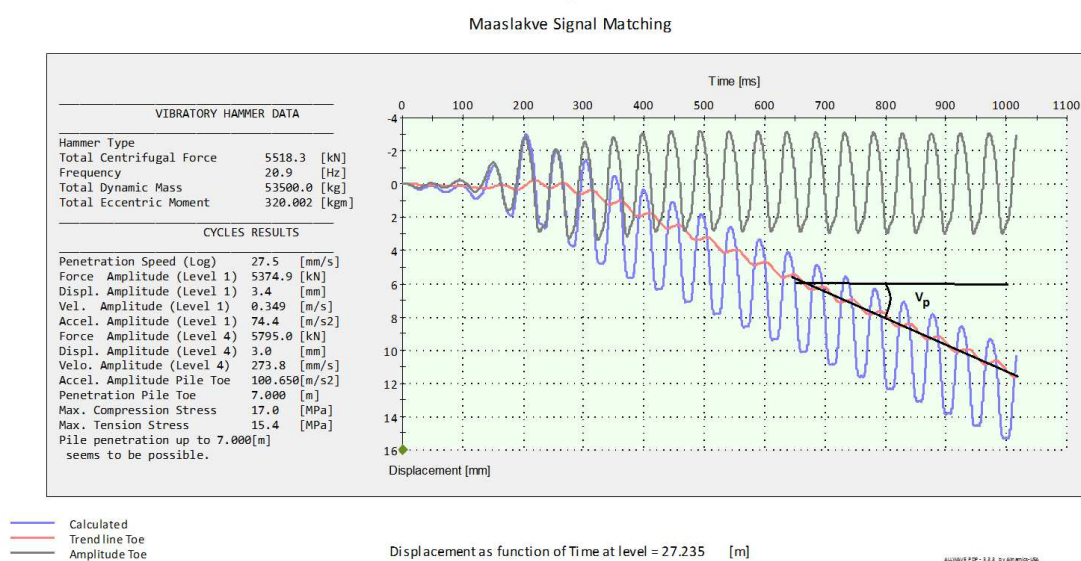


Figure IV-1: Single run example on pile toe displacement

Multiple runs

Multiple runs will recreate pile driving installation by performing consecutive single runs. It provides a wider perspective about the whole vibrodriving process, every selected penetration step. As outputs, (F, a, v, u) amplitude responses are provided.

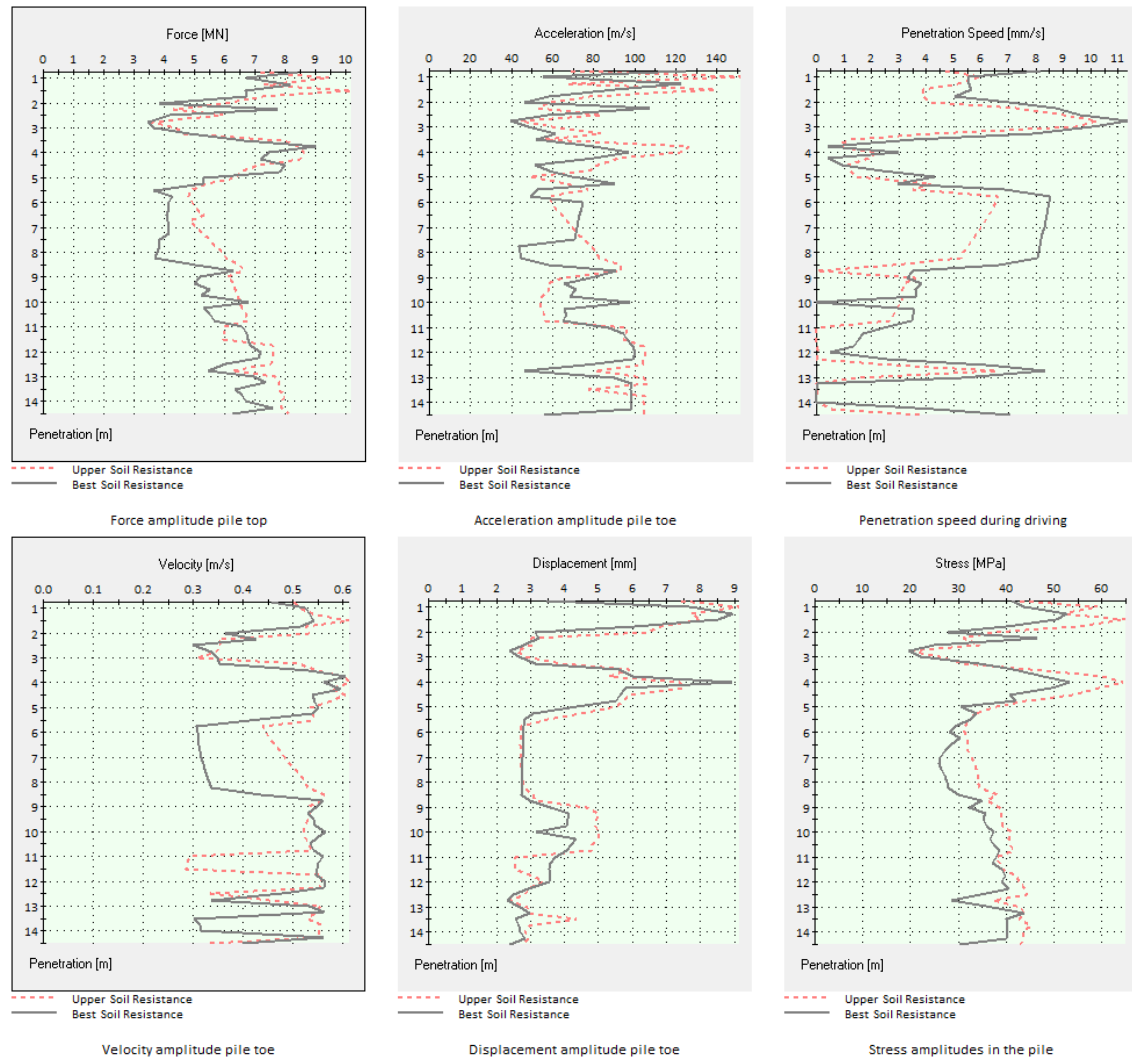


Figure IV-2: Multiple run example

b) Vibratory Driving Prediction: VDP

ANALYSIS OF THE VIBRODRIVEABILITY

First analysis to perform before starting a project, **aiming to determine at which penetration (multiple runs) refusal depth is predicted**. Different simulated frequencies can cover a wider range of possibilities. To increase certainty of predictions, upper and lower boundaries of soil resistances can be used and seen as security factors²⁰.

²⁰ Notice that soil investigations often produce a large scatter of results that other engineering branches may not be used to. Upper and lower boundaries are used to regroup main possible vibrodriving scenarios. Assessing acceptable risks is an important factor to take into account.

c) Vibratory Driving Analysis: VDA

MONITORING VIBRATORY DRIVING

Vibrodriving is monitored through accelerometers and strain gauges. This monitoring can be used to estimate soil dynamic behavior. Forces are obtained from stresses through Hooke's law, shown in Equation II-3. Velocities and displacements are obtained by simple and double integration of the measured acceleration. **Through VDA, (F, a, v, u) amplitudes responses are recorded.** Penetration speed is monitored as well, usually averaged each 25 cm pile penetration steps.

d) Vibratory Amplitude Matching: VAM

POST-ANALYSIS THAT RECREATES THE MONITORED SOIL MODEL.

Aims to model empirically the dynamic soil behavior by matching VDA measurements with VPD predictions. Simulations will match measurements through the comparison of main variables (F, a, v, u) amplitude responses and frequency applied²¹. Hopefully, pile bearing capacity could be assessed in the near future.

Due to the continuous applied force of vibratory driving, amplitude responses will integrate: vibrator forces, mobilized soil resistances and stress waves reflections. **An ambition can be set to extract soil resistances from these combined results**, allowing one to characterize soil dynamic behavior and quantify soil parameters.

Amplitude matching is performed through a series of VDP simulations that will match VDA measurements by adjusting soil parameters. Vibrator and pile specifications will be known beforehand, leaving soil dynamic behavior, the only unknown. When predictions and measurements agreed²², VAM is done. New soil parameters should be added to a database and continue the learning loop.

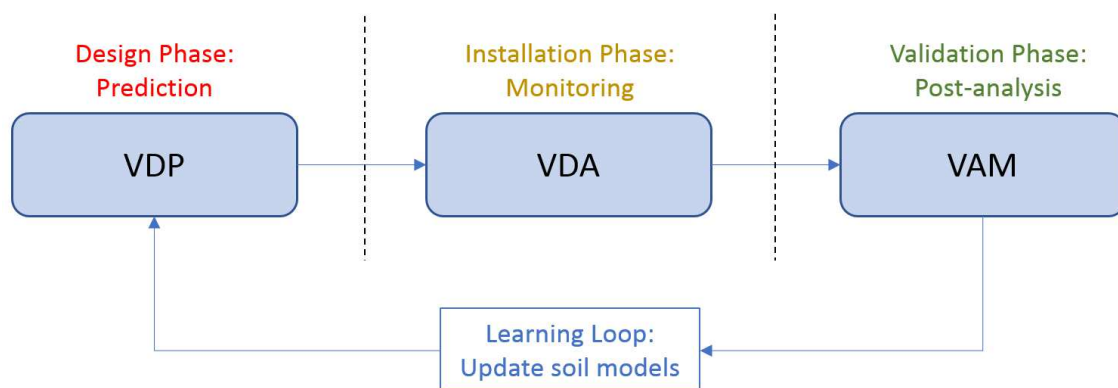


Figure IV-3: Phases of a vibratory project

²¹ Penetration speed isn't accurate enough to perform matchings still and will be indicative mainly.

²² Notice that perfect or even similar signal matching is not always possible. Some deviations will be presented and could not be further improved. See Section II.5.d).

2. Frequency analysis

FREQUENCY DURING VIBRODRIVING OSCILLATES BECAUSE OF ENCOUNTERED SOIL RESISTANCE OR A HAMMER OPERATOR TRYING TO OVERCOME A CRITICAL PENETRATION. CURRENT VIBRODRIVING PREDICTIONS ASSUME THAT A CONSTANT FREQUENCY IS APPLIED, WHEREAS THE REALITY IS COMPLETELY DIFFERENT: THERE IS A NEED TO PERFORM A FREQUENCY DEPENDENT ANALYSIS.

a) Frequency effects: Soil-Pile-Vibrator interactions and Resonance

A deviation has been observed by (Whenham & Holeyman, 2012), between generated vibratory force and measured force on the pile. To account for this phenomenon, the efficiency factor was introduced, a ratio between measured force on the pile and theoretical force generated by the vibrator ($\eta = F_{measured}/F_{vib}$). Their analysis concluded that soil-pile-vibrator interactions should be taken into account while resonance and anti-resonance may strongly affect pile response.

Soil-Pile-Vibrator interactions

As showed in Figure IV-4, ideal models include all vibrodriving actors, isolating soil parameters as key unknowns. However, the net effect of the static mass of the vibrator is not perfectly modeled, nor accurately accounted for as explained in Section II.3.d).

This mass, usually considered as a dead weight, depending on the hook pull of the crane.

Recording hook crane loads during driving is imperative, as stated by (Whenham & Holeyman, 2012). Next to that, the static part of the vibrator is connected to the vibrating part through elastomer pads (springs) of a non-specified stiffness from vibrohammers manufacturers. Therefore, the static part oscillates vertically when excited, generating second order inertia forces and dynamic loads.

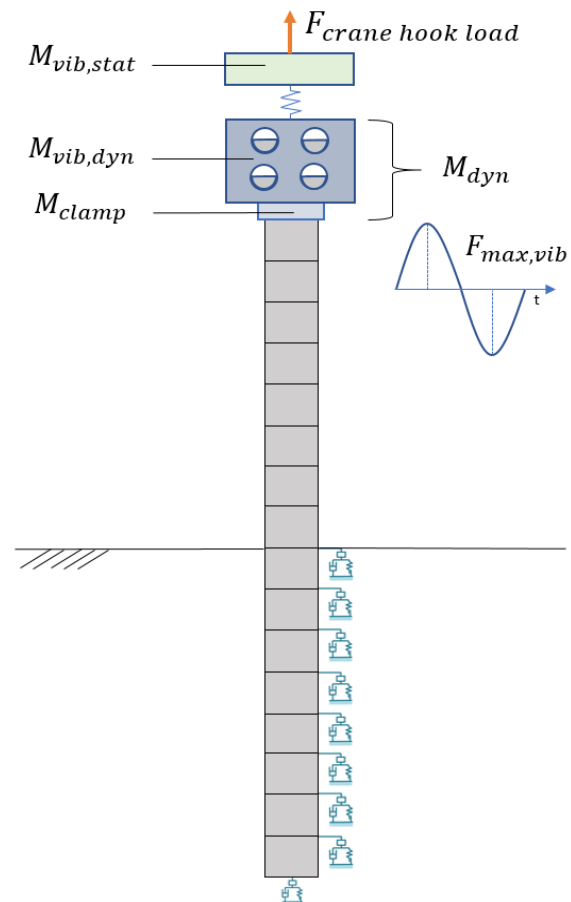


Figure IV-4: Ideal Vibrodriving Model

Mode of vibration / Resonance effects

Vibrator generated stress waves propagates through the pile interacting with soil resistances, as shown in Figure II-4. Stress waves summation of effects generate particular amplitude responses, as can be seen in Figure IV-5 with a high variability.

Given that vibratory pile-soil system, **superposition of stress waves, when sustained for a long period as it is the case, develop a standing wave pattern or mode of vibration.** This mode of vibration causes the response to be amplified at certain pile locations while it vanishes at others.

Within these modes of vibrations, resonance is defined as a frequency and penetration depth dependent phenomenon characterized by the amplification of the amplitude response (F, a, v, u) to a specific vibratory stress wave stimulus. Resonance is therefore related to a specific variable at a specific location, noting that resonance may occur only at certain pile levels. For example, sensors at the top level could measure a resonance situation, while antiresonance may take place at the toe level. Pile and vibrator specifications will be constant through the entire process. Therefore, besides soil parameters, frequency and pile penetration will be our two key operating variables.

