

École polytechnique de Louvain

Experimental investigation of the impact of the installation method on the lateral behavior of piles

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Abstract

In response to the latest IPCC reports on the climate emergency and the explosion of the energy markets, European countries are increasingly switching to renewable energy sources such as solar panels and wind turbines to provide their electricity. These energy sources are inexhaustible, more sustainable and contribute to the development of some world regions but their main drawback is that they are intermittent.

In addition, the European Union has a goal of achieving carbon neutrality by 2050, which means that in less than 30 years, the entire energy system must be revised to include more sustainable energy sources. Offshore wind turbines are a great opportunity to meet this challenge and reduce the environmental impact of energy production, although they are expensive and take time to repair.

In order to support the wind turbine at sea, foundations embedded in the seabed or floating are required. Currently, wind turbines are mostly installed in shallow waters up to 40-50 m deep and are mainly founded on monopile foundations. Two main installation techniques are considered nowadays : impact driving or vibratory driving.

Impact driving consists of hitting the pile top with a hammer, often hydraulic, to drive it into the seabed. This technique is widely used because it allows for a high bearing capacity and its application to a variety of soil conditions. However, the vibrations induced by impact driving cause significant disturbance to marine life or buildings near wind farms.

To overcome the negative environmental impacts, the economic costs and the slowness of impact driving, vibratory driving was developed. This technique takes advantage of the fact that the shear strength decreases under cyclic loading by a vibrator and makes it easier to drive the pile. However, the long-term strength of the vibrated pile is not well known at present and is still the subject of research and experimentation in full-scale and laboratory settings.

In this context, this thesis investigates the influence of the embedded length, the drop height of the hammer and the relative density of the dry sand on the pile's lateral behaviour during the driving process with the goal of being able to compare these results later on with those obtained during vibratory driving.

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List of symbols

English

A	-	Factor to account for cyclic or static loading condition
AL	kPa	Measured axial load
a_p	g	Pile vibratory acceleration amplitude
A_s	mm^2	Sieve area
A_v	m^2	Effective shear area of the beam
c	kPa	Cohesion of the soil
C_1, C_2, C_3	-	Coefficient determined from fig. 6.9
coeff	-	Conversion coefficient
D	mm	Pile diameter
d	mm	Specimen diameter
d_i	mm	Equivalent particle diameter
D_p	m	Average pile diameter from surface to depth
d_s	mm	Sieve mesh size
dx	m	Infinitesimal thickness
e	-	Material void index
e_{max}	-	Maximum material void ratio
e_{min}	-	Minimal material void ratio
E_p	MPa	Young's modulus of the pile
f	Hz	Frequency of the vibrator
F	kN	Force
$F_{initial}$	Volt	Value measured at the load cell
f_d	Hz	Driving frequency
F_i	Volt	Value measured at the load cell for each measuring point i
F_s	kN	Net load applied to the soil
g	m/s^2	Gravitational field acceleration
G_{max}	kPa	Maximum shear modulus
G_p	MPa	Shear modulus of the beam
G_s	kPa	Secant shear modulus
h	mm	Specimen height
H	m	Depth
h_{laser}	mm	Laser height
h_c	mm	Height of the consolidated specimen

h_d	m	Drop height
H_r	mm	Effective depth of the densimeter
I_D	-	Compactness or density index
I_p	m^4	Second moment of inertia of the pile
k	kN/m^3	Initial modulus of subgrade reaction determine from fig. 6.10
K_p	-	Passive earth pressure coefficient
L	mm	Embedded length
LP	kPa	Measured lateral pressure
m	kg	Mass of the mould and its content
M	kNm	Moment
M_1	kg	Original dried mass
M_2	kg	Dried mass of the reject on the 0.063 mm sieve
M_c	kg	Mass of the clamping device
m_d	kg	Dry mass of the soil sample
me	$kg \cdot m$	Eccentric moment of the vibrator
M_h	kg	Hammer mass
m_m	kg	Mass of the test mould
M_p	kg	Mass of the pile
M_s	kg	Mass of the static weight
M_v	kg	Mass of the exciter block
N	kN	Axial forces
P	kPa	Total mean stress
p_u	kN/m	Ultimate bearing capacity at depth H
p_{ud}	kN/m	Ultimate resistance at deep depth
p_{us}	kN/m	Ultimate resistance at shallow depth
P'	kPa	Effective mean stress
P'_c	kPa	Effective mean stress applied during the consolidation
P_s	kg	Mass of the sieve remaining in the bottom
q	kPa	Deviator
r	mm	Initial sample radius
r	mm	Pile radius
S	m^2	Corrected surface
S_0	m^2	Initial cross-sectional area of the specimen
s_p	mm	Pile vibratory displacement amplitude
t	min	Time elapsed since the start of sedimentation
T	s	Signal period
T_c	kN	Retaining force
t_p	mm	Pile wall width
u	kPa	Pore pressure
u_{bottom}	kPa	Pore pressure measured at the specimen bottom
u_{top}	kPa	Pore pressure measured at the specimen top
V	cm^3	Initial volume of the sample
V_{air}	mm^3	Volume of air present in the specimen just after its preparation.
V_0	mm^3	Volume of the specimen after its compaction
v_c	m/s	Propagation velocity
V_C	mm^3	Volume of the specimen just before the beginning of the triaxial test
VD	mm	Measured vertical displacement of the piston
v_i	m/s	Impact velocity
V_m	m^3	Internal volume of the test mould
v_s	m/s^2	Shear wave velocity

W	mm^2	Surface of the hysteresis loop
W_c	$N \cdot m$	Hammer kinetic energy before impact
X	mm	Loading height
y	m	Lateral deflection
Z	mm^3	Bending stiffness

Greek

ΔAL	kPa	Membrane correction on axial load
$\Delta l_{initial}$	Volt	Initial value measured at the displacement sensor
Δl_i	Volt	Value measured at the displacement sensor for each measuring point i
ΔLP	kPa	Membrane correction on lateral pressure
Δu_{end}	kPa	Measured excess pore pressure at the end of the n^{th} cycle
$\Delta u_{iinitial}$	Volt	Initial value measured at the back pressure at the sensor
Δu_i	Volt	Value measured at the back pressure at the sensor for each measuring point i
ΔV_{con}	mm^3	Volume of water expelled during the consolidation.
$\Delta V_{iinitial}$	Volt	Initial value measured on the volumeter
ΔV_{sat}	mm^3	Volume of water introduced in the specimen during the saturation.
ΔV_i	Volt	Value measured on the volumeter for each measuring point i
ϵ_1	%	Axial normal strain
ϵ_3	%	Radial normal strain
ϵ_a	%	Axial deformation
ϵ_r	%	Radial deformation
ϵ_v	%	Volume deformation
η	$mPa \cdot s$	Dynamic viscosity of water at test temperature
η_c	%	Drop efficiency
γ	%	Maximum shear strain
$\gamma_{d,max}$	kN/m^3	Maximum dry density particle
$\gamma_{d,min}$	kN/m^3	Minimum dry density particle
γ_{max}	kPa	Maximum shear strain amplitude
γ_{min}	kPa	Minimum shear strain amplitude
γ_d	kN/m^3	Obtained dry density particle
κ	-	Curvature strain of the beam element
λ	-	Diameter scaling ratio
λ_σ	-	Stress scaling ratio
λ_d	-	Damping ratio
λ_F	-	Force scaling ratio
λ_M	-	Moment scaling ratio
ω	-	Cross-sectional rotation
ω_v	rad/s	Angular frequency of the vibrator
ϕ	$^\circ$	Friction angle of the soil
$\rho_{d,max}$	kg/m^3	Maximum dry volumic mass of the material
$\rho_{d,min}$	kg/m^3	Minimal dry volumic mass of the material
ρ_d	kg/m^3	Dry density of the material
ρ_s	kg/m^3	Density of solid particles
ρ_w	Mg/m^3	Density of the sedimentation fluid at the test temperature
σ'	kPa	Effective normal stress
σ	kPa	Total normal stress
σ_1	kPa	Axial normal stress
σ'_1	kPa	Effective axial normal stress

σ'_3	kPa	Effective radial normal stress
σ_3	kPa	Radial normal stress
σ_v	MPa	Vertical earth pressure at a given depth
τ	kPa	Total shear stress
τ'	kPa	Effective shear stress
τ_{max}	kPa	Maximum shear stress
τ_{min}	kPa	Minimum shear stress

Definition

- **Reject** : Part of the grains retained in a sieve.
- **Cumulative reject** : All the grains retained in sieves with larger holes than the sieve considered.
- **Sieve** : Container made of wire cloth or perforated sheet metal defining a mesh of holes.
- **Deviatoric stress** : Difference in total vertical stress from a state of uniform isotropic stress.
- **Failure**: When the maximum deviatoric stress is reached
- **Undrained shear strength** : Equal to half the deviatoric stress at failure in the unconsolidated, undrained triaxial compression test.
- **Density of a material** : Ratio of the mass of a certain volume of that material to the mass of an equal volume of water under the same conditions of temperature and pressure.
- **Pore water pressure** : Pressure exerted by underground water on soil particles. Two different types, hydrostatic and dynamic. Hydrostatic develops linearly with depth. Cyclic soil loads transfer dynamic pressure to the groundwater. This pressure is temporarily stored and will gradually dissipate completely.
- **Cohesion** : Interparticle force between grains. Prevents a soil to achieve liquefaction by maintaining contact between soil particles.
- **Toe resistance** : Soil reaction acting on the transversal bottom area of the pile, superficially and equally distributed. Overcoming this resistance and shaft 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.
- **Bearing capacity** : Maximum vertical force that can be applied to the foundation before failure.

- **Liquefaction** : When a sandy saturated soil is cyclically loaded. It occurs when the pore water pressure equals the contact force between the particles. The soil then behaves as a fluid like state which decreases soil resistance even further than cyclic loading.
- **Cyclic load** : As opposed to a monotonic load, cyclic loading means unequal loading with time distributed around a mean value in between a maximum and minimum value. That kind of loading fatigues the soil and degrades its strength to a residual value.
- **Effective stress** : Difference of the soil stress and the pore pressure.
- **SIMOX consortium** : Development and practical implementation of one or more innovative technologies for the installation of XXL monopiles, as an alternative to conventional impact hammering [33].
- **Monopile** : a large diameter steel tube which is driven by impact driving or vibratory driving to a depth of 5 to 10 times the diameter.
- **Hammer** : Device producing impacts or blows.
- **Impact driving** : Action of hitting with a hammer a linear element.
- **Vibrator** : Metal box containing the whole vibrating device.
- **Pile driver's helmet** : System covering the pile and receiving the hammer blows.
- **Vibratory driver's helmet** : System equipped with one or more hydraulic clamps allowing the vibrating box and the pile to be joined together.
- **Elastomer** : Device connecting the vibrating box to the crane hook and equipped with a vibration damping system.
- **Mass** : Mass capable of exerting a centrifugal force by rotation in a mechanical system.
- **Reject** : Residual pile driving.
- **Soil p-y waves** : Soil is modelled as a collection of independent spring. The load displacement behavior of the springs can be specified using parameters for common p-y curves [12].

CHAPTER 1

Introduction

The latest IPCC reports on the climate emergency and the explosion of energy market prices are now encouraging countries to turn to renewable energies and the construction of related infrastructure such as wind turbines, solar panels and hydroelectric dams. The main advantages of most renewable energies are that they are environmentally friendly and are inexhaustible energy sources. They also enable the decentralisation of energy production and contribute to the development of some regions of the world. However, the main challenges of solar panels and wind turbines are that they are intermittent, their production depending on the area of installation, the weather and the seasons, and they have a visual impact on the landscape.

Moreover, in order to meet consumer needs, it is often necessary to combine renewable energy with fossil energy. However, the production and consumption of fossil fuels in Europe account for 70% of greenhouse gas emissions [47]. It is therefore important to turn to innovative and increasingly efficient but environmentally friendly techniques with moderate investment costs to remain competitive in the market. In spite of the clearly established necessity, it is very difficult to find a balance between the installation of renewable energies as a measure to deal with the energy and climate crisis and acceptable environmental impacts.

To overcome these challenges and achieve carbon neutrality as soon as possible, the European Commission has imposed targets for the contribution of renewable energy sources to the total electricity production of all member states. The objective at European level is to reach 40% of renewable energy in the energy mix and to reduce greenhouse gas emissions by 55% by 2030 [47]. For Belgium, 13% of its total energy consumption should be covered by renewable energy in 2020 and 17.5% by 2030 [53].

To achieve this, Belgium has built wind farms in the Belgian EEZ part of the North Sea and along the border with the Netherlands. A total of 399 turbines are now operational in nine wind farms, with an installed capacity of 2.26 GW and an average production of 8 TWh. This represents about 10% of Belgium's total electricity demand. At the European level, Europe wants to reach an offshore wind capacity of at least 60 GW by 2030 and 300 GW by 2050. Offshore wind turbines are therefore a great opportunity to reduce the environmental impact of energy production, even if their cost remains high [53].

The principle of the offshore wind turbine is similar to that of an onshore wind turbine. Under the action of the wind, the propeller's blades turn and create mechanical energy. This energy drives a generator, which in turn produces an alternating electric current. It is then carried by underwater cables to a substation and distributed to the electricity grid [15].

The advantage of wind turbines is that they do not emit greenhouse gases or other pollutants during the electricity production. As the sea is a more open landscape than most onshore wind turbines environment, the wind encounters less obstacles and is therefore steadier, more regular and less turbulent. With the same power rating, the offshore wind turbine generates up to twice as much electricity as its onshore counterpart. However, installation and maintenance costs are high and repairs take time. Offshore wind turbines require more robust construction materials to withstand corrosion and the often violent and unstable movements of the waves. The propeller blades are longer to accommodate the weather conditions and the mast on which the propeller stands is also higher to capture the full power of the wind [15].

In order to keep the wind turbine in place, they are either fixed to the seabed or floating. The choice of foundation type depends mainly on the type of the soil and the seabed depth at the installation site. Fixed foundations are particularly used in shallow waters up to 80 m deep, while floating foundations are used for deeper waters beyond 50 m.

Currently, most wind turbines are installed in shallow waters up to 40-50 m deep and the main type of foundation is the monopile. This consists of a large diameter steel tube which is driven by impact driving or vibratory driving to a depth of 5 to 10 times the diameter below the sea floor [15].

Impact driving means hitting the top pile with a hammer to drive it into the seabed. The drop of the hammer on the pile head generates a wave that propagates along the pile to the toe. The energy generated at the pile head overcomes the lateral friction along the pile and causes plastic deformation of the soil at the toe. Pile driving occurs.

Impact driving is widely used as it leads to a higher bearing capacity and applies to a variety of soil conditions. The long history and experience of this driving technique means that it is currently preferred to vibratory driving. Nevertheless, the high amplitude vibrations induced by impact driving lead to significant disturbance of marine life or buildings near wind farms. To reduce the underwater noise associated with the installation of monopiles to national underwater noise regulations, the use of double bubble curtains and soundproof sleeves have proven to be partially effective, but are expensive solutions [29].

To counteract the negative environmental impacts, the economic costs and the slowness of impact driving, experiments with vibratory driving have been carried out in recent years. This technique takes advantage of the fact that the soil resistance decreases under cyclic loading provided by a vibrator installed at the top of the pile. The degradation of the soil strength makes it easier to drive the pile. Vibratory driving is based on the emission of forced longitudinal vibrations produced by a vibrator with an even number of eccentric masses arranged symmetrically to the vertical axis of the pile and rotating in opposite directions. These vibrations generate only a vertical force that allows the pile to be driven [71].

This technique is widely used in soft, saturated or cohesionless soils because of its speed and low noise. However, the long-term strength of the vibrated pile is usually not well known and is still being researched and experimented.

In the long term, the impact driving and vibratory driving methods will be compared experimentally by students on a reduced scale for different relative densities, saturated and unsaturated sand, two pile diameters (6 and 10 cm) and for several driven lengths in order to analyse the pile lateral behaviour according to the initial parameters.

Objective of this thesis

The purpose of this thesis is to produce a scale model of a foundation pile for an offshore wind turbine in order to test the influence of impact driving, the driven length, the drop height of hammer and the relative density of dry sand on the lateral behaviour of the pile. In order to study the behaviour of the pile driven by impact, this thesis is divided into two main parts :

- Theoretical part
 - Background on the current situation of offshore wind turbines in Europe.
 - Literature review on the different pile driving methods, the existing pile types and the available and future technologies for offshore wind turbines.
 - Definition of North Sea sand properties via different tests such as the Grain size distribution test, Density test, Proctor test, Triaxial test and Cyclic triaxial test.
 - Study of the influence of the sand drop height via a sand raining system in order to obtain different relative densities for the experimental part.
- Experimental part
 - Development of the physical and theoretical aspects of impact pile driving through simulations.
 - Presentation of the similitude laws, equipment and methods used for the scale model experiments.
 - Discussion of the results obtained by varying the relative density of the dry sand, the driven length and the drop height of the hammer during the impact driving process on a 6-cm diameter pile.

CHAPTER 2

Context

The European Union is the third largest energy consumer in the world, after China and the United States. More than 70% of the energy consumed in EU is of fossil origin (fig. 2.1). Currently, three quarters of Europe's greenhouse gas emissions comes from energy production and consumption. It is heavily dependent on the rest of the world for energy, particularly Russia and the Middle East for gas and oil. Some countries are rather dependent on nuclear power, notably Belgium, which produces half of its electricity from this source, and others, such as Germany, are essentially dependent on coal, which it is intended to phase out by 2038. The share of renewable energy sources in Europe's energy mix is constantly increasing. It is in 2019 at 36.5% (fig. 2.1)[47].

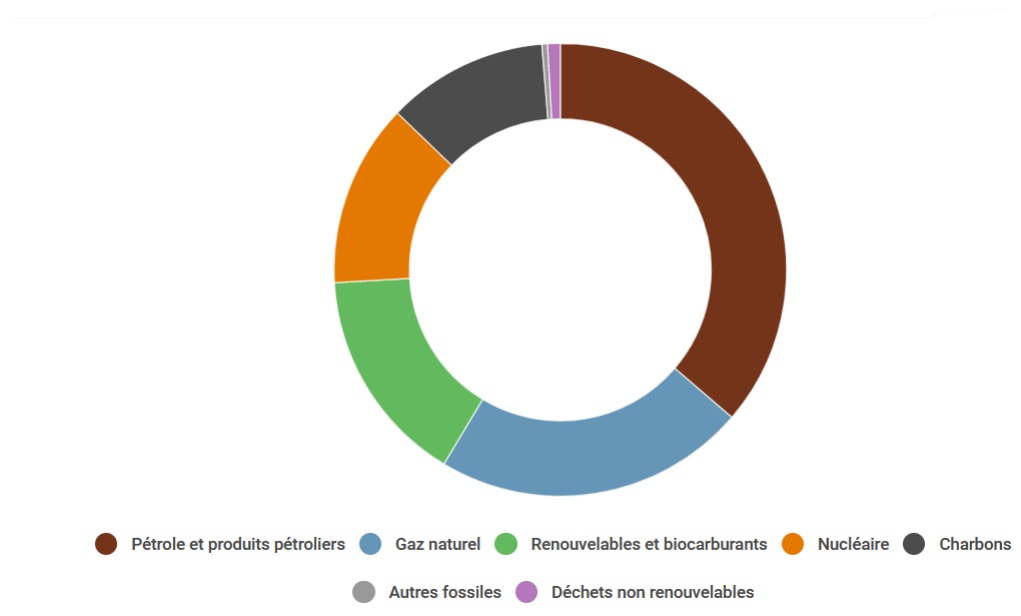


Figure 2.1: Available gross energy in EU in 2019 [47]

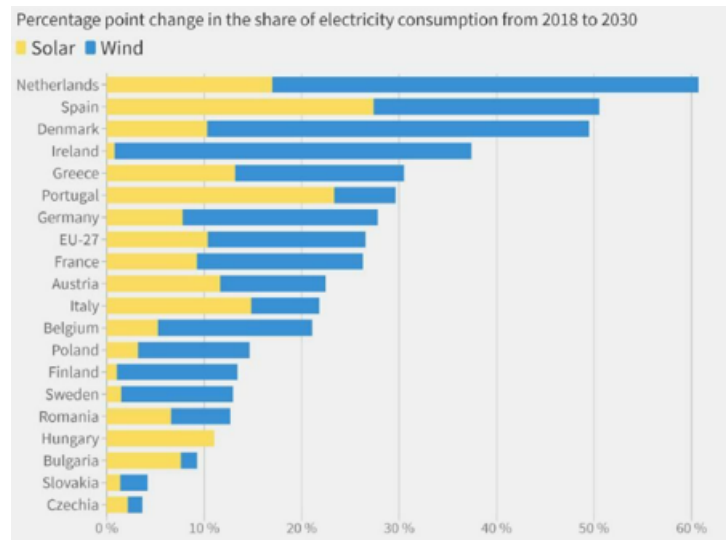


Figure 2.2: EU wind and solar deployment over the next decade [55]

Moreover, for several years now, the acceleration of global warming, environmental degradation and the melting of glaciers have been of concern to European countries. The challenges of reversing this trend and reducing greenhouse gas emissions are enormous. To meet them, the EU countries have put in place a Green Deal. This specifies, among other things, that European countries aim to achieve a 40% share of renewable energy in the energy mix by 2030 and to reduce greenhouse gas emissions by 55% (fig. 2.3). Currently, with the invasion of Ukraine and rising energy prices, Europe is seeking to become independent of Russian fossil fuels, diversify its energy supply and accelerate its transition to renewable energy and hydrogen. To achieve this, the EU is relying on innovative techniques. Its goal is to expand its offshore wind capacity. In 2020 alone, wind power in Europe increased by 9% and Belgium achieved a 28% growth in wind and solar power [47].

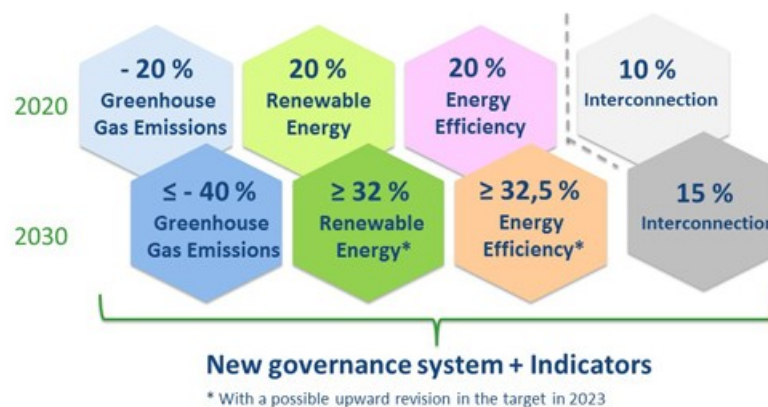


Figure 2.3: EU 2020 and 2030 targets for reducing greenhouse gas emissions [47]

By the end of 2020, 38% of Europeans' electricity production had switched from fossil fuel to renewable energy. Europe can count on a large potential offered by its seas : the North Sea, the Baltic, the Mediterranean and the Atlantic Ocean. By exploiting and expanding offshore wind, Europe can reduce its CO2 emissions and become carbon neutral by 2050 (fig. 2.4). As the geological conditions of the European basins differ, some technologies will be more suitable than others [28].

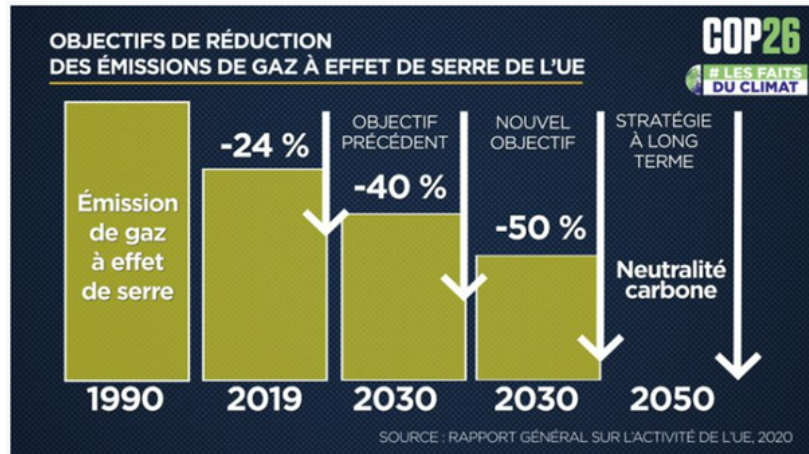


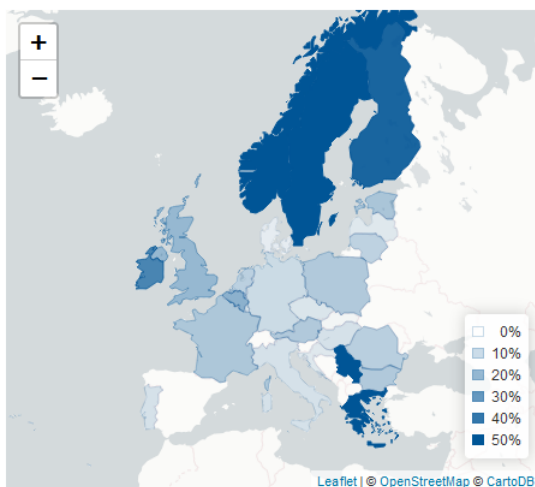
Figure 2.4: EU 2020 and 2030 targets for reducing greenhouse gas emissions [8]

The advantage of wind energy is that it is theoretically inexhaustible and has little negative impact on the environment because it only emits a very low amount of greenhouse gases compared to more classical energy sources. Wind turbines are therefore a perfect response to tackle climate issues and the growing need for energy.

Currently, the majority of wind turbines are built onshore, but the offshore wind turbine has many advantages making it attractive and explaining its expansion. Wind at sea is less turbulent, more regular and has a higher average speed. It allows for an increase in electricity production and production time (fig. 2.5). Offshore wind turbines produce twice as much as onshore wind turbines with an equal power rating. They also have the advantage of having less visual and noise impacts. Greater production capacity turbines can therefore be implemented. The location of offshore wind farms near the coast also reduces losses in the transmission of energy.

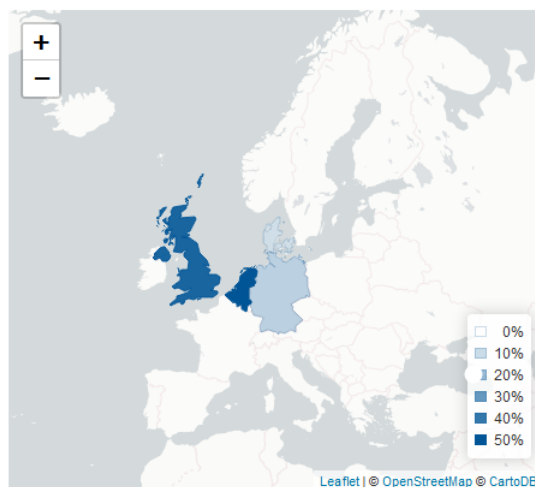
Onshore wind

17.1%



Offshore wind

43.2%



The capacity factor is the average power generated by wind divided by its peak capacity.

Figure 2.5: Onshore and offshore wind turbines capacity factors on 5th June 2022 [26]

In 2020, almost 2.9 GW of new offshore wind turbines have been installed in European waters (fig. 2.6). This brings Europe to a total of 25 GW of offshore wind capacity spread over 12 countries (fig. 2.9). In Belgium, the nine offshore wind farms currently have an installed capacity of 2.26 GW in the North Sea and cover 13% of the total Belgian electricity demand (fig. 2.7). By 2030, Belgium wants to triple its offshore wind capacity in the North Sea by opening a new wind farm called "Princess Elisabeth" and reach a capacity of 5.7 GW. Offshore wind power is climate-friendly and will enable Belgium to accelerate its transition to energy independence. As a comparison, 450 offshore wind turbines of 5 MW are needed to replace a nuclear reactor of 1100 MW since the wind is a random phenomenon [25][11][20].

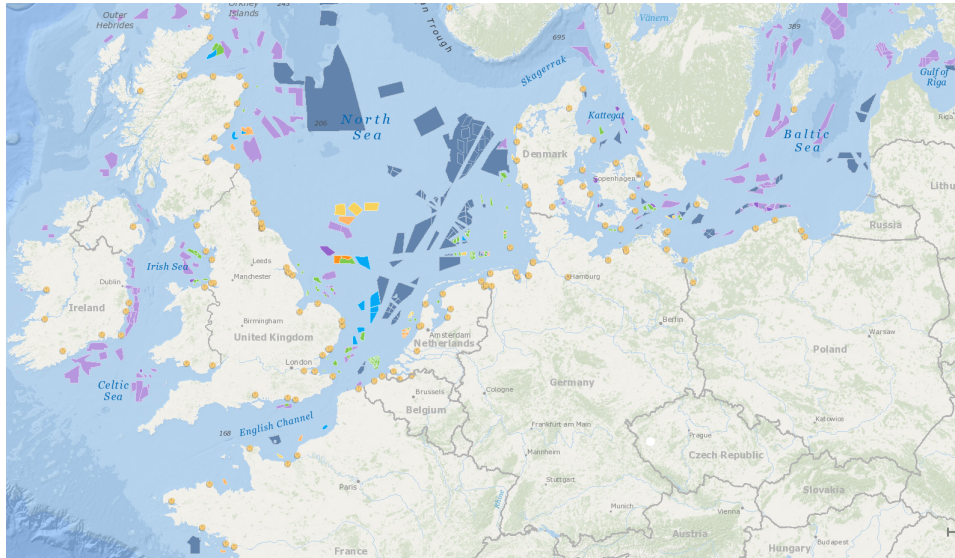


Figure 2.6: Localisation of windfarms in European waters [3]

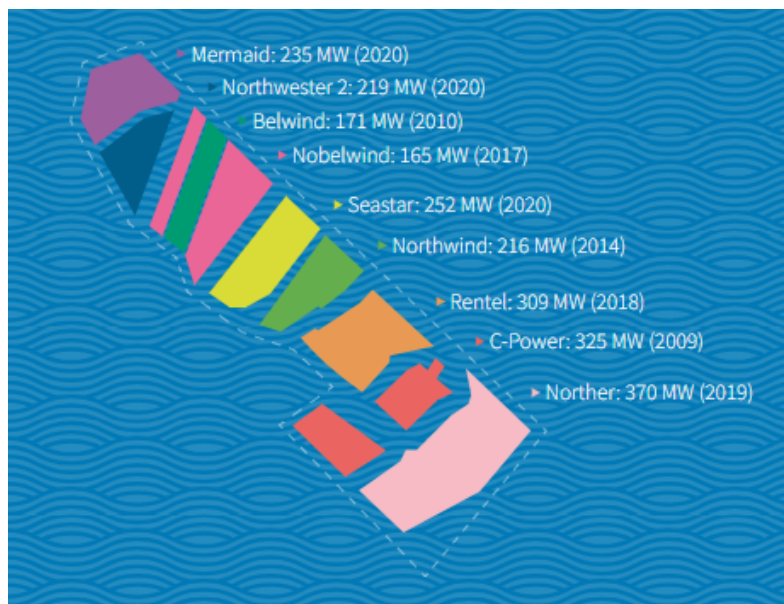


Figure 2.7: Electricity production capacity of wind farms in Belgium [11]

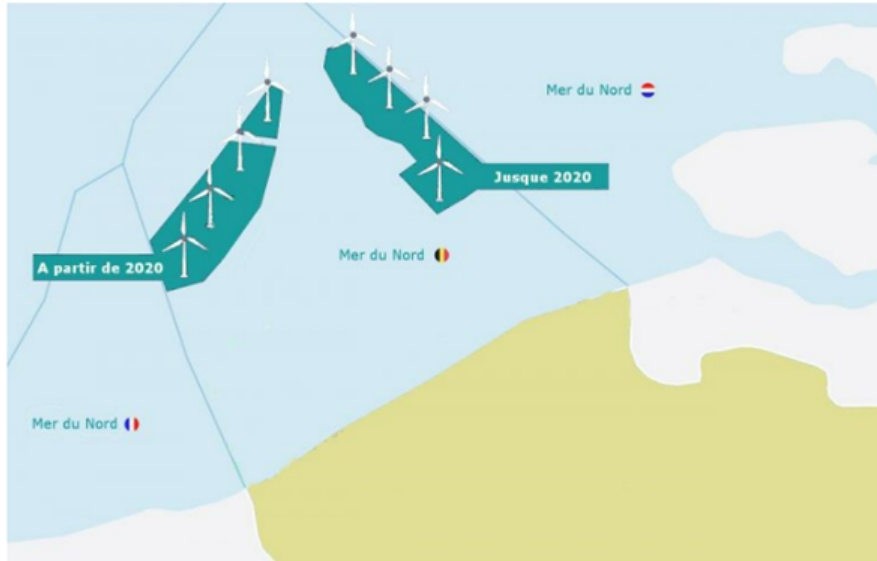


Figure 2.8: Location of new wind farms in Belgium [11]

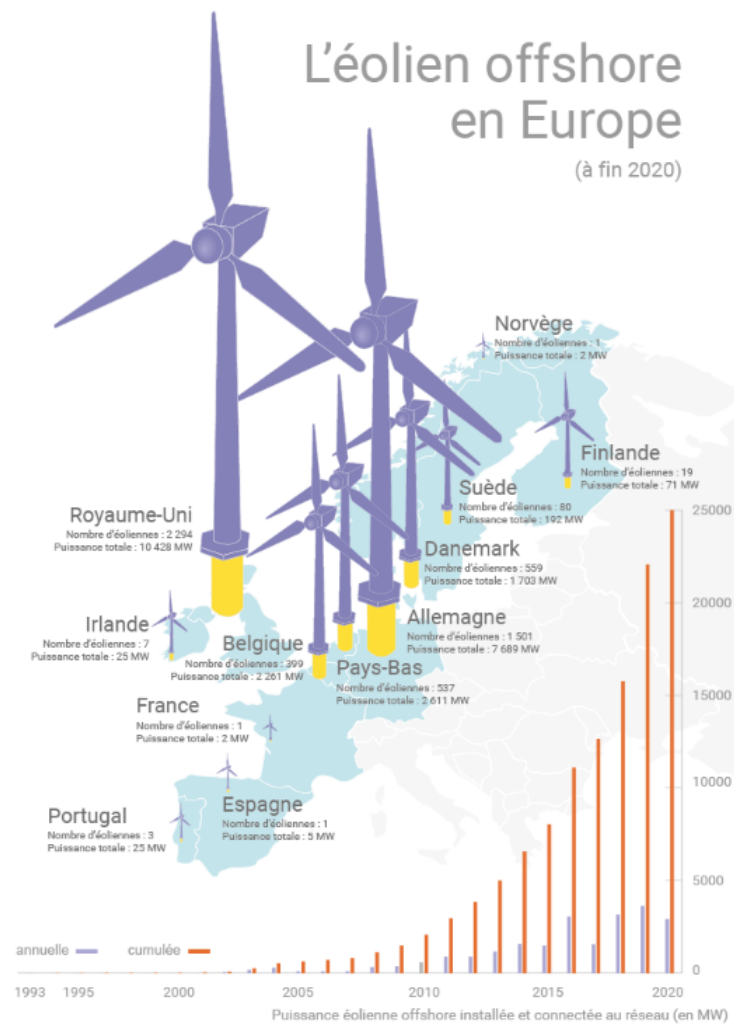


Figure 2.9: Amount of offshore wind turbines in Europe at the end of 2020 [20]

Until now, most wind farms are located in shallow waters and require infrastructure and facilities close to the shore and good wind conditions. To increase production, solutions for the installation of turbines in deeper waters between 50 and 1000 m will have to be considered, in particular with floating wind turbines. These will bring Europe closer to its clean energy goals. The production capacity of the largest turbines will also be increased from 9.5 MW currently to 15 MW in a near future [28].