A peak amplitude response is a **resonance condition, where stress waves effects add up to a higher response of the considered variable (F, a, v, u) at that location.** By contrast, a depression on the amplitude response is an **antiresonance condition, where stress wave effects add up to a minimum response of the considered variable (F, a, v, u) at that location.** Those resonance and antiresonance conditions can be shifted by minimal variations of soil parameters. These conditions are local within a defined range. Predicting and identifying those resonances, but more importantly anti-resonances, could lead to a more efficient vibrodriving²³.

In author's opinion, the superposition of stress waves effects under the action of the static mass may account for counter-intuitive deviations between generated and measured force. Such deviations may enable, for instance, penetration with a hard toe resistance, and early refusal for a softer one, using same parameters, as will be shown in section IV.3.c).

²³ Resonance and antiresonance can be understood as mode of vibration, which can be efficient or inefficient depending on the stable wave pattern developed.

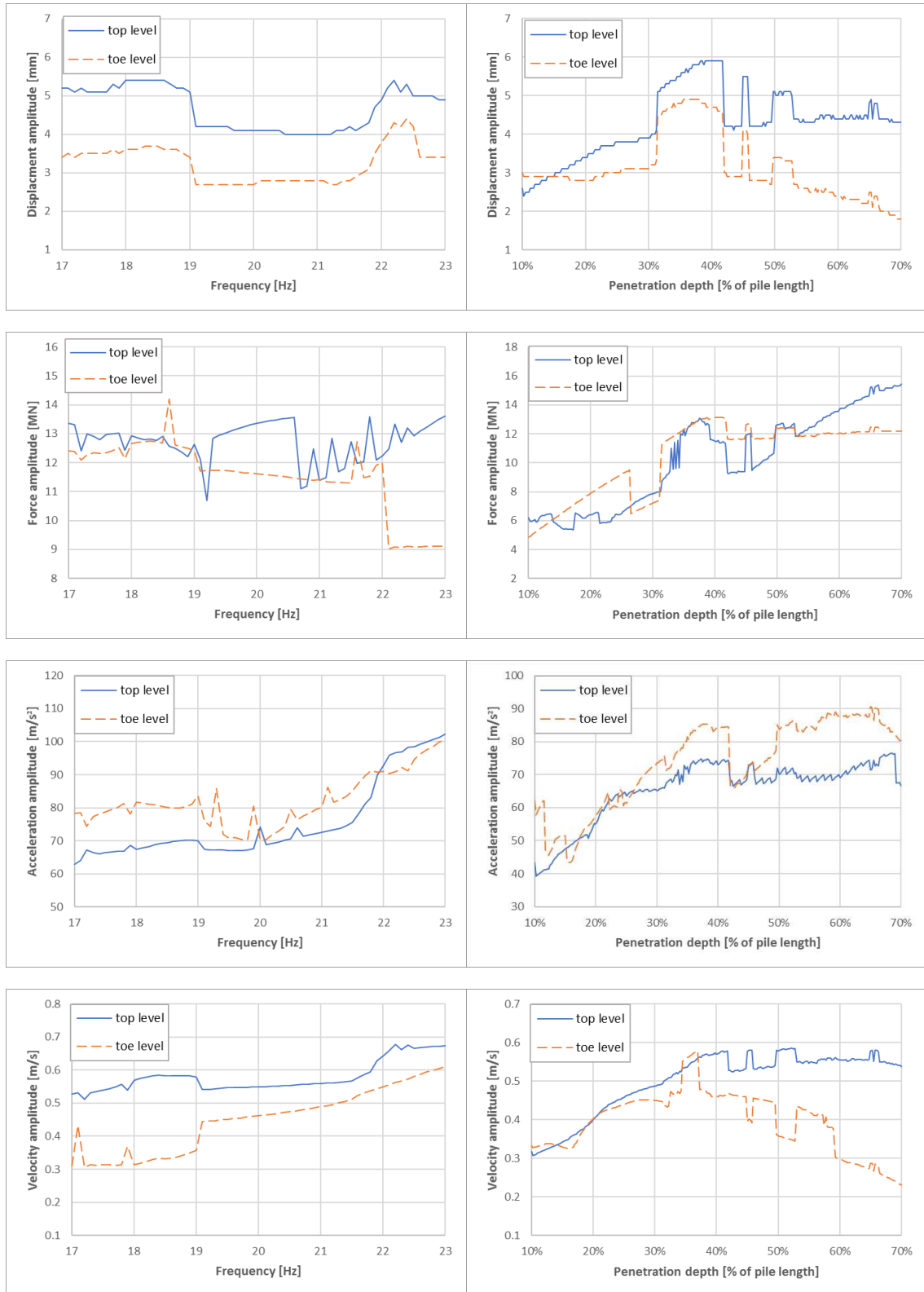


Figure IV-5: Displacement, force, acceleration and velocity amplitude responses vs. frequency at 50% pile penetration and vs. penetration depth at 19Hz frequency – 40m pile, default soil parameters, IDV soil profile

b) Frequency-Penetration Response Spectrum: FPRS

FPRS INTENDS TO COVER THE POSSIBLE VIBRODRIVING SCENARIOS BY PERFORMING SIMULATIONS IN A FREQUENCY-PENETRATION PLANE, AVOIDING FOR EXAMPLE CRITICAL FREQUENCY-PENETRATION COMBINATIONS THAT LEAD TO EARLY REFUSAL.

(F, a, v, u) amplitude response varies regarding frequency and penetration, given soil, pile and hammer data, as Figure IV-5 showed. Unexpected behavior that is not intuitive to spot, mainly because of stress waves summation of effects. Sometimes higher frequencies, therefore higher centrifugal forces, lead to an inefficient mode of vibration. Lower frequencies may develop a more efficient mode of vibration at some depths.

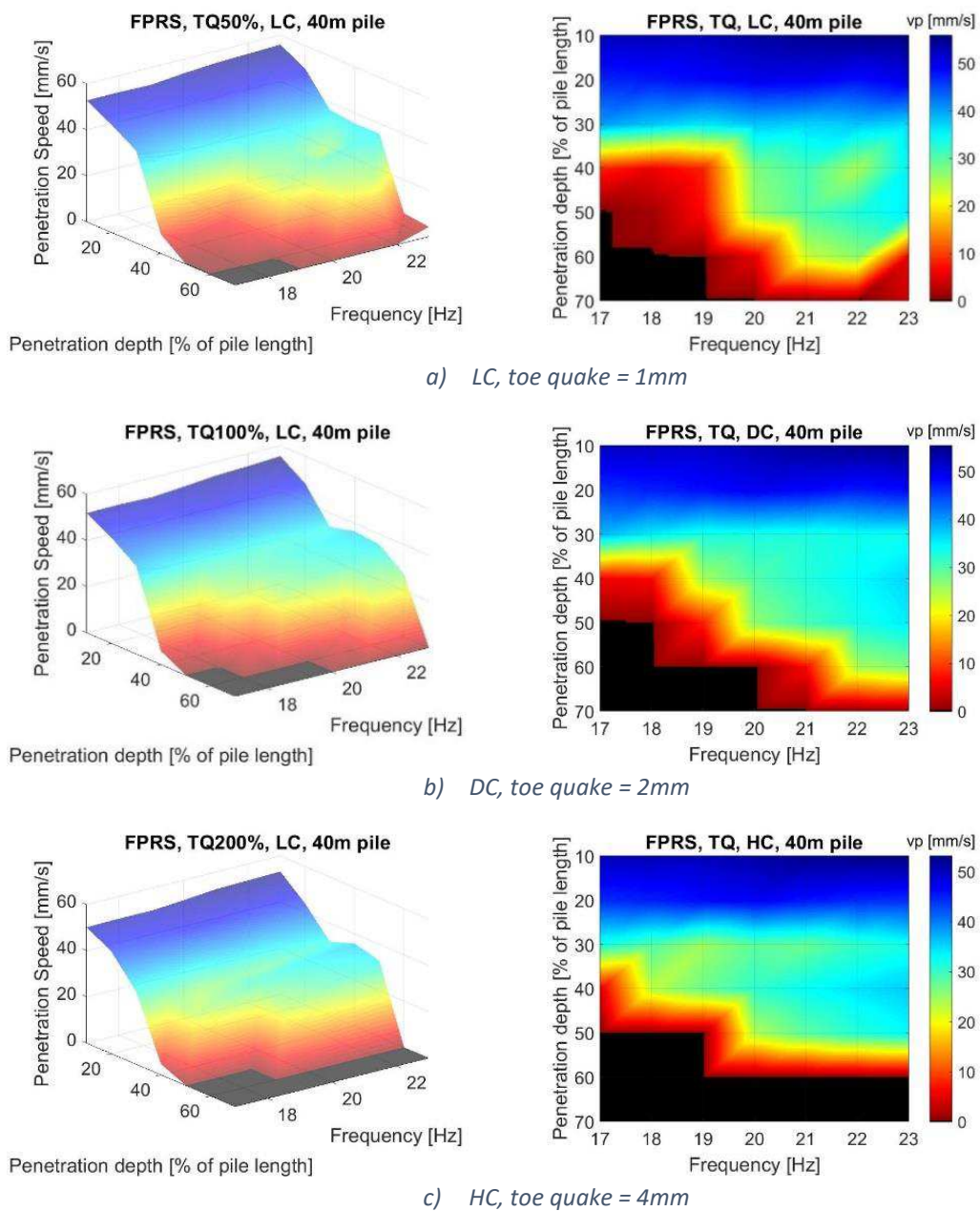


Figure IV-6: Sample FPRS: Penetration speed, 40m pile, Toe Quake, IDV soil profile

A frequency-penetration analysis, the result of which will be called “Frequency-Penetration Response Spectrum” (FPRS), gathers possible responses covering those two key operational variables. The FPRS is represented as a 3-dimensional graph, with x and y axes as frequency and pile penetration, respectively, and with z (color axis), as the amplitude response (F, a, v, u), see Figure IV-6.

The FPRS developed from the IDV soil profile combines 7 frequencies with 7 penetrations for the 3 cases (LC, DC and HC) of each considered soil parameter (e.g. quake, damping, beta factor). Simulated frequencies range from 17 to 23 Hz, with steps of 1 Hz, representing usual working frequencies for large vibrators. Pile penetrations range from 10 to 70% of the pile length²⁴, with steps of 10%. Values between the frequency-penetration grid intersections are calculated by interpolation. Therefore, 49 FP intersections have been simulated with a wave equation software to accommodate 3 sets of 4 soil parameters featured in 3 pile lengths for the IDV soil profile. A total of 1764 problems have thus been simulated to produce 36 FPRS in order to test the usefulness of the “vibrodriving trend” concept, introduced below.

c) Sensitivity analysis: Vibrodriving trends based

VIBRATORY DRIVING GATHERS ALL THE COMBINED VIBRATOR FORCES, SOIL REFLECTIONS AND SUMMATION OF EFFECTS SIMULTANEOUSLY INTO A SINUSOIDAL SIGNAL. ISOLATING THE RESPONSE OF A SPECIFIC SOIL LAYER IS EXTREMELY TIME-CONSUMING. IN ORDER TO FASTEN THIS PROCEDURE, VIBRODRIVING TRENDS AND SENSITIVITY ANALYSIS ARE INTRODUCED.

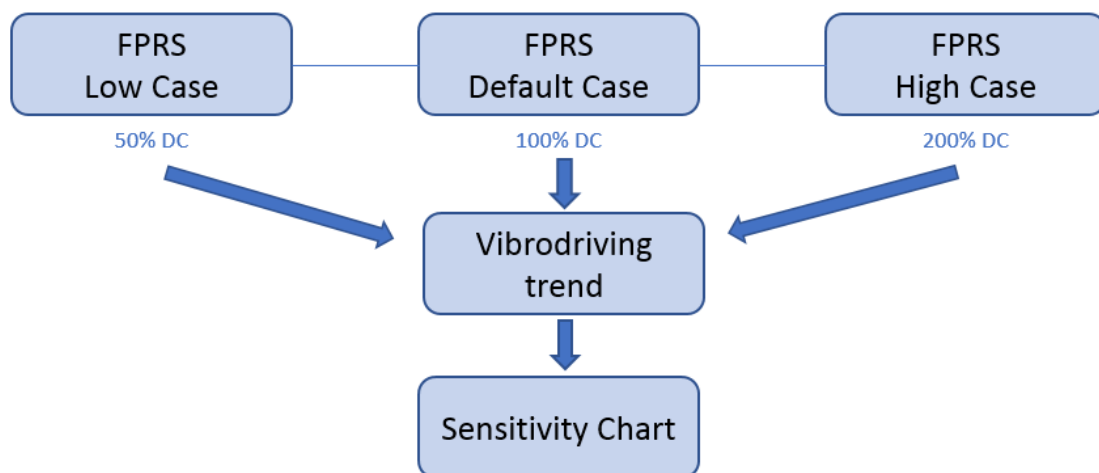


Figure IV-7: Sensitivity analysis procedure

²⁴ One of the main problems of vibrodriving analysis is its lack of a normalized reference. Bearing in mind that the goal is to compare different vibrodriving cases, % of the pile length and range of working frequencies intends to tend towards that objective.

Vibrodriving trends

Each analysis produces 3 FPRS according to the selected soil parameter for a given soil profile and a given pile length. Per Section IV.3.a) a selected soil parameter can be factored from their DC value by 200% (HC) and by 50% (LC) to cover a range where soil parameters are expected to vary. Comparing different cases from Figure IV-6, allows to establish 3 types of trends/correlations for each amplitude response (F, a, v, u), defined (in the case of acceleration) as:

$$if(a_{LC} < a_{DC} \& a_{DC} < a_{HC}) \Rightarrow +1$$

Positive effect on acceleration

Equation IV-1

$$if(a_{LC} > a_{DC} \& a_{DC} > a_{HC}) \Rightarrow -1$$

Negative effect on acceleration

Equation IV-2

$$if(\neq \text{Positive} \& \neq \text{Negative}) \Rightarrow 0$$

Unstable effect on acceleration

Equation IV-3

When positive or negative trends are spotted, the amplitude response (F, a, v, u) to a given soil parameter can be anticipated, increasing (positive) or decreasing (negative) the amplitude response. “Unstable” trends are usually associated with mixed trends, when variations of the considered soil parameter produce erratic changes on the amplitude response, disabling the indicative value of such a prediction tool.