The main technique used to install offshore wind turbine foundation is impact driving, using the energy developed by hammer blows to drive the pile in the seabed. Unfortunately, this technique generates high amplitude vibrations and sound waves that are harmful to marine animals.

In this context, the research started by this thesis compares the influence of impact driving and vibro-driving on the pile's lateral behaviour and its resistance at the base. The aim of this comparison is to find economical and efficient solutions to the disadvantages of impact driving, particularly in terms of the environment and productivity. Having an effective model to allow to simulate impact driving mechanisms on piles, will allow to prepare comparative studies with vibro-driving mechanisms.

3.1 Characteristics and operating principle of the offshore wind turbine

A standard wind turbine is composed of a mast, a nacelle and a three-bladed rotor(fig. 3.1). The rotation of the rotor generates loads with a specific frequency spectra. Wind direction sensors rotate the nacelle to align it with the wind direction. This maximises the amount of energy produced. The principle of a wind turbine is to transform the kinetic energy of the wind into electrical energy with the help of a rotor which then supplies energy to the electricity grid. Its energy comes from a natural, renewable and theoretically inexhaustible source of energy : the wind.

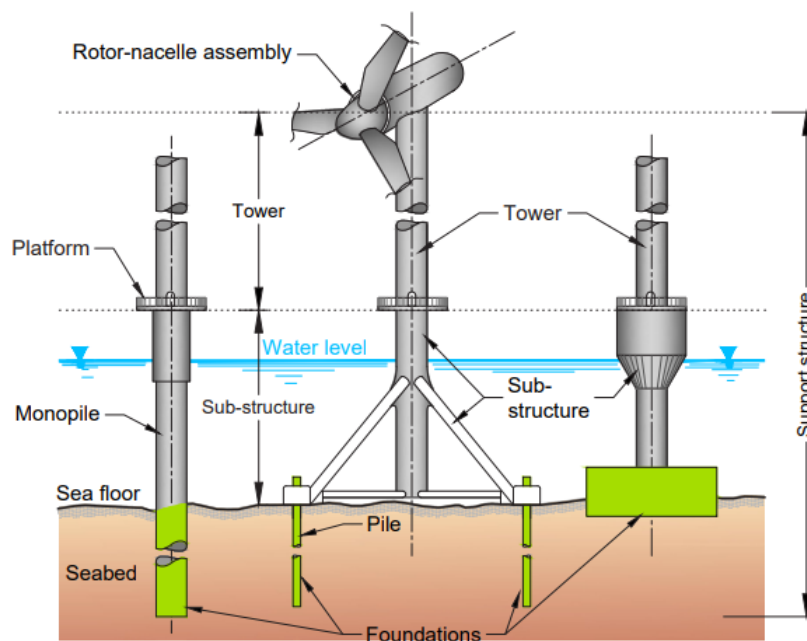


Figure 3.1: Components of an offshore wind turbine [30]

Its advantage over fossil fuels is that it does not emit greenhouse gases or other air pollutants during the production of electricity and requires little water. The CO_2 emissions associated with the design, transport, operation and dismantling of wind turbines are compensated in 6 to 9 months of operation. 85-90% of a wind turbine can be recycled and the aim is that by 2025 the majority of the blades will be recycled. Therefore, industries are striving to improve the efficiency, recycling and substitution of materials in wind turbines.

During the installation of offshore wind turbines, bottom trawling and dredging are prohibited in order to preserve the seabed. Air bubble curtains and hydro-sound dampers are also used to limit the negative impacts on the marine environment. Companies are working on wind turbines with less noise and reduced aerodynamic noise created by the movement of the blades in the wind. The standard life span of a wind farm is about 20-25 years. Currently, the efficiency of onshore wind farms is 40%, while for offshore wind farms it is 60%.

Regarding the design of a wind turbine, a wide range of shapes and sizes are available from manufacturers, but the standard composition consists of three blades mounted on a horizontal axis. Their power output can vary from 10 kW to 12 MW and their location can be both onshore and offshore. The latest models of wind turbines are 6 to 10 times more powerful and durable than the first ones.

The design of the turbine and thus the production of electricity depends on three parameters :

- The radius of the blades. In order to obtain a high electricity production, this must be large. If the blade radius is doubled, the power is multiplied by 4.
- The density of the air. This parameter is influenced by altitude, temperature and atmospheric pressure. Dense air near the coast and at sea drives the rotors more efficiently and increases electricity production.
- Wind speed. At sea, the winds are stronger. This means that wind turbines are more responsive and produce electricity more consistently than onshore turbines. As technology advances, wind turbines are able to operate at lower wind speeds.

3.2 Loads on an offshore wind turbine

Offshore wind turbines are rigid, slender structures subject to three types of loading :

- Permanent load : weight of the structure
- Variable load : maintenance load, installation load, vessel impact
- Environmental loading : wind, tides, hydrodynamic stress caused by swell and currents, ice in some regions of the world

The vertical loads are small while the lateral load and the overturning moment are high. There is a significant cyclic loading.

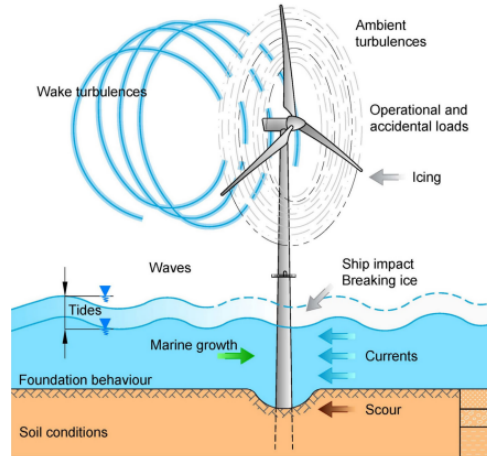


Figure 3.2: Modelling of an offshore wind turbine [30]

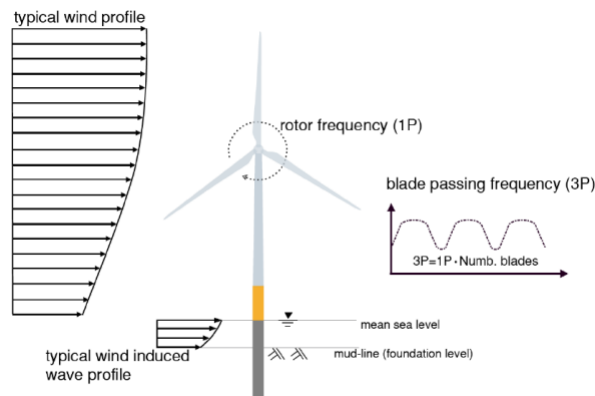


Figure 3.3: Typical loads applied on offshore wind turbine [28]

In addition, while the composition of offshore wind turbines is standard, the choice of foundation type varies depending on the soil, the depth of the seabed at the place of installation and the industries located nearby.

3.3 Offshore wind turbine foundations

A foundation is defined as the part of the structure that supports the weight of the structure and transmits the load to the underlying soil or rock.

Two main categories of foundations are represented in the literature : fixed and floating foundations. Fixed foundations transmit environmental forces directly to the ground through a rigid connection. They are mainly used in shallow waters (up to 80 m depth). Floating foundations, on the other hand, transmit forces through cables connected to foundations anchored in the seabed. The connection to the seabed is therefore more flexible and the structure can move more. This type of foundation makes it possible to reach greater water depths (beyond 50 m).

At present, wind turbines are mainly installed on fixed foundations in shallow waters, up to 40-50 m deep. Floating wind farms are very much in the minority at the moment, but are an important area of research. These could be installed further offshore where the wind is more regular and stronger but are proving a challenge in horizontal stability and power transmission.

For each of these categories, different structural types of foundation can be considered, depending on the state of the ground and the depth of the sea :

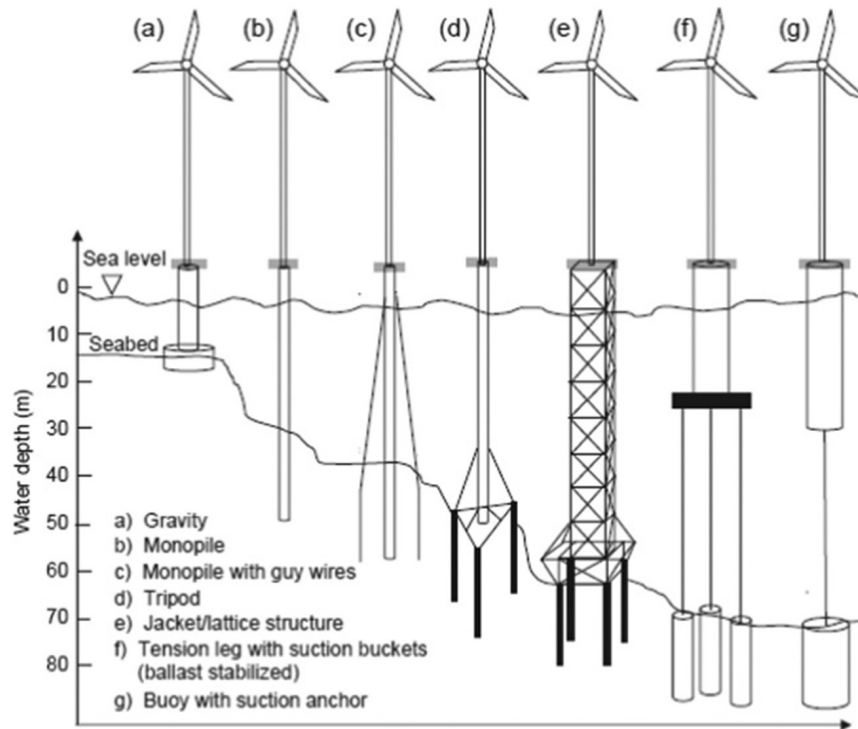


Figure 3.4: Different types of foundation for offshore wind turbine [9]

- Gravity structures are usually massive concrete or mixed steel structures sunk to the seabed using gravel, sand or stones as ballast. They are mainly used in shallow waters (typically 10 m) but can be installed up to 40 m deep. Designs are being developed to use gravity structures in deeper waters. The great advantage is also that gravity structures do not need to be driven into the ground. They can therefore be used for clay and rocky soils where other types of foundations reach their limit [24].
- Nowadays, monopiles are the main foundations used for offshore wind turbines. The monopile is essentially a hollow tubular steel element with a large diameter ranging from 4 to 10 m driven deep into the sand to transmit wind and wave loads directly to the ground. They are mainly used for water depths up to 40 m and are installed vertically. Their main advantages are their simple design and quick installation. The pile is usually driven or vibrated with a hydraulic or diesel hammer into cohesionless soil. In rocky ground, pre-drilling is carried out and the pile is held in place by grouting [9].
- Tripods and jacket structures are braced structures inherited from the oil and gas industry. They allow for greater water depths (up to 50 m) but are generally less economical and much more difficult to install. Environmental actions are mainly transferred to the ground by axial loading [9].

- Caisson foundations are suitable for sandy soils. The foundation consists of a cylindrical caisson with a connection to the tower at the top and a suction piston at the bottom. The caisson is placed on the ground and then by suction and under its own weight the foundation penetrates into the ground. This type of foundation is suitable for water depths up to 30 m [22].
- Floating structures are still in the design phase and are not widely used at the moment. A few small floating wind farms are emerging (about 120 MW of cumulative global capacity by 2022), but larger projects are being commissioned. This technology will mainly be used in deeper waters (beyond 50 m) and far from the coast [69].

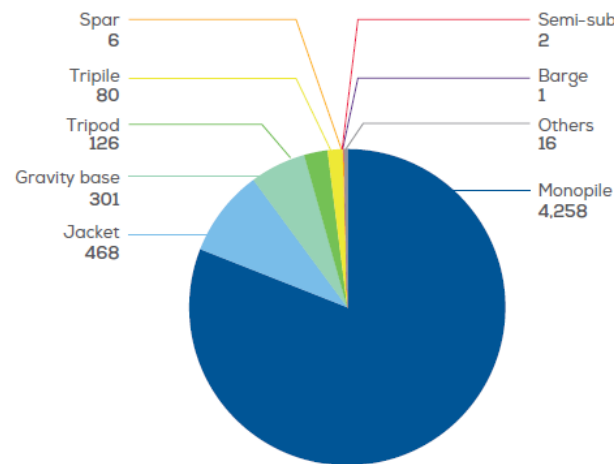


Figure 3.5: Type of foundation presents in European waters in 2019 [27]

This thesis will focus solely on monopile foundations for offshore wind turbines as this is the most common type of foundation currently used and offers the best value for money. These monopiles are described in terms of their characteristics and principles in the following section.

3.4 Monopile

The monopile foundation is a large diameter steel tube driven into the ground to a depth of about 5 to 10 times its diameter. Over the years, the diameters used have increased as the installation depth and power rating of the wind turbines got larger. Whereas previously the maximum diameter was about 6 m, today it reaches more than 9 m in the deepest waters.

In order to place the wind turbine on top of the monopile, a typical configuration is to place a transition piece (TP) between the two structures. This part usually contains equipment such as cathodic protection, access to the vessel for maintenance and the cabling that allows the energy generated by the turbine to reach the grid. The connection between the parts is mostly made out of grout.

A typical installation of a wind turbine supported by a monopile can be seen below :

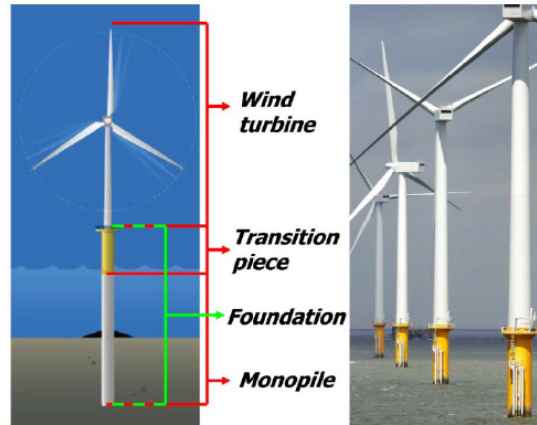


Figure 3.6: Components of monopile offshore wind structures [28]

To design such a foundation, two main design codes are followed by the industry : the Det Norsk Veritas (DNV) and American Petroleum Industry (API) guidelines. The main design technique used in these guidelines is p-y analysis, which was originally developed to design foundations for offshore oil and gas platforms. These structures have very different element sizes and loads to single pile foundations. The above guidelines are written for small diameter (about 0.6 m) and flexible piles as used in structures such as jacket structures. In contrast, monopiles are rigid, stiff and have large diameters. In addition, wind turbines are subject to cyclic horizontal loads, unlike oil and gas foundations which are typically axially loaded. These guidelines cannot be used as such which has led the research community to conduct tests to adapt these codes specifically for monopiles [14].

In addition, the three dominant limit states when designing a monopile foundation are : the ultimate limit state, the service limit state and the fatigue limit state (fig. 3.7)[46]. However, in the case of wind turbines, a frequency analysis problem also comes into play in the design. As wind turbines are very dynamic sensitive structures, the natural frequency of the tower must be different from the loading frequency in order to avoid locking.

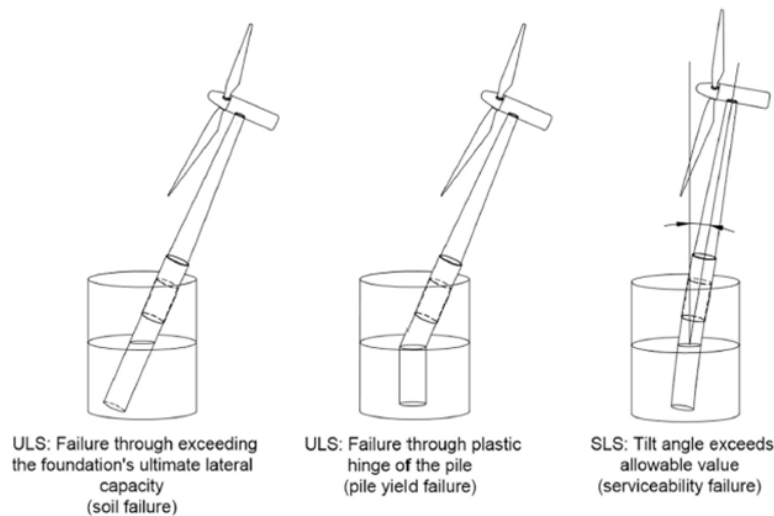


Figure 3.7: Limit states of offshore wind turbines [46]

A structure designed with own frequencies matching these excitation frequencies will be close to resonance or enter resonance and can cause high fatigue of the structure or failure. At nominal speed, the rotor should not excite any resonance mode of the structure.

For a wind turbine, the frequency ranges to avoid are the rotor and blade frequencies. Since most modern wind turbines have three blades, the blade frequency is three times the rotor frequency. These are called 3P and 1P respectively. Three main frequency ranges are therefore studied : the "soft-soft" range below the 1P frequency, the "soft-stiff" range between the 1P and 3P frequencies and the "stiff-stiff" range above the 3P frequency (fig. 3.8). Among these, the "soft-soft" range is generally not used in real projects because it leads to piles that are too thin and flexible, resulting in unacceptable deformations. This is also the frequency range in which environmental loads such as wind and waves act, leading to resonance risk. On the other hand, the stiff-stiff frequency range generally leads to piles that are too rigid and therefore uneconomical. For the reasons stated above, the vast majority of monopiles are designed in the soft-stiff region [46]. It is the optimal region for structure design at constant nominal speed.

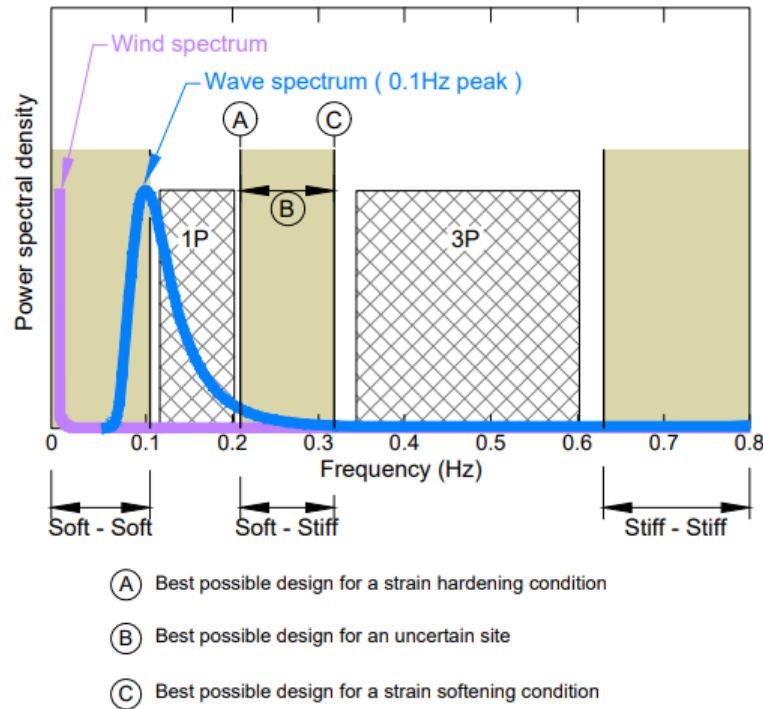


Figure 3.8: Functioning spectrum [30]

Due to the relatively small frequency range in this region, the stiffness of the foundation must be determined accurately to ensure the true natural frequency of the entire structure. The foundation design is an iterative process as the loads on the foundation will vary with the foundation stiffness. In order to design the foundation, the properties of the soil and the monopile are accurately determined by preliminary laboratory studies. The process of soil stiffness degradation by cyclic loading is also studied.

3.5 Pile installation processes

To drive piles into the ground, two methods with different principles can be considered : vibro-driving and impact driving.

3.5.1 Impact pile driving

In order to drive the pile into the ground, it is loaded onto a lifting barge and transported by ship to its intended location. The pile is then erected on the seabed at the intended installation spot. A clamp is used to keep the pile vertical and in the right place. Holding and guiding the pile is an important step in driving. Finally, the hammer is installed at the top end of the rigid pile and drives it at a rate of about 40 blows per minute, striking an anvil so as not to deform the pile head. The greater the drop height, the greater the force. Different types of hammers exist with the main parameters being the weight of the ram and the maximum energy per blow. Generally, hydraulic hammers are used. The advantage of hydraulic power is that it can generate large forces without increasing the size of the hammer considerably. This hydraulic power also allows real-time control of the energy delivered by the hammer, the rate of impact and the speed of the hammer on impact. As the hammer falls, a wave is generated and propagates along the pile to its tip. The kinetic energy at the head of the pile is used to overcome the lateral friction along the pile and cause plastic deformation of the soil at its base. Driving causes noise pollution with a negative impact on the marine environment and high stresses in the pile reducing its lifespan.

The driving of a pile requires a pile driver and a guidance system.

Guiding system

This system is used to guide the pile driver to hit the pile. Two methods are possible : the pile driver or the pile guide. With the pile driver, the ram slides on a rail system on a hydraulic carrier. Positioning and control of the verticality is easily ensured. However, this system is not suitable for large diameter piles. For the pile guide, the ram is suspended from a crane.



Figure 3.9: Pile driver and pile guide [13]

Hammer

Different types of hammers are possible for driving : free-fall hammers, diesel hammers, double-acting hydraulic hammers and double-acting compressed air hammers with rapid strikes. In order to choose the right hammer, a driveability analysis must be carried out. Different pile-driving models exist for this analysis and new models are still under development [70]. Soil properties also have an important impact on the driveability analysis. While sandy and clayey conditions are well determined [14][56][21], new soil zones are still being investigated in the future offshore monopile development areas, especially the brittle rocks [44]. A comprehensive geotechnical survey and laboratory tests are used to accurately characterise the soil [17].



Figure 3.10: IHC hammer and helmet for impact driving [39]

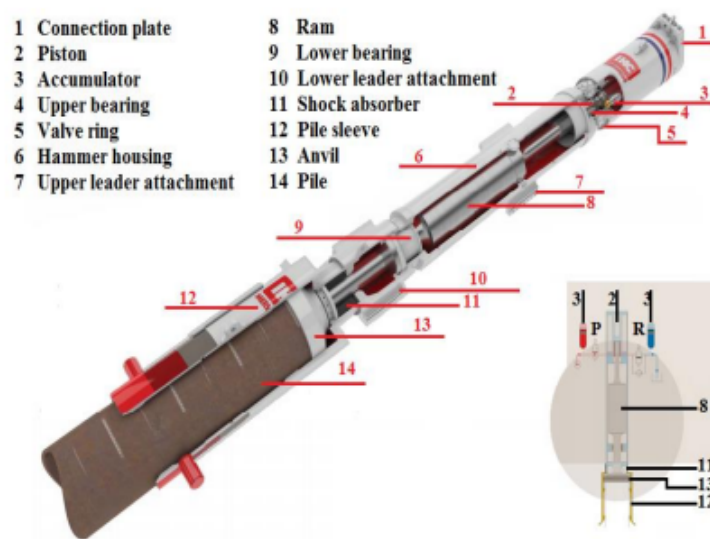


Figure 3.11: Hydraulic hammer [39]

3.5.2 Vibratory driving

To overcome the negative environmental impacts, economic costs and the slowness of pile driving, a solution has been studied in recent years : vibratory driving. The principle of this technique is to use the fact that the soil resistance decreases under cyclic loading conditions. This degradation of soil strength facilitates pile driving, requiring a lower surcharge than that required for non-degraded soils.

In soft soils, vibratory driving is preferred to impact driving due to its quick installation and low environmental impact. In contrast, in difficult soil conditions, impact pile driving is widely used because it can generate high energy but induces noise and strong vibrations in the soil and therefore cannot be used near buildings. Vibratory driving is therefore mainly used in granular or soft, saturated and cohesionless soils and not so much in cohesive soils such as clay soils. This is because soft soils liquefy easily when subjected to cyclic loading, whereas moderately stiff saturated clay soils or dense sands have a less pronounced cyclic strength degradation.

Vibratory driving uses longitudinal forced vibrations to push the pile into the ground. The vibrations are generated by a vibrator with an even number of eccentric masses arranged symmetrically to the vertical axis of the pile and rotating in opposite directions. If the masses rotate at a constant speed and in opposite directions, the horizontal components of the centrifugal force cancel out and a vertical force of sinusoidal intensity is produced (fig. 3.12) whose amplitude is a function of the sum of the eccentric moments of each mass me and the angular frequency ω . A stationary wave regime is then obtained [37].

$$F_v(t) = me \cdot \omega_v^2 \cdot \sin(\omega_v t) = me \cdot (2\pi f)^2 \cdot \sin(2\pi f t) \quad (3.1)$$

Under the action of its own weight and downward vibrations, the soil resistance decreases and the pile sinks into the ground. Indeed, the cyclic movement induces a degradation of the soil resistance and, in a saturated environment, the increase in pore pressure leads to liquefaction and a total loss of resistance. The speed of penetration can reach several metres per minute. Penetration continues until the refusal depth is reached, i.e. when the forces are no longer sufficient to overcome the ground resistance. At this point, penetration is no longer possible or becomes too slow because the vibrator is no longer able to provide the energy necessary to overcome the resistance to penetration of the soil.

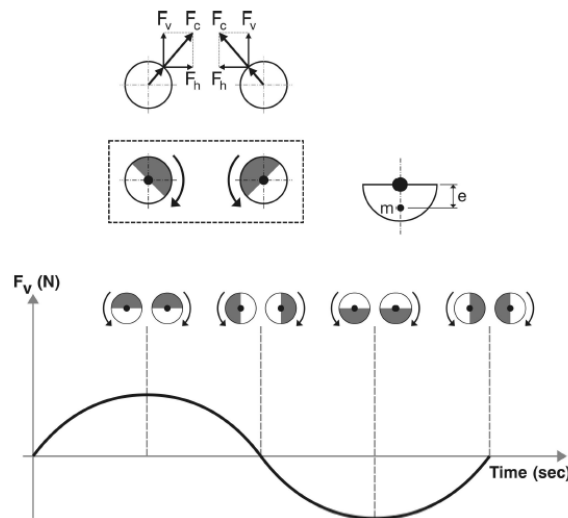


Figure 3.12: Working principle of the vibrator [37]

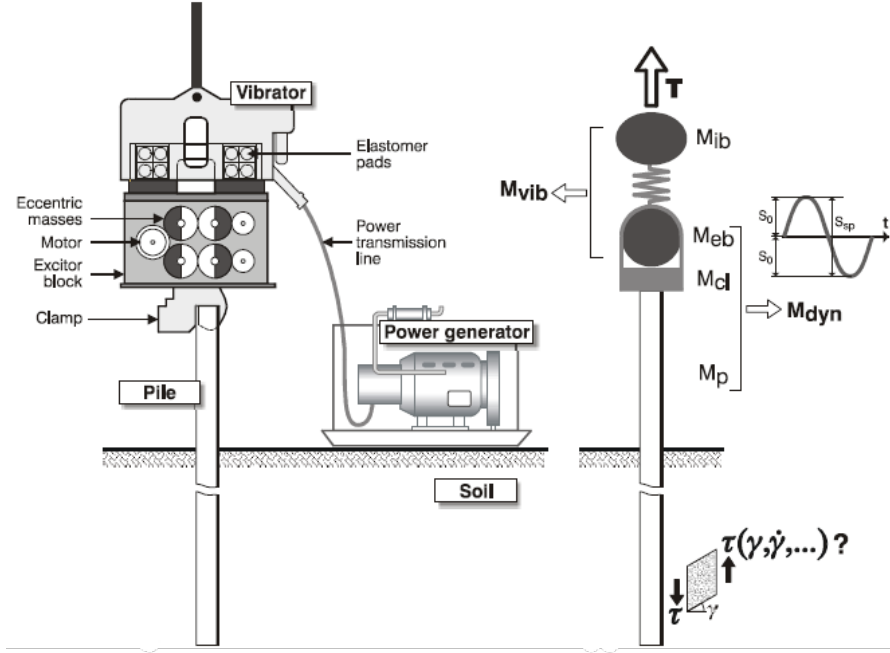


Figure 3.13: Different parts of a hydraulic vibrator [37]

Vibratory driving of a pile into the ground requires the use of a guidance system and a vibrator. The guidance system for vibratory driving is identical to that for impact driving.

Vibrator

The vibrator is composed of an exciter block containing an even number of eccentric masses arranged symmetrically around the vibrator axis and generating purely vertical vibrations as long as the centre of gravity of the rotating masses coincides at all times with the neutral axis. The vibrator also has a suspension support consisting of an isolating mechanism that filters the vibrations generated in the vibrating box by elastomers. The weight of this system corresponds to the static part of the applied load. A helmet sized according to the vibrator power and type of profile is applied on the pile top. During the driving operation, the penetration of the pile can be slowed down by controlling the holding force T_c applied by the carrier.

The whole system includes a static and cyclic term. The static term is represented by the vibration-isolated bias mass corresponding to the weight of the suspension device. The cyclic part consists of the pile, the exciter block and the clamping device.

The total load applied during vibratory driving on the ground is expressed in eq. (3.2) but can sometimes be reduced by the restraining force T_c .

$$F_s(t) = (M_p + M_c + M_v + M_s) \cdot g + m_e \cdot \omega_v^2 \cdot \sin(\omega_v t) - T_c \quad (3.2)$$

The vibrator is supplied by a hydraulic generator fed by a variable flow unit. Hydraulic power in vibratory hammering provides a good power-to-weight ratio, great flexibility of use and high performance compared to electric vibrators. The vibrators are classified according to the vibration frequency and the eccentric moment. Depending on the hydraulic vibrator, the eccentric masses rotate at speeds between 1200 and 3000 rpm, i.e. frequencies of 20 to 50 Hz (table 3.1).

The adverse effects on the environment during the vibratory process must be kept to a minimum. To achieve this, the eccentricity of the vibrator during the vibratory process is adjusted by modifying the phase shift of the feeder pairs to reduce it.

On the basis of frequency and eccentric moment, the different vibrators are generally separated into three classes as described in table 3.1.

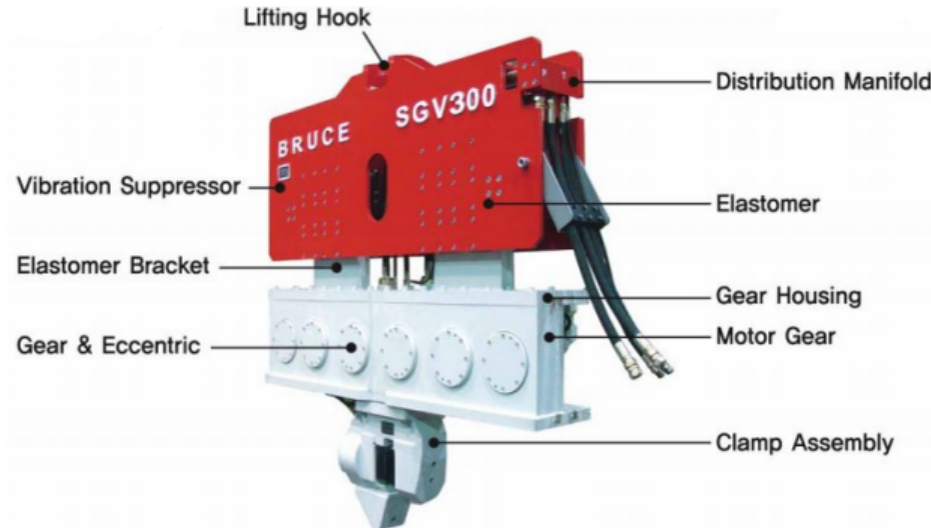


Figure 3.14: Detailed scheme of vibrator [13]

Type	Frequency range [Hz]	Eccentric moment [kg.m]	Maximum centrifugal force [kN]	Free hanging double amplitude [mm]
Standard frequency	21-30	up to 230	up to 4600	up to 30
High frequency	33-42	6 to 45	400 to 2700	13 to 22
Variable eccentricity	33-38	10 to 54	600 to 3300	14 to 17

Table 3.1: Vibrator types [37]

The free hanging double amplitude S_0 corresponds to the double amplitude of movement for a free hanging vibrator whose vibrating mass is formed of the exciter block and the clamping device.

$$S_0 = \frac{2 \cdot m_e}{M_c + M_v} \quad (3.3)$$

This amplitude is reduced when the pile is connected to the vibrator due to the increase of the vibrating mass. During the vibratory driving operation, the soil resistance further reduces the value of this amplitude [71].

For the choice of vibrator, the main characteristics are the eccentric moment m_e , the frequency f and the static surcharge M_s . Rodger and Littlejohn (1980) [62] proposed a table (fig. 3.15) helping the selection of the vibrator characteristics based on the soil conditions and the type of pile.

Cohesive soils	Dense cohesionless soils		Dense cohesionless soils	
All cases	Low point resistance	High point resistance	Heavy piles	Light piles
High acceleration. Low displacement Amplitude	High acceleration.	Low frequency Large displacement amplitude		High acceleration
Predominant side Resistance	Predominant side resistance	Predominant end resistance		Predominant side resistance
Requires high acc. for either shearing or thixotropic transformation	Requires high acceleration for fluidization	Requires high displacement amplitude and low frequency for maximum impact to permit elastoplastic penetration		Requires high acceleration for fluidization
Recommended parameters				
$f_d > 40\text{Hz}$ $a_p = 6\text{-}20\text{g}$ $s_p = 1\text{-}10\text{mm}$	$f_d = 10\text{-}40\text{Hz}$ $a_p = 5\text{-}15\text{g}$ $s_p = 1\text{-}10\text{mm}$	$f_d = 4\text{-}16\text{Hz}$ $a_p = 3\text{-}14\text{g}$ $s_p = 9\text{-}20\text{mm}$		$f_d = 10\text{-}40\text{Hz}$ $a_p = 5\text{-}15\text{g}$ $s_p = 1\text{-}10\text{mm}$

Figure 3.15: Choice of vibratory parameters [62]

The technique of installing monopiles by vibro-driving is quite similar to that of pile driving, except that instead of driving the pile with a hammer, an equal amount of phased motors are attached to the top of the pile and induce vertical waves in the pile. In non-cohesive saturated soils, this can lead to liquefaction of the soil and a much faster and quieter installation process than driving [73]. This technique is already used in onshore geotechnical engineering to install, for example, sheet piles [71].

On the other hand, this technology is not widely used for the installation of offshore monopiles, as the two main monopile design standards, DNV (Det Norske Veritas) and API (American Petroleum Institute), are based on the data provided by the piling log. On the few projects where vibratory driving is used, the pile is impact-driven for the last 10 m after vibratory driving in order to get the pile certified.

The lack of data on the long-term properties of vibrated piles has prompted the scientific community to study the difference between vibrated and driven piles (fig. 3.16). Different capacities have been investigated, such as end load capacity [29], monotonic lateral capacity [31] and single and two-way cyclic lateral capacity [64]. Finally, tests were carried out to see the impact on soil density changes around the pile after installation [61]. These tests were also carried out in different soil types (sand, clay) with different properties (saturated, dry or dense, loose). Two techniques have mainly been used : numerical simulations and model tests, but some tests were carried out at full scale [31].