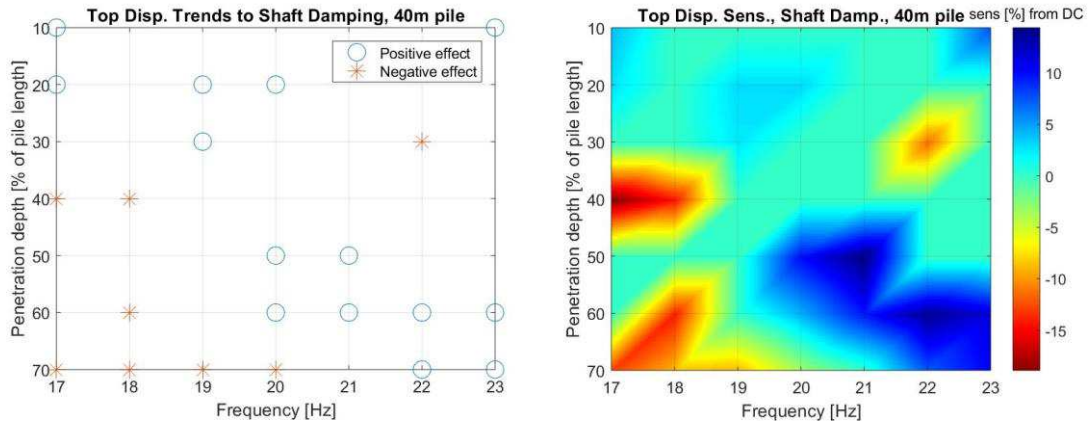


Figure IV-8: Top displacements trends (left) and sensitivities (right) regarding Shaft Damping

Sensitivity charts

A sensitivity chart characterizes the sensitivity of a tracked amplitude response with respect to a given soil parameter, using the following definition of sensitivity:

$$se [\%] = (a_{HC} - a_{LC}) / 2a_{DC}$$

Equation IV-4

$$se^{\uparrow} [\%] = a_{HC}/a_{DC} - 1 \quad \text{Equation IV-5}$$

$$se^{\downarrow} [\%] = 1 - a_{LC}/a_{DC} \quad \text{Equation IV-6}$$

Equation IV-4 is used to calculate values for the displayed in Figure IV-8 sensitivity chart. This chart provides insight on the average amplitude response reaction to a parameter variation, within parameter boundaries. Equation IV-5 and Equation IV-6 are used for VAM and quantify more accurately the amplitude response reaction to an increase or decrease of a soil parameter, respectively. The reaction is limited within the simulated boundaries, where an amplitude response deviation regarding DC will be shown. Having a HC (here 200% of DC), $max, r = 200$ and a LC (here 50% of DC), $min, r = 50$, is possible to interpolate the sensitivity by a selected variation $50 < var < 200$ as follows, for increasing or decreasing a parameter:

$$se_{new}^{\uparrow} = se^{\uparrow} \left(\frac{var - 100}{max, r - 100} \right) \quad \text{Equation IV-7}$$

$$se_{new}^{\downarrow} = se^{\downarrow} \left(\frac{100 - var}{100 - min, r} \right) \quad \text{Equation IV-8}$$

Finally, amplitude response can be calculated as:

$$a_{new}^{\uparrow} = a_{DC}(1 + se_{new}^{\uparrow}) \quad \text{Equation IV-9}$$

$$a_{new}^{\downarrow} = a_{DC}(1 + se_{new}^{\downarrow}) \quad \text{Equation IV-10}$$

Therefore, to simulate the effect of a parameter variation a simple interpolation of its sensitivity can be done. This tool allows to perform several instantaneous simulations for each separate (F, a, v, u) amplitude response. Inserting sensitivities in an Excel sheet, VAM can be quickly performed on all (F, a, v, u) amplitude responses at the same time.

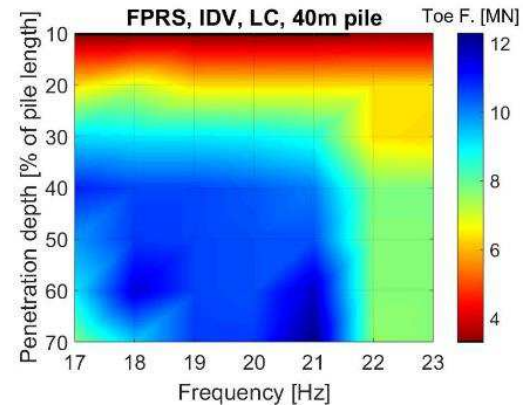
d) VDP implementation example

For VDP, this simple predicting tool can identify optimal and critical frequencies along penetration providing insight about the most efficient driving path within the frequency-penetration plane. Can be directly read from sensitivity charts. The example described below is inspired from the Anholt project where two offshore test monopiles were successfully vibrodriven to target depth.

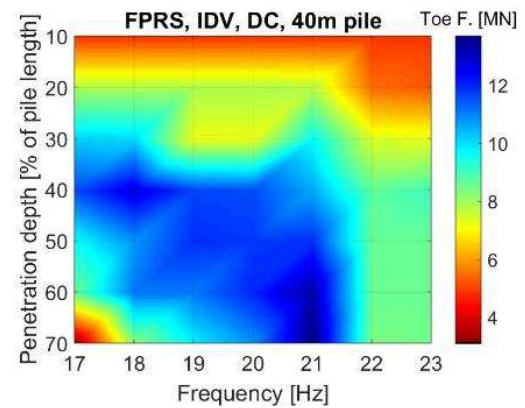
A site-specific sensitivity analysis was performed using as HC an additional 5MPa and, as LC, a subtracted 5MPa from the default (IDV) q_c soil profile. Friction ratio, $FR(\%) = q_c/f_s$, has been maintained at 0,5% to synthesize sandy soils f_s for both LC and HC. Pile and vibrator parameters are presented in Table IV-1. Those boundaries intend to cover the scarcity of possible soil resistances shown in Figure IV-9 and Figure IV-10.

FPRS charts will predict for instance reaching forces at toe level and so, the most efficient frequency-penetration path. Sensitivity charts will define which direction and magnitude (F, a, v, u) amplitudes will vary regards to a parameter variation (more resistance /damping). Therefore, if sensitivity charts are developed at top level, they can be directly compared to monitored data while vibrodriving is performed.

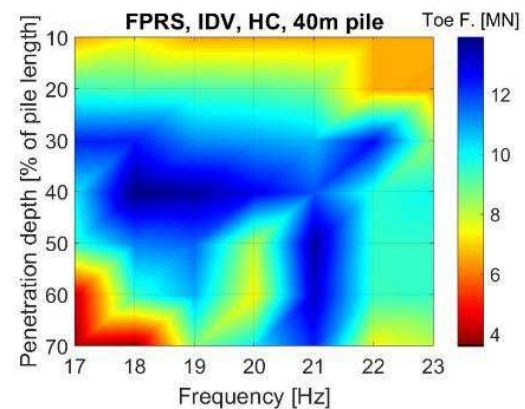
This combined tool enables to indicate an efficient path through toe level FPRS and to provide hints on soil behavior through top level Sensitivity charts. As shown in Figure IV-9, FPRS indicate that toe forces at deeper penetrations are highest for an optimal counter-intuitive frequency of 21Hz.



a) LC, $q_{c,LC}=q_{c,DC} - 5\text{MPa}$



b) DC, $q_{c,DC}$



c) HC, $q_{c,LC}= q_{c,DC} + 5\text{MPa}$

Figure IV-9: FPRS on Toe Force, IDV upper & lower bound, 40m pile

Analyzing Figure IV-10, the sensitivity chart on top forces indicates that in pile penetrations till 40%, a strong top force sensitivity could be compared to a stronger/softer soil. At deeper pile penetrations, a damped sensitivity could hardly reveal a stronger/softer soil. Combined analysis that could be used for monitoring engineers to react and advise hammer operator's which frequencies could improve results.

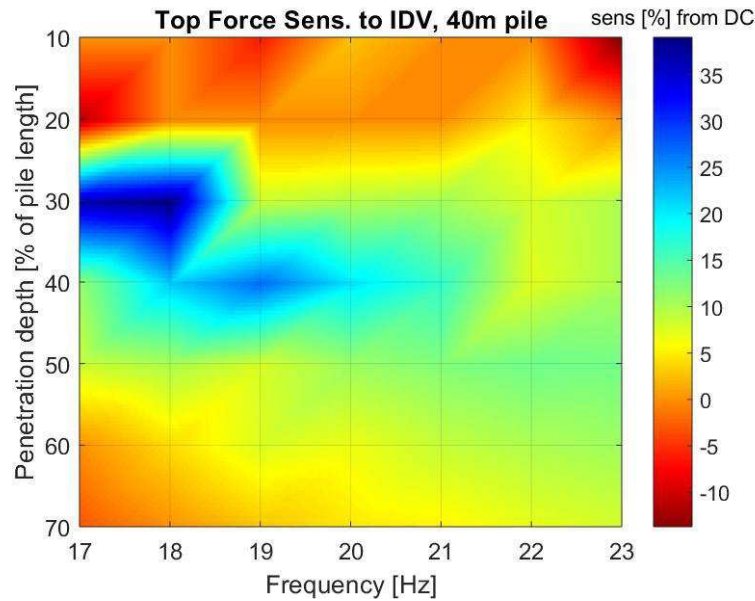


Figure IV-10: Sensitivity chart, Top Force, IDV upper & lower bound, 40m pile

e) VAM implementation example

For VAM, this tool enables to fasten the matching. Because VAM is a post-installation analysis, frequency applied, and penetrations are known beforehand, reducing the frequency-penetration plane to a line. Because of this reduction, a finer discretization of penetration depth becomes feasible to generate a more precise match. Steps of 25cm are recommended since it's the usual recording steps for vibratory driving. Once soil parameters are optimized to match measured (F, a, v, u) amplitude responses, final fine tuning with wave equation software is possible with generally similar results, as will be later shown by a case study.

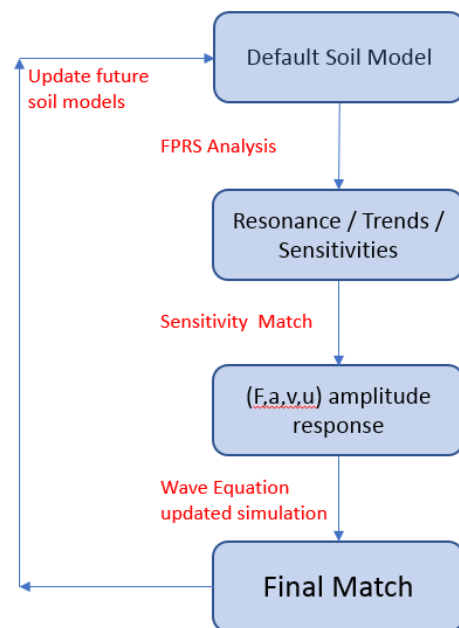


Figure IV-11: Vibratory Amplitude Matching procedure

As Figure IV-11 shows, while matching modelled amplitudes to measured ones, experience-based vibrodriving parameters should emerge. Resonance frequencies could vary from their initial position or even disappear as soil parameters get updated.

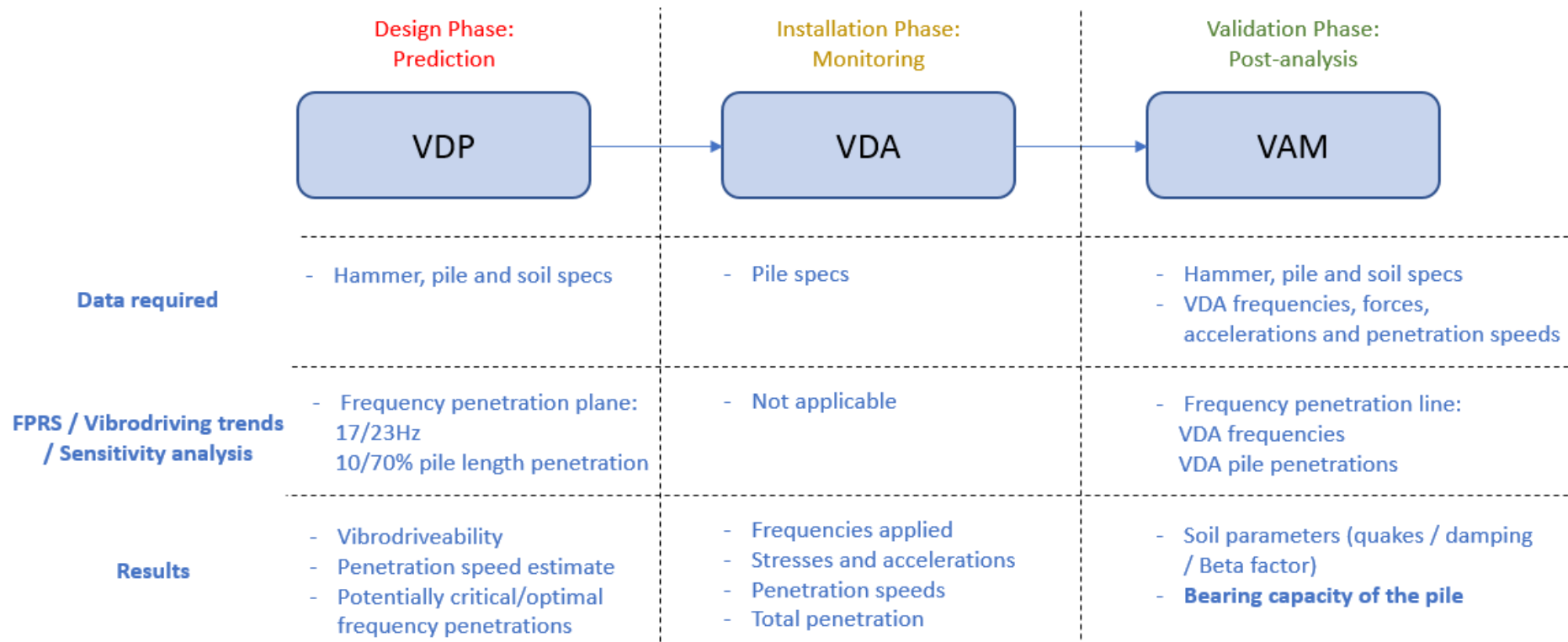


Figure IV-12: Phases of a vibratory project

3. Parametric analysis

A PARAMETRIC ANALYSIS IS PERFORMED TO EXPLORE SOIL PARAMETERS SENSITIVITY WITHIN AN EXPECTED VARIATION RANGE. ANALYSIS SIMULATES OFFSHORE-LIKE CASES.

a) Input parameters

Pile and vibrator parameters

Piles consist of constant tubular large diameter cross-sections, inspired from medium and lower offshore monopile dimensions. Vibrator shown in Table IV-1 is chosen from proposed Cape Holland²⁵ technical sheet based on selected monopile dimensions.