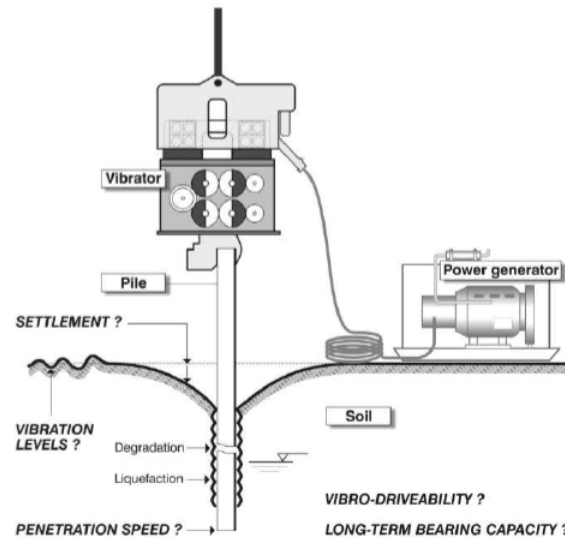


Figure 3.16: Vibrodriving: engineering issues [37]

3.6 Comparison of the installation techniques

Before investigating the behaviour of impact and vibration driven monopiles in a more detailed way through small-scale laboratory experiments, it is important to consider the different advantages and drawbacks of both techniques.

First, one of the main reasons for installing through impact driving is the common belief that the ultimate load capacity of impact driven monopiles is higher than that of vibrated monopiles. The difference in ultimate load capacity of vibratory piles is generally estimated to be about 60-85% of that of impact piles. Research is underway to determine whether the use of a lower frequency at the end of driving could help bring the ultimate load capacity closer to that of impact piles. At present, the lack of field testing and the limited data base leads to unpredictable behaviour of vibratory driving in terms of penetration speed and rejection depth.

Another advantage of driven piles is the long history and experience of this type of installation in the oil and gas industry. This extensive know-how leads to very precise driving studies and a large number of design standards and recommendations. On the other hand, vibratory piling in the offshore industry has still a low level of industrial maturity and many unknowns remain. This leads to questions about the long-term behaviour of the pile and the capacity to reach the originally planned depth.

On the other hand, one of the disadvantages of pile driving is the noise emitted during installation. The underwater noise due to the hammer blows exceed the hearing impairment threshold of marine mammals living in the surrounding area. In order to avoid disturbing marine life and harming animals, most European countries regulate the maximum allowable noise emissions in the direct environment of the pile driver. To comply with these restrictions, noise attenuation systems are put in place, such as bubble curtains and noise reduction boxes. This leads to more complicated systems and a more expensive and slower process. This is where vibrated piles have a great advantage over driven piles. Indeed, the vibratory system emits lower noise levels as shown in fig. 3.17[51] :

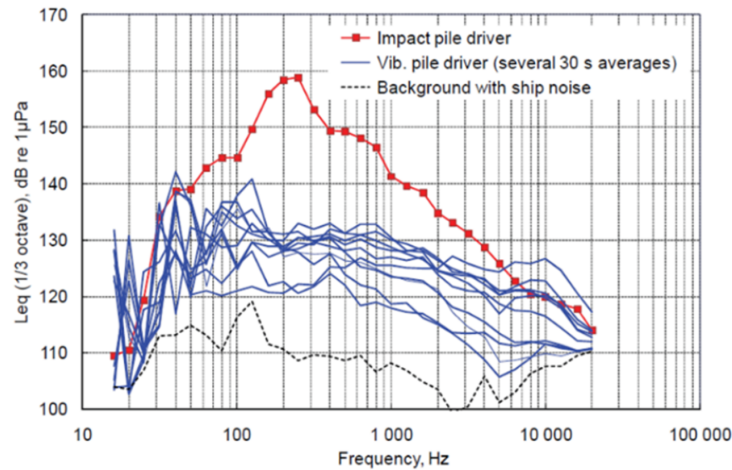


Figure 3.17: Noise levels for impact and vibratory driving [29]

Vibratory driving is a more suitable technique for work in sensitive areas such as the seabed. Noise reductions of up to 20 dB can be achieved by using vibratory driving rather than impact as the vibrations are lower and the process takes less time [73].

Another advantage of vibratory driving is the driving speed. Indeed, by using this technique, the installation time of a vibrated pile is reduced by up to 30% compared to traditional methods [18]. As barge time is one of the largest costs in offshore pile installation, this method leads to significant economic gains and improved efficiency. This technique also saves energy due to its fast installation time.

In addition, vibratory driving is an efficient technique for sandy soils, saturated or not, and alluvial soils, but is unfavourable for compact, dry or soils with high penetration resistance. In this case, it is essential to control driving to avoid exposing the pile to excessive stresses that could deform it.

In terms of the economic impact of the installation process, the income per year is quite significant. As the stresses sustained by the pile during installation are lowered, the vibratory driving extends its operational life and the income on the longer term. One of the main parameters in the design is the fatigue life of the monopile as it is subjected to cyclic loads throughout its life. Due to the high stress levels applied to the pile during the driving process, the fatigue life of the pile will be negatively affected from the beginning. Vibratory driving, on the other hand, allows for lower stress levels and can therefore extend the life of the offshore wind turbine.

A final advantage of vibratory driving over impact driving is that the vibrator is fixed to the top of the pile. This means that it is not necessary to use external tools to set the pile up as in the case of impact driving where a crane is used to pre-install the pile. This fixing is also advantageous as it can be used to extract the pile by applying traction while vibrating it. This is of great interest when wind turbines are taken out of service. Finally, this clamped connection allows better control of the pile during driving. As the pile cannot go deeper than the depth allowed by the crane operator, there can be no slippage of the pile. It also allows for control and correction of the inclination during driving [29][40].

As a result of this comparison, one of the biggest drawbacks of vibratory monopiles is the lack of experience and understanding of their impact on the long-term capacity of the pile. Therefore, studies of their impacts are being conducted so that engineers can make informed choices when selecting the pile installation method.

3.7 Major geotechnical methods to characterise the soil

In order to correctly design the piles to be driven by driving or vibratory driving, in situ tests and soil identification tests must be carried out beforehand to characterise the soil and its bearing capacity. Below is a list of tests carried out during a geotechnical survey campaign in order to define the mechanical properties of the soil and to design the pile parameters according to these properties.

3.7.1 Grain size distribution test

The particle size distribution test is used to determine the geometric characteristics of the sand particles and to establish their sieve curve. This test provides information on the mechanical, thermal and hydraulic behaviour of the soil and provides information on unsaturated soils.

The sieving process consists of the separation of soil particles into particle size classes by means of successive test sieves consisting of a mesh of wires allowing the passage of particles whose dimensions are smaller than the size of the openings. At the end of the process, the aim is to measure the proportion of the initial soil present at each opening size using the weights.

When the grain size is less than 300 μm , a sedimentometry test must be performed. This method is based on the fact that, in a liquid medium at rest, the settling speed of fine and very fine grains depends on their size.

For a grain of a given size, the settling velocity first increases rapidly under the effect of gravity and then stabilises at a constant value when the resistance of the fluid to the fall of the grain is equal to its weight. From Stokes' law, it is possible to determine the size of the grain :

$$d_i = 0.005531 \cdot \sqrt{\frac{\eta \cdot H_r}{(\rho_s - \rho_w) \cdot t}} \quad (3.4)$$

This test is described in more detail in section 4.1 and this section presents the results obtained on North Sea sand chosen for the experiments carried out throughout this thesis.

3.7.2 Maximum and minimum voids ratio

The range of void ratio provides a general basis for the comparative evaluation of material properties for cohesionless soils. The engineering properties of granular soil, such as density, strength and coefficient of permeability, are directly affected by the particle size distribution and grain shape, as well as by the void ratio.

The stress-strain behaviour of sand depends on the physical nature and condition of the sand. The nature of the sand is generally described by material properties such as particle size distribution, fines content and grain shape. The state of the sand, on the other hand, indicates

the physical conditions under which the sand exists and is defined by the relative density D_r , the effective stress state and the fabric. Relative density D_r is a state parameter indicating the density of a given sand sample or deposit relative to its range of possible densities obtained using maximum and minimum void tests.

The minimum voids index is obtained by compacting a sample of the soil using a standard procedure. The maximum voids index is obtained by pouring the soil through a funnel into a standard mould and limiting the height of fall of the grains as much as possible.

Through this test, the relative density of a soil of void ratio e is defined as :

$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}} \quad (3.5)$$

The table 3.2 shows some density values and the characteristics of the soil according to the density value.

Relative density I_D [%]	Description
0-15	Very loose
15-35	Loose
35-65	Medium
65-85	Dense
85-100	Very dense

Table 3.2: Relative density values and their description

This test is described in more detail in section 4.2 and this section presents the results obtained on North Sea sand chosen for the experiments carried out throughout this thesis.

3.7.3 Proctor test

The Proctor test specifies for a given soil and standardised compaction the influence of the water content. The soil under investigation is compacted at different water contents w by means of a Proctor tamper. For each water content, the dry density of the compacted material is measured. These measurements are then plotted on a graph showing the variation of γ_d as a function of w .

The graph represents the bell-shaped Proctor curve which contains a high point meaning that for a given compaction energy, there is an optimum water content that results in a maximum density. Beyond this optimal amount of water, the soil starts to liquefy. In the case of offshore wind turbines, and especially during vibratory driving, this test is useful because the cyclic loading during vibratory driving deforms the soil and liquefies it. This process can be accelerated if the optimal water content is known. In the case of sandy soils, the Proctor curve is flattened which means that compaction is practically impossible and does not depend on the water content.

This test is described in more detail in section 4.3 and this section presents the results obtained on North Sea sand chosen for the experiments conducted throughout this thesis.

3.7.4 Measurement of permeability at constant load

This test is performed on a reworked sample, compacted in a CBR mould. Water flows through the sample under a constant hydraulic load. By applying Darcy's law, the permeability of the soil can be deduced. The permeability coefficient of a saturated soil is a soil characteristic that depends mainly on its grain size, nature, structure, void index and temperature.

The finer or more compact the soil, the smaller the pores, the greater the friction and pressure drop and the smaller the permeability coefficient.

3.7.5 Oedometer and compressibility tests

The oedometer and compressibility tests are used to study the consolidation, i.e. the change in volume due to the expulsion of water, or the settlement of intact or reworked soil samples subjected to permanent vertical loads. The tests are carried out by measuring the deformations at constant load of a cylindrical specimen with zero lateral deformation and drained on the inner and upper side. With this test, it is possible to measure the permeability of the specimen during the consolidation phase and the parameters that characterise the rate of consolidation as a function of the initial state of the stresses and overloads adopted.

This test makes it possible to establish two types of curves and to determine several parameters for a given sample :

- The consolidation curves which give the settlement of the sample as a function of time under application of a constant stress.
- The compressibility curve which shows the total settlement as a function of the logarithm of the applied stress.
- The effective pre-consolidation stress
- The compression index C_c
- The swelling index C_s
- The swelling stress

3.7.6 Triaxial test

The triaxial test consists of applying a constant hydrostatic stress and an increasing deviatoric stress to a cylindrical specimen. This test makes it possible to determine the cohesion and the angle of internal friction in the short and long term by drawing Mohr's circle using tests carried out at different pressures.

Three types of triaxial tests exist :

- The unconsolidated undrained (UU) test to determine short term characteristics. This test consists of applying the load rapidly, at a constant strain rate.
- The consolidated undrained test with pore pressure measurement ($C_u + u$). The shear characteristics are measured under undrained conditions and the specimen is assumed to be completely saturated.

- The consolidated drained (CD) test perfectly reproduces the geostatic conditions of the soil in situ. The sample undergoes consolidation and is compressively sheared slowly enough to allow the pore pressures created by the shear to dissipate. The axial strain rate is kept constant.

3.7.7 Cone Penetration Test (CPT) and Standard Penetrometer Test (SPT)

These tests are applicable to all fine and coarse soils with an average element size not exceeding 20 mm. The penetration length is limited to the reaction force of the equipment. These tests are used to design pile parameters and to classify soils.

The static penetration test consists of driving into the ground, at a constant speed, a string of rods terminated by cone and measuring continuously the penetration resistance q_c , the total resistance and/or the friction of a sleeve as a function of depth. The injection of the drilling mud eliminates the lateral friction of the rods in the soil.

In contrast, the dynamic penetration test consists of driving a string of rods with a protruding tip at the bottom into the ground by continuous beating, while injecting drilling mud between the borehole wall and the rods and noting the number of blows necessary to make the tip penetrate the ground by 10 cm.

3.8 Future of monopile installation

As mentioned earlier, monopiles are by far the most commonly used foundation for offshore wind turbines in the North Sea. Their simplicity and lower manufacturing costs, as well as ease of transport by ship and installation, have allowed companies to continue in this direction. However, as larger and heavier turbines are developed and wind farms are installed in deeper waters, the technical and economic limitations of single pile foundations are being tested against other types of foundations. It is therefore becoming essential to extend the dimensions of current monopiles to allow the use of larger wind turbines in deeper waters and in more severe environmental situations.

To achieve this, future offshore wind turbines will have the following characteristics and will have to meet new targets :

- Turbines up to 15 MW with rotor diameters up to 230 m.
- Resistance to extreme wind and wave loads caused by hurricanes or typhoons.
- Water depth up to 65 m or more.

These new requirements lead to monopile designs with bottom diameters between 8 and 11 m, lengths up to 120 m and wall thicknesses up to 150 mm. The final weight of such monopiles can reach up to 2400 tonnes. Since the weight of such monopiles is significant, research is being carried out to reduce the weight of such piles by using stronger steels or by increasing the slenderness ratio $\emptyset/t$ to between 100 and 190. The results of a study show that monopiles with slenderness ratios up to 160 are realistic and applicable to deep water and large turbines [58].

In addition, future monopiles for wind turbines will no longer be equipped with transition pieces but will have a direct connection to the turbine tower. In the past, this transition piece was used to achieve verticality tolerance and to connect all kinds of accessories. However, the removal of the transition piece eliminates the need for a bolted connection and subsequently reduces maintenance costs, particularly for large diameter monopiles that may have more than 150 bolts.

Furthermore, thanks to the SIMOX consortium, alternative installation technologies to impact driving are being researched, developed and tested for the installation of future XXL monopiles. SIMOX is researching and testing the following installation technologies with different levels of technological maturity (TRL) and knowledge gaps [23] :

- Vibro is a vibratory drive with purely vertical excitation.
- Concept Gentle Driving of Piles (GDP) combines a vertical vibratory excitation with a high frequency torsional jolt.
- JET facilitates vibratory techniques by fluidising the internal soil column of the pile.
- Blue Piling (BP) is a modification of conventional impact piling that reduces noise. This technology uses the impact of a large mass of water over a long period of time to create a pushing force on the pile.
- DECOM are pile driving technologies that uses vibration or pressure.

Finally, in order to meet the foundation requirements of the new generation of 15 to 20 MW wind turbines, monopile transport companies have also had to adapt and reinvent themselves to increase their capacity for wind turbine installation vessels.

This is the case of Offshoretronic, which uses a double-crane lifting and transport technique for the installation of the ultra-long XXL monopiles, which can carry up to 12 monopiles. With the continued growth of wind turbines, monopiles are also becoming larger in diameter and longer than 100 m. This new generation of piles makes the current vessels obsolete and requires new lifting equipment. Due to the large dimensions of XXL monopiles, it is necessary to have several lifting points during the lifting process in order to avoid the risk of buckling and damage to the monopile [57].



Figure 3.18: New drillship for Ultra XXL monopiles installation [57]

Characterisation of the North Sea Sand

4.1 Grain size analysis of soil

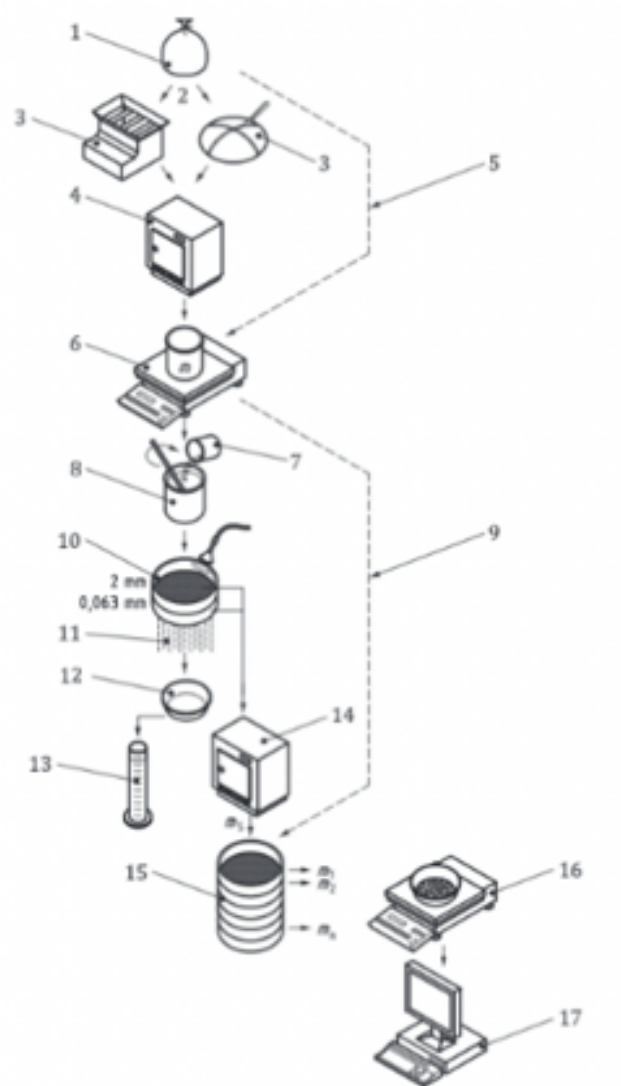
4.1.1 Description and definition of the process

In order to determine the geometric characteristics of the sand particles and to establish their particle size curve, a particle size analysis by sieving was carried out. The granulometric analysis makes it possible to determine and observe the different diameters of the grains that constitute a granulate. The particle size distribution test provides information on the mechanical, thermal and hydraulic behavior of the soil. This test is regulated by European standards [1][4].

The sieving process consists of the separation of soil particles into particle size classes by means of successive test sieves consisting of a mesh of wires or perforated metal sheet allowing the passage of particles whose dimensions are smaller than the size of the openings. At the end of the process, the aim is to measure the weight proportion of the initial soil present at each size of opening.

As our soil contains very few fines (particles with a diameter of less than 0.063 mm), we only use the sieve test for aggregates (particles with a diameter of more than 0.063 mm). The soil contains less than 10% of fine particles, so a sedimentation test is not necessary. The test is carried out with a dry sand mass, i.e. the sand is heated to a temperature of 105°C until its mass is constant. It is then cooled and weighed three times at 1 hour intervals to ensure that the mass is constant. This mass is called M_1 .

4.1.2 General sieve test procedure



Legend

- | | |
|-------------------------------------|--|
| 1. Sample | 10. Separation and washing |
| 2. Dry sample method | 11. Washing |
| 3. Division or quartering | 12. Collection of material < 0.063 mm (optional) |
| 4. Drying | 13. Sedimentation test (optional) |
| 5. Wet sample method | 14. Drying of rejects |
| 6. Weighing | 15. Sieving |
| 7. Dispersing agent | 16. Weighing of material on each sieve |
| 8. Wetting and stirring | 17. Calculation of test results |
| 9. Dry sieving (alternative method) | |

Figure 4.1: General sieve test [54]

4.1.3 Operational procedure ¹

1. Washing : the objective is to clean the soil, i.e. to remove all the fine particles contained in the sample. Vibration is not sufficient to bring down the fines. Indeed, these grains are so fine that they stick to grains of a larger diameter and remain blocked in sieves that do not correspond to their diameter. To remove these fines :
 - a) Place a certain amount of sand in a container and add the necessary water to cover it. The soaking time is prolonged until the sand clumps are easily separated.
 - b) Mix this quantity of sand energetically until complete separation and suspension of the fine particles is achieved.
 - c) Soak a 0.063 mm sieve with water and place a protective sieve (1 mm or 2 mm opening) on top. Fill the upper sieve with sand and continue washing until the water passing through the 0.063 mm sieve is clear.
 - d) Dry the reject at 105°C on the 0.063 mm sieve until the mass is constant. Then allow to cool and weigh. The mass obtained is M_2 .
2. Sieving :
 - a) Construct the sieve column consisting of a number of nested sieves with sieve openings decreasing in width from top to bottom and held together by a cover and clamps. This column ends at the bottom with a 0.063 mm sieve and then a bottom. The 0.063 mm sieve is used to collect the fine particles not removed during the washing process.
 - b) Place the washed and dried sand contents and mechanically vibrate the sieve column with a mechanical vibrator for about 10 minutes. The vibration moves the grains down through the sieves until they are blocked by the sieve of the mesh size corresponding to the grain diameter. To avoid overloading the sieve, it is important not to exceed the fraction retained at the end of the sieving operation on each sieve, which is expressed as follows :

$$\frac{A \cdot \sqrt{d}}{200} \quad (4.1)$$
 - c) The sieving operation is complete when a further sieving does not change the reject mass by more than 1% by weight.
3. Weighing : determination of the remaining mass on each sieve. To do this :
 - a) Weigh the reject on the sieve with the largest mesh size and record its mass R_1 .
 - b) Pour the contents of the sieve into a container and weigh the empty sieve.
 - c) Carry out the same procedure for all the sieves in the column to obtain the mass of the different fractions of material retained and record these masses $R_2, R_3, \dots, R_n$.
 - d) Weigh the material at the bottom, if any, and record its mass P.

¹Niclaes, Jens Recueil de procédure d'essais, Ecole Polytechnique de Louvain, 2021.

4.1.4 Calculation and expression of results

1. Record the different masses before and after washing and drying on a sheet of paper.
2. Calculate the mass of the rejects on each sieve as a percentage of the original dry mass M_1 .
3. Calculate the cumulative percentage of the different sieves of the original dry mass passing through all the sieves up to but not including the 0.063 mm sieve.
4. Calculate the percentage of fines f passing through the 0.063 mm sieve according to the following equation :

$$f = \frac{(M_1 - M_2) + P_s}{M_1} \cdot 100[\%] \quad (4.2)$$

For dry sieving, the percentage of fines is expressed as follows :

$$f = \frac{P_s}{M_1} \cdot 100[\%] \quad (4.3)$$

The results are valid if the sum of the masses R_i and P differs from M_2 by less than 1%.

4.1.5 Results

- Mass of dried sand $M_1 = 1713.8g$
- Mass of fine particles $P_s = 0.32g$

Sieve opening width [mm]	Sand mass [g]	Percent passing [%]
4	4.58	99.98
1	68.38	99.71
0.63	60.32	95.72
0.4	377.56	92.2
0.315	429.02	70.17
0.2	685.1	45.14
0.125	83.26	5.17
0.08	4.52	0.31
0.063	0.42	0.04
0	0.32	0.019

Table 4.1: Experimental results of percentage passing as a function of mesh size

4 – Characterisation of the North Sea Sand

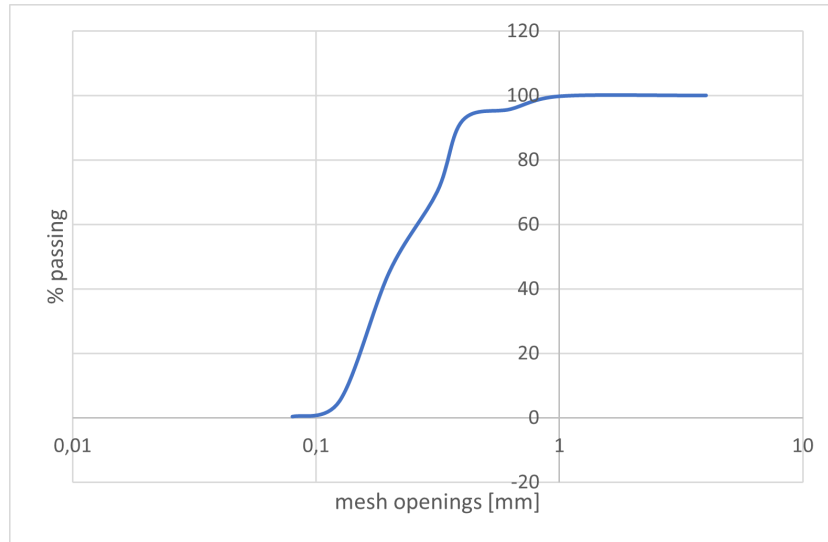


Figure 4.2: Experimental granulometric curve

A comparison can be made with the grain size given in the VEMAT data sheet for marine sand.

Sieve opening width [mm]	Average flow [%]	Max flow [%]	Min flow [%]
4	100	100	100
2.8	100	98	100
2	99	94	100
1	97	88	99
0.5	92	82	99
0.25	18	3	33
0.125	0	0	10
0.063	0.2	0	0

Table 4.2: VEMAT results of percentage passing as a function of mesh size

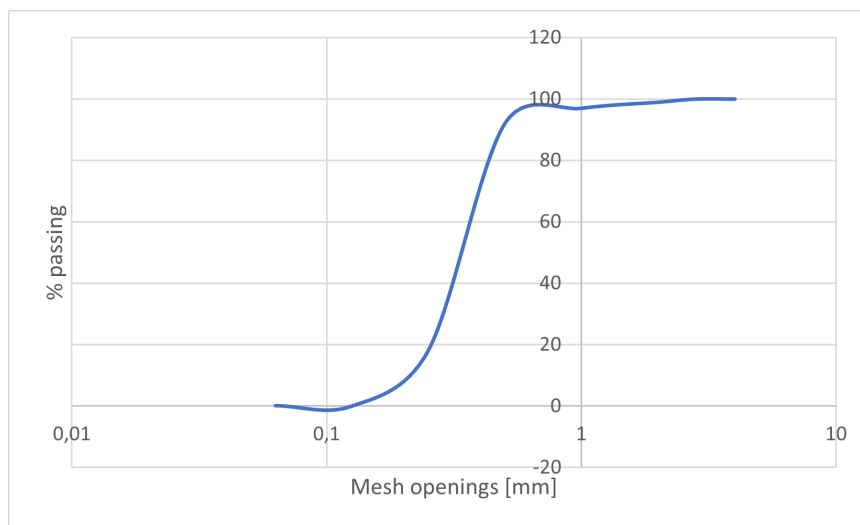


Figure 4.3: VEMAT granulometric curve

4.1.6 Discussion

The particle size curve obtained experimentally is quite similar to that provided by the supplier's data sheet. The shape of the curve and the particle size values collected show that it is indeed a sandy soil. The majority of grain size is between 0.25 and 4 mm.

The sieve test was carried out in order to design the opening of the three plates of the sand raining system.

4.2 Maximal and minimal void ratio test

4.2.1 Description and definition of the process

For a non-coherent soil, its strength is highly dependent on its relative density. For the further experiments in this thesis, it is interesting to have an overview of the range of densities achievable with the chosen sand. To achieve this, the procedure given in standard NF P94-059 [5] is followed.

The test consists of determining the minimum and maximum densities of non-coherent soils. To do this, the soil is successively placed in a loose state (e_{min}) and then in a compact state (e_{max}) in a standardised mould according to a specific procedure.

The norm NF P 94-059 is only applicable to soils with some specific characteristics :

- The largest elements must have a diameter lower or equal to 50 mm.
- The percentage of soil passing through a sieve with rectangular openings whose sides are 31.5 mm long must be higher or equal to 70%.
- The percentage of soil passing through a sieve with rectangular openings whose sides are 80 μm long must be lower or equal to 12%.

In our case, all the elements constituting the chosen sand have a grain size of less than 4 mm (see Section 4.1.6)

4.2.2 Equipment required

- A scale
- A cylindrical test mould + its extension
- A levelling ruler
- A funnel
- An overload for compaction

4.2.3 Operational procedure

1. Dry a sand sample in an oven at 105°C until its mass is constant, i.e. when all the water has evaporated and the sand is completely dry.
2. Choose the right size of mould according to the grain size of the soil and measure its volume. In our test, a Proctor test mould is used with the following dimensions :

Dimension	Symbol	Value	Units
Height	H_m	117	mm
Diameter	D_m	100	mm
Volume	V_m	918.89	cm^3
Mass	m_m	3957.9	g

Table 4.3: Dimension of the mould

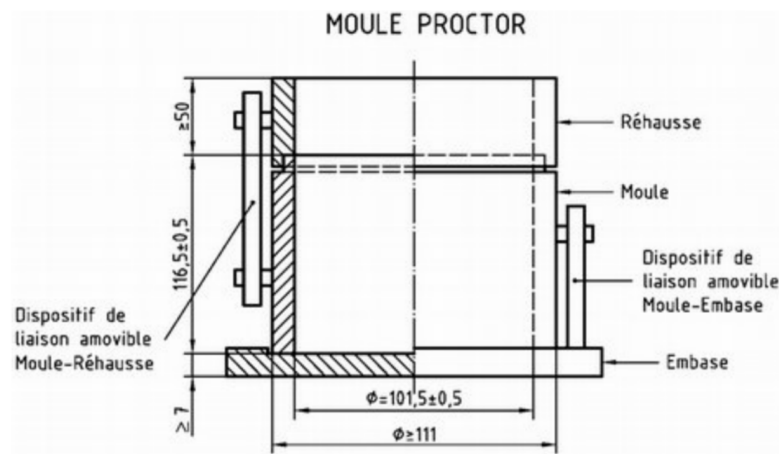


Figure 4.4: Proctor mould [5]

3. Weigh the empty mould.
4. Place the sand layer by layer in the mould using a funnel in order to have a zero height of fall. Raise the funnel at a constant speed, keeping the drop height to a minimum, and use the levelling ruler to remove the excess sand.
5. Weigh the mould and its contents to determine the mass of the material and its minimum density.
6. To determine the maximum density, add the extension to the mould and fill the mould in uniform layers. Between each layer, compact the sand with a surcharge of the same diameter as the mould in a fixed number of shots. Repeat the operation until there is a surplus of sand and remove the extension and any sand protruding from the top of the mould.
7. Weigh the mould and its contents to determine the mass of the material and its maximum density.
8. Repeat the above operations three times to obtain an average density.

4.2.4 Calculation and expression of results

The density index is calculated as follows :

$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}} = \frac{\frac{1}{\rho_{dmin}} - \frac{1}{\rho_d}}{\frac{1}{\rho_{dmin}} - \frac{1}{\rho_{dmax}}} \quad (4.4)$$

The maximum material void index can be obtained as follows :

$$e_{max} = \frac{\rho_s}{\rho_{dmin}} - 1 \quad (4.5)$$

and the minimal material void index is equal to :

$$e_{min} = \frac{\rho_s}{\rho_{dmax}} - 1 \quad (4.6)$$

For each test sample, the dry density of the sample is calculated using the values of the mass and volume of the empty mould and the mass of the filled mould as follows :

$$\rho_d = \frac{m_d}{V_m} = \frac{m - m_d}{V_m} \quad (4.7)$$

The data recovered during the experiments can be found here under :

Name	Total weight [kg]	Soil weight [kg]	Soil density [kg/m^3]
$e_{max,1}$	5.39	1.44	1562.49
$e_{max,2}$	5.41	1.45	1574.9
$e_{max,3}$	5.41	1.45	1578.38
$e_{min,1}$	5.64	1.68	1826.5
$e_{min,2}$	5.63	1.67	1815.4
$e_{min,3}$	5.63	1.67	1827.47

Table 4.4: Experimental data for minimal and maximal density calculation

By convention, the value recovered as minimal and maximal density achievable with this sand is taken as the arithmetic mean of the results.

For the maximal density, if the difference between the measured values is superior to 2%, a new trial has to be made. In our case, the maximal difference between two measures is 11.1kg/m³ while 2% of the mean density is equal to 36.43 kg/m³. This condition is thus verified for all our trials.

The value retained for the maximal and minimal densities is finally the following :

Density	Value [kg/m^3]
$\gamma_{d,min}$	1571.92
$\gamma_{d,max}$	1821.46

Table 4.5: Value of minimal and maximal densities

These data obtained will be useful for the future calculation of the relative density with the rainfall system but also for the triaxial and cyclic tests.

4.2.5 Discussion

When compared to the value of the dry sand density given in material catalogues, which is equal to $1600 \frac{kg}{m^3}$, the density values obtained experimentally are very close.

4.3 Normal proctor Test

4.3.1 Description and definition of the process

The Proctor test is a geotechnical test that consists of wetting a material at various water contents and compacting it in a standard mould, using a standard tamper, according to a conventional process and energy. For each of the water content values considered, the dry density of the material is determined and the curve of the variations of this density as a function of the water content is drawn. This test is often used in public works when using embankments. The Proctor test is regulated by standard NF P 94-093 [6].

In addition, the experimental protocol is repeated several times on samples with different water contents in order to define several points on a curve $(\gamma_d; \omega)$. This curve has a bell shape and has a maximum whose abscissa is the optimal water content and ordinate the optimal dry density [16].

4.3.2 Equipment required :

- A compaction base
- A proctor mould with a volume of 944 cm^3 (fig. 4.6)
- A manual compaction tamper (fig. 4.5)
- A levelling ruler
- An oven at 105°C
- A measuring cylinder
- A balance with a capacity of 20 kg

4.3.3 Operational procedure

1. Take a large quantity of sand to perform all the tests. The quantity must be sufficient since it is not allowed to reuse the same material for the determination of several points on the Proctor curve. The curve requires at least 5 tests.
2. Sieve to check that the rejection at 20 mm is less than 25% otherwise the test cannot be carried out.
3. Weigh the Proctor mould without the riser;
4. Incorporate a certain amount of water and mix manually until the water is well distributed within the sand. It is advisable to have a difference of about 2% of water content between each point and a maximum of 4%.

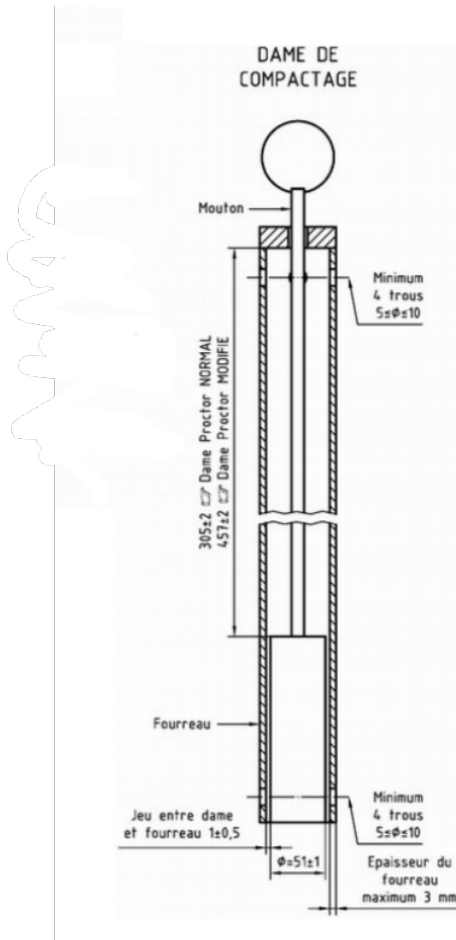


Figure 4.5: Manual compaction tamper [6]

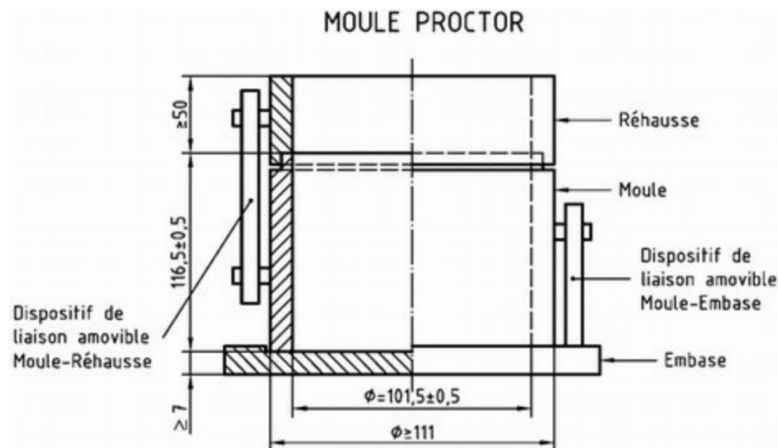


Figure 4.6: A proctor mould [6]

5. Before introducing the sand into the mould, tighten the mould, the base and the top.
6. Introduce the sand in three layers so that each layer is slightly more than a third of the height of the mould after compaction.

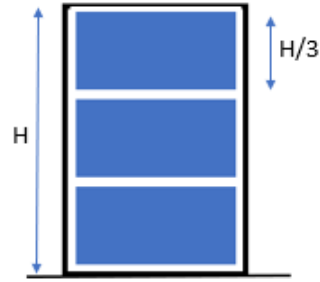


Figure 4.7: Layer configuration in relation to the height of the Proctor mould

7. Compact each layer with the compaction ram by applying 25 shots according to the following scheme :

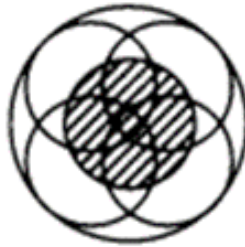


Figure 4.8: Configuration of shots to compact the layers

For each layer, three sequences of eight shots are distributed near the edge of the mould and the 25th is placed in the centre.

8. After the last layer has been compacted, remove the riser and the excess material with the trowel at the mould.
9. Weigh the mould and the material together.
10. Then remove the compacted sand and extract a representative sample to determine the water content by taking a small portion from either side of the edges of the mould at the 2nd layer.
11. Weigh the sand sample to determine its wet mass m_h .
12. Dry the sample in an oven at 105°C. When its mass is constant, weigh the dry sample to determine m_s .
13. Repeat the same procedure 5 times, adding 2% water to the sand each time.
14. Draw the curve representing the dry densities obtained as a function of the water content.

4.3.4 Calculation and expression of results

This Proctor test calculates the water content, the density of the dry material taking into account the actual volume of the mould used and the mass of dry material.

The net wet mass is calculated as follows :

$$m_{netwet} = m_{fullmould} - m_{empty mould} \quad (4.8)$$

The water content ω (%) is calculated as follows :

$$\omega = \frac{m_h - m_s}{m_s} [\%] \quad (4.9)$$

The wet γ_h and dry γ_s densities of the sample are obtained as follows :

$$\gamma_h = \frac{m_{netwet}}{V_{internalmould}} \quad (4.10)$$

$$\gamma_s = \frac{\gamma_h}{1 + \omega} \quad (4.11)$$

4.3.5 Results

Tests	$m_{net,wet}$ [g]	m_h [g]	m_g [g]	ω [%]	γ_h [kN/m ³]	γ_s [kN/m ³]
Test 1	1508.2	80.7	79.1	2.02	15.98	15.66
Test 2	1560.2	104.42	100.41	3.99	16.53	15.89
Test 3	1607.6	212.35	200.77	5.77	17.03	16.1
Test 4	1652.5	146.99	136.57	7.63	17.51	16.24
Test 5	1668.8	270.81	245.39	10.36	17.68	16.02

Table 4.6: Results of Normal Proctor Test

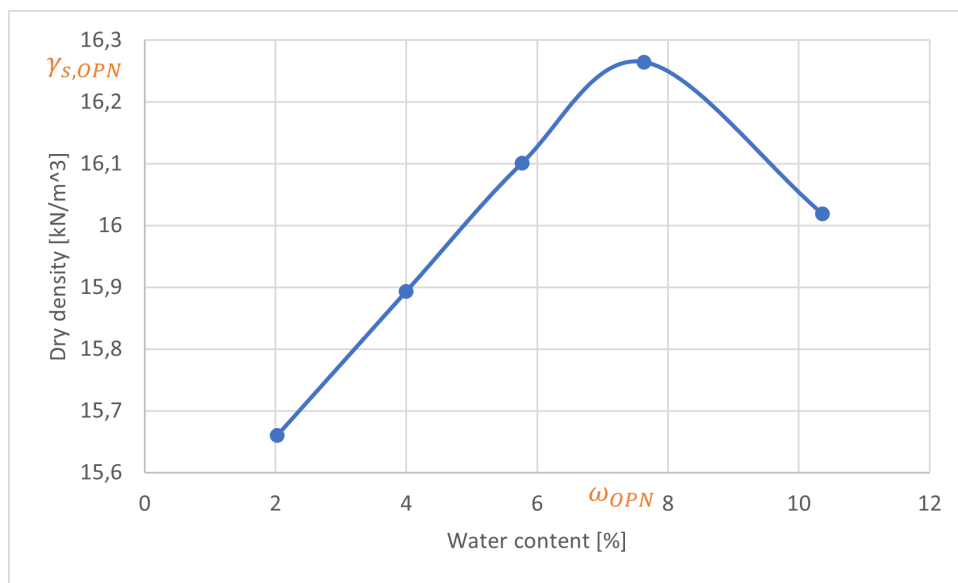


Figure 4.9: Normal Proctor curve

4.3.6 Discussion

The curve obtained has a bell shape. At the beginning of the curve the sand is dry and loose. It has a low compactness and a pile can be driven in easily. As we approach the maximum, the sand has a higher and higher water content and if we drive a pile into it, it becomes more and more complicated. When the sand has reached its optimum compactness, it is dense, very hard and almost impenetrable. Beyond the maximum, the sand liquefies because the water content is too high and the sand loses its compactness. A pile can easily be driven in again, but the load-bearing capacity decreases. The optimal value for water content is 7.63%.

In the context of our project, the normal proctor test is useful to quantify the amount of water needed to saturate the soil and know when the soil has reached the liquefaction. This data is also useful to know the water content that facilitates pile placement by driving or vibratory driving while maintaining good mechanical properties of the soil. These data can be used next year as a continuation of this thesis to perform saturated soil tests. For the purpose of this thesis, the data from the normal proctor test was used in the saturated cyclic tests.

4.4 Triaxial test

4.4.1 Description and definition of the process

The triaxial test consists of applying a deviatoric (or axial) stress to a cylindrical sample, surrounded by a flexible and impermeable membrane and subjected to an isotropic lateral pressure to simulate an isotropic stress condition and then shearing to failure [52]. The triaxial test is regulated by standard NBN EN ISO 17892-8 [2].

The triaxial test uses a cylindrical specimen of soil, ranging from 38mm to 100mm diameter and the height is generally twice the size of the diameter. The soil sample is placed on a metal base with a porous stone inside a sealed enclosure. On its lateral surface, the sample is covered with an waterproof membrane. Above the sample, a porous stone is placed as well as an upper base on which a piston rests.

The triaxial apparatus consists of a jack, a triaxial cell, various pressure devices and a measuring chain. The press is electromechanical and speed-controlled and constitutes the loading zone. The pressurisation of the water-filled cell allows the application of a confining stress. The cell is equipped with two electrical pressure transducers to measure the confining pressure and the pressure inside the sample [10].

The sample is subjected to :

- A horizontal hydrostatic pressure σ_3 imposed through the cell water.
- A vertical force V , via the jack, which penetrates the interior of the enclosure and acts on the upper surface of the sample.

During the test, the following measurements are obtained :

- The imposed hydrostatic lateral pressure σ_3 which is kept constant.
- The vertical force V transmitted by the piston.

- The vertical displacement of the piston Δl .
- The confining change of volume.
- The change in water volume of the sample Δv via the pore pressure u in the sample.

The application of the imposed pressures is carried out via a pressure-volume controller. The application of the vertical load is usually achieved by moving the press platen on which the triaxial cell is placed at a constant speed. The triaxial cell must be able to withstand the applied cell pressure without excessive loss of cell fluid. The soil sample must be confined by an elastic membrane whose function is to effectively prevent the penetration of the cell fluid into the specimen [63].

4.4.2 Equipment required

- A triaxial cell (fig. 4.10)
 1. Alternative position of the force measuring device
 2. Air evacuation
 3. Linear vertical displacement transducer
 4. Jack
 5. Top plate
 6. Soil sample
 7. Rubber membrane
 8. O-rings
 9. Cell pressure measurement and control device
 10. Triaxial cell

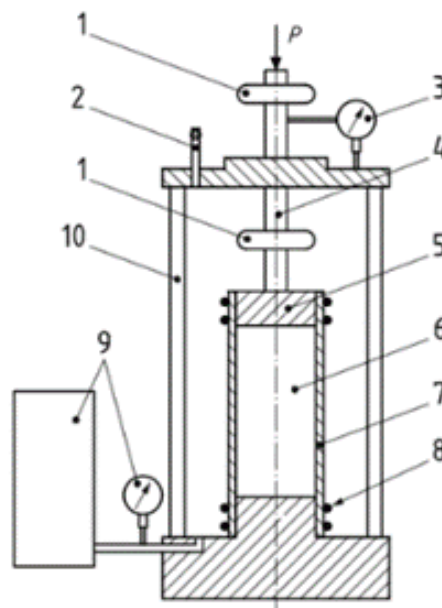


Figure 4.10: A triaxial cell [2]

- A load cell
- A pressure measuring device (fig. 4.11)
- A data acquisition and processing system to have and visualise in real time the evolution of the various test parameters

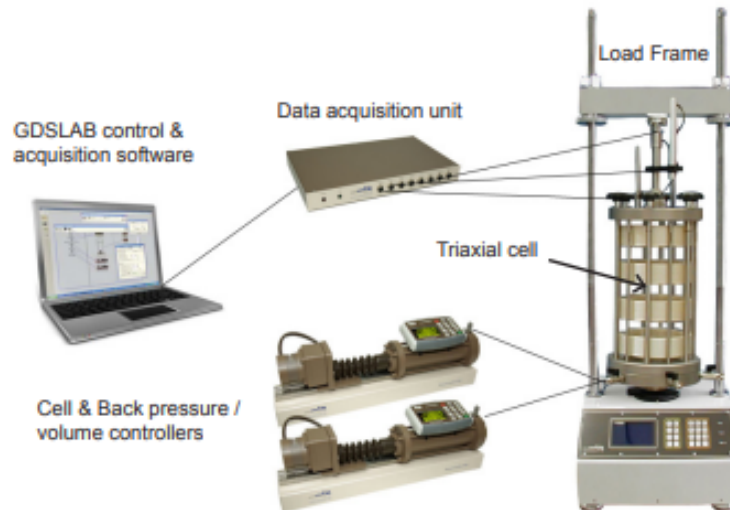


Figure 4.11: GDS Triaxial Testing System components [60]

4.4.3 Operational procedure

1. Thread the membrane onto the base and place the O-rings. Place a porous stone on the base pedestal of the cell.
2. Place the mould in position and pull the membrane out and over the top of the mould. Apply vacuum between the mould and the membrane to ensure that the membrane takes the shape of the mould and check that there are no creases in the wall of the membrane.



Figure 4.12: Suction device to apply vacuum between the mould and the membrane [66]

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3. Place the sand in several layers of equal height and mass. To achieve the closest value to e_{min} , slightly compact the different layers evenly. To get as close to as possible e_{max} , use a funnel and pour the sand into it, raising the funnel as you go along to limit the influence of the height.
4. Place a porous stone and the upper base on top of the mould and fold the membrane over the base. The porous stones allow to saturate the specimen while removing the water inlet effects.
5. Apply vacuum to the sample by increasing the vacuum pressure during demoulding so that the sample holds in place.



Figure 4.13: Preparation of the sand sample [66]

6. Place the cell on the sample and lower the piston until good contact is made with the cylinder at the top of the sample.
7. Fill the cell with water by gravity and ensure that all air is removed from the cell. Ensure that the cell has a good seal before starting the test;
8. Place the cell on the triaxial press and ensure that the LVDT is set to zero.
9. Apply a desired containment pressure. When the pressure is reached, start the test. The specimen is then subjected to axial deformation at constant speed. After the test has been started, ensure that there is no water leakage from the triaxial cell to the outside, to the pipework and its connections.

4.4.4 Calculation and expression of results

The stress state obtained during a triaxial test is displayed in fig. 4.14 :

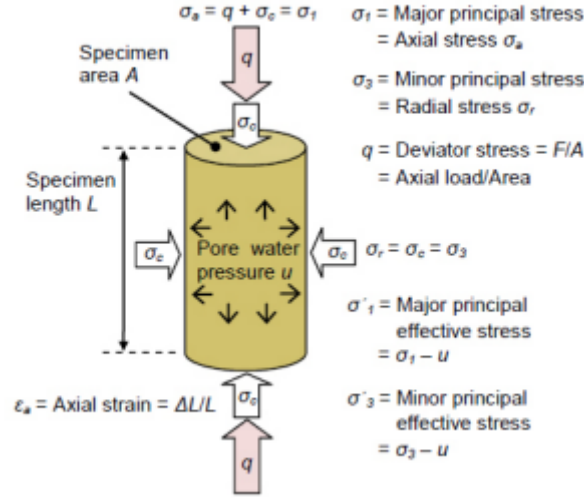


Figure 4.14: Specimen stress state during triaxial compression [60]

The different data acquired during the triaxial test can be calculated as follows [63] :

1. Deviator stress q

$$q = \sigma_1 - \sigma_3 \quad [kPa] \quad (4.12)$$

Since the test records the force, we have :

$$q = \frac{F}{S} \quad [kPa] \quad (4.13)$$

$$S = S_0 \cdot (1 + (1.5 \cdot \epsilon_a)) \quad [m^2] \quad (4.14)$$

$$F = (F_i - F_{initial}) \cdot coef \quad [kN] \quad (4.15)$$

2. Pore pressure u

For an undrained test, the change in pore pressure is obtained as follows :

$$\Delta u = (\Delta u_i - \Delta u_{initial}) \cdot coef \quad (4.16)$$

3. Volumetric strain ϵ_v

$$\epsilon_v = \frac{(\Delta V_i - \Delta V_{initial}) \cdot coef}{V} \cdot 100 \quad (4.17)$$

4. Axial strain ϵ_a

$$\epsilon_a = \frac{(\Delta l_i - \Delta l_{initial}) \cdot coef}{h} \cdot 100 \quad (4.18)$$

5. Radial strain ϵ_r

$$\epsilon_r = \frac{(\Delta r_i - \Delta r_{initial}) \cdot coef}{r} \cdot 100 \quad (4.19)$$

4.4.5 Results

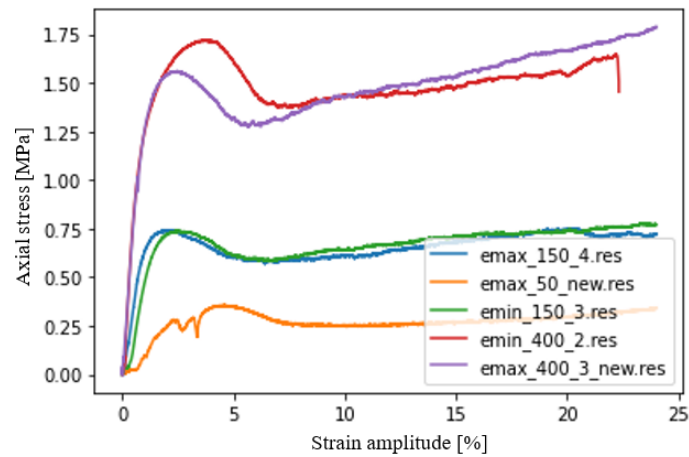


Figure 4.15: Axial stress as a function of strain amplitude for a triaxial test

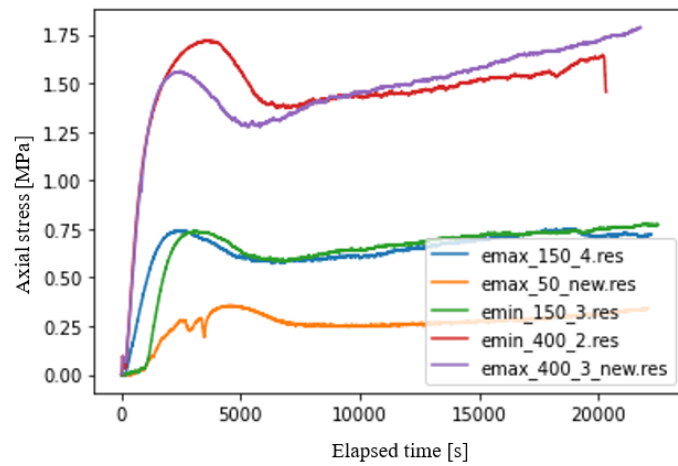


Figure 4.16: The evolution of axial stress during a triaxial test

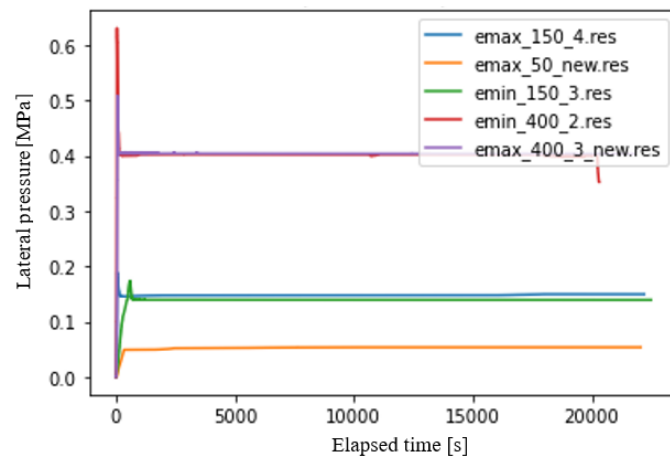


Figure 4.17: The evolution of lateral pressure during a triaxial test

4 – Characterisation of the North Sea Sand

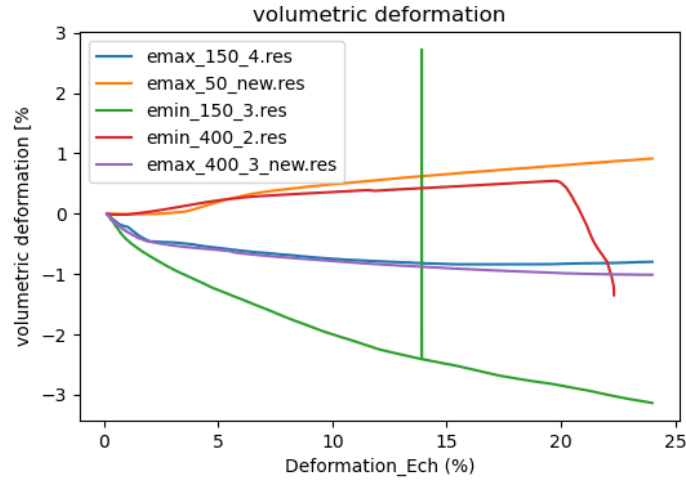


Figure 4.18: The evolution of the volumetric deformations during a triaxial test

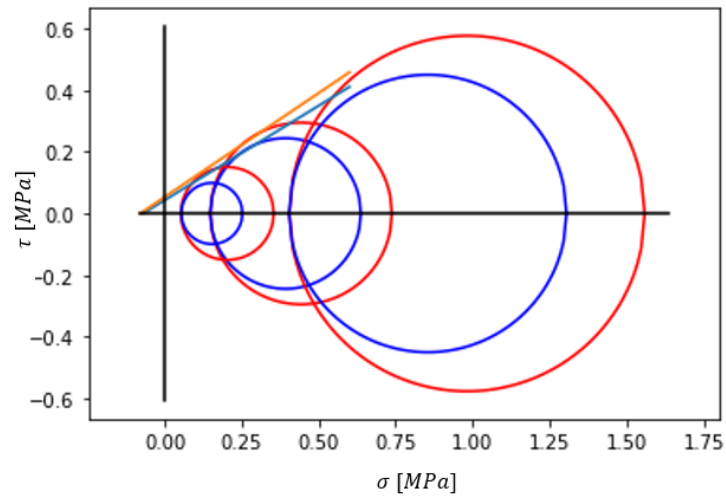


Figure 4.19: Mohr circle for dry loose sand

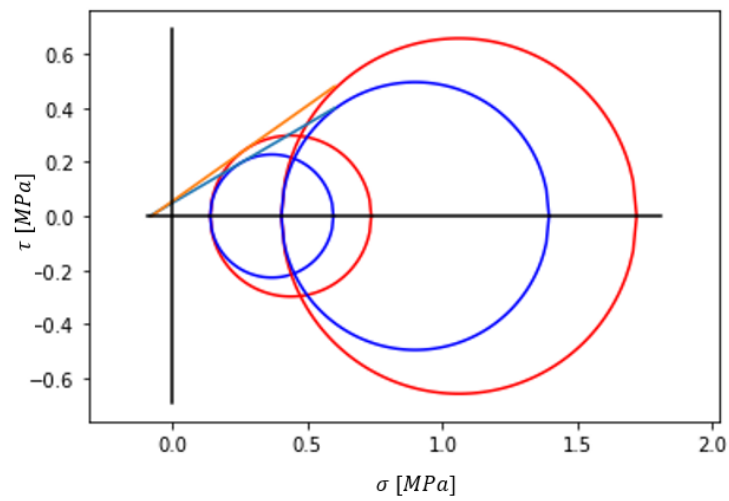


Figure 4.20: Mohr circle for dry dense sand

Test name	$e_{max,50}$	$e_{max,150}$	$e_{max,400}$	$e_{min,150}$	$e_{min,400}$
Weight [g]	92.81	91.2	90.41	104.93	109.24
Relative density [%]	28.8835	16.5073	20.4747	84.9527	98.5918
$\sigma_{1,peak}$ [MPa]	0.3537	0.7378	1.5591	0.7359	1.7188
$\sigma_{3,peak}$ [MPa]	0.0525	0.1478	0.4047	0.1397	0.4028
$\sigma_{1,res}$ [MPa]	0.252	0.6368	1.3049	0.5955	1.3943
$\sigma_{3,res}$ [MPa]	0.0537	0.1478	0.4039	0.1396	0.4027
Young modulus [MPa]	0.0354	0.6842	1.278	0.0177	2.4004

Table 4.7: Principal stresses for different densities and lateral pressure

The relative density values are an approximation because the exact specimen volume has not been measured before to launch of the triaxial test.

4.4.6 Discussion

As the test progresses, the stress-strain curves (σ_1 - σ_3 ; ϵ) are recorded. The shear strength often corresponds to the maximum of the deviator reached during triaxial compression and is obtained when the specimen breaks. These maxima then define a Mohr circle tangent to the intrinsic curve. To obtain the Mohr line and the mechanical characteristics, several tests are performed at different cell pressures. The stress state is represented as a Mohr circle and the cohesion (c) and the angle of friction (ϕ) can be derived.

Cohesion represents an electrostatic force between particles. The greater the cohesion, the more difficult it is for particles to lose contact with each other. Sand grain particles are large enough to avoid cohesive forces. The friction angle helps to quantify a soil's shear strength. It defines the angle of the Mohr coulomb line and depends on a few factors, primarily grain size distribution, angularity and particle interlocking. An angular and coarse sand has a higher friction angle than a fine grained and well-rounded sand [32].

The cohesion and friction angle values for dense and loose sand are the following :

Type of sand	Loose sand	Dense sand
Peak cohesion [MPa]	0.0433	0.0491
Residual cohesion [MPa]	0.0539	0.0554
Peak friction angle [°]	34	35.29
Residual friction angle [°]	31.52	30.3

Table 4.8: Cohesion and friction angle for loose and dense sand

The table 4.8 shows that for loose and dense sand, the cohesion value is close to zero. This value can be explained by the fact that the triaxial test has been done with sea sand so there are shells in the sample. That can explain that the cohesion is not perfectly equal to zero. In general, for sand, the cohesion is equal to zero and the friction angle is equal to 30-40° for sand with a spread out grain size. In view of the values found in the literature, the values obtained during the triaxial tests can be validated and used in the following sections of this thesis.

The fig. 4.15 shows the evolution of stress-strain during a triaxial test. The values of peak stresses and residual stresses are included in table 4.7. These values allow to represent the Mohr circles (fig. 4.19 and fig. 4.20) for dense and loose sand. The blue circles show the residual values i.e. the values when the stress-strain curve is a quasi constant line. The red Mohr circle shows the peak values i.e. the values when the stress-strain curve increases until the maximal axial stress and before it decreases until the quasi constant line (fig. 4.21).

The elastic phase lasts as long as the curve forms an increasing linear line. The Young's modulus can be defined in this phase. Here, the dense sand samples have a higher Young Modulus than loose sand samples. Then the plastic phase begins. The failure can be defined in terms of peak or residual values. At peak, the soil carries the maximum possible stress and at residual state, the soil has undergone a very large strain. The friction angle and cohesion can be defined using the peak or residual values. In general, the peak parameters are used to characterise a soil because the strains are relatively small and stresses are well below the peak.

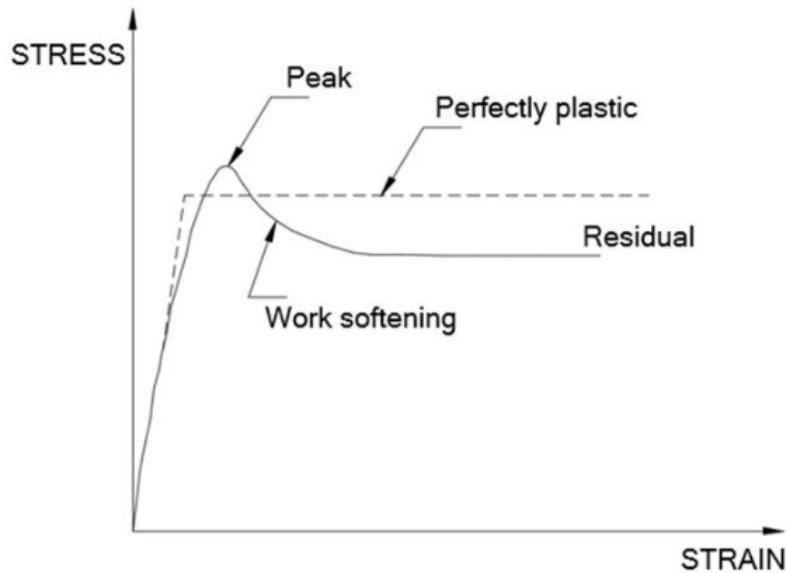


Figure 4.21: Elastic-perfectly assumption of Mohr-Coulomb model [67]

Through the triaxial test, the Mohr Coulomb criterion can be defined. It is a mathematical model that describes the strength of materials such as soils when subjected to normal or shear stresses. In general, this criterion applies to materials whose compressive strength is much higher than their tensile strength.

The following criterion therefore describes an envelope of stresses that the material can withstand :

$$\tau = c + \sigma' \cdot \tan(\phi) \quad (4.20)$$

The cohesion being close to zero and the specimen being dry, the previous equation becomes :

$$\tau = \sigma \cdot \tan(\phi) \quad (4.21)$$

On the fig. 4.15, during the peak phase, the minimum density curves are higher than maximum densities curves for the same cell pressure. During the residual phase, it is the opposite. The $e_{max,50}$ sample have two drops of axial stress because the top pore stone broke during the triaxial test. There are only two tests for the minimal density because the triaxial cell was down to realise the last test.

The fig. 4.17 shows the evolution of the lateral pressure during a triaxial test. The lateral pressure remains constant during the triaxial test. That means that there are no leaks and exchange of water between the cell and the sample, which shows that the membrane is well sealed. The sample is not saturated.

4.5 Cyclic test

4.5.1 Description and definition of the process

The cyclic triaxial test measures the dynamic properties of the soil as the strength and deformation properties. The cylindrical sample with a height of 200 mm and a diameter of 100 mm is prepared by the wet tamping method in 10 layers following the sub-compaction procedure. It is then surrounded by a thin rubber membrane and placed between two loading plates. After saturation, the specimen is consolidated by applying a lateral and an axial stress. Cyclic axial deformation is then applied to the soil sample while keeping the lateral stress constant. In a triaxial test, the principal directions are oriented continuously in the horizontal and axial directions and the stress distribution is assumed to be uniform throughout the specimen.

At the end of the cyclic triaxial test, from the measurements of the axial and lateral stresses and the vertical strain, the secant shear modulus G_s and the damping ratio λ can be calculated. The lateral deformation can be deduced from the vertical deformation. During the test, the pore pressure can be observed.

Furthermore, in the case of saturated soil without cohesion, cyclic loading by a vibrator induces soil liquefaction around the pile. This liquefaction of the soil leads to a reduction in shear strength and facilitates pile driving under gravity forces. This degradation and transition to liquefied soil can be studied through cyclic triaxial tests. Soil degradation is strongly influenced by the magnitude of the shear strain, the relative density and the initial stress state.

In these cyclic triaxial tests, deformation is controlled in order to better analyse soil degradation because degradation progresses normally whereas at controlled stress, liquefaction occurs abruptly making degradation analysis complicated. The tests are performed on dry, saturated samples under undrained conditions. The resulting pore pressure is measured continuously.

In our case, the cyclic triaxial test is regulated by standard EUAF-FGBM-SOIL-TM-701 : Static triaxial testing and EUAF-FGBM-SOIL-TM-751 : Preparation of sand and silt specimens using Ladd's under-compaction provided by Fugro. The "calculation and expression results" part is based on the thesis of Jean-François Vanden Berghe [71]

4.5.2 Equipment required

- For compaction :
 - Compactor
 - Mould
 - Balance
 - Caliper
- For consolidation, saturation and cyclic test :
 - Loading frames
 - Cyclic triaxial cell
 - Load cell
 - Pressure transducers
 - Linear vertical displacement transducers
 - Volume change apparatus
 - Pressure controllers
 - De-aired water
 - Bottle of CO_2

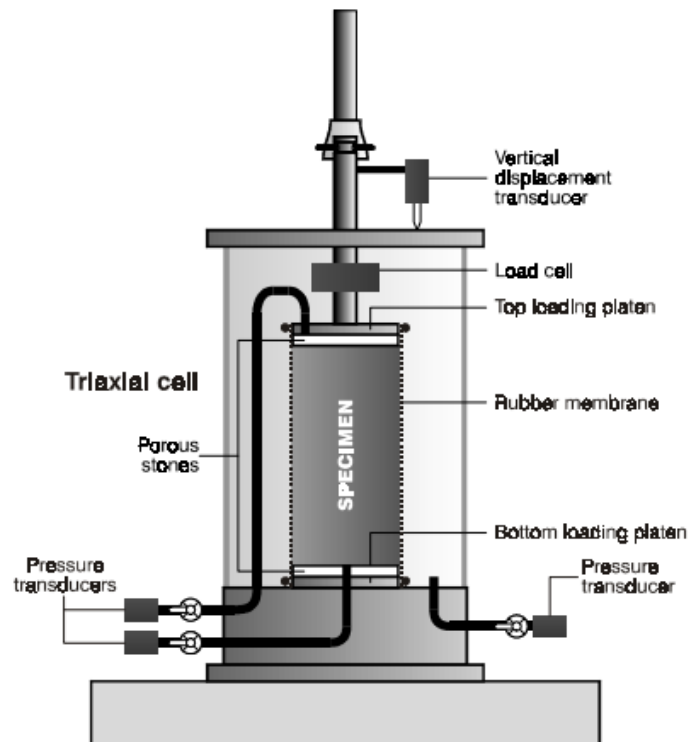


Figure 4.22: Cyclic triaxial cell [71]

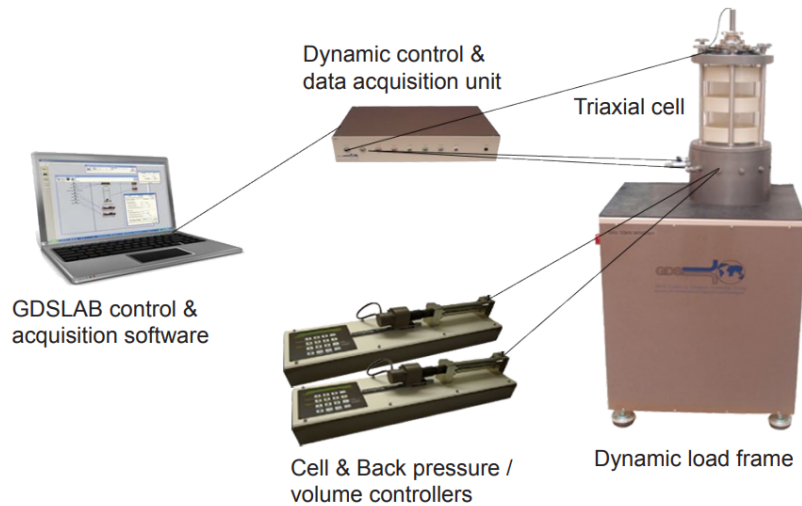


Figure 4.23: GDS Dynamic Triaxial Testing System components [42]

4.5.3 Operational procedure

- Undercompaction.

Each specimen is prepared using the 10-layer wet tamping method following the subcompaction process. This method is necessary in order to obtain a more uniform density throughout the sample and reproducible results. Undercompaction takes into account the fact that the compaction of successive layers of soil also compacts the underlying layers. The initial layers are therefore compacted to lower densities than the subsequent layers, so that the final density of each layer in the sample is approximately uniform.

1. The compaction is carried out in a cylindrical mould formed by three interconnected cups. Fix a split mould separate from the triaxial cell in order to obtain better control of the sample preparation and a high relative density.
2. Place the dry sand in a plastic bag and add water to the sand to achieve a water content value as recommended in table 4.9 or the optimum for the Proctor test. Seal the plastic bag and thoroughly mix the sand. Leave the sealed bag to achieve an equilibrium of water content in the whole sample. The waiting period for fine sands is two hours. If at any point during the sample's preparation and compaction the procedure is interrupted, the sample should be sealed.

Particle size majority	Recommended water content
Fine silt	16%
Medium silt	17%
Coarse silt	12%
Fine sand	10%
Medium sand	8%
Coarse sand	6%

Table 4.9: Recommended water contents for saturated sample

For dry sample, the two previous steps are not necessary.

3. Measure the mould internal diameter, height and the membrane thickness.
 4. For the first layer, enter a value for undercompaction that ranges between zero for dense specimens and 15% for very loose specimens. The required mass and height of each layer is calculated automatically.
 5. Before taking the required sample mass, shake the bag well. Keep the bag sealed during the tamping operation to avoid losing moisture content.
 6. Adjust the tamping rod guide yoke to obtain the computed height of the first layer within $\pm 0.05\text{mm}$. Fasten firmly the locking screws on the yoke.
 7. Place all the sand required for the first layer in the mould. Use the tamping rod to compact the sand to the required height. Compact the sand in a diagonal manner. The diagonal tamping is very important to ensure a homogeneous specimen in the horizontal direction. Lack of homogeneity in this direction will lead to non-repeatability of results. For the last few coverages, it might be necessary to hit the tamping rod with a rubber mallet in order to compact the material into a dense state.
 8. After compaction, weight the filled mould and place it on the base of the triaxial apparatus.
 9. Remove the sample from the mould by carefully releasing the individual cups from the mould in turn.
 10. Place the rubber membrane around the sample and place a porous stone on top of it.
 11. Close the cell and fill it with water.
- Saturation with CO_2 .