	Parameters	Units	
Offshore monopiles (3 pile lengths)	Diameter	[m]	4
	Thickness (constant)	[mm]	50
	Length	[m]	30, 40 and 50
	Parameters	Units	
Vibrator CV 640 VLT	Vibrator Dyn. Mass	[tons]	119
	Vibrator Static Mass	[tons]	72
	Nominal me	[kg.m]	640

Table IV-1: Parametric analysis pile and vibrator parameters

Soil profiles

Simulated soil profiles range from simplified cases to add step by step layers of complexity till a complete soil profile. Three academic and one typical soil profiles have been considered for parametric analyses, as shown in Table IV-2 and are respectively identified by 2 or 3 letter acronyms. All soil profiles have a Friction ratio of 0,5%, in order to simulate sandy soil, where soil liquefaction can be developed, allowing to reach the full potential of the driving technique. The IDV soil profile, (5-40MPa) has been selected based on Anholt study case, (LeBlanc Thilsted, 2014).

	Type	Description	Depth [m]	Qc [MPa]
FE	Free Toe	Free hanging pile	/	0
BE	Fixed Toe	Highly resistant soil layer	/	200
OT	Toe Soil	Medium resistant soil layer	/	20
IDV	Shaft and Toe	Increasing resistant soil	0 – 20	5 to 40 (linear)
		profile with depth	above 20	40 (constant)

Table IV-2: Parametric analysis soil profiles

²⁵ Worldwide vibrohammer manufacturer.

Soil parameters

As stated in Section II.2, main soil parameters defining soil model will vary in our scope of study. Those are quakes, damping values and beta factors. A sensitivity analysis will be performed to understand the influence of those parameters within a typical offshore model. Three sets of soil parameters are presented in Table IV-3, defining the explored range of the parameters variations.

	Low Case (LC) (50% DC)	Default Case (DC) (100% DC)	High Case (HC) (200%DC)
Toe Quake	1mm	2mm	4mm
Shaft Quake	1mm	2mm	4mm
Shaft Damping	50% Equation II-32	Equation II-32	200% Equation II-32
Toe Beta Factor	0.5	0.585	0.65

Table IV-3: Parametric analysis soil parameters range

Shaft damping values depend on depth and soil resistance, therefore they are related to Equation II-31 and Equation II-32. Notice as well that toe Beta factor has been varied in a different range than the other soil parameters. Due to its largely superior influence, extending excessively its range could be an unrealistic approach.

b) Independent research software comparison

(Whenham & Holeyman, 2012) calculated through finite element simulations the force transfer ratio, which they define as the ratio of the amplitude of the force at the pile head to the amplitude of the vibrator input force ($\eta = F_{measured}/F_{vib}$). These authors demonstrated that the force transfer ratio is also prone to resonance and anti-resonance, as shown in peaks in Figure IV-13. Their simulations were performed using an elastic body assumption, based on 1D stress wave propagation.

The Allwave-PDP dedicated wave equation software was tested on the same case, using equivalent values. Instead of the original pile (type IV at Limelette site (Whenham & Holeyman, 2012), an open ended tubular equivalent pile has been selected, adopting the same pile length, cross sectional area and toe conditions (a. o. a spring stiffness $k_{toe} = 10MN/m$, $k = F_{uy1}/u_q = 500MPa/m$ corresponding to the original $k = 10MN/m$).

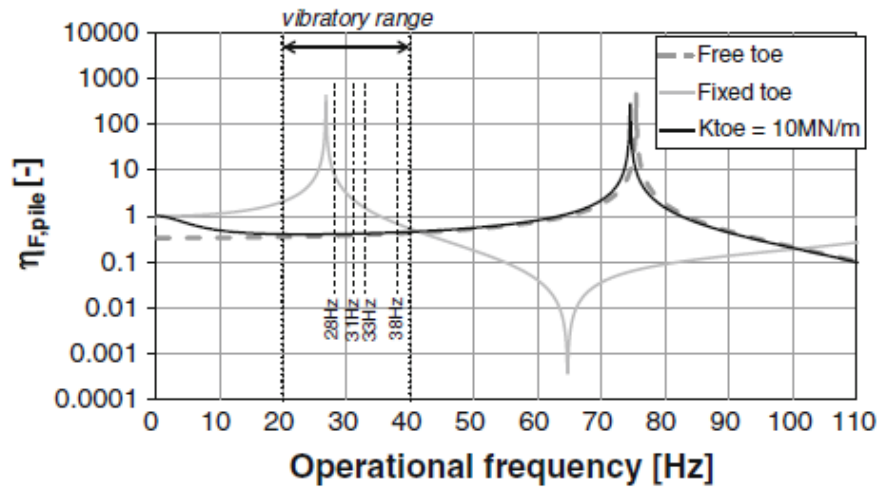


Figure IV-13: Force transfer ratio, (Whenham & Holeyman, 2012)

Parameters	Units	Original Pile (Whenham)	Equivalent Open ended pile (Author)
Section	[m ²]	0.0199	0.02 ($\phi=1\text{m}$, $t_h=6.4\text{mm}$)
Length	[m]	20	20
Vibrator Dyn. Mass	[kg]	5770	5770
Suppressor housing Mass	[kg]	2000	2000
Nominal m_e	[kg.m]	36	36
$K_{toe}=10$	[MN/m]	10 (500MPa/m)	10 ($u_q=2\text{mm}$, $F_{u1}=1\text{MPa}$)
Free toe resonance	[Hz]	76	76
$K_{toe}=10$ resonance	[Hz]	75	76
Fixed Toe resonance	[Hz]	27	26

Table IV-4: Results comparison of Allwave-PDP results with (Whenham & Holeyman, 2012) (Limelette site, Pile Type IV)

Results summarized in Table IV-4 show similar resonance frequencies for the free toe (76Hz), fixed toe (27-26Hz) and for $k_{toe} = 10\text{MN/m}$ (75-76Hz). Anti-resonance identified by (Whenham & Holeyman, 2012) for the fixed toe condition could not be verified by the Allwave-PDP software because of excessively low force amplitude (0.1% of vibrator theoretical force per Equation II-39). Even tough, general convergence of results suggests that both approaches are correct.

c) Mode of vibration: Resonance and anti-resonance

Refusal is encountered when toe force and/or toe displacement are insufficient to overcome the toe spring resistance (Figure II-10). As shown in Figure IV-5, (F, a, v, u) amplitude **resonances do not necessarily develop at the same frequency–penetration combinations for all variables**. Therefore, to enable penetration at critical depths, energy must be properly distributed between toe displacement and toe force.

Even though general predictions could show that penetration is possible, antiresonances have to be scrutinized since they could reveal under which critical combination of soil parameters and frequencies, vibrodriving could face early refusal. Several combinations can be fine-tuned but based on impact driving soil parameters and a future larger database, it is hoped that solutions would converge. Vibratory Amplitude Matching (VAM) represents a first avenue to explore possible ranges of soil parameters.

Variable	Forces		Displacements		Pen. speed	Toe resist.
Units	[kN]		[mm]		[mm/s]	[kN]
Level	Top	Toe	Top	Toe	/	Toe
LC	10550	11606	4.5	2.9	Refusal	12410
DC	12863	14572	5.8	4.1	4.9	14520
Trend	Unstable	Positive	Unstable	/	Unstable	/

Table IV-5: Forces and displacements amplitude responses on refusal antiresonance on IDV, for Toe Beta Factor, 40m pile, at 17Hz, 40% pile penetration

Variable	Forces		Displacements		Pen. speed	Toe resist.
Units	[kN]		[mm]		[mm/s]	[kN]
Level	Top	Toe	Top	Toe	/	Toe
LC	11466	12685	5	3.5	4	12410
DC	10550	11606	4.5	2.9	Refusal	12410
Trend	Unstable	Unstable	Unstable	Unstable	Unstable	/

Table IV-6: Forces and displacements amplitude responses on refusal resonance on IDV, for Shaft Quake, 40m pile, at 17Hz, 40% pile penetration

Refusal antiresonances and amplitude response depressions have been spotted from the 1764 and are presented in Table IV-5 and Table IV-6. As shown, when a refusal situation is encountered, varying certain soil parameters enables sometimes to generate sufficient toe forces and/or toe displacements to allow further penetration.

Results from Table IV-5 show that enough Toe displacement (>2mm) is in place to overcome elastic regime of the Toe spring. Despite natural thinking, penetration is possible at that location for a high toe resistance while refusal takes place for a lower one. Similar results are extracted from Table IV-6. Lower Shaft quakes (1mm) allow to develop full shaft static resistance sooner, where vibrodriving would seem more complicated. However, again due to stress wave superposition effects, higher force amplitude response can be developed at the toe, and thus penetration.

4. Proposition of a pile vibrodriving formula

EVEN THOUGH THE FPRS AND SENSITIVITY ANALYSIS PRESENTED BEFORE INTEND TO FASTEN VIBRATORY PREDICTIONS AND ANALYSIS, THOSE CAN BE STILL BE TIME DEMANDING APPROACHES. THE FOLLOWING METHOD INTENDS TO GIVE A DIRECT ASSESSMENT OF THE POSSIBLE REFUSAL DEPTH.

a) New terms

THE DRIVING FORMULA IS FULLY EMPIRICAL. IT USES THE FOLLOWING RATIOS THAT COULD PROVIDE A GENERAL PERSPECTIVE OF THE INTERACTIONS THAT WILL TAKE PLACE WHILE VIBRODRIVING.

Relative Pile wavelength

Ratio between vibrator period and pile period as follows:

$$\lambda_r = \frac{T_{vib}}{T_{pile}} = \frac{1/f_{vib}}{2L_{pile}/c_{steel}} [-] \quad \text{Equation IV-11}$$

Determines how many times stress waves will travel through the whole pile and back in one vibrator cycle, usually between 2 and 8 for large offshore monopiles. Will be related to toe resistance.

Relative Soil wavelength

Ratio between vibrator period and penetrated pile period, as follows:

$$\lambda_{r,soil} = \frac{T_{vib}}{T_{pile,pen}} = \frac{1/f_{vib}}{2z_p/c_{steel}} [-] \quad \text{Equation IV-12}$$

Determines how many times stress waves will travel back and forth, only through the penetrated pile in one vibrator cycle. This index is penetration dependent. Will be related to shaft resistance.

Relative acceleration

Ratio between centrifugal force and dynamic mass, as follows:

$$a_{rel} = \frac{F_{vib}}{M_{dyn}} = \frac{m_e(2\pi f)^2}{M_{pile} + M_{vib,dyn}} \left[\frac{m}{s^2} \right] \quad \text{Equation IV-13}$$

Simplified approach assuming the pile to a rigid body in order to quantify the provided acceleration to a given dynamic system. Related to the whole dynamic system.

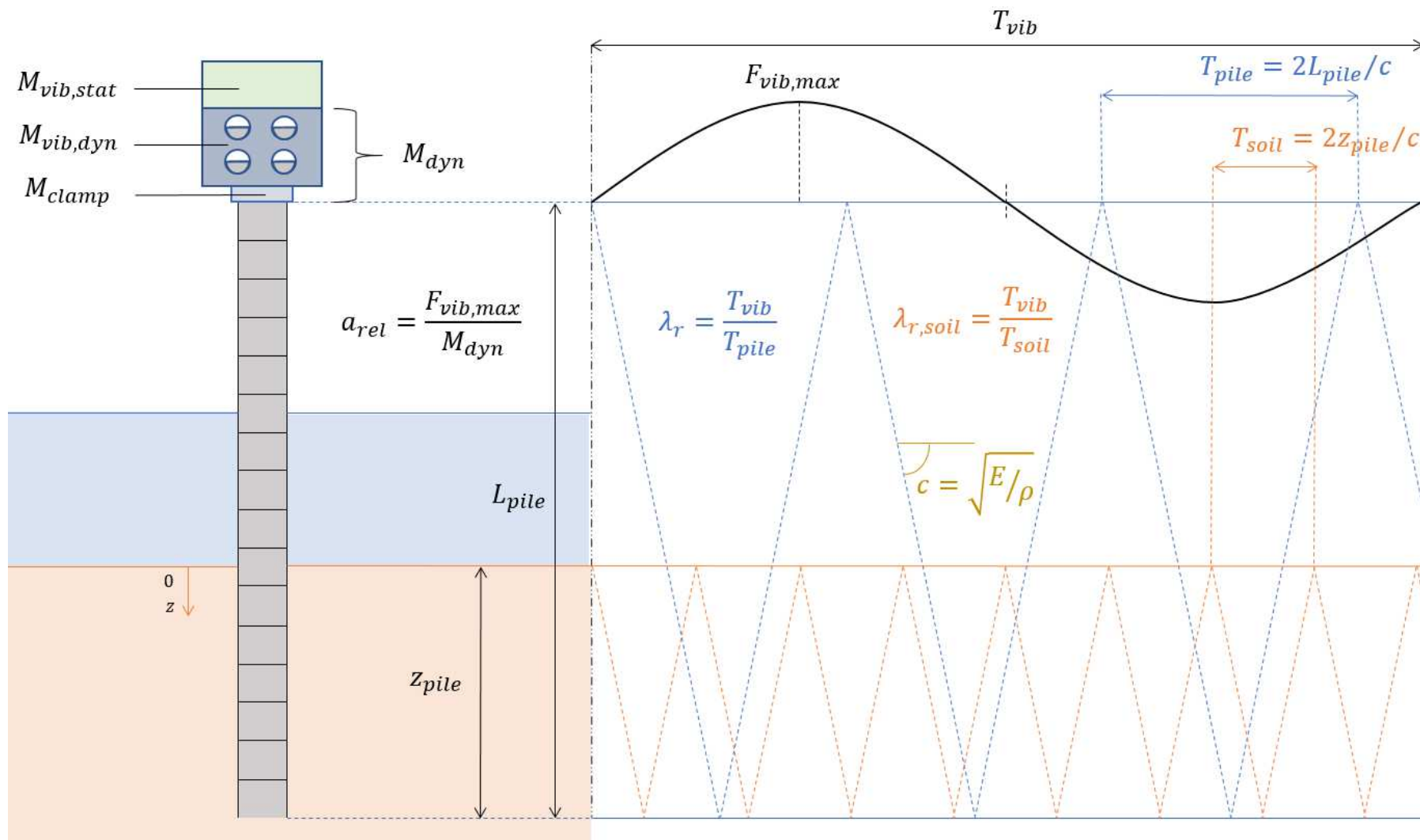


Figure IV-14: Relative indices representation

b) Methodology

Starting from an initial proposed refusal depth, calculation based on CPT soil investigation, of soil resistances at that penetration, for n soil layers or linear measures:

$$R_{Shaft,initial} = \sum_{i=1}^n f_{s,i} \cdot (z_{bottom,i} - z_{top,i}) \cdot (\chi_{in} + \chi_{out}) \cdot \beta_{Shaft} \quad \text{Equation IV-14}$$

$$R_{Toe,initial} = q_{c,z_{prop}} \cdot A \cdot \beta_{Toe} \quad \text{Equation IV-15}$$