As CO_2 dissolves better in water than air, these specimens can be flushed with CO_2 before flushing with water in order to assure a better saturation. To do this, proceed as follows :

1. Connect the top pedestal line to a saturated drainage tube and note the water level in the tube.
2. Connect the base pedestal line to a CO_2 gas cylinder.
3. Increase gradually the CO_2 flush pressure until you see CO_2 moving through the drainage tube at a rate of 2-3 bubbles per second. During this process, the cell pressure is regulated to maintain a positive difference between lateral stress and pore pressure at all times.
4. Once the flush has been sufficient (at least 20 min), close, remove and replace the gas pressure line by the back pressure line.
5. Elevate gradually the back pressure until you reach the final CO_2 flush pressure to replace any CO_2 in the sample with demineralised water. So that the remaining pore gases are dissolved into pore water. The water is introduced from the bottom of the test tube, allowing air bubbles to leave the test tube from the top.

6. Stop the flushing as soon as an increase in the water level in the drainage tube is observed.
 7. Close the back pressure line and transfer this line to the top cap line for use in subsequent saturation.
- Saturation by increasing the cell pressure and back pressure :
 1. Close the back pressure valve and apply a suitable cell pressure.
 2. Calculate the degree of saturation using the Skempton method. This method consists of slightly increasing the lateral pressure while keeping the drain closed and measuring the corresponding increase in pore water pressure.

$$B = \frac{\Delta u}{\Delta \sigma_3} \quad (4.22)$$

If the measurement of the B value and corresponding time increment indicate the sample to be saturated, proceed to the consolidation stage. A B value higher than 0.95 proves that the specimen is saturated.

Base pressure at the start of the cyclic stages must be equal or above the pore water pressure at which satisfactory B-value was achieved. The increase in back pressure and cell pressure must be sufficiently slow to ensure an even distribution of pore water pressure in the sample.

- End of saturation checks
 1. At the end of the saturation stage, close the valves to the specimen and the valve to the cell so that the following actions do not apply any changes to the cell or base pressures.
 2. Close the valve connecting the base pressure transducer to the back pressure pump.
 3. Connect the base pressure transducer to the cell's pressure pump and the cell pressure transducer.
 4. Change the cell's pressure controller to the base pressure at the end of the saturation stage.
 5. Change the offset in the cell pressure transducer to read the same value as the base pressure transducer.
- Isotropic consolidation

Consolidation of the soil specimen is necessary to establish the desired stress state and apply the correct effective stresses prior to the cyclic loading.

For soft material, small cell pressure steps or slow ramps should be used. The sample through the ram and the deviator stress remains at 0 kPa. Water should be allowed to drain freely from the specimen.

During isotropic consolidation, the changes in sample volume is equal to the volume of water that was removed from the sample since the saturation phase has been already done. Data from the last consolidation stage should be used to calculate the rate of shear.

1. Consolidate until the excess pore pressure is at least 95% dissipated.
2. Once 95% dissipation is achieved, measure the volume of water being squeezed out of the sample for a further period of between one and four hours
3. Record the volume readings at the start and end of the time period on the appropriate lab form.
4. Terminate the isotropic consolidation if the rate of volumetric strain is less than 0.02% per hour, which equates to the following values for various diameter samples assuming a height/diameter ratio in the range 2 to 1 :

Diameter [mm]	Height [mm]	Area [mm ²]	Volume [cm ³]	Volumetric strain [%/hr]	Volume [cm ³ /hr]
38	76	1134.11	86.19	0.02	0.02
50	100	1963.50	196.35	0.02	0.04
70	140	3848.45	538.78	0.02	0.11
100	200	7853.98	1570.80	0.02	0.31

Table 4.10: Volumetric strain rates indicating end of isotropic consolidation

5. Note the rate of volumetric strain at the end of the consolidation stage on the test forms.

■ Cyclic phase

After the soil has reached equilibrium, the sample drainage is closed to impose an undrained condition (i.e. no volume change) and the sample is sheared by a symmetrical cyclic variation of the axial deformation. The axial strain amplitude ϵ_a is 0.15% and the cyclic displacement frequency is 1 and 5 Hz. The tests are undrained while the pore pressure is measured continuously at the top and bottom of the specimen.

For sand samples, the sample liquefies during the cyclic phase when the pore water pressure becomes equal to the cell pressure.

During the tests, the following measurements are made continuously :

- The vertical displacement measured by an LVDT transducer attached to the piston.
- The axial force applied by the piston. This force is measured with a load cell placed inside the triaxial cell to avoid interface friction between the piston and the load cell.
- The pore pressure measured at the top and bottom of the specimen.
- The pressure of the cell

■ End of the test

1. For sand samples, photographs should be taken at the end of the test while the sample is still in the triaxial cell and before the pressure is removed.
2. Empty the cell and remove the sample from the triaxial cell base as quickly as possible to minimise water absorption by the porous discs.

3. At the end of each test, the back pressure tester should be emptied and rinsed with tap water before being refilled with deaerated water to prevent corrosion of the equipment.

4.5.4 Calculation and expression of results

- Regulation and data acquisition

At the end of the consolidation, the volume of the specimen is calculated as follows :

$$V_C = V_0 + \Delta V_{sat} - V_{air} - \Delta V_{con} \quad (4.23)$$

During the test, the volume of the specimen is supposed constant due to the undrained condition. Assuming that the specimen stays cylindrical during the test, the specimen height h and diameter d are calculated as follows :

$$h = h_c + VD \quad (4.24)$$

$$d = \sqrt{\frac{4 \cdot V_C}{h \cdot \pi}} \quad (4.25)$$

Assuming that the stress and strain distributions are homogenously distributed and that the vertical and lateral stresses are the principal stresses, the stresses and strains are calculated as follows :

$$\epsilon_1 = \frac{VD}{h_c} > 0 \quad (4.26)$$

$$\epsilon_3 = \frac{d_c - d}{d_c} \quad (4.27)$$

$$\gamma = \epsilon_1 - \epsilon_3 \quad (4.28)$$

$$\sigma_3 = LP + \Delta LP \quad (4.29)$$

$$\sigma_1 = \sigma_3 + \frac{4 \cdot AL + \Delta AL}{d^2 \cdot \pi} \quad (4.30)$$

$$\tau = \frac{\sigma_1 - \sigma_3}{2} \quad (4.31)$$

The pore pressure is measured as follows :

$$u = \frac{u_{top} + u_{bottom}}{2} \quad (4.32)$$

The effective stresses are obtained as follows :

$$\sigma'_1 = \sigma_1 - u \quad (4.33)$$

$$\sigma'_3 = \sigma_3 - u \quad (4.34)$$

The effective and total mean stresses and the deviator are calculated as follows :

$$P' = \frac{\sigma'_1 + 2\sigma'_3}{3} \quad (4.35)$$

$$P = \frac{\sigma_1 + 2\sigma_3}{3} \quad (4.36)$$

$$q = \sigma'_1 - \sigma'_3 \quad (4.37)$$

- Representation of the stress states in the soil

When the soil is saturated, the external loads applied to the soil balance the contact forces and the pressure u . The contact forces give the soil its strength and stiffness. The state of stress in the soil is represented by the external load applied to the soil, the pore pressure and the effective stress state. The following relationships describe these stresses for each stress direction and at each point in the soil :

$$\sigma = \sigma' + u \quad (4.38)$$

$$\tau = \tau' \quad (4.39)$$

An increase in pore pressure leads to a decrease in effective stress and vice versa.

During a cyclic loading of large amplitude, the sand grains are reorganised and the amplitude and direction of the contact force between the grains are modified by the cyclic movements. As a result of this reorganisation, the grains get more loose and try to reduce the volume of the specimen. If the soil is saturated and if the boundary conditions prevent drainage, this trend results in an increase in pore pressure leading to a reduction in effective stresses and shear strength. This increase in pore pressure causes the specimen to liquefy.

The term liquefaction is related to all phenomena leading to an increase in the pore pressure of sands under undrained loading conditions. It is defined as the total loss of shear strength which happens when pore pressure increases so much that it becomes equal to the total stress. This liquefaction phenomenon is also due to a reorganisation of the granular structure aimed at reducing the volume of the soil and to an increase in the pore pressure reducing the effective stresses to a value close to zero. Under these conditions, the sand has no shear strength and behaves like a viscous liquid.

- Stress-strain relationship during cyclic shear

During symmetrical cyclic strains of amplitude γ_a , the relationship between stress and shear strain is represented by a hysteresis loop. The shape of the loop is usually characterised by the secant shear modulus G_s defined as the slope between the two ends of the loop.

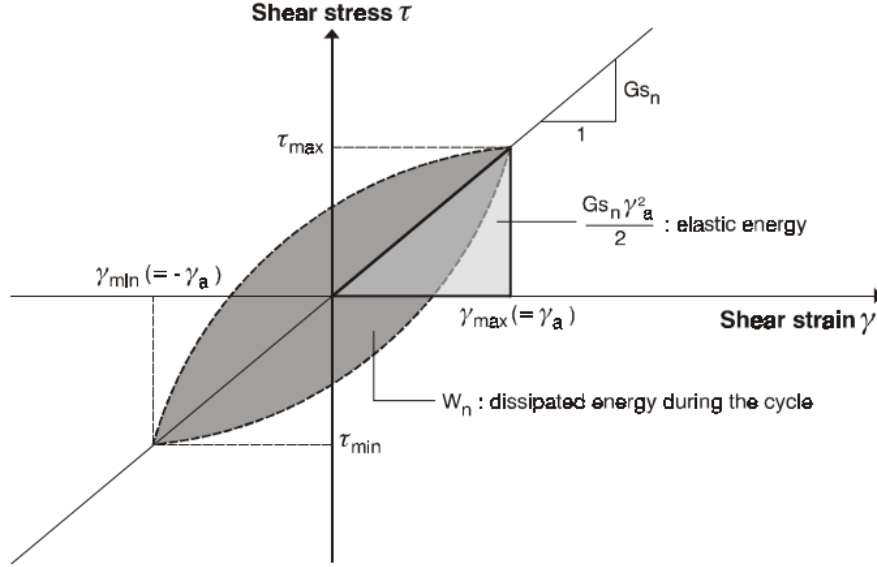


Figure 4.24: Characteristics of hysteresis loop during cyclic shearing [71]

$$G_s = \frac{\tau_{max} - \tau_{min}}{\gamma_{max} - \gamma_{min}} \quad (4.40)$$

$$\lambda_d = \frac{1}{2\pi} \cdot \frac{W}{\gamma_{max} \cdot \tau_{max}} \quad (4.41)$$

The secant shear modulus G_s and the damping ratio λ_d depend on the strain amplitude γ_a , void ratio e , average stress P' , plasticity index PI , overconsolidation ratio OCR and cycle number N .

The maximum shear modulus G_{max} is equal to the tangent shear modulus at the origin and the limiting shear stress τ_c is equal to the shear stress at failure under monotonic loading :

$$\tau = \frac{G_{max} \cdot \gamma}{1 + \frac{G_{max} \cdot \gamma}{\tau_c}} \quad (4.42)$$

- Influence of the deformation amplitude

- Low deformation amplitude

If the shear strain amplitude of a cyclic load is small, the behaviour of the soil is linear elastic and the hysteresis loop looks like a straight line. In this case, the secant shear modulus corresponds to the elastic shear modulus of the soil G_{max} and represents the slope of this line. The damping ratio is zero.

In this particular case, the soil being a linear elastic material, G_{max} is defined using wave theory :

$$G_{max} = \rho \cdot v_s^2 \quad (4.43)$$

The maximum shear modulus is independent of the amplitude of the shear strain and the number of cycles.

- Large deformation amplitude

The amplitude of the shear strain strongly influences the shape of the hysteresis loops. Indeed, increasing the shear strain amplitude implies a decrease in the shear secant modulus and an increase in the damping ratio.

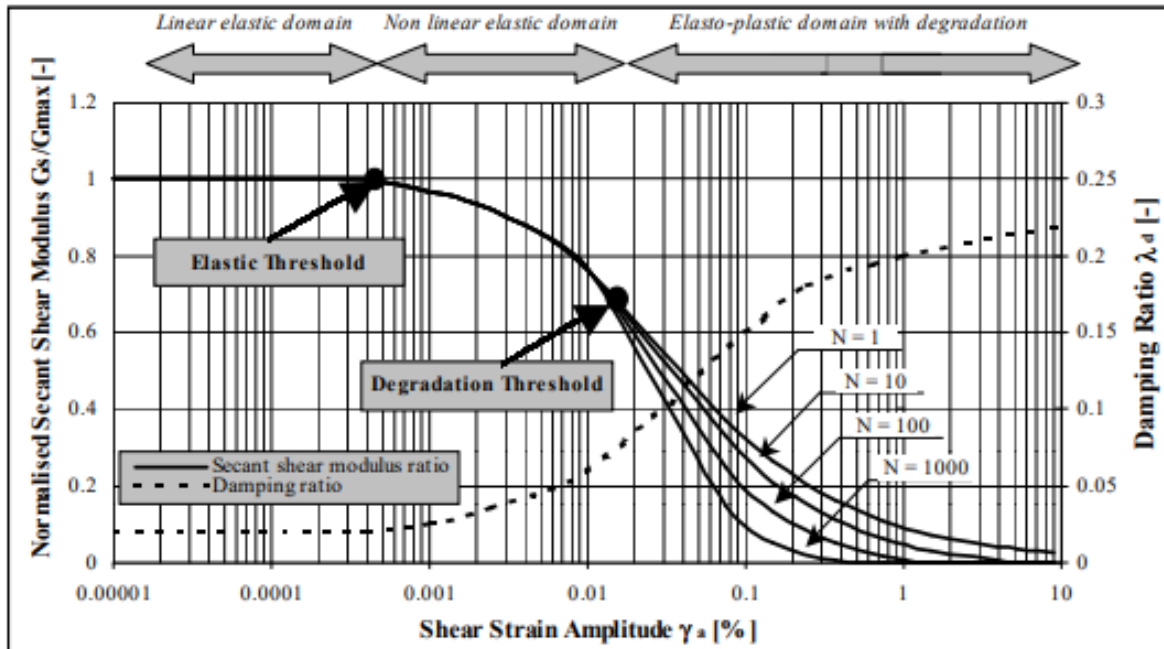


Figure 4.25: Relationship between secant shear modulus ratio G_s/G_{max} and damping ratio λ_d as a function of the shear strain amplitude γ_a [71]

In the above graph, three soil behaviours are observed as a function of the deformation amplitude :

- * Linear elastic behaviour
- * Elasto-plastic behaviour without degradation
- * Elasto-plastic behaviour with degradation

The graph (fig. 4.25) shows that the behaviour of the soil is linear elastic under cyclic loading with low strain amplitudes. The secant shear modulus is equal to 1 and the value of the damping ratio is small and represents intrinsic viscous damping.

With increasing deformation amplitude, the behaviour of the soil is non-linear elastic and hysteresis loops are formed. This increase implies a decrease in the secant shear modulus G_s and an increase in the damping ratio. The transition between linear elastic and non-linear elastic behaviour is called the elastic threshold.

At high strain amplitudes, the grain structure deteriorates and a loss of strength at each cycle is observed. This loss of strength implies a reduction in the secant shear modulus and an increase of the damping ratio with the number of cycles. The deformations lead to plastification of the contact areas between the grains and cause them to slip relatively to each other.

- Degradation of soil strength

During cyclic loading under conditions of high controlled deformation, the pore pressure gradually increases and the secant shear modulus decreases with the increasing number of cycles. These conditions lead to a degradation of the stiffness of the sample and a progressive decrease in the slope and area of successive hysteresis loops.

The degradation of the soil can also be observed on the basis of the energy dissipated during the cycles, i.e. on the basis of the surface of the hysteresis loops.

The dissipated energy W_n is calculated by integrating the hysteresis loop on one period starting from the zero shear strain during the compression phase of the cycle. Due to the soil degradation, this integration is not performed on a closed loop as because the shear stress at the end of a cycle is smaller than the value at the beginning of the cycle.

The accumulated energy $W_{tot,n}$ is the total energy dissipated since the beginning of the cyclic test and is calculated by adding the energy dissipated during each cycle.

$$W_{tot,n} = \sum_{i=1}^n W_i \quad (4.44)$$

The degree of liquefaction $d_{liq,n}$ is defined as the ratio between the excess pore pressure at the end of the cycle and the effective mean stress P'_c applied during the consolidation.

$$d_{liq,n} = \frac{\Delta u_{end}}{P'_c} \quad (4.45)$$

4.5.5 Results

The software supports the measurements of the vertical displacement transducer, the 3 pressure transducers and the vertical load cell. It converts the voltages obtained into physical values and calculates the corresponding stresses and strains. Based on the measured vertical strains and stresses, the program controls the load frame motor to obtain a cyclic axial deformation of the specimen and regulates the cell pressure to keep the total average stress constant.

The following table 4.11 shows the different characteristics used to perform the eight cyclic tests. The shear strain is fixed at 0.15 % and the frequency varies between 1Hz which is the frequency of the wind and 5Hz which is the maximum frequency of the cyclic triaxial cell. The low consolidation pressure models the actual pressure in the model pile driving experiments.

N°	Type of soil	D_r [%]	Saturation	Frequency [Hz]	Strain [%]	Consolidation pressure [kPa]
1	Loose	34	Saturated	5	0.15	50
2	Loose	30	Saturated	1	0.15	50
3	Loose	59	Dry	5	0.15	50
4	Loose	59	Dry	1	0.15	50
5	Dense	58	Saturated	5	0.15	50
6	Dense	49	Saturated	1	0.15	50
7	Dense	92	Dry	5	0.15	50
8	Dense	92	Dry	1	0.15	50

Table 4.11: Principal characteristics of the cyclic triaxial tests

The fig. 4.26 describes the characteristics for the sample preparation and calculate automatically the mass and the height of the hammer for the compaction. All the sheets with the different characteristics for the preparation of the different samples can be found in the appendix E. The table 4.11 summarises the important characteristics of the different compacted samples.

4 – Characterisation of the North Sea Sand

Client			Borehole			Test no		
Site			Sample			System		
Job no			Sample depth (m)	-		Specimen depth (m)	-	

Ladd undercompaction preparation sheet - Triaxial tests

Test Type	CIUc		Instrumentation				
Min dry density (Mg/m ³)		Max density (Mg/m ³)		Relative density, D _r (%)	No data		
Suggested U _{ni}	15,0	Membrane Thickness (mm)		Estimated density, D _r (%)			
Total mass (gr)	945,05						

REQUIRED SPECIMEN CONDITIONS

Dry Density (Mg/m ³)	Specimen Diameter (mm)	Number of layers (n)	Specimen height (mm)	Initial Moisture Content (%)	Particle density Assmd. (Mg/m ³)	Mould height (mm)	Under compaction of first layer U _{ni} (%)
1,64	69,99	9	140,00	7,0	2,65	181,59	15,0
Initial void ratio e _i	Degree of saturation S _r (%)	Volume of voids (cm ³)	Saturated bulk density (Mg/m ³)	Specimen Area (mm ²)	Specimen Volume (cm ³)	Total mass (gr)	Difference in mould height (mm)
0,62	30,12	143,43	1,755	3846,80	538,55	945,05	41,59

UNDER COMPACTION CONDITIONS

Layer (n)	U _n	Final layer height (mm)	Accumulative mass (g)	Incremental mass (g)	Required yoke setting (mm)
1	15,0	17,89	105,01	105,01	163,70
2	13,1	33,15	210,01	105,01	148,44
3	11,3	48,42	315,02	105,01	133,18
4	9,4	63,68	420,02	105,01	117,91
5	7,5	78,94	525,03	105,01	102,65
6	5,6	94,21	630,03	105,01	87,38
7	3,8	109,47	735,04	105,01	72,12
8	1,9	124,74	840,05	105,01	56,86
9	0,0	140,00	945,05	105,01	41,59

INITIAL MOISTURE CONTENT		INITIAL SAMPLE MASS		NOTES
Date sample in oven		Date		
Tin number		Mould and specimen (g)	2546,68	
Wet mass and tin (g)		Mould only (g)	1601,55	
Dry mass and tin (g)		Specimen mass (g)	945,14	
Mass of tin only (g)		Bulk density (Mg/m ³)	1,755	
Moisture content (%)		Sample Preparation by		

SAMPLE DESCRIPTION

Figure 4.26: Measurements performed during a typical specimen preparation

4 – Characterisation of the North Sea Sand

Test name	LOOSE-5Hz	LOOSE-1Hz	DENSE-5Hz	DENSE-1Hz
Bulk density [Mg/m^3]	1.76	1.755	1.828	1.802
Mass of the mould and specimen [g]	2554.62	2546.69	2586.64	2579.07
Mass of the mould [g]	1606.08	1601.55	1603.97	1602.46
Mass of the specimen [g]	948.54	945.14	982.67	976.61
Volume [mm^3]	538.01	538.55	537.55	541.82
Water content [%]	7	7	7	7
Dry density [Mg/m^3]	1.648	1.64	1.708	1.685
e [-]	0.4283	0.4349	0.3775	0.3971
D_r [%]	34	30	58	49

Table 4.12: Main characteristics of samples for saturated sand

Test name	DENSE-1Hz	DENSE-5Hz	LOOSE-1Hz	LOOSE-5Hz
Bulk density [Mg/m^3]	1.8	1.8	1.71	1.71
Mass of the specimen [g]	1051.17	1051.17	998.61	998.61
Volume [m^3]	583.98	583.98	583.98	583.98
Water content [%]	0	0	0	0
Dry density [Mg/m^3]	1.8	1.8	1.71	1.71
e [-]	0.3075	0.3075	0.3763	0.3763
D_r	92	92	59	59

Table 4.13: Main characteristics of the samples for dry sand

The following table summarises the steps performed during a typical saturated cyclic test.

Stage	Name	Cell Pressure	Back pressure	Remarks
1	Setup	50 kPa	40 kPa	Back pressure closed
2	CO2 flush	50 kPa	40 kPa	Back pressure closed; CO2 pressure = 40 kPa; 20 minutes
3	H2O flush	50 kPa	40 kPa	V higher than V voids
4	Saturation	↗600 kPa	↗590 kPa	1h
5	B-value Check	↗700 kPa	↗690 kPa	1h; B-value>0.95
6	Isotropic consolidation setup	700 kPa	690 kPa	BP = PWP
7	Isotropic consolidation	↗740 kPa	690 kPa	till $\Delta v < 100$ on 1h
8	Strain controlled cyclic	740 kPa	690 kPa	f = 1Hz; $\Delta h = 0.0015 \cdot (140 - \Delta h_{con})$; $\Delta h = 0.209841$

Figure 4.27: Measurements performed during a typical specimen saturation and consolidation

Soil behaviour under a cyclic loading for dry sand

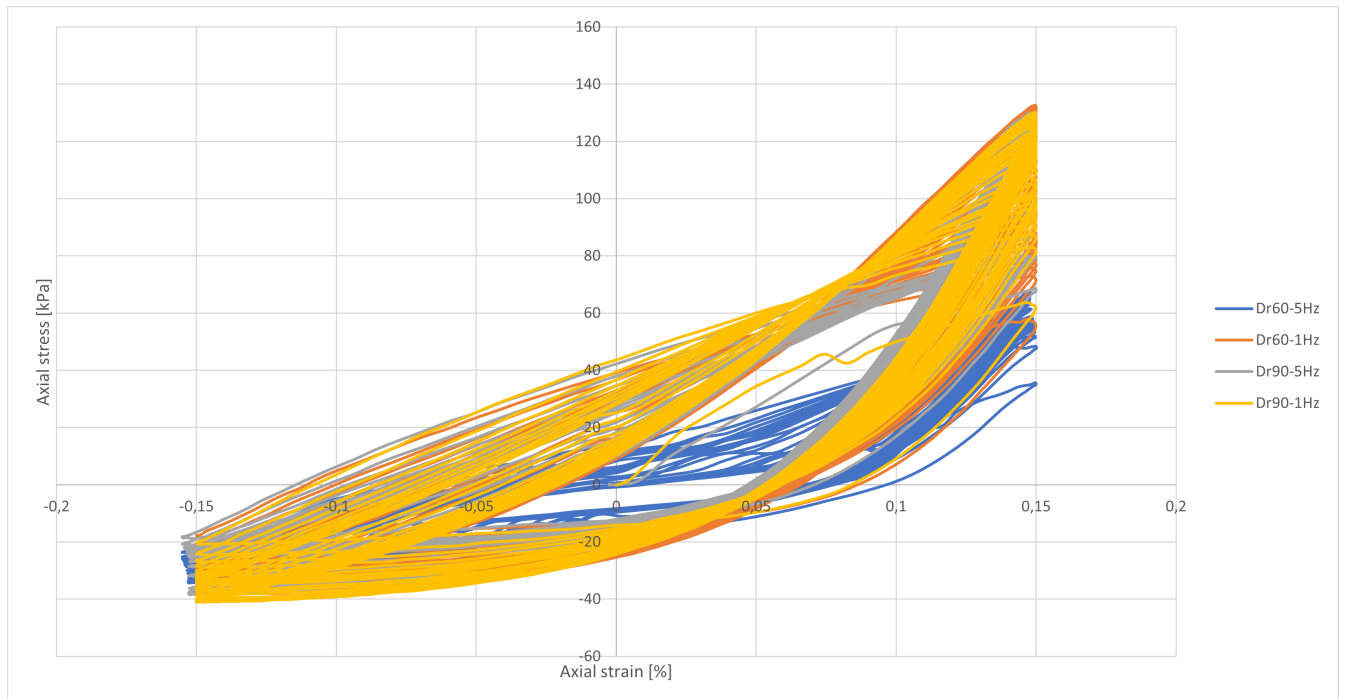


Figure 4.28: Hysteresis loops during a cyclic triaxial test for dry sand

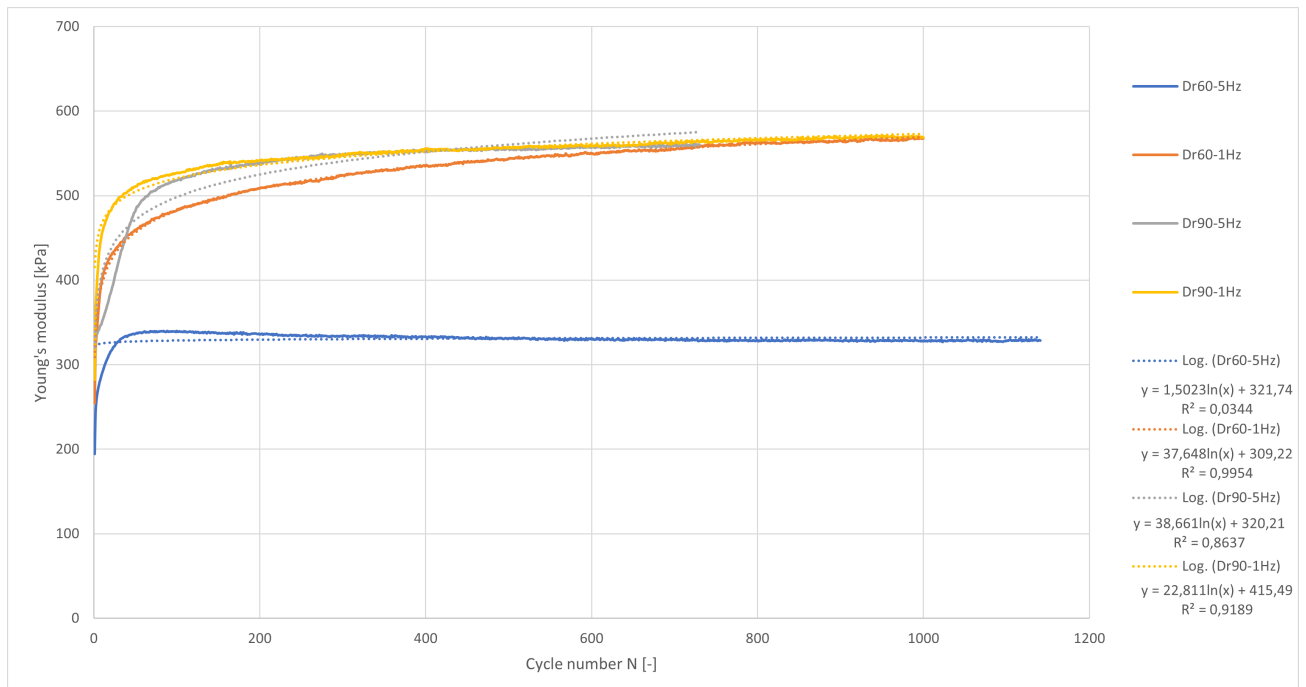


Figure 4.29: Evolution of the young modulus during a cyclic triaxial test for dry sand

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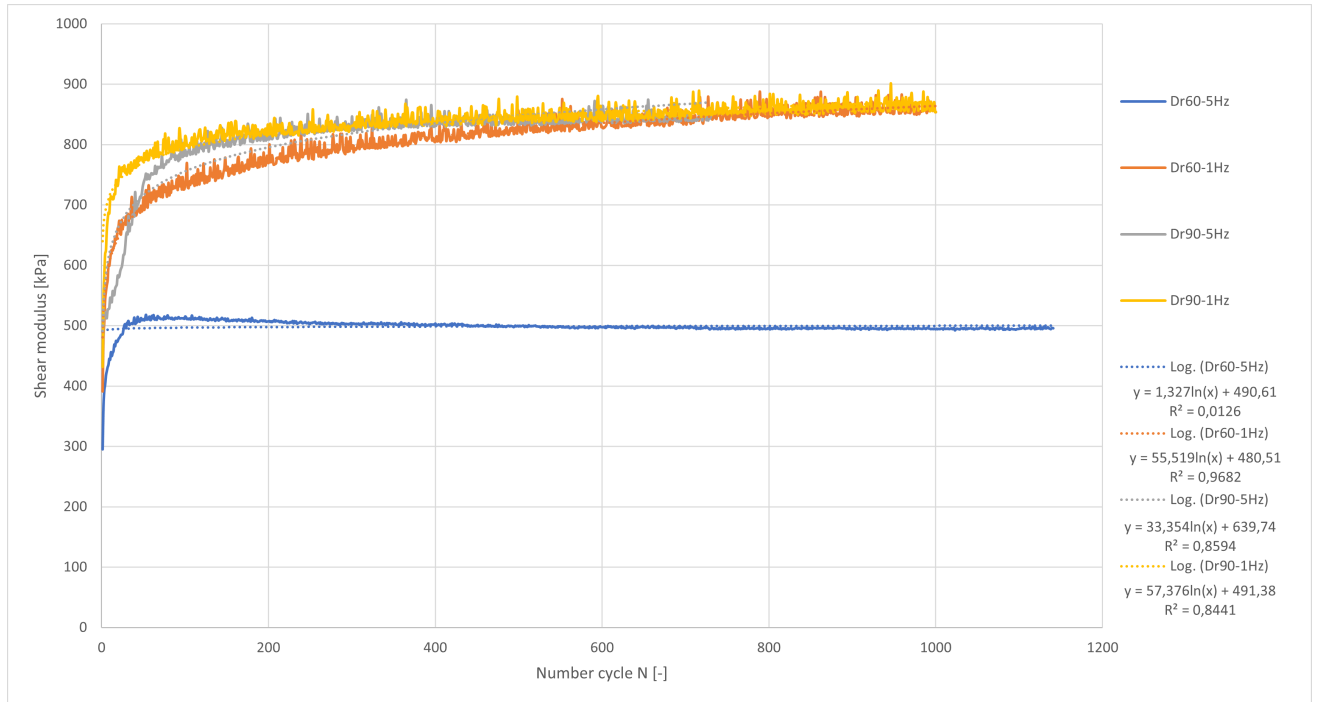


Figure 4.30: Evolution of the shear modulus during a cyclic triaxial test for dry sand

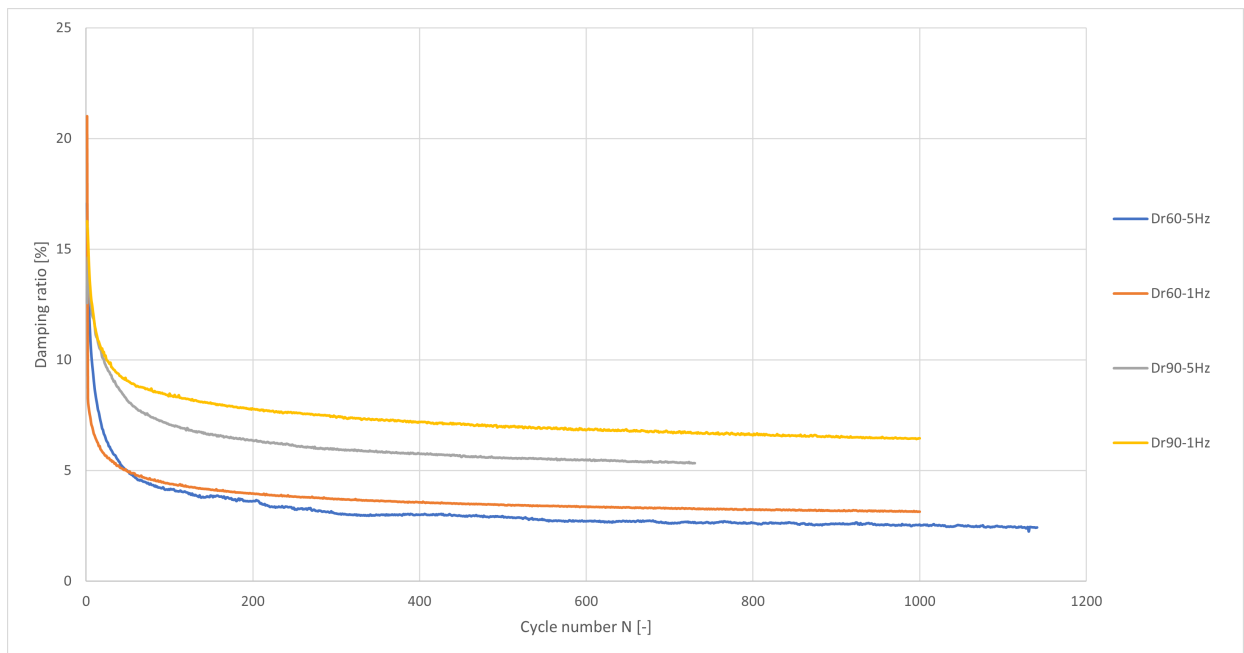


Figure 4.31: Evolution of the damping ratio during a cyclic triaxial test for dry sand