Those soil resistances will be associated to previous relative indices, trying to quantify the vibrodriving response²⁶, as follows:

$$z_{refusal,initial} = c \cdot a_{rel} \left(\frac{b \cdot \lambda_{r,soil}}{R_{Shaft}} + \frac{a \cdot \lambda_r}{R_{Toe}} \right) \quad \text{Equation IV-16}$$

Once, new refusal depth is calculated, shaft and toe resistance are recalculated, and new refusal depth as well. Following iterations should converge to a final result. However due to extreme soil variability, iterations do not converge properly, where several solutions. To avoid this uncertainty first penetration that fulfill the following condition will be considered as refusal:

$$\text{if } z_{calculated} \leq z_{prop} \quad \Rightarrow \quad z_{refusal} = z_{calculated} \quad \text{Equation IV-17}$$

A calculation example can be found in Appendixes VIII.4. Based on measurements from Maasvlakte study case, it appears that shaft resistance has a major influence on vibrodriving regards toe resistance. To implement this hypothesis into the formula, optimization parameters were set for $a = 0,5$ (toe) and $b = 2$ (shaft). Parameter c is defined as follows, depending on the pile weight:

$$c = \min \left\{ \frac{160}{200/M_{pile}}, 500 \right\} \quad \text{Equation IV-18}$$

c) Limitations and accuracy

Notice that fitting parameters a , b and c , are calibrated for Riffgat and Maasvlakte offshore projects, treated in Section V.2²⁷. Formula should be treated with caution, as the accessible database grows, further accuracy could be provided.

²⁶ This method is fully empirical, based on parameters that provide an overview of the vibrodriving case. Under no circumstances they should be interpreted as physical phenomena only roughly related.

²⁷ Further optimization with a larger offshore projects database is welcome.

5. Conclusions

Frequency penetration behavior

- ✓ Resonances at a specific pile location could be antiresonances at other pile levels
- ✓ Sensitivity analysis provides a deeper understanding on how soil parameters affect a vibrodriving model
- ✓ Software checked with independent research

VDP

- ✓ FPRS covers wider range, reassembling the main possible scenarios
- ✓ FPRS and sensitivity charts can be combined to understand how a pile top amplitude response variation affects toe level amplitude response
- ✓ Avoiding critical frequency-penetration locations or areas
- ✓ Providing optimum frequency penetration paths

VAM

- ✓ Sensitivity analysis allows to fasten VAM procedure
- ✓ As the frequency-penetration line is known in advance (VDA), simulated penetration can be furtherly discretized increasing VAM accuracy

Pile driving formula

- ✓ Alternative direct formula for first stage design purposes (vibrohammer selection)
- ✓ Enabling to assess quickly a new vibrodriving situation

V. Study cases analysis

IN THIS THESIS WE WILL CONFRONT OUR TOOLS WITH 2 REAL STUDY CASES, ONE FROM ROTTERDAM, NEARSHORE AND ANOTHER ONE FROM RIFFGAT, OFFSHORE. THOSE TWO PROJECTS HAVE BOTH DIFFERENT AND SIMILAR SOIL TYPES. FOR A WIDER PERSPECTIVE, SEVERAL SIMILAR CLOSE BY CASES SHOULD BE ANALYZED.

1. Rotterdam study case

Maasvlakve study case is a very interesting vibratory research project to improve vibrodriving understanding. This project is split in 2 phases, first one in Maasvlakte where an upper section of a pile was vibrodriven nearshore. Once vibrator, pile and soil behavior were tested, a second phase will vibrodrive the complete pile in several offshore locations. Data collected from the first phase, enable us to analyze vibrodriving and start a database.

a) Overview of the project

Pile and vibrator²⁸

	Length [m]	Top diameter [m]	Bottom diameter [m]	Wall thickness [mm]
1 st section	12,2	3,2	4	40
2 nd section	2,5	4	4	55
3 rd section	4,2	4	4	55
4 th section	4,2	4	4	55
5 th section	4,2	4	4	60

Table V-1: Maasvlakte monopile dimensions, part 1

The upper part of the pile has approximately 27 m length and 4 m diameter, see Table V-1. To drive this upper monopile part, Cape Holland selected the Vibrohammer CV-320VLT-U. Inputted parameters can be seen in Table V-2.

	Parameters	Units
Vibrator CV 320 VLT	Vibrator Dyn. Mass	[tons] 54
	Vibrator Static Mass	[tons] 30
	Nominal me	[kg.m] 320

Table V-2: Maasvlakte vibrator dimensions

²⁸ Pile and vibrator complete specifications can be found in Appendixes.

Soil

Test location was onshore, on a man-made sand bank, near the North Sea. Figure V-1 provides the CPT record performed at that exact location.

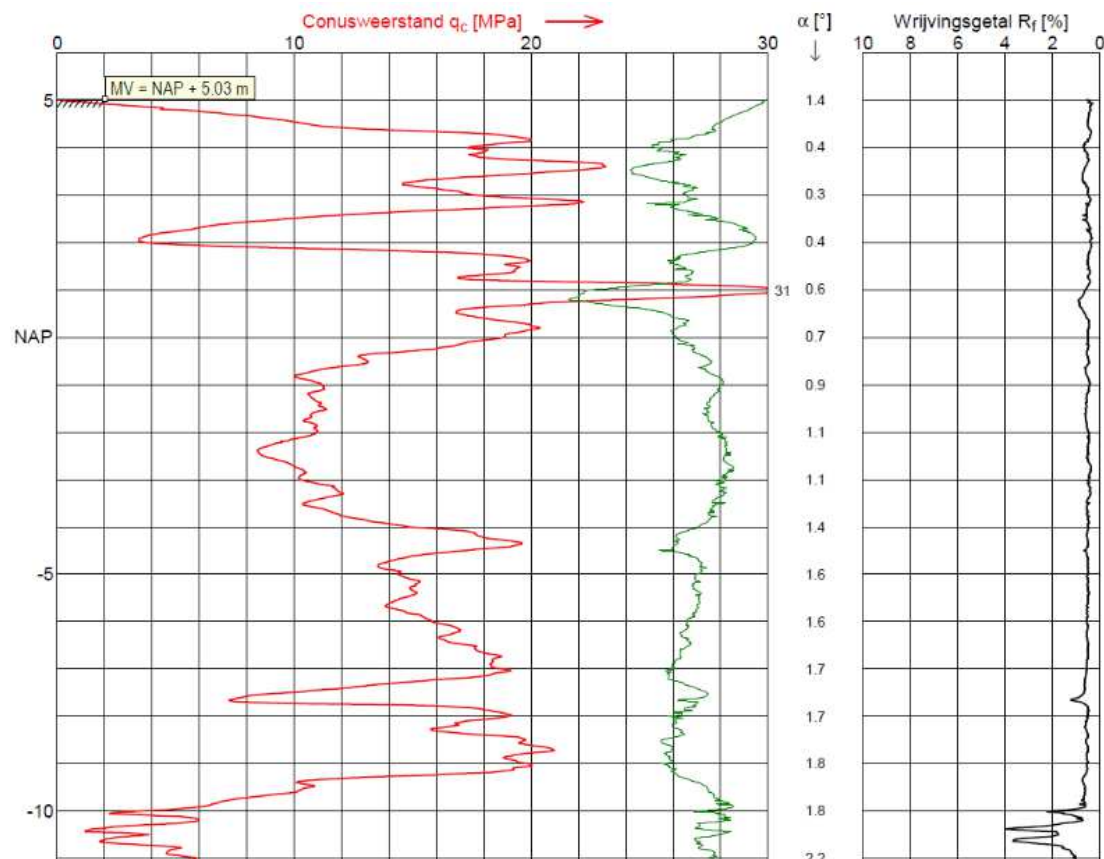


Figure V-1: Maasvlakte CPT soil investigation

The soil investigation features a CPT test, with an overall constant friction ratio of 0,5%, typical of sands. First 5m are composed of dry sands above the phreatic level, and saturated ones at deeper levels.

This is a suitable soil that can be liquefied and that vibrodriving projects look for since it can develop the largest soil resistance degradation and therefore reach target depths.

b) VDA monitoring

Mainly 3 different frequencies were applied, as shown in Figure V-2. The pile stood up for 30 minutes at 4 m depth, nearly refusing on a very dense sand layer characterized by a peak cone resistance of more than 30MPa. Once the pile overcame that penetration, it finally refused at 14,65m after a total driving time of 1 hour and 31 minutes.

Pile was monitored by 2 strain sensors and 2 accelerometers placed 3,25m below pile tip. Penetration speeds were recorded each 25cm pile penetrations were performed, having the average penetration speed of that section, as can be seen in Figure V-2.

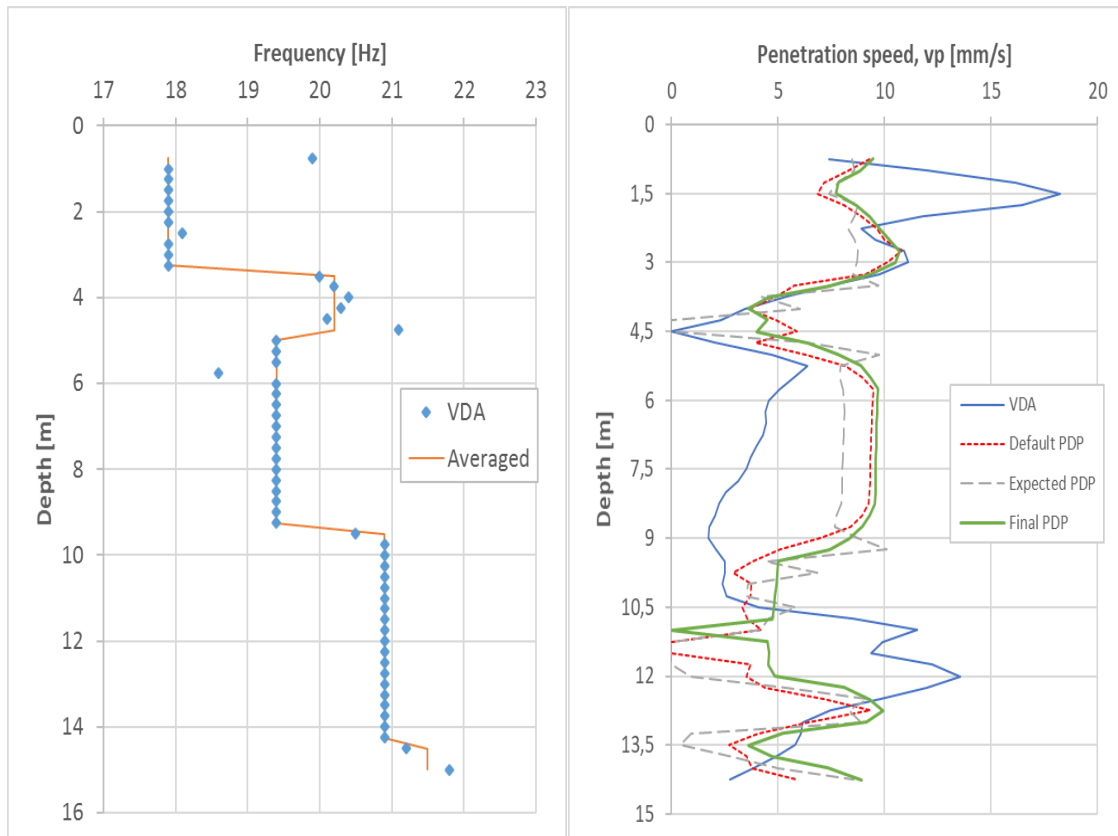


Figure V-2: Maasvlakte VDA frequency and penetration speed logs

c) VAM

Values presented in Table IV-3 were used as HC, DC and LC to perform a sensitivity analysis. Interpolating its boundaries results, proportionally, with the parameter variation, ranging from LC till HC, instantaneous results can be generated.

Because frequencies and penetration are known in advance from Figure V-2, a reduced sensitivity analysis following the same frequency penetration path can be simulated. This VDA focused sensitivity analysis prepared to expedite the VAM features 59 penetration increments (from 0.75m to 14.25m, with steps of 25cm), and 4 values of the frequency (17.9/20.2/19.4/20.9Hz) per averaged curve (orange) shown in Figure V-2. Resulting trends and sensitivities have been used to anticipate the amplitude response (F, a, v, u) to a parameter variation at each grid point.

As can be seen from Figure V-3, trends are non-stable and pretty much chaotic. Reasoning the effect of a soil parameter within a vibrodriving model is therefore not the best approach. The resulting improved matched profile can be found in Figure V-4. Displayed as a grey curve, these first improved match profiles can be produced using

linear interpolation at each grid depth, i.e. without time-consuming finite element simulations.

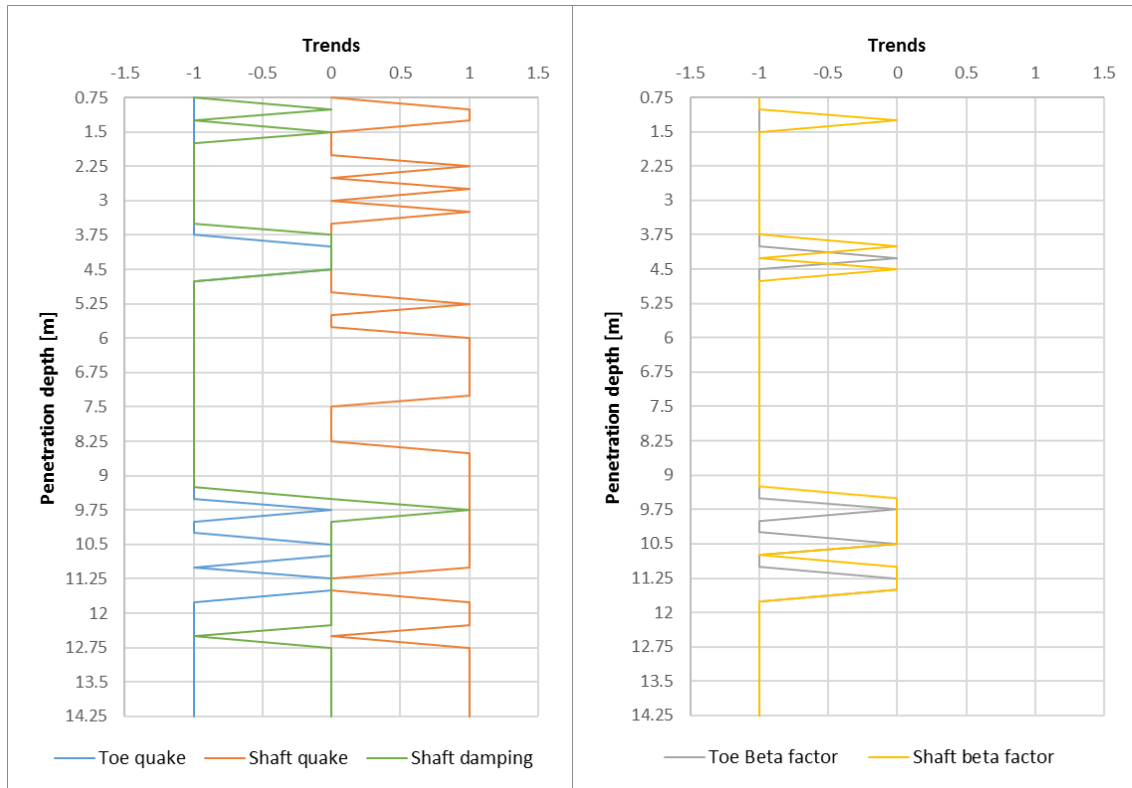


Figure V-3: Resulting trends for the VAM analysis

Once a satisfactory sensitivity match is achieved, a dedicated wave equation software is used to simulate the matched parameters. The wave equation simulation better takes into account the effective stress waves interactions that the sensitivity interpolation was linearly approximating, providing a more reliable amplitude response profile.

This intermediate step is quite efficient since several matches can be done consecutively, as long as the sensitivities have been developed beforehand. The first match (grey curve per interpolation) is comparable to the PDP final match (green curve per wave equation simulation) shown in Figure V-4. Deviations (Expected interpolated/VDP simulated) amount to 5%, 3%, 5% and 2% for each (F , a , v , u) variable amplitude, respectively.

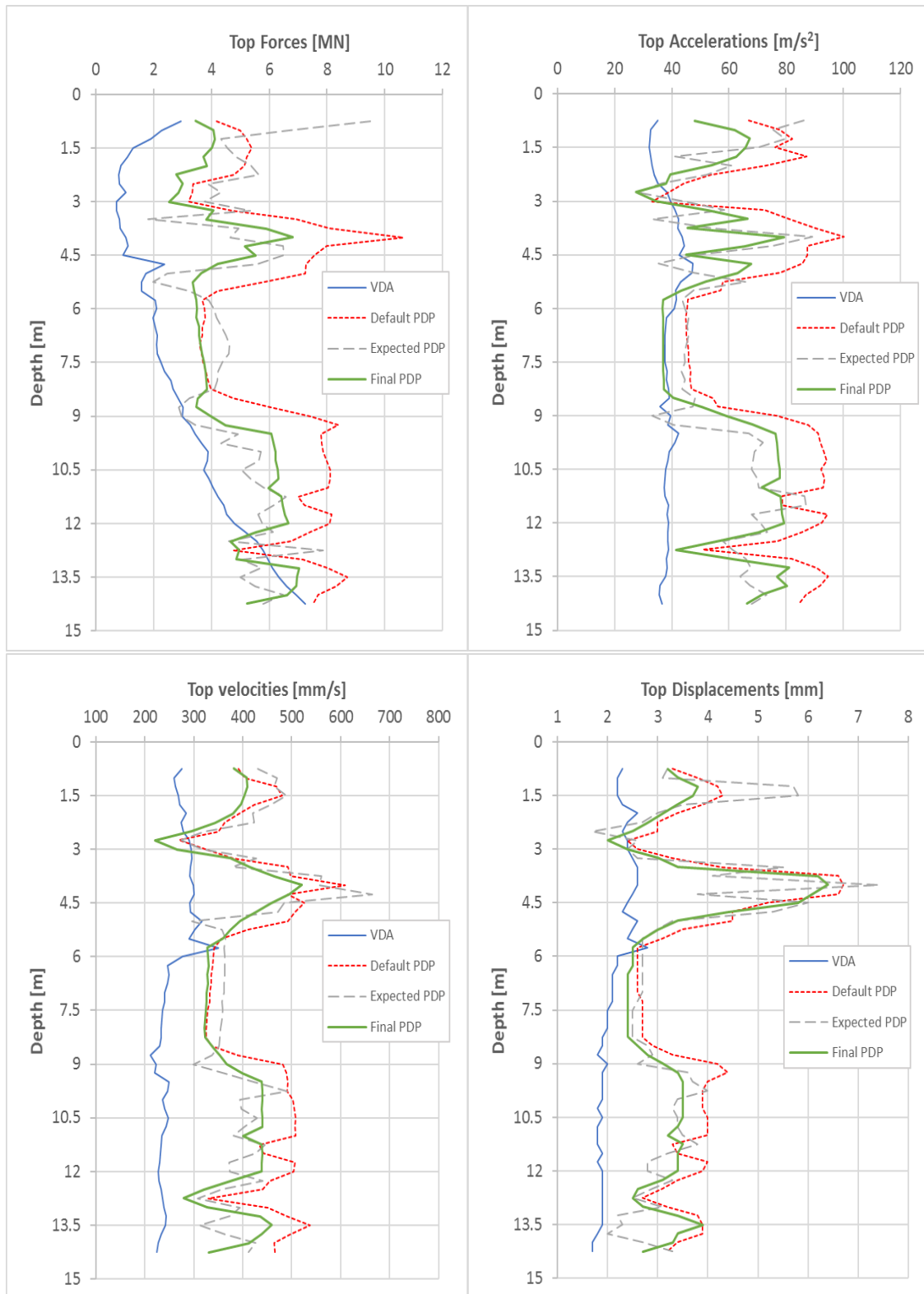


Figure V-4: Amplitude profile matching main results: Forces, Accelerations, Velocities, and Displacements

After trying several combinations, the most generally accurate match performed on all (F, a, v, u) variables, is presented in Figure V-4. Notice that deviations are still in place, revealing a still inaccurate match. Those deviations may be related to the hook crane load and improper static mass modelling. Matching separately each soil layer could increase accuracy as well as exponentially increase the signal matching times.

Parameters	Units	Default	Variation [%]	Final
Toe quake	[mm]	2	200	4
Shaft quake	[mm]	2	50	1
Shaft Damping	[Ns/m ³]	Equation II-32	50	50% Equation II-32
Toe Beta factor	[-]	0,5	80	0,4

Table V-3: Final matched parameters

Main deviations occur near 4m penetration depth where a near refusal situation held the pile for over 30 minutes, with unstable results. Predicted profiles agreed better with the measured ones between 5 and 10m penetration. There, penetration speed increased in spite of an increasing q_c soil profile. That remarkable result was identified while performing the parametric analysis. In authors' opinion, it could be linked to a more efficient amplitude response yet to be modelled with an improved soil model. Refusal penetration was calculated using the finally adjusted soil parameters at 17m depth with a vibrohammer working at 20,9Hz.

2. Riffgat study case

Off the North Sea coast of Germany, in 2012, an offshore windfarm park was build. The foundations were monopiles, driven in 2 phases, first by vibratory hammers and secondly by impact hammers to target depth. **The reason to change hammers was a client concern about the soil resistance leftover after disturbing it vibrodriving.** Therefore, impact hammers were preferred for the final stage due to its proven response.

a) Overview of the project and initial data

Pile and vibrator

In general, the piles consist of two distinct parts. The conically shaped top part will accommodate the transition to the monotower supporting the gondola. The bottom part (entering the subsurface) is of constant diameter. The length of the top part as well as the corresponding diameters and final pile toe level vary per location. Both parts can have multiple variations in wall thickness (see example in Appendixes).

Location	Section	Length [m]	Diameter top [mm]	Diameter bottom [mm]
WEA 7	part 1	0-21.29	4700	6500
	part 2	21.29-62.4	6500	6500
WEA 8	part 1	0-14.60	4700	5700
	part 2	14.60-56.2	5700	5700
WEA 16	part 1	0-14.6	4700	5700
	part 2	14.6-60.2	5700	5700
WEA 25	part 1	0-22.58	4700	6500
	part 2	22.58-60.40	6500	6500
WEA 28	part 1	0-22.58	4700	6500
	part 2	22.58-71.60	6500	6500
WEA 29	part 1	0-22.58	4700	6500
	part 2	22.58-64.8	6500	6500
WEA 30	part 1	0-14.60	4700	5700
	part 2	14.60-65.40	5700	5700

Table V-4: General monopile dimensions

Hammer specifications

For Riffgat project, a new hammer was developed, the Super Quad Kong. This vibrator consists of 4 vibrators APE 600, synchronized, in order to deliver enough force to drive the large diameter piles. As additional mass to the Super Quad Kong 40 tons were added, accounting for the base frame used to fix the 4 synchronized vibrators.

Model	Frequency [Hz]	Centrifugal force [kN]	Eccentric moment [kg.m]	Dynamic mass vibro [tons]
APE 600	23.3	4946	230	19
SQK	23.3	19752	920	76

Table V-5: Riffgat hammer parameters

Driving Data

The piles were driven at target depth with the Super Quad Kong vibrator at all locations. On average the seabed is encountered at LAT -20 m, with 20 meters of water table are expected on average.

Pile	Driving time [hh:mm:ss]	Pile length [m]	Self- Penetration [m]	End Penetration [m]	Vibro Penetration [m]	Av. Speed [mm/s]
Maasvlakte	1:31:00	27.2	0.5	14.7	14.2	2.6
WEA 07	0:53:30	62.4	7	20	13	4.0
WEA 16	0:37:52	60.2	3	22	19	8.4
WEA 25	0:44:31	60.4	4	20	16	6.0
WEA 28	0:27:28	71.6	3	21	18	10.9
WEA 29	0:23:55	64.8	3	22	19	13.2
WEA 30	0:32:30	65.4	1	23	22	11.3

Table V-6: Main vibrodriving data from Maasvlakte and Riffgat projects

b) Optimization of the pile driving formula

Pile driving formula iteration independent values are shown in Table V-7. It is interesting to notice that a relative top acceleration, a_{rel} , are usually between 25-30 m/s^2 , being itself a predesigning value. If a vibrohammer do not provide at least 25 m/s^2 , vibrodriving could be compromised.

	M_{pile} [tons]	L_{pile} [m]	f_{vib} [Hz]	T_{pile} [ms]	λ_r [-]	a_{rel} [m/s ²]
Maasvlakte	125.7	27.2	21.6	10.5	4.4	23.9
WEA 07	645	62.4	23	24.1	1.8	25.1
WEA 16	625	60.2	23	23.3	1.9	25.8
WEA 25	629	60.4	23	23.4	1.9	25.1
WEA 28	742	71.6	23	27.7	1.6	21.8
WEA 29	675	64.8	23	25.1	1.7	23.6
WEA 30	606	65.4	23	25.3	1.7	25.9

Table V-7: Pile driving formula iteration independent values

Optimization parameters used for the pile driving formula were $a = 0.5$ and $b = 2$. Parameter c is calculated for each case using Equation IV-18 as shown in Table V-8. WEA28, WEA29 and WEA30 had bottom clay layers where less soil fatigue can be developed. This was modelled by a reduction of the shaft Beta factor, from 0,1 (sands) to 0,2, representing more accurately soil behavior. Toe Beta factor was not modified since for soft clays, its influence is very limited.

	c [-]	$\lambda_{r,soil}$ [-]	Sim. R_{Toe} [MN]	Sim. R_{Shaft} [MN]	Achieved Penetration [m]	Sim. Refusal [m]
Maasvlakte	100.8	8.9	7.6	2.5	14.7	13.5
WEA 07	500	4.8	12.6	16	20	23.5
WEA 16	500	5.4	18.6	14.7	22	21
WEA 25	500	4.9	20.6	11.5	20	23
WEA 28*	500	5.6	2.1	25.2	21	20
WEA 29*	500	5	2	23.7	22	22.5
WEA 30*	484.4	3.8	2.6	19.2	23	29.5

Table V-8: Pile driving formula results

General results achieved to calculate within an acceptable range refusal depth, in author's opinion. In average, 7% deviations from achieved penetration with a scatter ranging from 18% till -28%. As previously explained, this formula can only be used for limited purposes.