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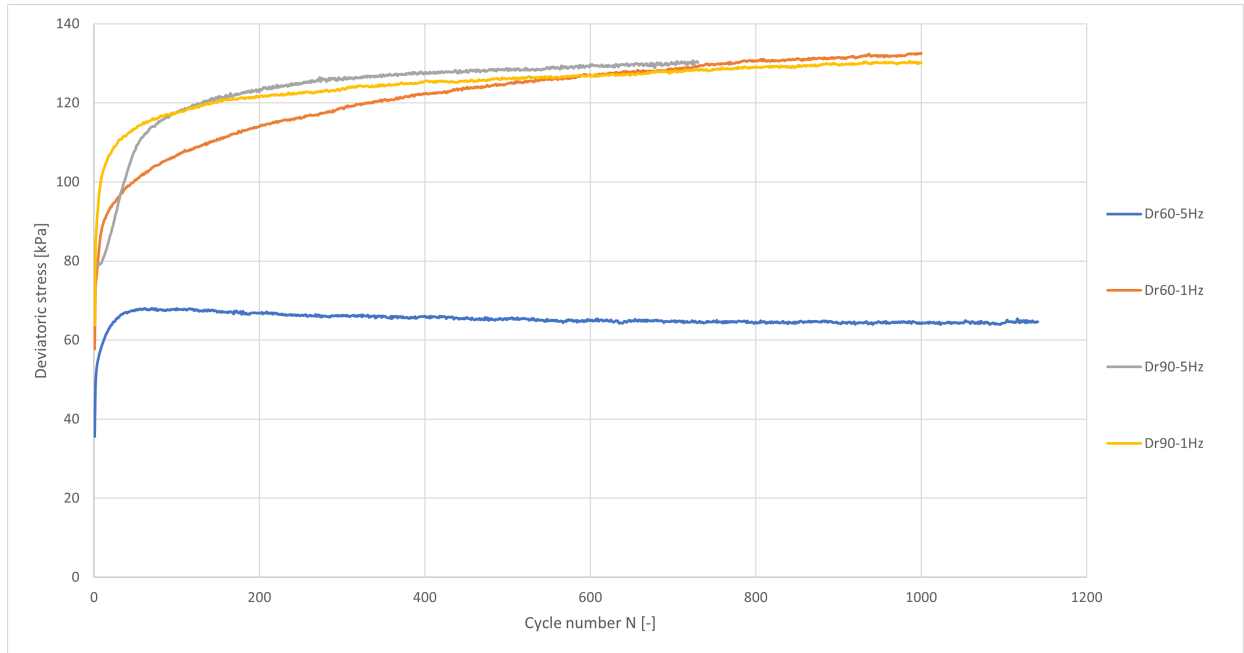


Figure 4.32: Evolution of the deviatoric stress during a cyclic triaxial test for dry sand

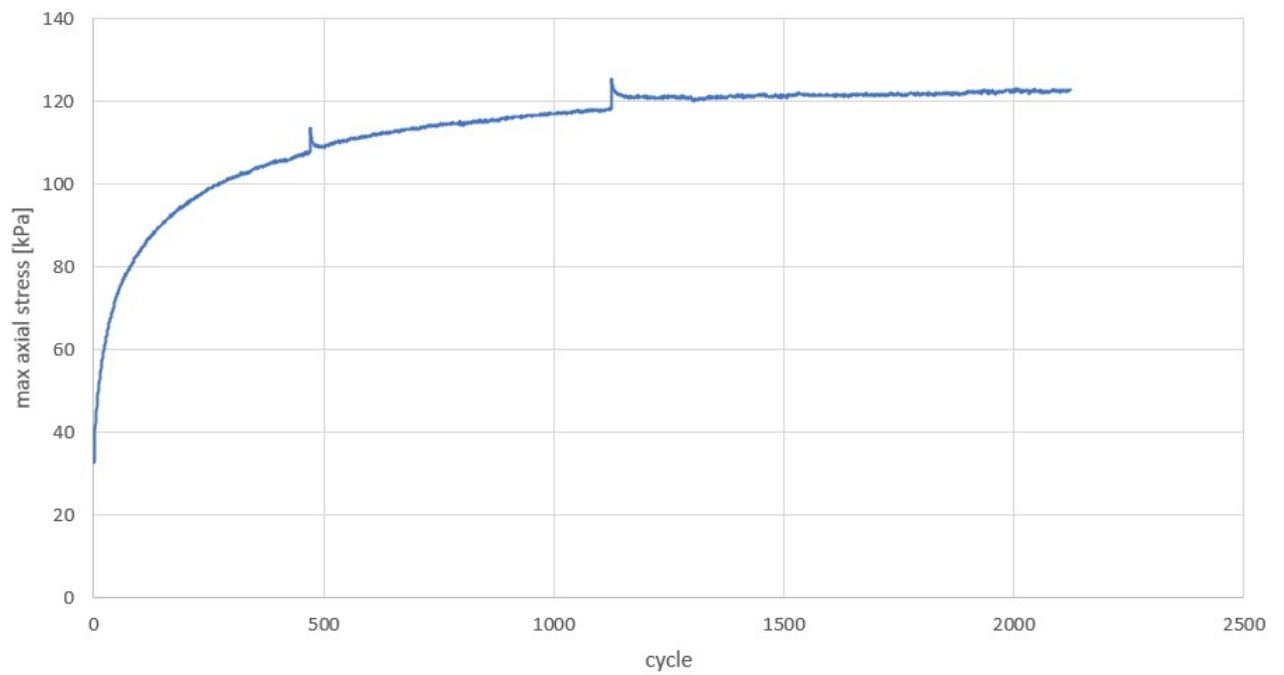


Figure 4.33: Possible healing properties of dry sand during cyclic loading

Soil behaviour under a cyclic loading for saturated sand

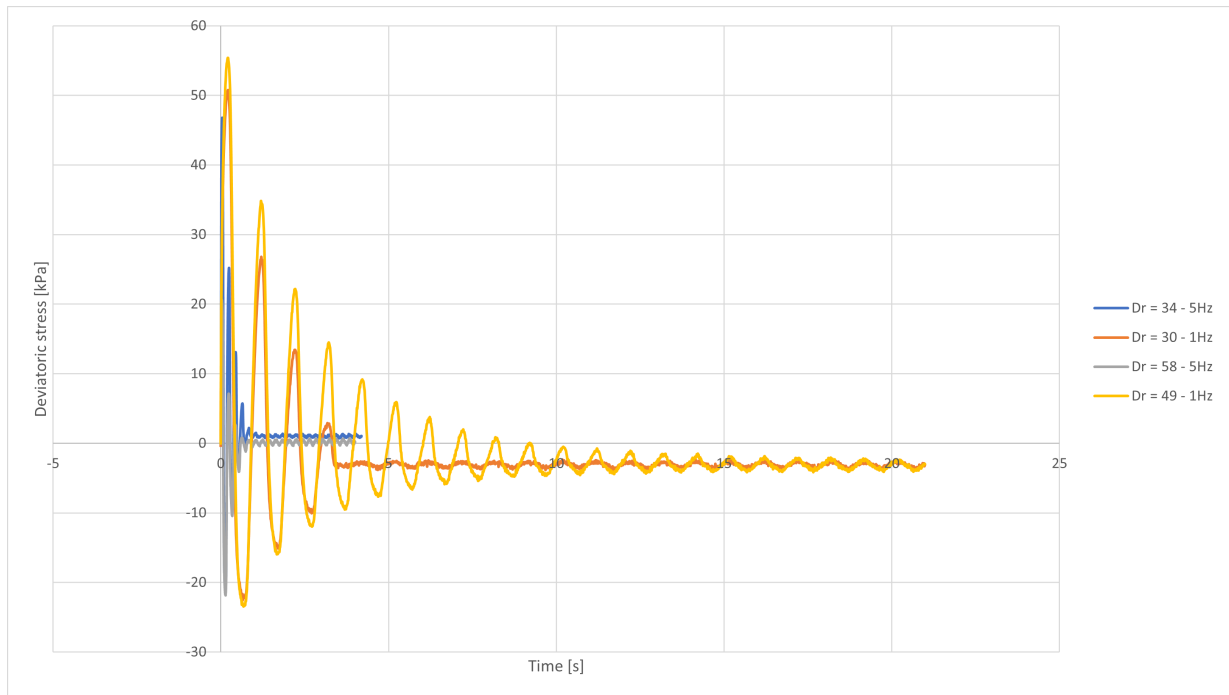


Figure 4.34: Evolution of the deviatoric stress as a function of the time for saturated sand

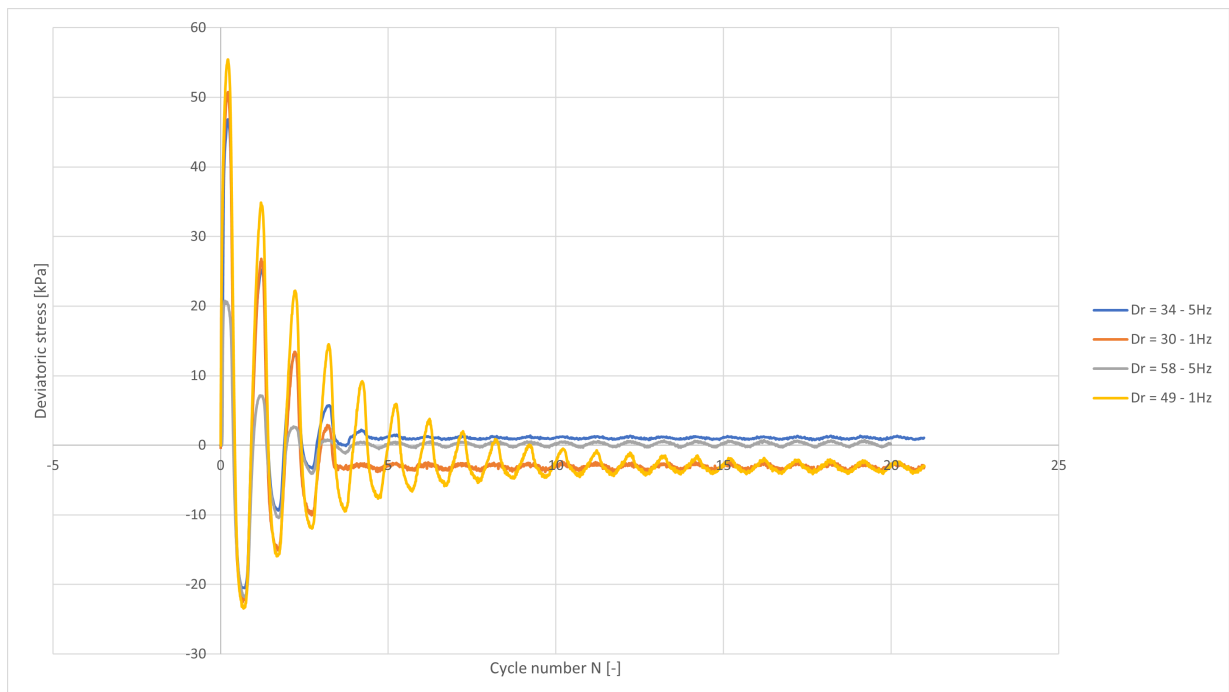


Figure 4.35: Evolution of the deviatoric stress during a cyclic triaxial test for saturated sand

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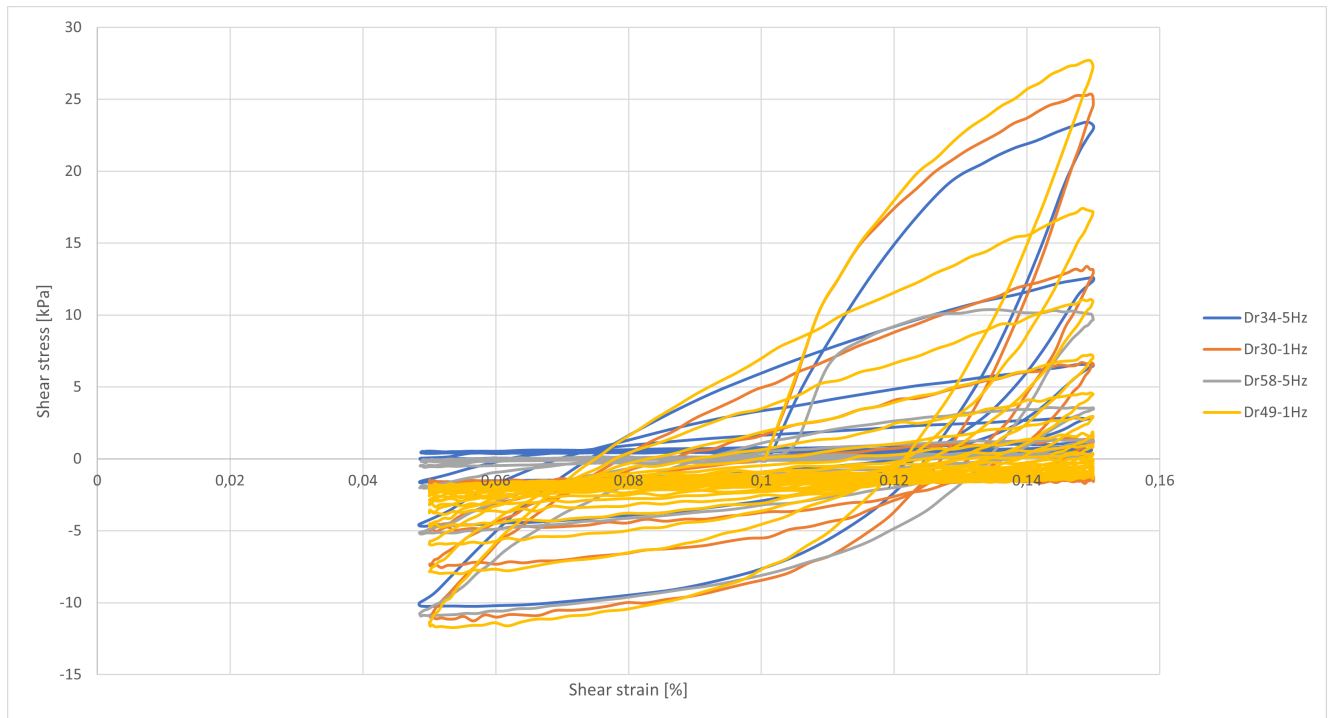


Figure 4.36: Hysteresis loops during a cyclic triaxial test for saturated sand

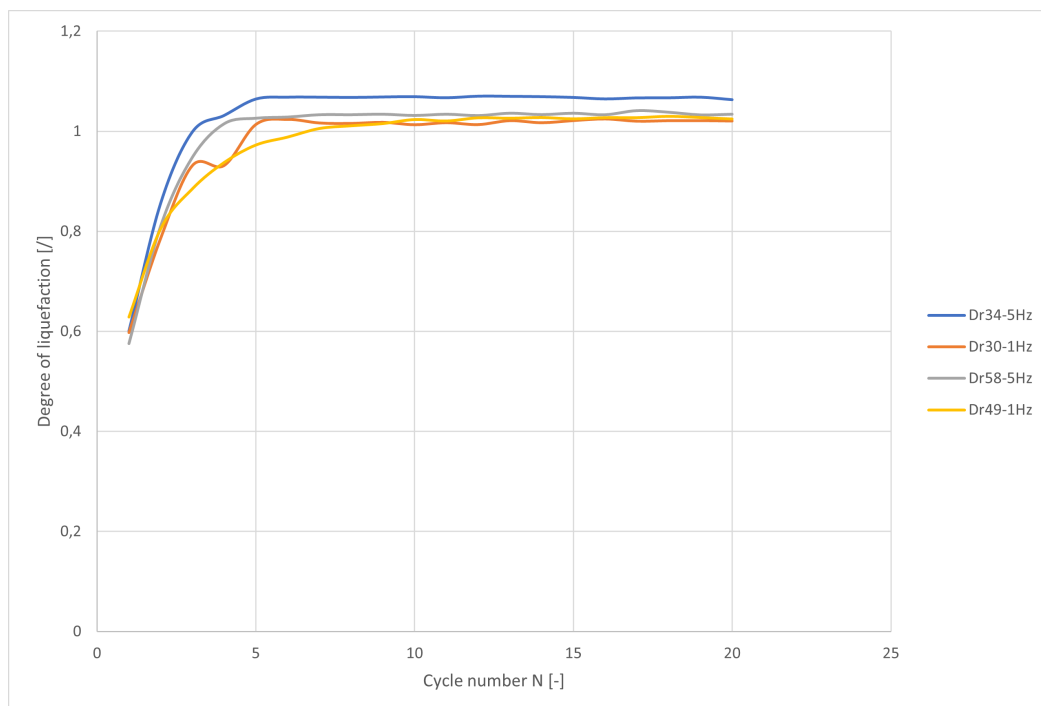


Figure 4.37: Evolution of the degree of liquefaction during a cyclic triaxial test for saturated sand

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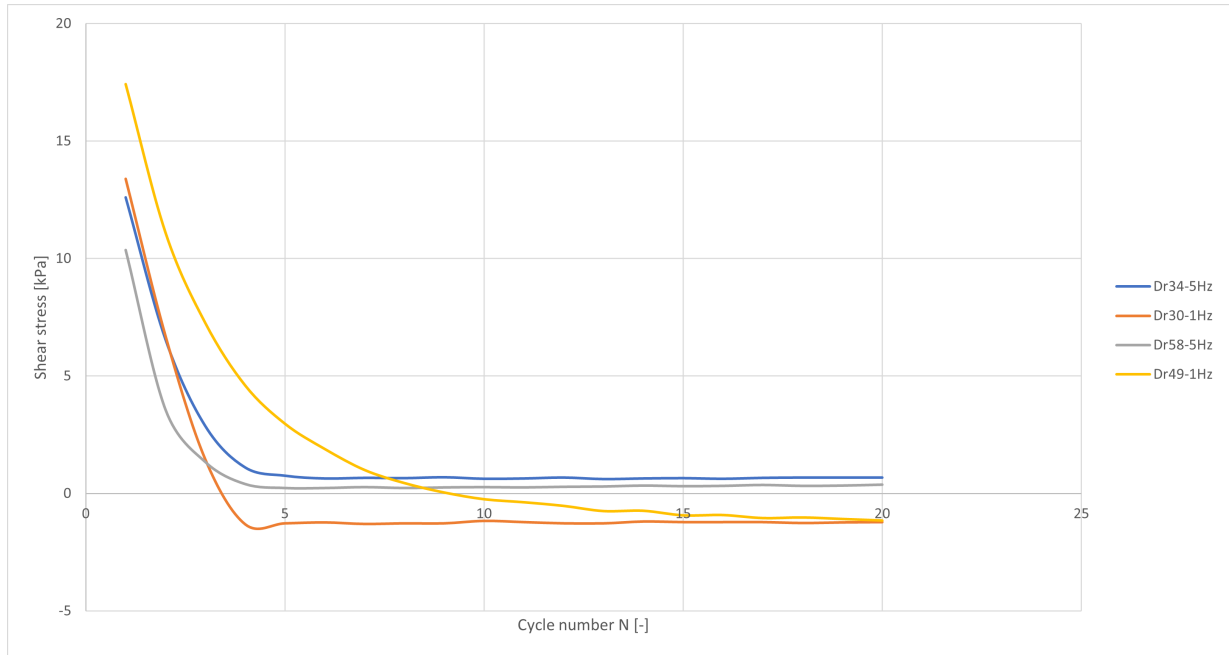


Figure 4.38: Evolution of the shear stress during a cyclic triaxial test for saturated sand

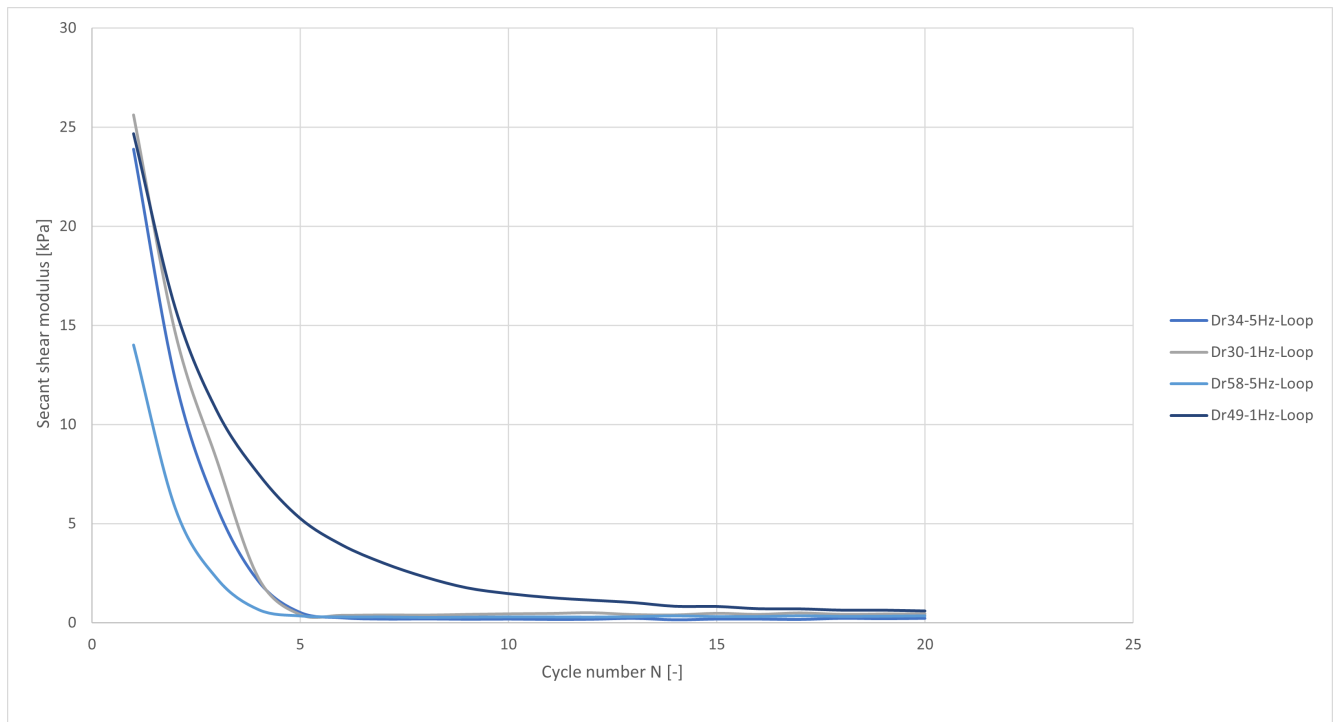


Figure 4.39: Evolution of the secant shear modulus during a cyclic triaxial test for saturated sand

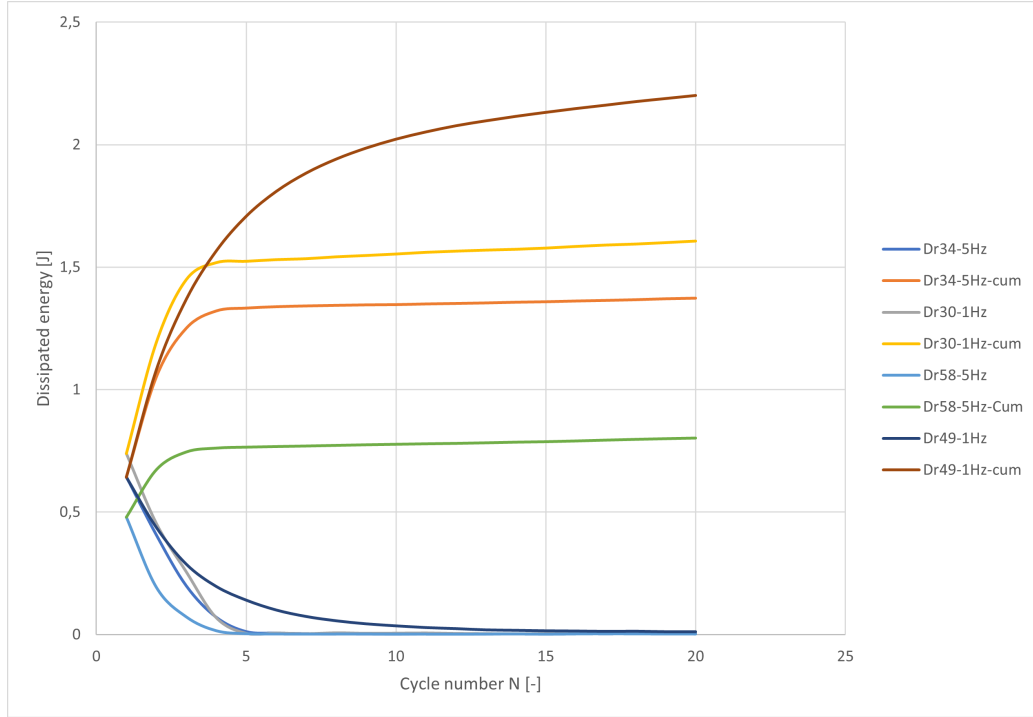


Figure 4.40: Evolution of the dissipated energy during a cyclic triaxial test for saturated sand

Test name	Dense-1Hz	Dense-5Hz	Loose-1Hz	Loose-5Hz
Max stress [kPa]	27.7041	10.3665	25.3749	23.4007
Min stress [kPa]	-11.7291	-10.9166	-11.2615	-10.2513
Shear modulus [MPa]	34.8	20.4	26.8	24.9
Liquefaction [Cycle]	8	4	5	4

Table 4.14: Main characteristics for saturated sand during a cyclic triaxial test

4.5.6 Discussion

Due to their very different behaviour under cyclic loading, dry and saturated samples will be analysed separately.

Graph analysis for dry sand

Cyclic loading in the vertical direction is applied directly on the soil samples using strain-controlled method. The slope of the secant line connecting the extreme point on the hysteresis loop is the dynamic Young's modulus, E , which is given by :

$$E = \frac{\sigma}{\epsilon} \quad (4.46)$$

The shear modulus is expressed as follows :

$$G = \frac{E}{2(1 + \nu)} \quad (4.47)$$

where the Poisson's ratio is equal to 0.4 for dry sands.

The fig. 4.28 shows the evolution of the hysteresis loops between the maximum shear strain γ and the maximum shear stress τ for different frequencies and relative densities. As the sand is dry, the liquefaction do not occur because there is not a variation of the pore pressure in the specimen.

For each frequency and density, the hysteresis loops presents two fixed points and the corresponding values of stresses and strains are the following :

Test name	S-Dr59 1Hz	S-Dr59 5Hz	S-Dr92 1Hz	S-Dr92 5Hz
Shear strain γ_1 [kPa]	0.064	0.189	5.889	0.052
Shear stress τ_1 [kPa]	-1.992	-10.298	-4.461	-3.911
Shear strain γ_2 [kPa]	0.085	0.287	5.924	0.068
Shear stress τ_2 [kPa]	28.135	10.698	69.131	47.891

Table 4.15: Shear strain and stresses for fixed points of hysteresis loops for dry sand

At each cycles excepted the first one, the hysteresis loops cross these points. The fixed points correspond to the disappearance of the shear stresses i.e. when the axial σ'_1 and the radial σ'_3 stresses are equal. At this points, the deformation corresponding to the equilibrium state does not depend on the number of cycles.

The values of shear modulus and young modulus are higher for dense sand and a frequency of 1Hz. It is due to the fact that the extreme values of the hysteresis loops are higher than for loose sand and high frequency. The values of shear and young modulus increase until a plastic phase is reached. At this moment, the curve is constant. The Young and shear modulus can only be calculated in the elastic phase of the cyclic triaxial test.

The fig. 4.31 shows the evolution of the damping ratio during a cyclic triaxial test. For dense sand, the damping ratio is higher than for loose sand. The frequency do not have a great influence on the value of the damping ratio. The damping ratio decreases during the triaxial cyclic test because the dissipated energy decreases and the shear stress increases.

Finally, it can be noted that on fig. 4.33, where the cyclic test was stopped then started again twice, the first few cycles after restarting had a higher resistance. Then the resistance came back following the same trend curve as before the stopping. This phenomenon might be related to sand healing properties, but haven't been explored further due to a lack of time.

Graph analysis for saturated sand

The fig. 4.36 shows the evolution of the hysteresis loops between the maximum shear strain γ and the maximum shear stress τ for different frequencies and relative densities.

The deviator stress is represented by the difference between the major and minor principal stresses which is equal to the axial load applied to the specimen divided by the cross-sectional area of the specimen. The major principal stress in the specimen is equal to the deviator stress plus the cell pressure and the minor principal stress in the specimen is equal to the cell pressure. During the cyclic triaxial test, the pore pressure increases and the effective stresses decrease. The signal is not symmetric and decreases rapidly after the first few seconds and cycles. The deviator stress is higher at low frequency and high relative density. At high frequency, the

deviator stress tends to zero after four cycles. The signal shape (fig. 4.34, fig. 4.35) is similar for time and cycles because the relationship between the frequency and time can be expressed as follows :

$$f = \frac{1}{T} \quad (4.48)$$

The fig. 4.36 shows the evolution of the hysteresis loops between the maximum shear strain γ and the maximum shear stress τ for different frequencies and relative densities. As the cycles progress, the maximum and minimum shear stresses decrease progressively and reach zero after 4 cycles at 5 Hz and after 5 and 8 cycles respectively for loose and dense sand at 1Hz. For each frequency and density, the hysteresis loops presents two fixed points and the corresponding values of stresses and strains are the following :

Test name	Dr34 5Hz	Dr58 5Hz	Dr30 1Hz	Dr49 1Hz
Shear strain γ_1 [kPa]	0.171	0.196	0.114	0.205
Shear stress τ_1 [kPa]	1.659	-0.248	-1.604	-6.346
Shear strain γ_2 [kPa]	0.215	0.239	0.168	0.244
Shear stress τ_2 [kPa]	-0.3	0.117	-2.162	-1.399

Table 4.16: Shear strain and stresses for fixed points of hysteresis loops for saturated sand

At each cycles excepted the first one, the hysteresis loops cross these points. The fixed points correspond to the disappearance of the shear stresses i.e. when the axial σ'_1 and the radial σ'_3 stresses are equal. At this points, the deformation corresponding to the equilibrium state does not depend on the number of cycles.

The maximum and minimum shear strain of each cycle correspond to the minima pore pressure. The sample goes from expansion to contraction when the displacement changes direction. The inflexion point of each branch of the loop correspond to the maxima pore pressure. At this points, the necessary energy to dilate is higher than to contract. During the expansion phase, the specimen volume tries to increase to reach the imposed deformation but the undrained conditions prevent volume change. This translates into a reduction of pore pressure and an increase of stresses. When the deformation direction changes, the stress distribution changes, the stresses between grains decrease and the grains find their place before the expansion. The grains go to an equilibrium state and try to reduce the sample volume and the pore pressure increases. At each additional deformation imposed, the grains are more free and can move without contact with other grains. At this time, no more stress between grains is observed and no more resistance can be obtained. The liquefaction is reached.

The fig. 4.37 shows the evolution of the degree of liquefaction during a cyclic triaxial test. The degree of liquefaction is defined as the ratio between a particular excess pore pressure and the initial effective mean stress imposed to the specimen during the consolidation P'_0 . This definition corresponds roughly to the maximum possible value of degree of liquefaction.

The curve of the degree of liquefaction increases until it reaches a value of 1 or higher depending on the different frequencies. When this value is reached, the effective stresses disappear and no more stresses exist between the grains and the soil loses its strength which tends to zero. This state corresponds to liquefaction and the soil behaves like a liquid. This phenomenon is frequent in saturated, loose and sandy soils cyclically loaded because this soil has a tendency to compress when a load is applied. In response to soil compressing, the pore

water pressure increases, the volume decreases and the water attempts to flow out from the soil to zones of low pressure. There is no shear strength. At the opposite, a dense sands tend to expand in volume.

At 5Hz, liquefaction is reached at the 4th cycle for loose and dense sand while for loose and dense sand at 1Hz, the number of cycles is respectively 5 and 8 cycles.

The fig. 4.38 shows the evolution of the shear stress during a cyclic triaxial test. The shear stress decreases until a value close to zero at 5Hz and below zero at 1Hz. As the pore pressure increases during the cyclic triaxial test, the shear stress decreases because it is the difference between the main stresses.

The fig. 4.39 shows the evolution of the secant shear modulus of the specimen during a cyclic triaxial test. The degradation of the secant shear modulus represents the decrease of the difference between the maximum and minimum shear stresses. The secant shear modulus depends on the deformation amplitude and the number of cycles. It is the slope between two extremities of the hysteresis loop. As the effective stresses decreases with the increase of pore pressure, the secant shear modulus decreases at each cycle until a value close to zero. The soil stiffness deteriorate and the behaviour is non linear. When the curve turns, the assumption of homogeneity of stresses and strains is no more respected because a weakness zone appears at the specimen top.

The degradation is influenced by the number of cycles and the energy dissipated during the cycles. The shear strain amplitude influences strongly the degradation. If the shear strain is high, the degradation occurs faster. The increase of pore pressure causes a decrease of soil stiffness and energy. As shows on fig. 4.40, the dissipated energy for dense sand is higher than loose sand at the same frequency. For dense sand, the grain structures are more reorganised and the grains have more frictional sliding between their surface. This phenomena induce more energy dissipation than for loose sand where the grains are freer. The quantity of energy dissipated by the frictional mechanism is independent of the frequency of loading while the energy dissipated by viscous mechanism is proportional to the frequency of loading.

4.5.7 Bender element method

A bender element is a piezoelectric transducer which can convert electrical energy into mechanical energy and vice versa. This means that whenever these elements are supplied with a voltage they are deforming on a specific way and that when they are deformed, produce a small voltage [65].

As long as the shear strain amplitude of the wave γ_a remains in the linear elastic domain, this method allows the shear wave velocity to be measured and the maximum shear modulus G_{max} to be derived. Indeed, the theory of elasticity allows to define the following relation :

$$G_{max} = \rho \cdot V_s^2 \quad (4.49)$$

G_{max} is an important soil property which helps to understand the elastic behaviour of the soil and to evaluate its response to dynamic loading as vibrations. The maximum shear modulus is given at very small value of strain and can be obtained only with a bender element test or resonance column.

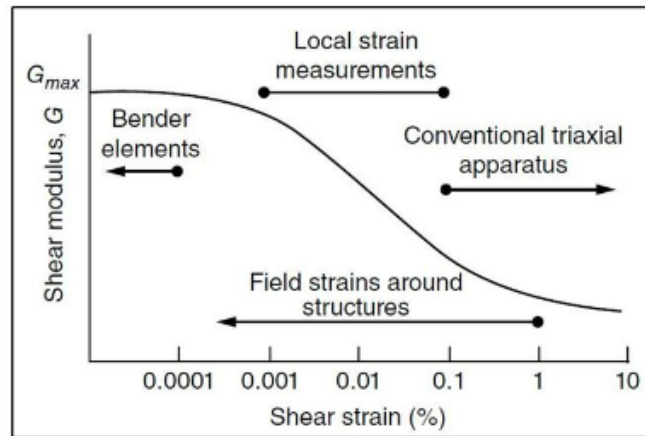


Figure 4.41: Various test available to determine the shear modulus of a soil [65]

Shear wave velocity is measured using bending elements consisting of two thin piezoceramic plates. These are rigidly bonded together in such a way that, when tension is applied to the ends of the elements, one plate expands while the second contracts. These deformations cause the elements to bend. The two bending elements (a shear wave transmitter and receiver) are placed at opposite ends of the sand sample (fig. 4.42). A voltage pulse is applied to the transmitter inducing a shear wave which then reaches the receiver at the other end of the specimen (fig. 4.43). This wave induces distortion in the receiver element and produces another voltage pulse. The input and received voltages are used to determine the time taken for the wave to travel the distance between the tips of the bending elements and thus to define the speed of the shear wave. Two other devices are also needed to perform the Bender Element test :

- A function generator to create the sinusoidal curve of the transmitting signal and that produce the deformation of the transmitting element.
- A digital oscilloscope to record the signal captured by the receiving element. It synchronises the two electrical signals in order to be comparable.