VI. Conclusions and guidelines

Vibrodriving modelling is performed through the well accepted 1-dimensional stress wave theory. Method of characteristics solution was applied to simulate how stress waves propagate through different pile interfaces and interact with soil. Those stress waves can define soil dynamic behavior by monitoring strains and accelerations. In order to so the vibrator static mass influence has to be monitored and integrated in future models as an external semi-dynamic mass.

Soil can be modelled through several tests and parameters. In order to generate a starting soil model, predictions should use, according to the author, a CPT. This is a type of soil investigation capable of testing non-altered toe and shaft resistance continuously. In case different soil tests are available, correlations have been proposed for the researcher to perform their own CPT based simulations. Particle shape test could also be useful for defining the vibrodriving potential of a soil, but a database and calibration is still needed.

An extensive parametric analysis was performed to understand the influence of each soil element spring/dashpot/fatigue into the vibrodriving model. Among several soil parameters, only 5 were chosen, representing toe and shaft governing parameters. Once pile, vibrator and soil have been defined, a new perspective was presented: The Frequency Penetration Response Spectrum and Sensitivity Charts.

Those new tools were developed for VDP to cover the range of possible vibrodriving scenarios, and, for VAM to fasten the time-demanding matching simulations in a simplified manner. Results showed that tools were able to indicate efficient/inefficient mode of vibrations in a frequency-penetration plane and being able to predict the effect of a parameter in the vibrodriving model.

An example of a real study case was finally performed with general positive results. Predictions of parameters effects were below 5% deviations, enabling to perform VAM faster with an acceptable accuracy. The author did not further improve the matching accuracy due to the fact of an incomplete measurement. Therefore, varying extremely soil parameters could have led to a misinterpretation of e.g. static mass into soil dynamic behavior. At this first stage of analysis, a valid perspective was shown and proven useful.

A driving formula was presented in order to have a fast response and being able to predesign a vibratory project. As already stated, further researches and data are needed to stablish trends and calibrate optimization parameters.

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VIII. Appendixes

1. Rotterdam case data

a) Pile data

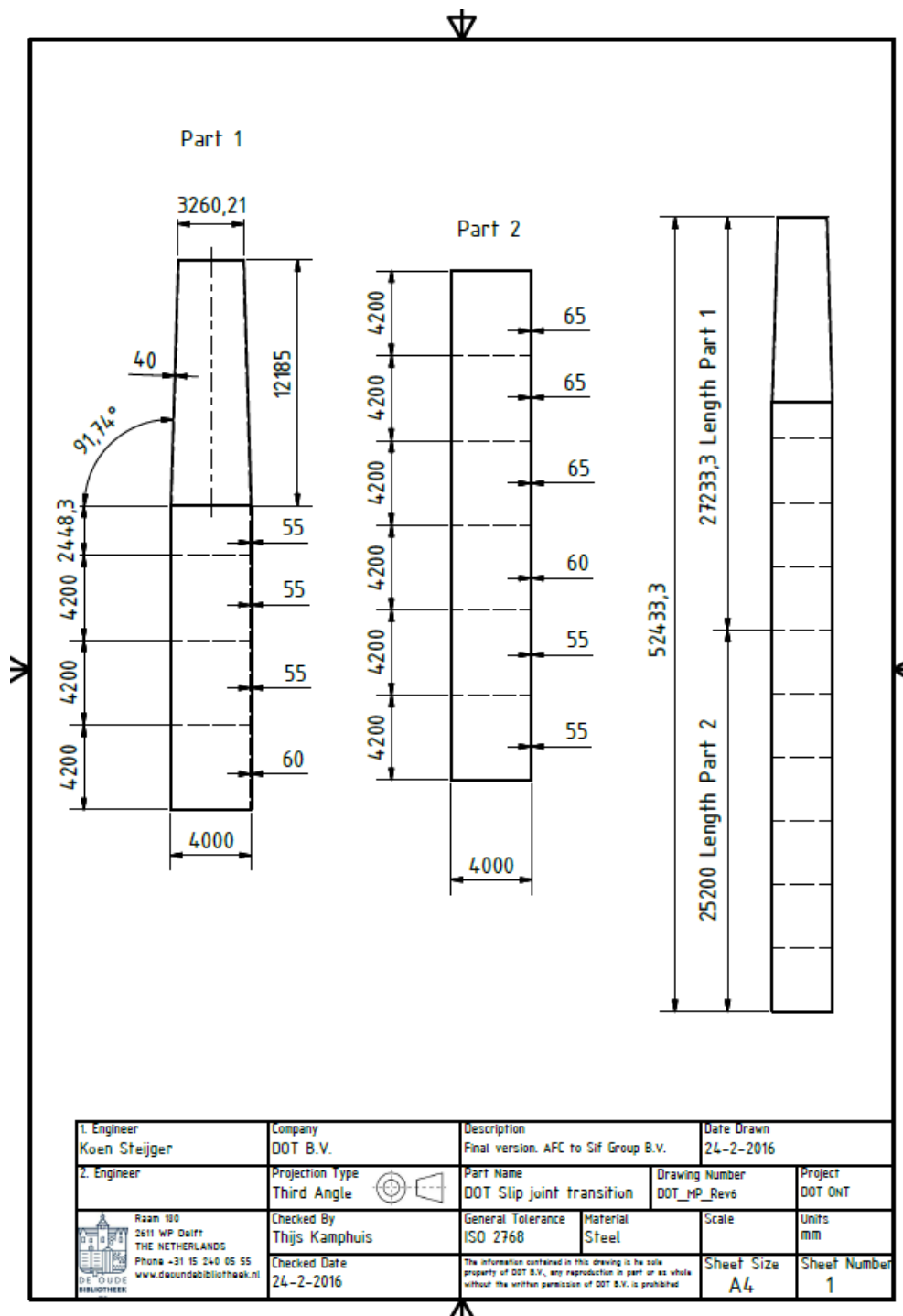


Figure VIII-1: Rotterdam pile data (only part 1)

b) Hammer Data



CV-320 VLT-U SINGLE

Vibro Hammer

SPECIFICATIONS

		CV-320 VLT-U
Eccentric moment	kgm	320
Max. Centrifugal force	kN	6,853
Max. Line pull	Te	500
Max. Safe Working Load (SWL) ¹	Te	427
Max. Hydraulic flow	l/m	2,240
Max. Operating pressure	bar	350
Max. Frequency	rpm	1,400
Weight ²	Te	88
Dynamic weight ²	Te	54
Min. number of clamps		
OC-250	qty	4
Min. pile diameter (OC-250) ³	m	Ø 1.7
Max. pile diameter (OC-250)	m	Ø 2.9
Min. Power units		
PU-1180	qty	2
PU-2400	qty	1
Dimensions		
Length	m	6.6
Width	m	2.7
Height ²	m	5.3
Vibro set ⁴ (lxwxh)	m	8.2x3.6x6.5

¹ The SWL can vary depending on the Dynamic Amplification Factor (DAF) for the specific project.

(A higher DAF results in a lower SWL)

² Including base frame and clamps

³ Outside diameter based on a wall thickness of 50 mm

⁴ Operational set in park stand

Certification: Lloyd's Register CLAME
(Code for Lifting Appliances in a Marine Environment)

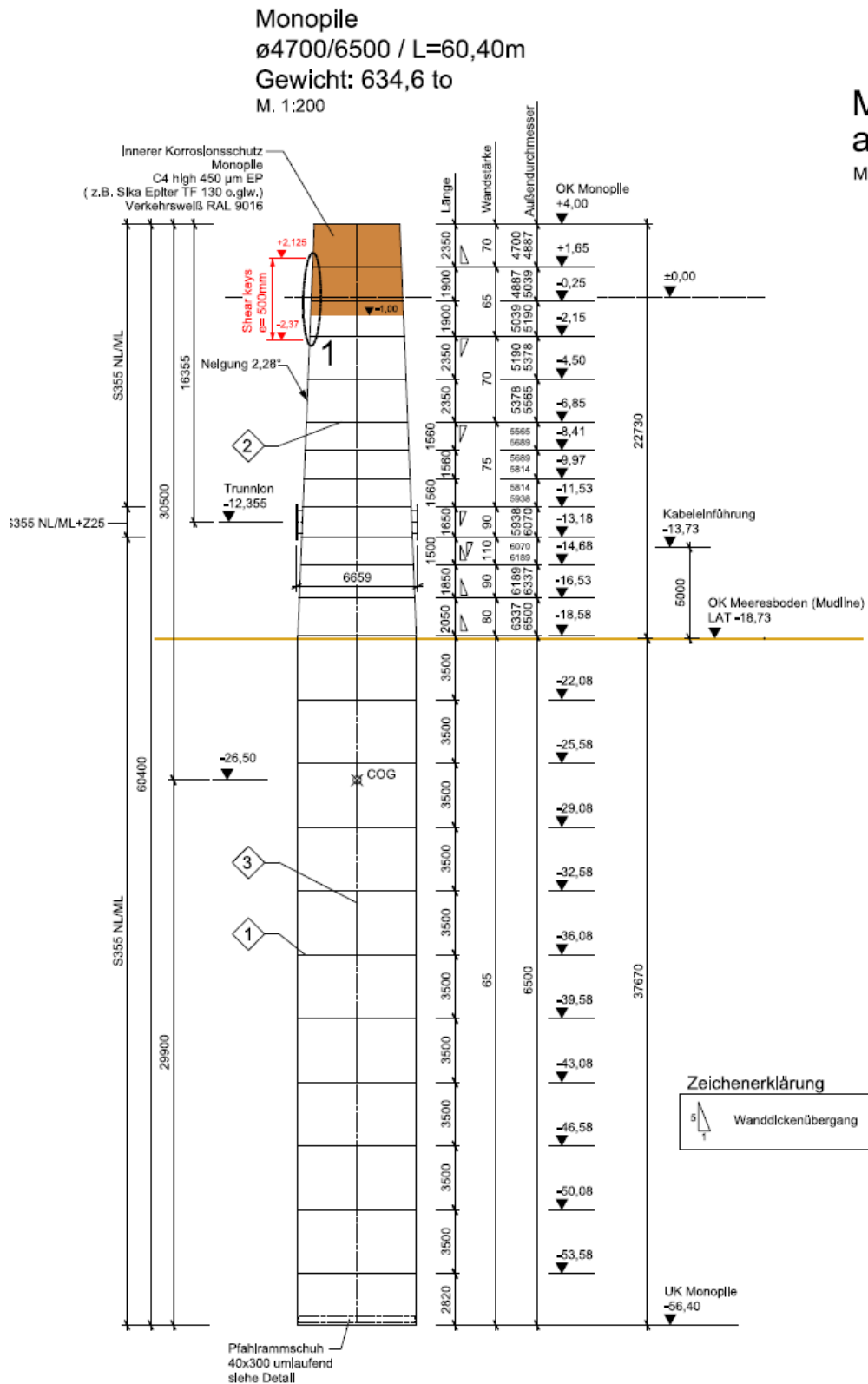


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Figure VIII-2: Rotterdam Vibro Hammer Specifications, (Cape Vibro Hammers VLT)