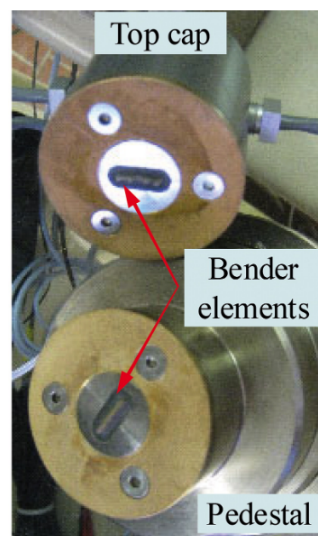


Figure 4.42: Position of bender elements on the specimen [41]

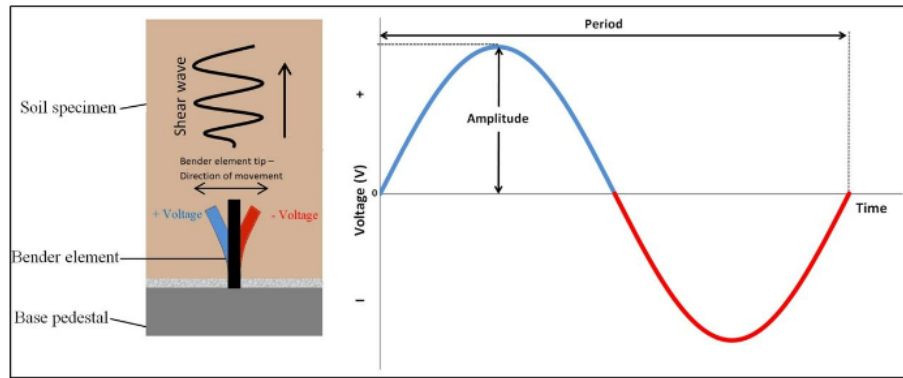


Figure 4.43: Bender element deformation and propagation of the shear wave through the sample [65].

The amplitude of the shear deformation generated by the bending elements is not a fixed value. It depends on the stiffness of the soil and varies along the specimen due to the damping of the waves.

As the bending element test is not a destructive test, it can be combined with other types of tests as cyclic triaxial test.

The preparation, saturation and consolidation of the specimen are the same as for the cyclic triaxial test but two thin piezoceramic plates are placed above and below the sample to measure the shear wave velocity and shear modulus. The two bender elements, transmitter and receiver, must be parallel in order to achieve a good quality received signal. At any phase, a Bender Element test can be done and the small-strain shear modulus G_{max} can be measured.

The Bender Element test starts by providing a sine wave signal to the transmitting element. The element is deforming towards one side, then to the other and finally comes back to the starting position. The magnitude of this deformation is controlled by the amplitude of the signal which usually ranges between 1 V and 12 V. If the received signal is not captured very clearly by the receiver, the amplitude is increased. The transmitter and receiver operate in the same frequency to have comparable results. The software combine the repeated signals to produce an average waveform which is clean of any random noise that is affecting it. The Peak-to-Peak method is used to calculate the time travel and the shear wave velocity [65].

Results

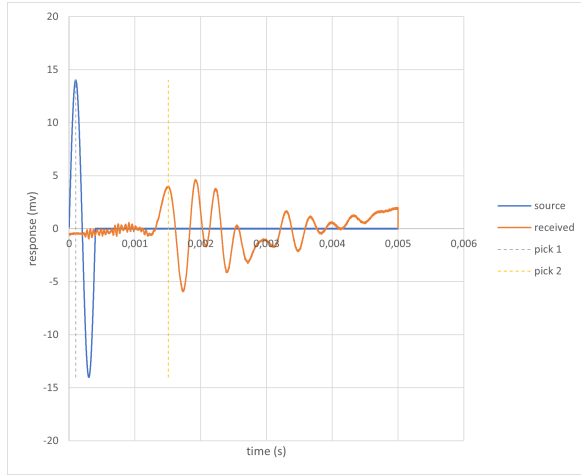


Figure 4.44: Bender element - P04 - S6 - SATURATION

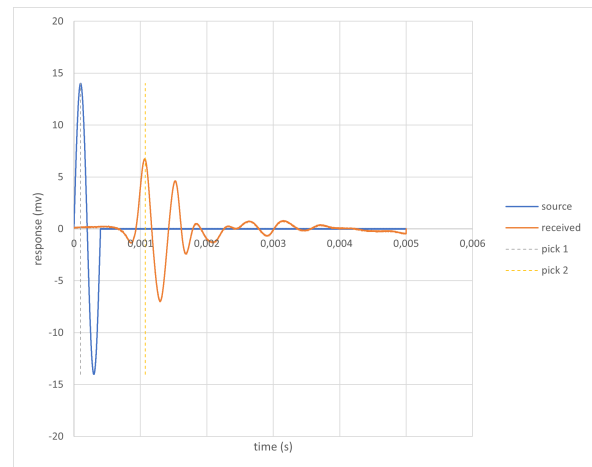


Figure 4.45: Bender element - P04 - S3 - CONSOLIDATION

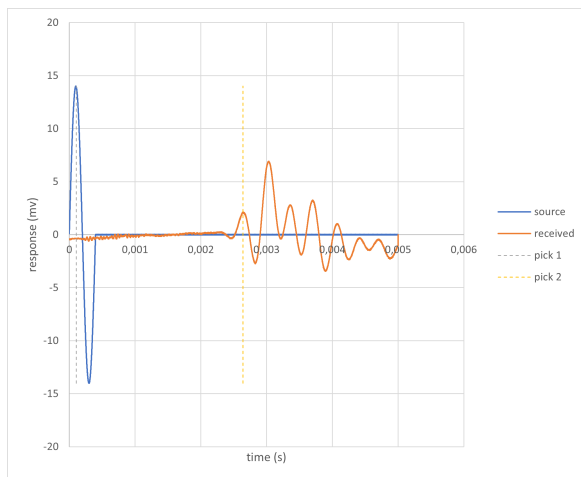


Figure 4.46: Bender element - P04 - S6 - CYCLIC

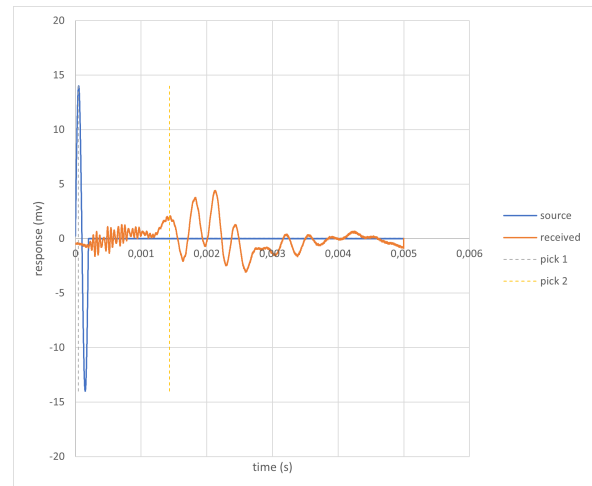


Figure 4.47: Bender element - P05 - S7 - SATURATION

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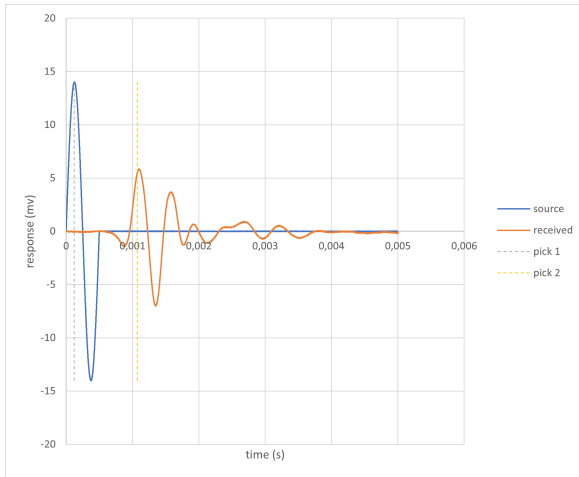


Figure 4.48: Bender element - P04 - S6 - CONSOLIDATION

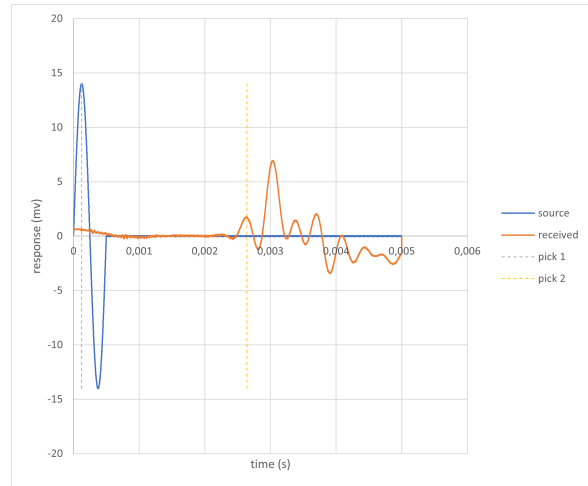


Figure 4.49: Bender element - P05 - S4 - CYCLIC

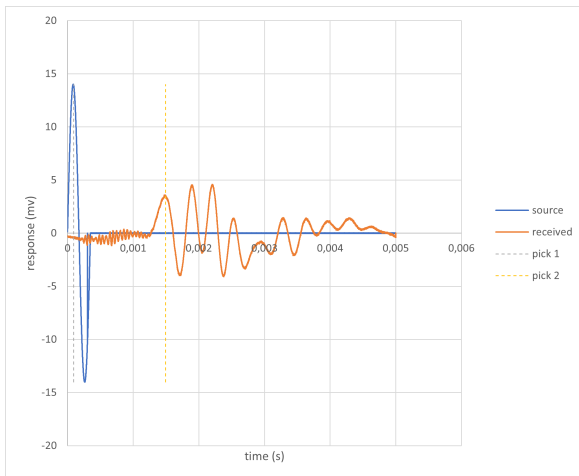


Figure 4.50: Bender element - P035 - S5 - SATURATION

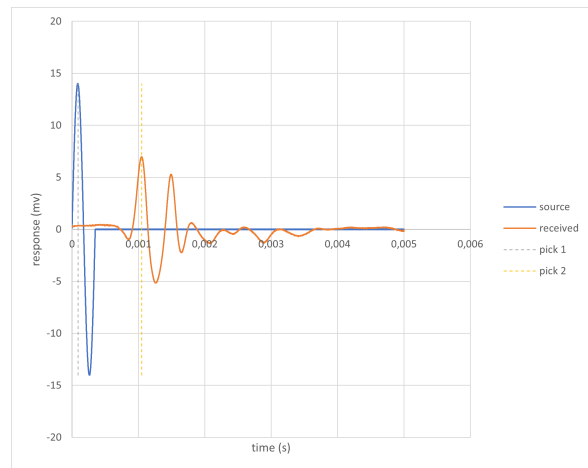


Figure 4.51: Bender element - P035 - S5 - CONSOLIDATION

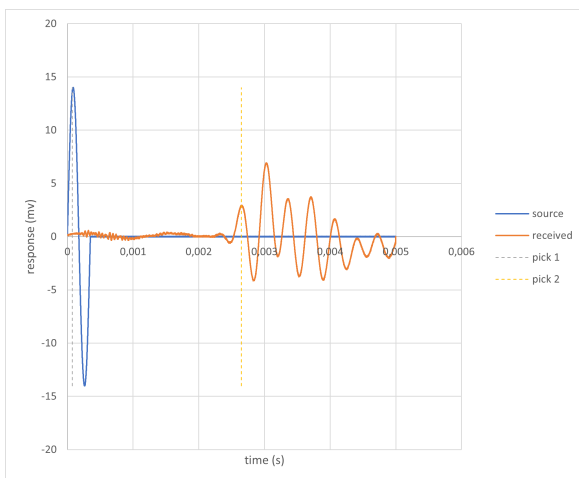


Figure 4.52: Bender element - P035 - S5 - CYCLIC

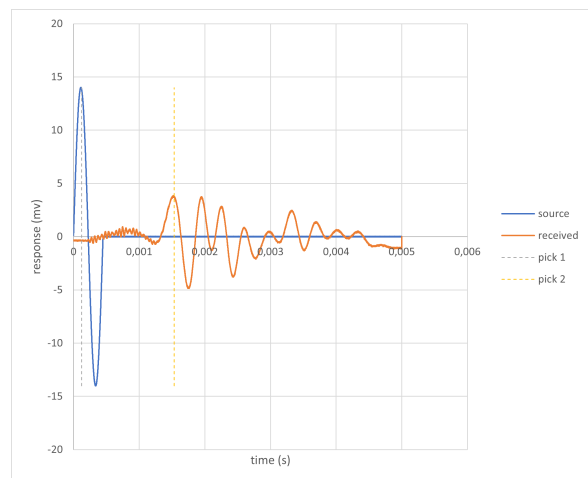


Figure 4.53: Bender element - P045 - S6 - SATURATION

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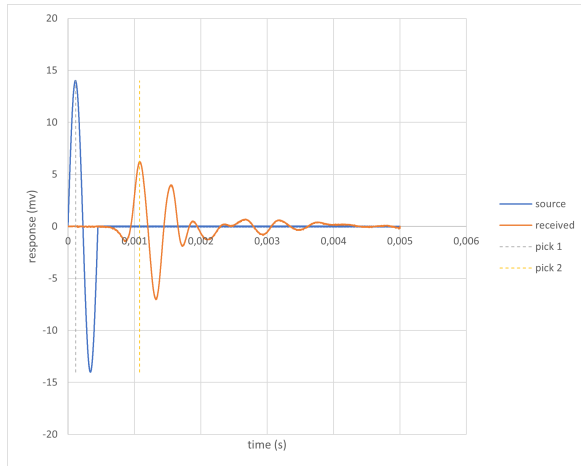


Figure 4.54: Bender element - P045 - S2 - CONSOLIDATION

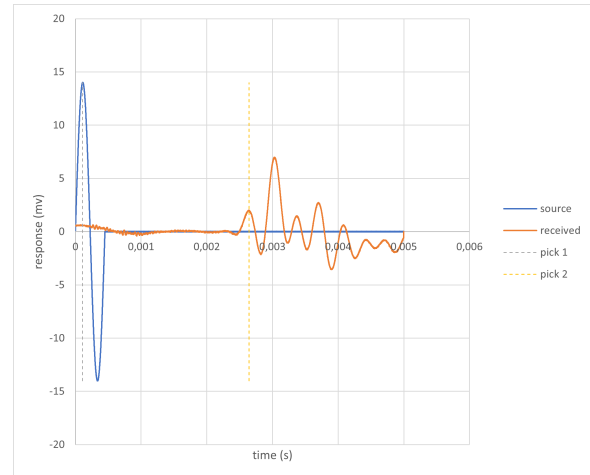


Figure 4.55: Bender element - P045 - S5 - CYCLIC

Test name	BE-P04-S6-SAT	BE-P04-S3-CON	BE-P04-S6-CYC
First pick [ms]	0.0995	0.0995	0.1075
Second pick [ms]	1.5085	1.072	2.6415
Travel time [ms]	1.409	0.9725	2.534
Wave velocity [m/s]	95.1	137.79	52.88
G [MPa]	17.03	35.75	5.27

Table 4.17: Main characteristics with bender elements for a period of 4 ms for saturation, consolidation and cyclic phases

Test name	BE-P05-S7-SAT	BE-P05-S2-CON	BE-P05-S4-CYC
First pick [ms]	0.046	0.1225	0.1225
Second pick [ms]	1.432	1.1025	2.6415
Travel time [ms]	1.386	0.98	2.519
Wave velocity [m/s]	96.68	136.735	53.20
G [MPa]	17.6	35.21	5.33

Table 4.18: Main characteristics with bender elements for a period of 5 ms for saturation, consolidation and cyclic phases

Test name	BE-P035-S5-SAT	BE-P035-S5-CON	BE-P035-S5-CYC
First pick [ms]	0.092	0.092	0.0765
Second pick [ms]	1.493	1.049	2.649
Travel time [ms]	1.401	0.957	2.5725
Wave velocity [m/s]	95.65	140.02	52.09
G [MPa]	17.23	36.92	5.11

Table 4.19: Main characteristics with bender elements for a period of 35 ms for saturation, consolidation and cyclic phases

Test name	BE-P045-S6-SAT	BE-P045-S2-CON	BE-P045-S5-CYC
First pick [ms]	0.1225	0.115	0.1075
Second pick [ms]	1.5315	1.0795	2.6415
Travel time [ms]	1.409	0.9645	2.534
Wave velocity [m/s]	95.1	138.93	52.88
G [MPa]	17.03	36.35	5.27

Table 4.20: Main characteristics with bender elements for a period of 45 ms for saturation, consolidation and cyclic phases

Discussion

During the consolidation phase, the shear wave velocity is higher than for saturation and cyclic phases for all time period and the shear modulus is maximal.

The sand raining system

5.1 Description and definition of the process

In order to carry out small-scale laboratory tests, a sand raining system has been set up to reproduce a sandy soil that reflects as closely as possible the properties of natural sand deposits. This method allows the relative density to be controlled and maximised and relatively homogeneous sand samples can be obtained repeatedly. Thanks to this system, very reliable test results can be obtained. To achieve this, there are two methods :

- The stationary pluviator, which contains one or more sieves or meshes to distribute the flow of sand from the hopper to the desired area;
- The mobile pluviator which moves around the tank and creates a deposit at its bottom by means of an opening and without using a sieve. It does not achieve a relative density greater than 70-90%.

Rainfall consists of a rain of cohesionless soil particles in a mould. It is obtained by gravity and with the help of appropriate diffusers. To obtain a relatively homogeneous sand, the system is composed of a hopper to store the sand, a plate with orifices to vary the intensity of the deposit, and two diffusion sieves with a capacity to produce uniform layers of sand in a rapid manner. These sieves or metal grids have different opening widths and are oriented at 45° in the vertical axis to each other. When testing, it is important to ensure that there is no obstruction of the diffuser before the sand settles in the tank and that the minimum opening widths are respected [7].

Through this rainfall principle, a desired relative density can be obtained. This is an important parameter governing the behaviour of the soil and therefore of the pile during the subsequent tests. A high relative density makes it possible to obtain uniform and repetitive sand beds. The placement of sand by rainfall is therefore a critical process as it must be as uniform as possible.

The main factors influencing the relative density are :

- The height of the sand fall
- The width of the sieve openings
- The rate and speed of fall of the sand particles/ deposition intensity
- The relative diffusion ratio

The drop height of the sand is the distance between the bottom plate and the top of the sand bed. This can be adjusted by lowering or raising the rainfall system using pulleys or chains.

The deposition intensity is the weight of soil falling per unit area and time. This is regulated by varying the effective area of the hopper through which the sand exits.

The relative diffusion ratio is the ratio between the opening width of the sieve and the average or maximum grain size [49].

When the process is started, the speed of the sand particles increases non-linearly with the drop height until a constant final speed is reached. However, independently of the drop height, the velocity increases linearly with increasing particle size. This velocity is reached when the gravitational forces equalise the drag forces [35].

Furthermore, the relative density is directly related to the kinetic energy at the moment of impact. Before reaching their limiting drop height, sand particles increase their kinetic energy as their drop height increases, which leads to an increase in relative density. The limiting drop height is reached when a further increase in height becomes negligible. At this point, the particles have reached their terminal velocity [68].

On the other hand, if the width of the openings is large, the falling velocity of the sand particles decreases, which results in a decrease of the kinetic energy and thus of the relative density. This loss of kinetic energy is related to the interference between particles when a large number of particles fall simultaneously. The relative ratio of the scatterers affects the interference between the particles at the outlet of the scatterers and therefore the relative density according to the same theory mentioned above. If the ratio is low, the sand will be more uniformly distributed as the relative density increases. However, an increase in the number of diffusers for a fixed drop height and deposition intensity leads to a decrease in the relative density. If the deposition intensity is too high, there is a greater risk of sand accumulation on the diffusers and this prevents free circulation of the sand, resulting in non-uniformity of the sand in the tank [34],[45],[48].

In this thesis, the sand raining system is used to test different heights to obtain low and high relative density. Subsequently, the relative densities obtained for a fixed drop height are used to compare the behaviour of the impact driven into loose and dense sand.

5.2 Equipment required

- Three plates with holes with different diameters. In our case, the plates have holes of 13 mm, 8 mm and 6 mm.
- 12 iron angles
- 4 metal strips
- A pulley
- A winch
- A balance
- A bucket
- A meter
- Several small containers



Figure 5.1: Sand raining system

5.3 Operational procedure

1. Weigh the small containers and measure their volume.
2. Place the small containers at the bottom of the tank.



Figure 5.2: Position of small containers

3. Fill the buckets with sand and weigh them so that the density in the tank can be compared with that in the containers. Empty them through the sand raining system.
4. Measure the distance between the last plate at the bottom of the rainfall system and the layer of sand that is gradually forming at the bottom of the tank. This distance should remain constant throughout the experiment until the sand reaches the top of the containers. If this is not the case, adjust the distance using the winch.
5. Measure the height of sand reached at different points to obtain an average height for the calculation of the tank volume.
6. Empty the tank and repeat the experiment for different drop heights.

5.4 Calculation and expression of results

The relative density is calculated as follows :

$$D_R = \frac{\gamma_{d,max} \cdot (\gamma_d - \gamma_{d,min})}{\gamma_d \cdot (\gamma_{d,max} - \gamma_{d,min})} \cdot 100 \quad \% \quad (5.1)$$

The values of $\gamma_{d,max}$ and $\gamma_{d,min}$ are calculated in section 4.2.

5.5 Results

The results for a drop height of 15 cm for the small containers are represented in table 5.1 :

Container	Sand mass [kg]	Diameter [cm]	Height [cm]	Volume [cm^3]	Relative density [%]
Container 1	4.41	15.38	15.05	2797.22	2.1474
Container 2	4.343	15.32	14.95	2754.61	2.179
Container 3	4.386	15.37	15	2781.89	2.176
Container 4	4.356	15.28	15.05	2760.98	2.674
Container 5	4.351	15.35	14.9	2757.36	2.792
Container 6	4.305	15.33	14.9	2751.37	3.383
Container 7	4.378	15.35	15	2775.86	2.427
Container 8	4.337	15.33	14.9	2751.37	2.028

Table 5.1: Relative density obtained for a height of drop of 15 cm

The results for a drop height of 30 cm for the small containers are represented in table 5.2 :

Container	Sand mass [kg]	Diameter [cm]	Height [cm]	Volume [cm^3]	Relative density [%]
Container 1	4.47	15.38	15.05	2797.22	11.9165
Container 2	4.403	15.32	14.95	2754.61	12.0965
Container 3	4.448	15.37	15	2781.89	12.3199
Container 4	4.415	15.28	15.05	2760.98	12.3924
Container 5	4.422	15.35	14.9	2757.36	14.4673
Container 6	4.394	15.33	14.9	2751.37	11.4708
Container 7	4.435	15.35	15	2775.86	11.7768
Container 8	4.397	15.33	14.9	2751.37	12.3002

Table 5.2: Relative density obtained for a height of drop of 30 cm

5 – The sand raining system

The results for a drop height of 60 cm for small containers are represented in table 5.3 :

Container	Sand mass [kg]	Diameter [cm]	Height [cm]	Volume [cm^3]	Relative density [%]
Container 1	4.574	15.38	15.05	2797.22	28.2425
Container 2	4.511	15.32	14.95	2754.61	29.2828
Container 3	4.569	15.37	15	2781.89	31.3247
Container 4	4.567	15.28	15.05	2760.98	36.2742
Container 5	4.513	15.35	14.9	2757.36	28.8942
Container 6	4.512	15.33	14.9	2751.37	30.2607
Container 7	4.544	15.35	15	2775.86	29.004
Container 8	4.503	15.33	14.9	2751.37	28.8623

Table 5.3: Relative density obtained for a height of drop of 60 cm

The results for a drop height of 90 cm for small containers are represented in table 5.4 :

Container	Sand mass [kg]	Diameter [cm]	Height [cm]	Volume [cm^3]	Relative density [%]
Container 1	4.651	15.38	15.05	2797.22	39.8596
Container 2	4.58	15.32	14.95	2754.61	39.8387
Container 3	4.649	15.37	15	2781.89	43.3466
Container 4	4.618	15.28	15.05	2760.98	43.9342
Container 5	4.606	15.35	14.9	2757.36	43.0492
Container 6	4.564	15.33	14.9	2751.37	38.2326
Container 7	4.616	15.35	15	2775.86	39.9372
Container 8	4.569	15.33	14.9	2751.37	38.9896

Table 5.4: Relative density obtained for a height of drop of 90 cm

These results can be compared with the relative density values obtained by taking into account the filled volume of the tank on a sand height varying between 15 and 20 cm. The table 5.5 shows the different relative densities obtained as a function of the height of fall.

Height of drop [cm]	Relative density [%]
15	10.8654
30	44.3769
60	82.2705
90	85.8499

Table 5.5: Relative density obtained for all height of drop for the volume of the tank on a sand height varying between 15 and 20 cm

5.6 Discussion

In this thesis, only the influence of the drop height on the relative density was tested. The other parameters of the raining system such as the diameter of the openings, the number of plates and openings are fixed and identical for all experiments.

Looking at the different results, it appears that the relative density is mostly influenced by the drop height rather than by the position of the containers in the tank. This shows that this

method of raining sand allows a relatively homogeneous relative density to be obtained over the entire surface of the tank and in a repeated manner.

Furthermore, when comparing the different tables, the relative densities increase with the drop height. This is because sand particles acquire more kinetic energy at higher drop heights.

Finally, an important difference can be observed between the relative densities obtained in the small containers and in the tank. At present, no plausible explanation is given for this. However, if the relative densities in the tank are taken into consideration, it can be observed on fig. 5.3 that at heights of 60 and 90 cm, the relative densities increase slightly. If a height slightly higher than 90 cm is taken, the limit height would be reached. At this point, an increase in drop height will have a negligible influence on the relative density value.

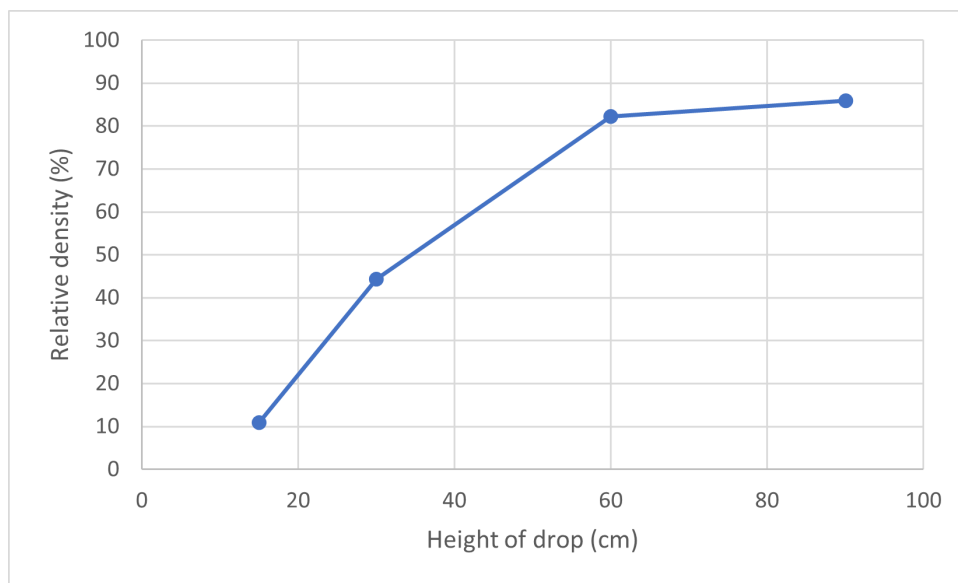


Figure 5.3: Relative density in the tank for a sand height between 17 and 20 cm in function of drop height

For further experiments, especially during pile driving, two types of relative densities will be used to compare their influence on the pile behaviour. A relative density corresponding to a loose soil and one corresponding to a dense soil. To achieve this, a drop height of 30 and 60 cm will be used. Indeed, during the tests with the rainfall system, these heights gave interesting results and were sufficiently distant to have an insight into the influence of the relative density later on.

6.1 Drop hammer

The drop efficiency can be calculated as the equilibrium between the kinetic and potential energy :

$$\eta_c = \frac{W_c}{Mgh_d} = \frac{1}{2} \frac{v_i^2}{gh_d} \quad (6.1)$$

6.2 Stress wave theory

The impact pile driving can be analysed from stress waves created in the pile that run through it.

ASSUMPTIONS

- The pile is discretized in small elements to design the stress wave propagation in the pile.
- The stress wave speed is constant.
- Cross-section remains plane during deformation.
- As the input forces are only vertical, the stress waves also have only a vertical direction.
- Each pile segment has an undisturbed stress wave transmission.
- Unit compression ϵ_x due to the axial displacement u : $\epsilon_x = \frac{du}{dx}$

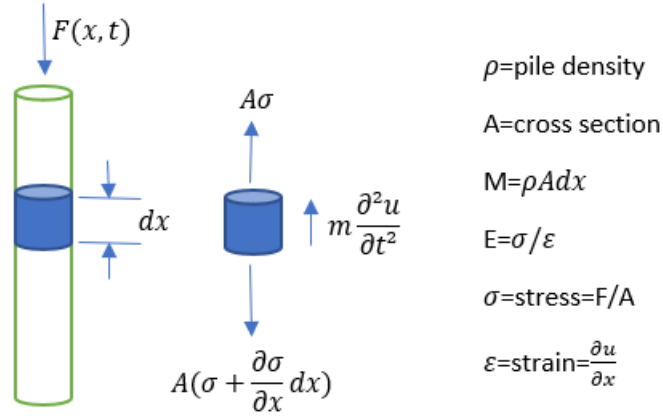


Figure 6.1: Stress wave forces on an infinitesimal element

Assume that the axial displacement is a function of the x-position of the defined point relative to the pile in its initial situation and the time t :

$$u_x = u_x(x, t) \quad (6.2)$$

If the pile material is assumed to be elastic, its behaviour is described by Hooke's law of axial compression and axial compressive stress :

$$\sigma = E \cdot \epsilon = E \cdot \frac{\partial u}{\partial x} \quad (6.3)$$

If we consider a free pile element between two cross-sections at x and x+dx, the axial translation equilibrium is written :

$$A \left(\sigma + \frac{\partial \sigma}{\partial x} dx \right) = A \cdot \sigma + ma \quad (6.4)$$

The Newton's law of motion gives :

$$F = ma = \rho A \cdot dx \frac{\partial^2 u}{\partial t^2} \quad (6.5)$$

The corresponding compressive force on the pile is written :

$$F = \sigma \cdot A = EA \cdot \frac{\partial u}{\partial x} \quad (6.6)$$

By combining the previous equation in eq. (6.4), the following equation is obtained :

$$EA \cdot \frac{\partial^2 u}{\partial x^2} dx = \rho A \cdot \frac{\partial^2 u}{\partial t^2} dx \quad (6.7)$$

$$\frac{\partial^2 u}{\partial x^2} - \frac{\rho}{E} \frac{\partial^2 u}{\partial t^2} = 0 \quad (6.8)$$

The following equation is the general equation of stress wave :

$$\frac{\partial^2 u}{\partial x^2} - \frac{1}{c^2} \frac{\partial^2 u}{\partial t^2} = 0 \quad (6.9)$$

The stress wave propagation speed is expressed as follows :

$$c = \sqrt{\frac{E}{\rho}} \quad (6.10)$$

By substitution in the wave equation, two fundamental equations appear from the second order homogeneous differential equation with partial derivatives :

$$f_1(z + v_c \cdot t) = f^+ \quad (6.11)$$

$$f_2(z - v_c \cdot t) = f^- \quad (6.12)$$

The first expresses a displacement waveform translating unchanged at constant speed $-v_c$ and the second at speed $+v_c$ along the x-axis.

6.3 Method of characteristics

The solution to a problem whose geometry, materials, boundary and initial conditions are known can be obtained by the superposition of downward and upward moving waves generated at particular points. These points have the particularity of modifying the incident wave and creating a reflected wave. This method is the method of characteristics known for solving partial differential equations.

The following solution considers that two waves travel in opposite directions :

$$u = f(x + v_c t) + f(x - v_c t) \quad (6.13)$$

The previous equation can be derived regarding time to obtain the velocity :

$$v = \frac{\partial u}{\partial t} = \frac{\partial f}{\partial (x + v_c t)}(v_c) + \frac{\partial f}{\partial (x - v_c t)}(-v_c) \quad (6.14)$$

By convention, the following velocities are obtained :

$$v^+ = \frac{\partial f}{\partial (x + v_c t)}(+v_c) \quad (6.15)$$

$$v^- = \frac{\partial f}{\partial (x - v_c t)}(-v_c) \quad (6.16)$$

The force can be expressed as follows :

$$F = -EA \frac{\partial u}{\partial x} = -EA \frac{\partial f}{\partial (x - v_c t)} - EA \frac{\partial f}{\partial (x + v_c t)} \quad (6.17)$$

By convention, the following forces are obtained :

$$F^- = -EA \frac{\partial f}{\partial (x - v_c t)} \quad (6.18)$$

$$F^+ = EA \frac{\partial f}{\partial (x + v_c t)} \quad (6.19)$$

The pile impedance is expressed as follows :

$$Z = \frac{EA}{v_c} = A\sqrt{E\rho} \quad (6.20)$$

The forces can be also expressed as follows :

$$F^- = Zv \quad (6.21)$$

$$F^+ = -Zv \quad (6.22)$$

As said in the assumptions, the stress wave has a constant speed and the transmission is undisturbed within each pile segment. This means that the only interaction with external forces can be designed at the interface. The soil interaction is modeled by a spring, a dashpot and a mass.

6.3.1 Stress wave boundary conditions

After an impact force due to the hammer drop, a stress wave propagates back and forth along the pile until it no longer has enough energy to propagate. The generated stress wave propagates downwards until pile toe at L/c where L is the pile length and c the constant speed. After, the wave is reflected and propagate upwards until reaching the pile top.

Condition	Model description
Free end $F(x=L,t)=0$	No friction and no bottom resistance
Fixed end $v(x=L,t)=0$	No friction and full bottom resistance
Soil friction F_F	Friction interface

Table 6.1: Main boundary conditions

The initial conditions in $x=0$ and $t=0$ can be described as follows :

$$F^+(x, t) = 0$$

$$F^-(x, t) = F_{hammer}$$

- Initial reaction

Assume an infinite bar at rest for which the initial displacement is given as follows :

$$u(x, 0) = f^+(x) + f^-(x) \quad (6.23)$$

$$v(z, 0) = v_c \left(\frac{\partial f^+}{\partial x} - \frac{\partial f^-}{\partial x} \right) \quad (6.24)$$

The limit conditions are fulfilled for :

$$f^+(x) = f^-(x) = \frac{1}{2} \cdot f(x) \quad (6.25)$$

The initial displacement dissociates into two waves of shape and amplitude equal to half the initial wave travelling in opposite directions.