2. Riffgat case data

a) Pile data



b) Soil data

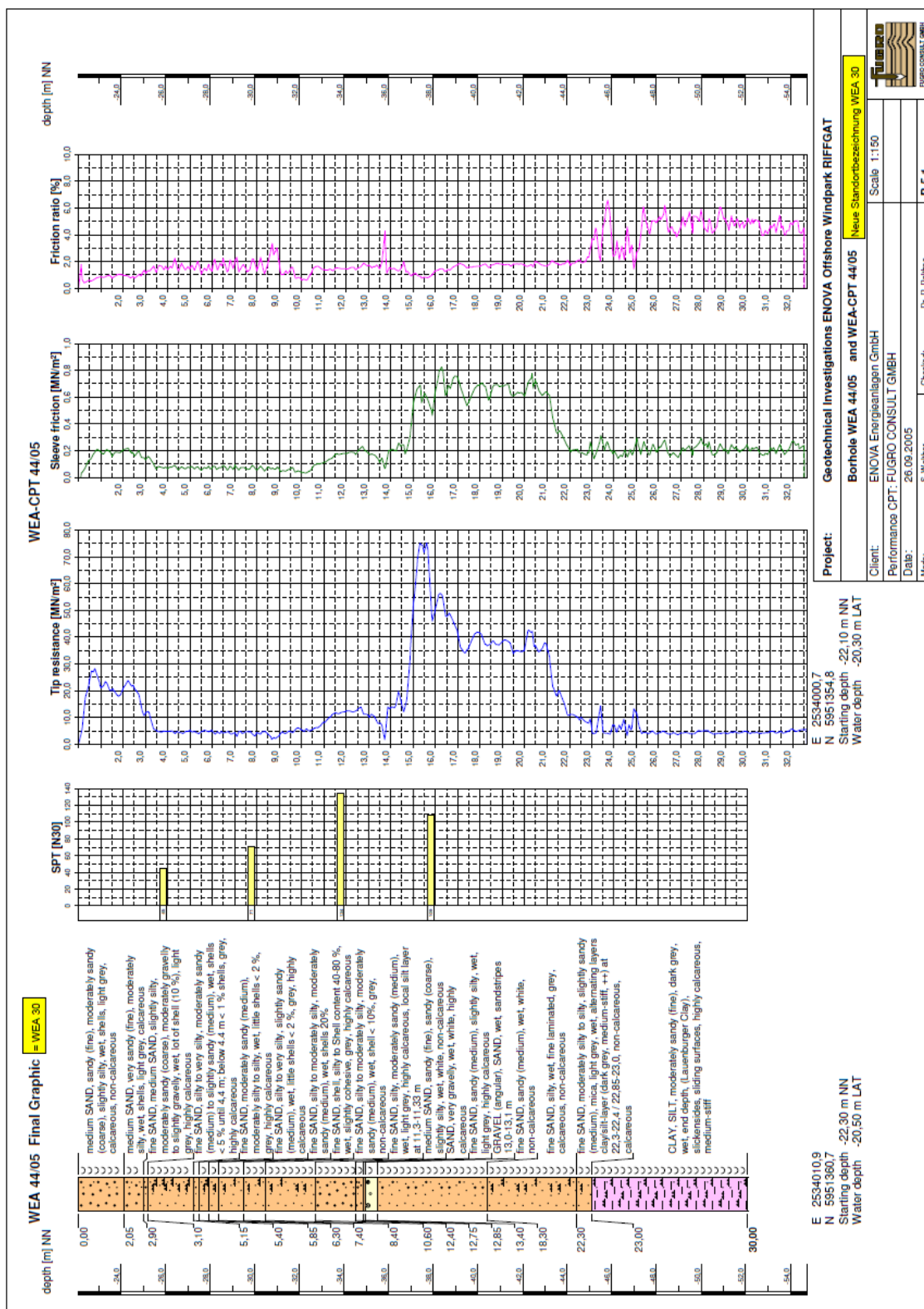


Figure VIII-4: Riffgat WEA 30 soil profile example

3. Particle shape determination test

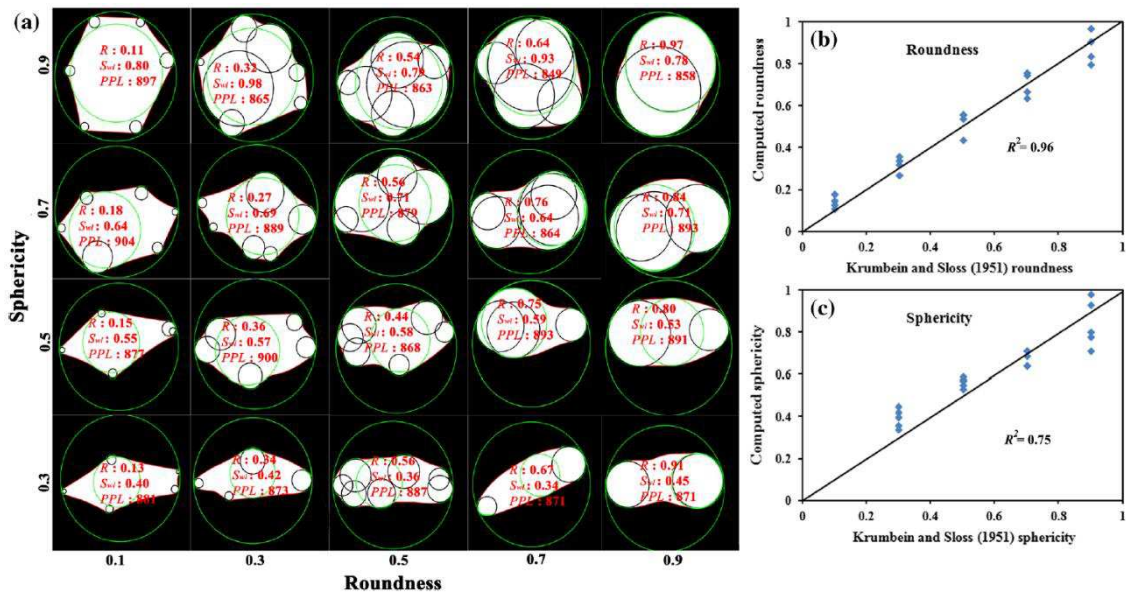


Figure VIII-5: Comparison of computed shape parameters, (Vangla, Roy, & Latha Gali, Image based shape characterisation of granular materials and its effect on kinematics of particle motion, 2017)

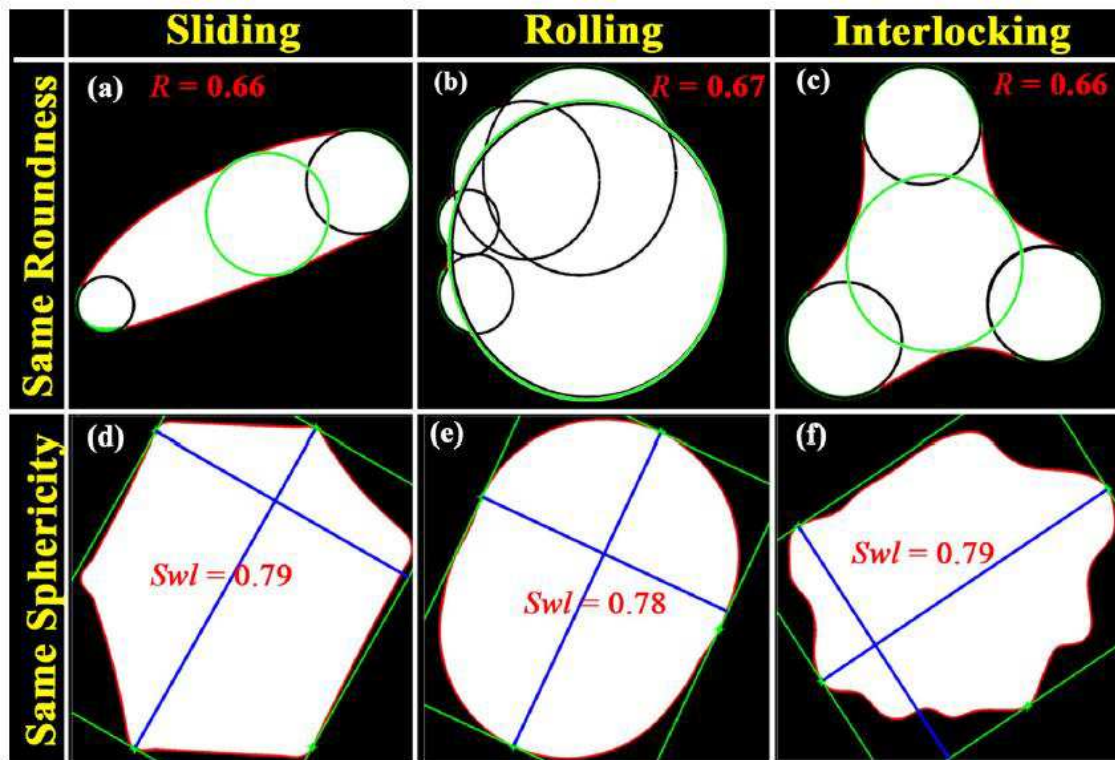


Figure VIII-6: Example of particles with same shape parameters but exhibiting different kinematic behavior, (Vangla, Roy, & Latha Gali, Image based shape characterisation of granular materials and its effect on kinematics of particle motion, 2017)

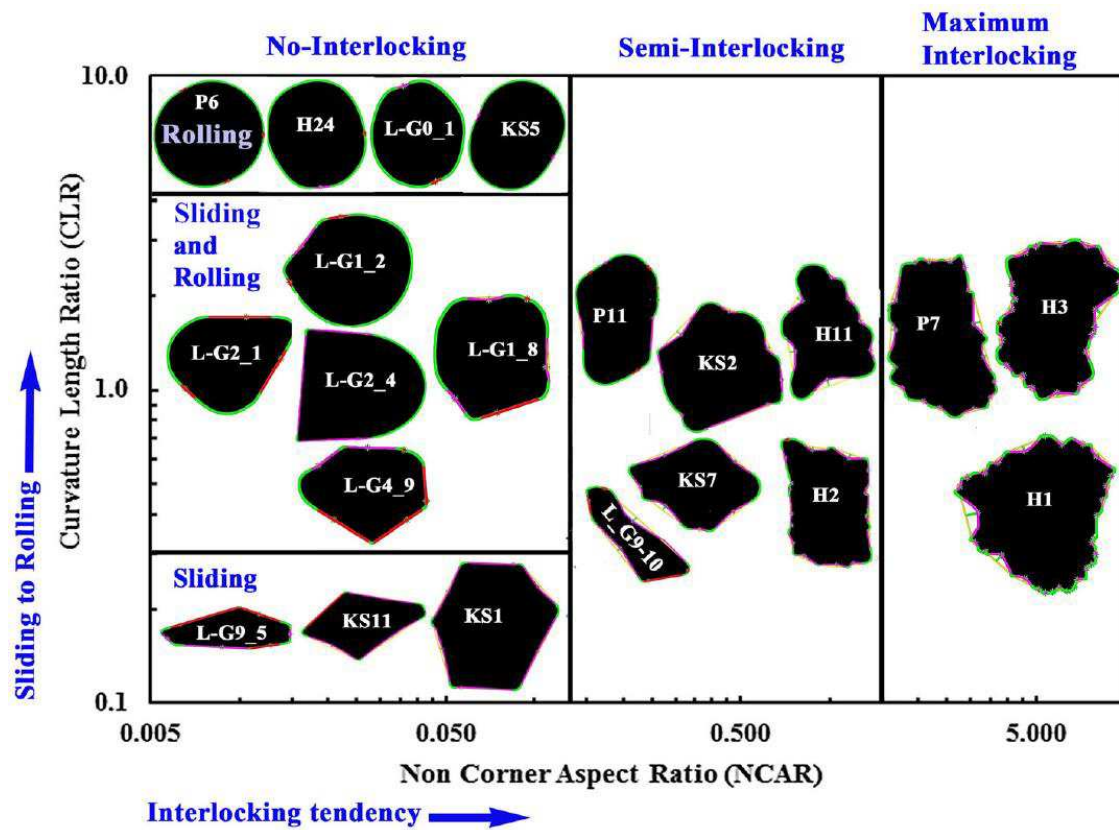


Figure VIII-7: Visual kinematic classification of typical particles, (Vangla, Roy, & Latha Gali, Image based shape characterisation of granular materials and its effect on kinematics of particle motion, 2017)

4. Pile vibrodriving formula calculation example

<i>Soil Parameters</i>		
Shaft Beta Factor	[-]	0,1
Toe Beta Factor	[-]	0,5
Max qc	[MPa]	40
Max qs	[MPa]	0,2
<i>Vibrator Parameters</i>		
Frequency	[Hz]	21,6
Vibrator dynamic mass	[tons]	161
Eccentric moment	[kg.m]	320
Centrifugal force	[kN]	5894,1
Relative top acc	[m/s ²]	20,5
Vibrator period	[ms]	46,3
Pile stress wave period	[ms]	10,5
Relative wavelength	[-]	4,4
<i>Pile Parameters</i>		
Pile length	[m]	27,235
Pile weight	[tons]	126
Average outside perimeter	[m]	0
Average inside perimeter	[m]	24,6
bottom area	[m ²]	0,74
<i>Optimisation parameters</i>		
Toe	a	0,5
Shaft	b	2
Vibrator	c	100,8

Figure VIII-8: Pile vibrodriving formula input parameters for Maasvlakte study case

Refusal depth proposed [m]	Toe resistance [kN]	Shaft resistance [kN]	Relative soil wavelength [-]	Refusal depth calculated [m]	Zcalculated - Zproposed	Refusal position
5,0	6792	988	23,9	408,67	403,67	
5,5	4857	1112	21,8	609,09	603,59	0
6,0	3967	1205	20,0	691,69	685,69	0
6,5	3981	1272	18,4	672,76	666,26	0
7,0	3996	1338	17,1	652,72	645,72	0
7,5	4010	1405	16,0	631,58	624,08	0
8,0	4025	1471	15,0	609,32	601,32	0
8,5	4657	1548	14,1	514,22	505,72	0
9,0	6123	1637	13,3	321,08	312,08	0
9,5	6771	1740	12,6	217,36	207,86	0
10,0	6771	1854	12,0	183,92	173,92	0
10,5	6771	1967	11,4	148,59	138,09	0
11,0	6771	2080	10,9	111,37	100,37	0
11,5	6771	2193	10,4	72,26	60,76	0
12,0	6582	2303	10,0	53,35	41,35	0
12,5	4024	2382	9,6	308,62	296,12	0
13,0	4331	2464	9,2	235,85	222,85	0
13,5	7600	2553	8,9	-176,64	-190,14	1
14,0	5582	2667	8,6	5,03	-8,97	0
14,5	3563	2781	8,3	184,79	170,29	0
15,0	1545	2895	8,0	362,65	347,65	0
15,5	1612	3006	7,7	301,99	286,49	0
16,0	1748	3117	7,5	231,65	215,65	0
16,5	1884	3229	7,3	159,44	142,94	0
17,0	7434	3365	7,0	-537,41	-554,41	1
17,5	8642	3503	6,8	-743,24	-760,74	0
18,0	8216	3641	6,7	-765,65	-783,65	0
18,5	7790	3779	6,5	-790,37	-808,87	0
19,0	6084	3888	6,3	-662,48	-681,48	0
19,5	5145	3993	6,1	-622,24	-641,74	0
20,0	5850	4098	6,0	-770,77	-790,77	0
20,5	6556	4202	5,8	-921,04	-941,54	0
21,0	7261	4307	5,7	-1073,05	-1094,05	0
21,5	6075	4433	5,6	-1019,61	-1041,11	0
22,0	2604	4583	5,4	-716,97	-738,97	0
22,5	1246	4700	5,3	-644,74	-667,24	0
23,0	700	4781	5,2	-652,87	-675,87	0
23,5	2901	4840	5,1	-966,34	-989,84	0
24,0	8075	4950	5,0	-1639,45	-1663,45	0
24,5	9259	5088	4,9	-1871,71	-1896,21	0
25,0	8102	5209	4,8	-1833,47	-1858,47	0
25,5	6945	5331	4,7	-1797,27	-1822,77	0
26,0	5788	5453	4,6	-1763,10	-1789,10	0
26,5	1254	5555	4,5	-1338,10	-1364,60	0
27,0	1926	5625	4,4	-1492,67	-1519,67	0
27,5	6494	5731	4,4	-2107,53	-2135,03	0
28,0	8966	5872	4,3	-2502,65	-2530,65	0
28,5	10416	6030	4,2	-2792,16	-2820,66	0
29,0	12803	6220	4,1	-3206,04	-3235,04	0
29,5	14800	6436	4,1	-3591,23	-3620,73	0

Figure VIII-9: Calculation results of pile vibrodriving formula for Maasvlakte study case