- Free end reaction

The free end condition in $x=L$ is expressed as :

$$\sigma(L, t) = 0$$

For the stress to be cancelled out at the $x=L$ point, by the combination of the upward and downward waves, it is necessary that the corresponding stresses have the same amplitude but the opposite sign.

$$F\left(L, \frac{L}{v_c}\right) = F^+\left(L, \frac{L}{v_c}\right) + F^-\left(L, \frac{L}{v_c}\right) = 0 \quad (6.26)$$

$$F^+\left(L, \frac{L}{v_c}\right) = -F^-\left(L, \frac{L}{v_c}\right) = -F_{hammer} \quad (6.27)$$

$$v^+\left(L, \frac{L}{v_c}\right) - v^-\left(L, \frac{L}{v_c}\right) = 0 \quad (6.28)$$

$$v\left(L, \frac{L}{v_c}\right) = v^+\left(L, \frac{L}{v_c}\right) + v^-\left(L, \frac{L}{v_c}\right) = 2v^+\left(L, \frac{L}{v_c}\right) \quad (6.29)$$

At the point of zero stress, the particle velocity is double that of the reflected tension wave or the incident compression wave. The stress wave travels downwards in compression and upwards in tension after the wave is reflected.

- Fixed end reaction

The fixed pile end condition ($x=L$) is expressed as :

$$u(L, t) = 0$$

$$v(L, t) = 0$$

There are two displacement waves of opposite sign and equal amplitude, one incident, the other reflected. For the velocity to be zero at a point, it is necessary that the velocity of the particles corresponding to the ascending wave cancels that corresponding to the descending wave.

$$v\left(L, \frac{L}{v_c}\right) = v^+\left(L, \frac{L}{v_c}\right) + v^-\left(L, \frac{L}{v_c}\right) = 0 \quad (6.30)$$

$$v^+\left(L, \frac{L}{v_c}\right) = -v^-\left(L, \frac{L}{v_c}\right) = -\frac{F^-}{Z} = -\frac{F_{hammer}}{Z} \quad (6.31)$$

$$F^-\left(L, \frac{L}{v_c}\right) = F_{hammer} \quad (6.32)$$

$$F^+\left(L, \frac{L}{v_c}\right) = -Zv^+ = F^-\left(L, \frac{L}{v_c}\right) = F_{hammer} \quad (6.33)$$

$$F\left(L, \frac{L}{v_c}\right) = F^+\left(L, \frac{L}{v_c}\right) + F^-\left(L, \frac{L}{v_c}\right) = 2F_{hammer} \quad (6.34)$$

The two deformation waves correspond in amplitude and sign at all times at the fixed point. The stress at this point is double the incident or reflected wave.

■ Soil friction

This condition corresponds to the action of the lateral friction F_F oriented upwards on a layer of limited thickness, located for example at the depth $x = x_F$. The force F_F is decomposed into two waves, upward and downward, generated symmetrically and simultaneously at the application point of the force F_F

$$F = F_1^+ + F_1^- = F_2^+ + F_2^- + F_F \quad (6.35)$$

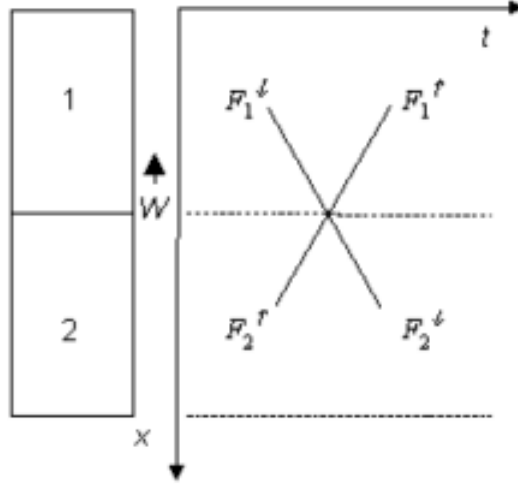


Figure 6.2: Representation of forces at the interface in contact with soil

By applying the continuity law, the velocity is the same at each interface.

$$v = \frac{F_1^-}{Z} - \frac{F_1^+}{Z} = \frac{F_2^-}{Z} - \frac{F_2^+}{Z} \quad (6.36)$$

Combining both previous equations, the forces can be expressed as follows :

$$F_2^- = F_1^- - 0.5 \cdot F_F \quad (6.37)$$

$$F_2^+ = F_1^+ - 0.5 \cdot F_F \quad (6.38)$$

If the force F_F is known at all times following an incident wave F_0^- , the resulting solution is calculated by superposing the incident wave on the symmetric wave system :

$$F^- = F_0^- - \frac{1}{2} \cdot F_F \quad (6.39)$$

$$F^+ = \frac{1}{2} \cdot F_F \quad (6.40)$$

6.4 Analysis of the hammer impact on a semi-infinite pile using a helmet

This section is based on the thesis of Prof A. Holeyman [36].

The hammer is considered by the mass M , the helmet by the spring coefficient k and the pile by the impedance I .

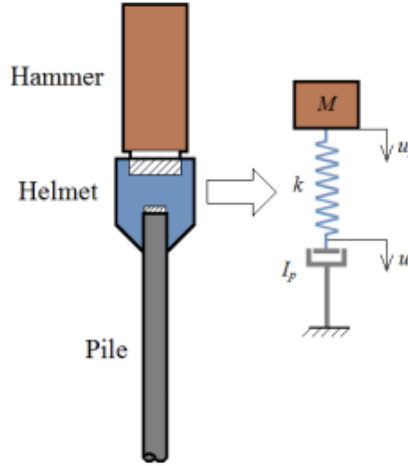


Figure 6.3: Hammer-helmet-pile modeling [36]

By calling u_1 and u the displacements relative to the initial impact position ($t=0$) of the hammer and the pile head respectively, it is easy to establish the vertical equilibrium equations of the hammer and the pile head, neglecting the action of gravity :

$$M \cdot \ddot{u}_1 + k(u_1 - u) = 0 \quad (6.41)$$

$$I\dot{u} - k(u_1 - u) = 0 \quad (6.42)$$

By introducing :

$$\omega_n = \sqrt{\frac{k}{M}}$$

$$\alpha = \frac{k}{2I}$$

The eq. (6.41) and eq. (6.42) becomes as follows :

$$\ddot{u}_1 + \omega_n^2 \cdot u_1 - \omega_n^2 \cdot u = 0 \quad (6.43)$$

$$\dot{u} - 2\alpha \cdot u_1 + 2\alpha \cdot u = 0 \quad (6.44)$$

By expressing u_1 as a function of u and by deriving twice with respect to time, we have :

$$u_1 = u + \frac{\dot{u}}{2\alpha} \quad (6.45)$$

$$\ddot{u}_1 = \ddot{u} + \frac{\ddot{u}}{2\alpha} \quad (6.46)$$

The substitution of u_1 and $\ddot{u}_1$ in the first equation by the obtained expressions gives the following equation :

$$\ddot{u} + 2\alpha \cdot \ddot{u} + \omega_n^2 \dot{u} = 0 \quad (6.47)$$

It is a third order differential equation whose the general solution is :

$$u = C_1 \cdot e^{\beta_1 t} + C_2 \cdot e^{\beta_2 t} + C_3 \quad (6.48)$$

where :

$$\begin{aligned} \beta_1 &= \alpha + \sqrt{\alpha^2 - \omega_n^2} \\ \beta_2 &= \alpha - \sqrt{\alpha^2 - \omega_n^2} \end{aligned}$$

The initial condition are as follows :

$$u(t=0) = 0$$

$$u_1(t=0) = 0$$

$$\dot{u}(t=0) = 0$$

$$\dot{u}_1(t=0) = v_i$$

Three cases are to be considered depending on whether the radical involved in the expression of β_1 and β_2 is positive, zero or negative.

- Case 1 : $\alpha^2 - \omega_n^2 > 0$ or $kM > 4I^2$

The β_1 and β_2 values are reals, negative. The solution of eq. (6.48) becomes :

$$u = C_1 e^{-\alpha t} e^{\beta t} + C_2 e^{-\alpha t} e^{-\beta t} + C_3 \quad (6.49)$$

with :

$$\beta = \sqrt{\alpha^2 - \omega_n^2} = \sqrt{\frac{k^2}{4I^2} - \frac{k}{M}} < \alpha = \frac{k}{2I}$$

By the condition $\dot{u}(t=0) = 0$, we obtain C_2 as function of C_1 and by $u(t=0)=0$, we obtain C_3 as function of C_1 .

The solution becomes :

$$u = C \left(-\frac{2\beta}{\alpha + \beta} \cdot e^{-\alpha t + \beta t} + \frac{-\alpha + \beta}{\alpha + \beta} \cdot e^{-\alpha t - \beta t} \right) \quad (6.50)$$

The eq. (6.44) gives the force in the pile $F = I \cdot \ddot{u}$ and the initial condition $\dot{u}(t=0) = v$ allow to find the C_1 value and the force :

$$F = v_i \frac{k}{\beta} \cdot e^{-\alpha t} \cdot \frac{e^{\beta t} - e^{-\beta t}}{2} \quad (6.51)$$

$$F = I \cdot v_i \cdot \frac{1}{\sqrt{\frac{1}{4} - \frac{I^2}{kM}}} \cdot e^{-\frac{\alpha t}{2I}} \sinh \left(\sqrt{\frac{k^2}{4I^2} - \frac{k}{M}} \cdot t \right) \quad (6.52)$$

The maximum force is reached at $t = \frac{1}{\beta} \tanh^{-1} \left(\frac{\alpha}{\beta} \right)$ and decreases asymptotically close to zero for infinite time.

$$\left(\frac{F}{I \cdot v_i} \right)_{max} = \frac{2\alpha}{\beta} e^{-\frac{\alpha}{\beta} \tanh^{-1} \left(\frac{\alpha}{\beta} \right)} \cdot \sinh \left(\tanh^{-1} \left(\frac{\beta}{\alpha} \right) \right) \quad (6.53)$$

It is a monotonic movement reaching an asymptotic value. As the pile and the hammer remain constantly in contact, the semi-infinite pile receives after an infinite time the totality of the hammer's energy, which finally comes to rest on a spring with no potential deformation energy.

- Case 2 : $\alpha^2 + \omega_n^2 = 0$ or $kM = 4I^2$

The β_1 and β_2 values are equal and corresponding to $-\alpha$. The solution of eq. (6.48) is :

$$u = (C_1 + C_2 \cdot t)e^{-\alpha t} \quad (6.54)$$

The force is obtained this way :

$$\frac{F}{I \cdot v_i} = 2\alpha \cdot t \cdot e^{-\alpha t} \quad (6.55)$$

The maximum force is reached at $t = \frac{1}{\alpha}$ and is defined like this :

$$\left(\frac{F}{I \cdot v_i} \right)_{max} = \frac{2}{e} \quad (6.56)$$

- Case 3 : $\alpha^2 - \omega_n^2 < 0$ or $kM < 4I^2$

The β_1 and β_2 are conjugated complex numbers. The solution of eq. (6.48) is obtained as follows :

$$u = e^{-\alpha t}(C_1 \cdot \sin(\beta t) + C_2 \cdot \cos(\beta t)) + C_3 \quad (6.57)$$

with $\beta = \sqrt{\omega_n^2 - \alpha^2}$ This case, which is the most common in impact driving is solved in the same way as the case 1.

With $\dot{u}(t=0) = 0$, we obtain :

$$C_1 = \frac{\alpha}{\beta} \cdot C_2$$

$$\dot{u} = \frac{\omega_n^2}{\beta} \cdot C_2 \cdot e^{-\alpha t} \sin(\beta t) \quad (6.58)$$

$$\ddot{u} = -\frac{\omega_n^2}{\beta} \cdot C_2 \cdot e^{-\alpha t}(-\alpha \sin(\beta t) + \beta \cos(\beta t)) \quad (6.59)$$

The eq. (6.58) substituted in $u_1 = u + \frac{\dot{u}}{2\alpha}$ gives :

$$\dot{u}_1 = \frac{\omega_n^2}{2\beta} \cdot C_2 e^{-\alpha t}(\sin(\beta t) - \cos(\beta t)) \quad (6.60)$$

With the initial condition $\dot{u}_1(t=0) = v_i$, we can find :

$$C_2 = -\frac{v_i \alpha}{\omega_n^2}$$

By substituting this constant in eq. (6.59) and by multiplying by the impedance to obtain the force, we obtain :

$$F = v_i \cdot \frac{k}{\beta} e^{-\alpha t} \sin(\beta t) \quad (6.61)$$

The eq. (6.61) corresponds to a damped oscillatory movement. At $t=0$, the force is equal to zero and is maximum at $t = \frac{1}{\beta} \cdot \tan^{-1} \left(\frac{\beta}{\alpha} \right)$. The maximum force value is given as follows :

$$\left(\frac{F}{I \cdot v} \right)_{max} = 2 \cdot \frac{\alpha}{\beta} \cdot e^{-\frac{\alpha}{\beta} \cdot \tan^{-1} \left(\frac{\beta}{\alpha} \right)} \sin \left(\tan^{-1} \left(\frac{\beta}{\alpha} \right) \right) \quad (6.62)$$

The helmet transmits only compressive forces. The contact between the pile and the hammer is broken. As at this moment the velocity at the head of the pile is cancelled out, this break in contact can only occur when the hammer rebounds. Part of the incident kinetic energy of the hammer is returned to it in the same form.

PDI Wave simulations

The PDI wave software allows to predict the effect of one blow on a given pile in a given soil. The parameters asked by the software are the following:

- hammer weight and drop height
- helmet weight and height
- pile length, area and diameter
- soil ultimate resistance and skin friction percentage

As the only parameters missing, the percentage of skin friction was defined as 5% and the soil ultimate resistance was calculated considering the soil under the pile is in the following stress state (fig. 6.4) :

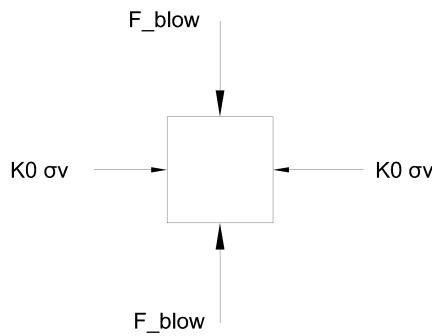


Figure 6.4: Assumed stress state for PDI predictions

As the base pile depth is known for each experiment, and the set per blow is given by PDI, the soil resistance for each blow can be calculated thanks to the following equation :

$$F = A_p \cdot (\sigma_h \tan(\phi) + (1 + \sin(\phi)) \cdot \gamma_d \cdot z_i) \quad (6.63)$$

By changing the soil resistance after each blow for three characteristic experiments, namely dense sand and a drop height of 10 cm, dense sand and a drop height of 5 cm and loose sand and a drop height of 10 cm, the following predictions could be drawn (fig. 6.5) :

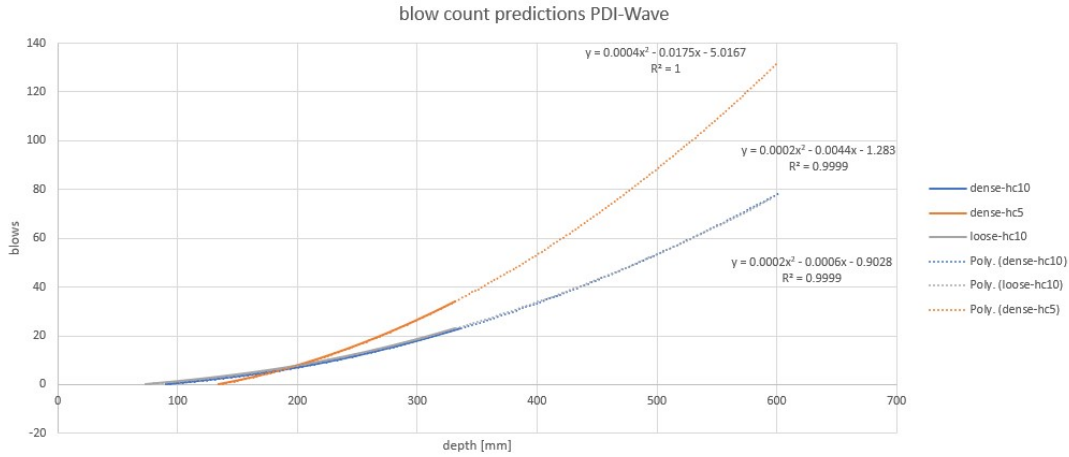


Figure 6.5: PDI predictions results

It is interesting to note that while the drop height has a substantial effect on the amount of blows needed to get to a given depth, the sand density does not look like changing anything. It might be explained by the assumptions as while the terms dependant on ϕ decrease, the effect of the lower density cancels it out.

This conclusion was not totally expected but still needs to be confirmed by the experiments.

It is also important to note that as the software is designed for larger pile simulations, some scale effects might lead to some inaccuracy. Namely, the software did not allow to divide the pile in segments bigger than 0.25 m long, and its timescale did not allow for the calculation of the complete blow. This effect is mainly important for the first blows when the pile has a large set depth. It stopped while the pile still had some substantial downwards speed.

6.5 Soil reaction for laterally loaded piles

Monopiles are subject to lateral loads and bending moments due to wind and wave forces. They are designed to have enough ultimate resistance against vertical and lateral loads. To ensure enough stiffness of the foundation and an acceptable pile-head deflection, monopiles are designed by use of p-y curve method.

A p-y curve describes the non linear relationship between the soil resistance acting against the pile wall, p, and the lateral deflection of the pile, y. The soil resistance, p, is given as the reaction force per unit length acting on the pile until reaching the ultimate soil resistance.

6.5.1 Winkler approach [12]

The pile is considered as a beam on an elastic foundation. The beam is supported by a number of uncoupled springs with spring stiffness given by p-y curves and the continuity is not taken into account.

Since the pile deflection is non linear, the soil resistance is modelled as uncoupled non linear springs with stiffness K_i acting on an elastic beam. K_i is a non linear load transfer function corresponding to E_{py} .

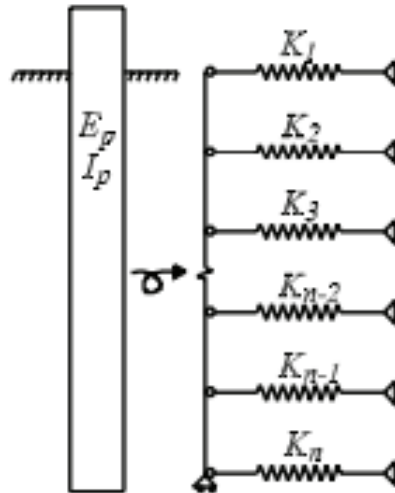


Figure 6.6: Winkler approach with the pile modelled as an elastic beam element supported by non-linear uncoupled springs. K is the stiffness corresponding to E_{py} [12]

Assumption

- The beam is straight and has an uniform cross section.
- The beam has a longitudinal plane of symmetry, in which loads and reactions lie.
- The beam material is homogeneous, isotropic and elastic.
- The Young's modulus of the beam material is similar in tension and compression.
- The beam deflections are small.
- The beam is not subjected to dynamic loading.

The equation for an infinitesimal small element, dx , located at depth x , subjected to lateral loading, can be derived from static equilibrium.

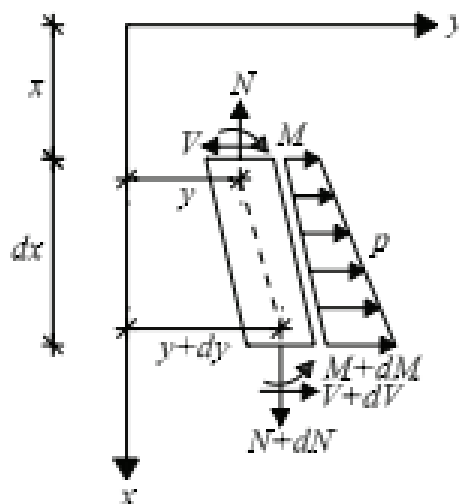


Figure 6.7: Sign convention for in infinitesimal beam element [12]

$$\frac{dM^2}{dx^2} + \frac{dV}{dx} - \frac{N d^2 y}{dx^2} = 0 \quad (6.64)$$

Following relations are used :

$$M = E_p \cdot I_p \cdot \kappa \quad (6.65)$$

$$\frac{dV}{dx} = -p \quad (6.66)$$

$$p(y) = -E_{py} \cdot y \quad (6.67)$$

Use of eq. (6.65) - eq. (6.67) and the kinematic assumption $\kappa = \frac{d^2 y}{dx^2}$ which is valid in Bernoulli-Euler beam theory the governing fourth-order differential equation for determination of deflection is obtained :

$$E_p I_p \frac{d^4 y}{dx^4} - N \frac{d^2 y}{dx^2} + E_{py} \cdot y = 0 \quad (6.68)$$

In eq. (6.68) the shear strain γ in the beam is neglected. This assumption is only valid for relatively slender elements. For short and rigid beams, the shear strain is taken into account. The following relations are used :

$$V = G_p \cdot A_v \cdot \gamma \quad (6.69)$$

$$\gamma = \frac{dy}{dx} - \omega \quad (6.70)$$

$$\kappa = \frac{d\omega}{dx} \quad (6.71)$$

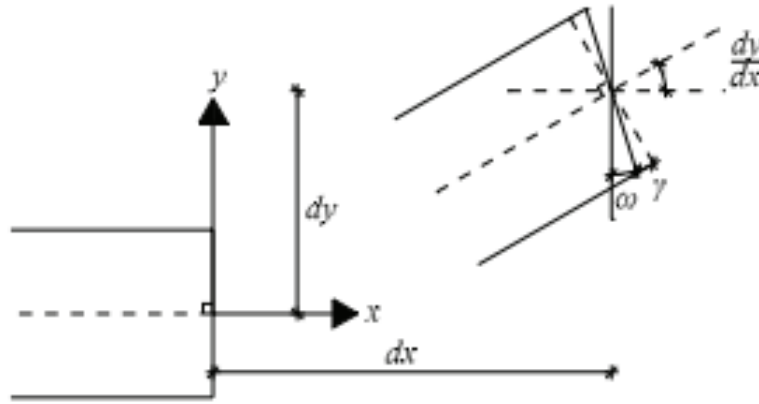


Figure 6.8: Shear curvature deformation of a beam element [12]

The effective shear area is defined so the two stress variations give the same shear force. For a pipe the effective shear area can be calculated as :

$$A_v = 2(D - t) \cdot t \quad (6.72)$$

By combining eq. (6.64) - eq. (6.67) and eq. (6.69) - eq. (6.71) two coupled differential equations can be formulated to describe the deflection of a beam :

$$G_p \cdot A_v \frac{d}{dx} \left(\frac{dy}{dx} - \omega \right) - E_{py} \cdot y = 0 \quad (6.73)$$

$$E_p I_p \frac{d^3 \omega}{dx^3} - N \frac{d^2 y}{dx^2} + E_{py} \cdot y = 0 \quad (6.74)$$

6.5.2 P-y curve for sand

This section presents the formulation proposed by API [59].

The ultimate resistance is approximated using the dimensionless parameters C_1, C_2 and C_3 :

$$p_{us} = (C_1 H + C_2 D_p) \cdot \gamma \cdot H \quad (6.75)$$

$$p_{ud} = C_3 \cdot D_p \cdot \gamma \cdot H \quad (6.76)$$

$$p_u = \min(p_{us}, p_{ud}) \quad (6.77)$$

The constants C_1 , C_2 and C_3 can be determined from fig. 6.9 :

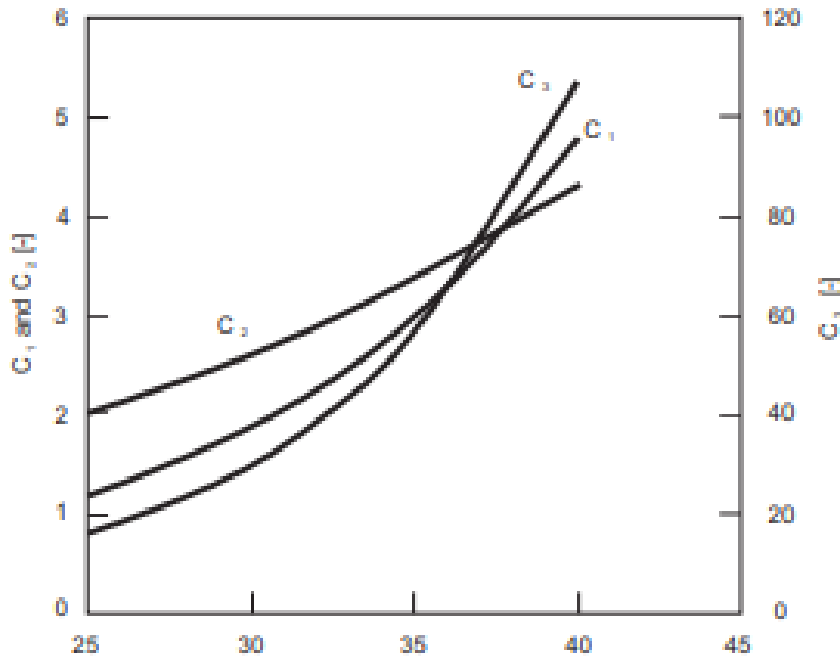


Figure 6.9: Variation of the parameters C_1 , C_2 and C_3 as function of angle of internal friction [59]

A hyperbolic formula is used to describe the relationship between soil resistance and pile deflection :

$$p(y) = A \cdot p_u \cdot \tanh\left(\frac{kH}{Ap_u} y\right) \quad (6.78)$$

The coefficient A for static loading can be determined by :

$$A = \left(3 - 0.8 \cdot \frac{H}{D_p}\right) \geq 0.9 \quad (6.79)$$

and for cyclic loading $A = 0.9$.

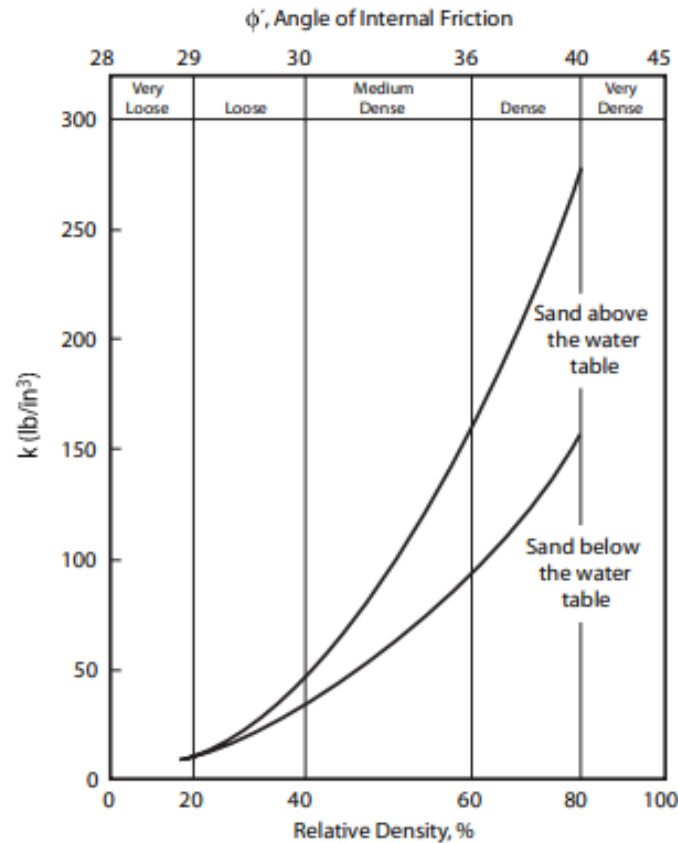


Figure 6.10: Relative density as a function of internal friction angle

Application of P-y curve theory to the small scale model

Solving the equations suggested by API [59] with the data obtained from the previous tests mentioned in this thesis to obtain the p-y curves for sand. The data used to solve the equations are shown in the following table 6.2:

Test name	ϕ [°]	C_1 [-]	C_2 [-]	C_3 [-]	D [m]	γ [kN/m ³]	k [kN/m ³]
Loose sand	34	2.8	3.35	48	0.06	15.72	115
Dense sand	35	3	3.5	54	0.06	18.21	135

Table 6.2: Data used to solve the equations and obtain the p-y curves

The internal friction angle is obtained by using the values (table 4.7) corresponding to the peak values derived from Mohr circles in section 4.4.5. The effective soil weight values are calculated in section 4.2.4. The C_1 , C_2 , C_3 and k are obtained by using the fig. 6.9 and fig. 6.10 for the corresponding internal friction angle.

By varying the depth to have different lateral bearing capacities and the lateral deflection to have different lateral soil resistances, the following graphs can be obtained for loose and dense sand :

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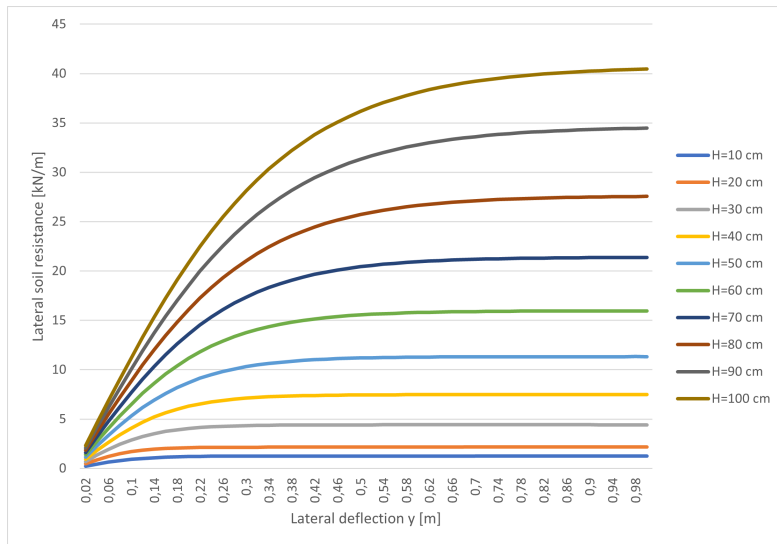


Figure 6.11: P-y curve for different depths for loose sand

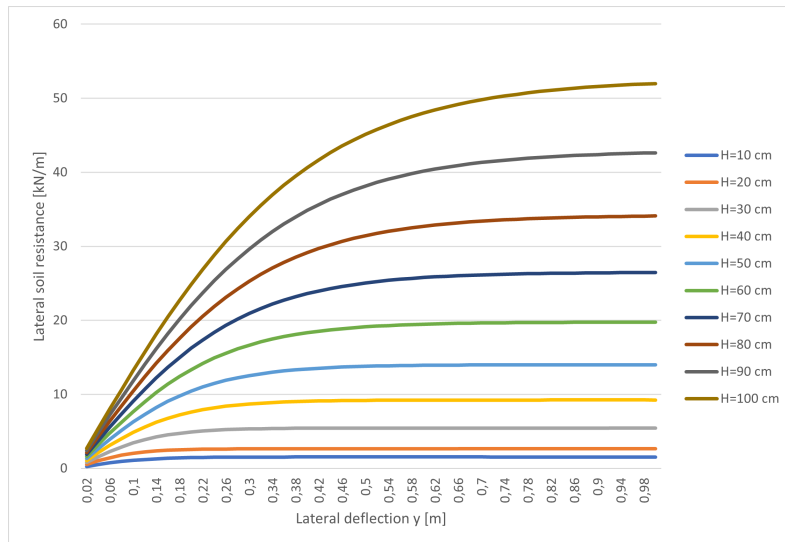


Figure 6.12: P-y curve for different depths for dense sand

Looking at the fig. 6.11 and fig. 6.12, as the soil depth and lateral deflection become greater, the soil lateral resistance increases, regardless of whether the sand is loose or dense. The lateral resistance of the soil increases rapidly to counteract the lateral displacement until it reaches its maximum value and then it remains constant despite the increase in lateral deflection. This means that the soil is no longer strong enough to take up the lateral displacement of the pile and the pile may overturn if the deflection becomes too great.

For small depths, the maximum lateral resistance of the soil is reached very quickly and is very low compared to the longer driving lengths. Furthermore, for the same lateral displacement, the maximum lateral soil resistance is reached more quickly for loose than dense soil. This means that for dense soil, the pile must be driven to a shallower depth to achieve the same lateral soil resistance as for loose soil. Depending on the density of the sand and the lateral displacements due to waves and wind, it is easy to predict how far the pile should be driven in order to obtain sufficient lateral soil resistance to avoid pile tipping.

Different methods can be used to get more insight into what the influence of the installation method is on the final lateral resistance of the monopiles. Respectively numerical modelling, full scale tests and model tests. The latter is the one we will focus on from now on as it is the one that has been undertaken in the UCLouvain civil works laboratory in Louvain-La-Neuve.

7.1 Similitude relationships

In order to get a correspondence between the small scale experiments and full scale effects, some non dimensional groups need to be kept between the two. These can be divided in different categories : the design and the action ratios.

The ratios from the first category are used in order to get the right dimensions while preparing the experiments : pile length, diameter, loading resultant height and pile thickness.

The second category of design ratios is used in order to interpret the results and transform them into full size relationships : the forces, moments and displacements will be discussed here.

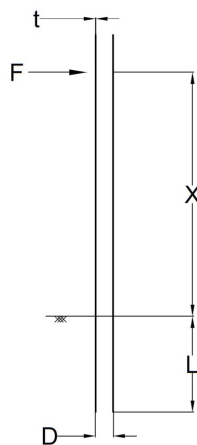


Figure 7.1: Scaled model dimensions

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The main parameters that have been taken into account for the design ratios are the pile length, diameter, steel thickness and loading resultant height (see fig 7.1).

The reference ratio used is the one relating the real size to small scale diameter of the pile. This ratio is kept at 100 :

$$\lambda = \frac{D_p}{D_m} = 100 \quad (7.1)$$

With p representing the prototype, full scale characteristic and m stands for the models' one. Which means that the tube with a diameter of 60 mm is scaling a 6 m diameter prototype pile. On the other side, a 110 mm diameter model scales a 11 m diameter monopile.

At first, these two sizes were chosen because they are respectively representing the mean actual size of monopiles installed in the North Sea and the future size of piles that the industry is expecting to install in order to accommodate bigger wind turbines. Unfortunately, only experiments on the smaller 60 mm piles have been performed.

From this relationship, other ratios can be derived that allow us to scale the embedded length and loading resultant height. These can be found in the literature from the database of existing windturbines.

The embedded length to diameter ratio usually lies in between 3 and 10, or somewhere around the following relationship (see [43] and [72]) :

$$L = 8 \cdot D - 5m \quad (7.2)$$

In our case, for the small 60 mm diameter pile, that means that the embedded length must respectively be in between 18 and 36 mm or close to 43 mm. A ratio of 5.5 and of 10 have been chosen for the experiments as they fit both sides of the relations spectrum.

The loading resultant height is a very difficult subject as the windturbine is subjected to variable cyclic loading due to the wind, wave, current and blade movement. As all these actions have variable intensity in time and all along the shaft of the windturbine, it is impossible to find one accurate loading resultant height. However, simplifying assumptions can be made to get the shear force and mudline moment that approximates those really experienced by the monopile and the surrounding soil.

Different relationships have been found in the literature in order to get this ratio. Some are related to the diameter of the pile and others are related to the embedded length.

The two relations are the following (see[14] and [19]) :

$$5 \leq \frac{X}{D} \leq 18.3 \quad (7.3)$$

$$\frac{X}{L} = 2.5 \quad (7.4)$$

These relationships give respectively the following values for the 60 mm pile :
 $300 \text{ mm} \leq X \leq 1098 \text{ mm}$ and $X = 825 \text{ mm}$.

The steel thickness has also quite a big impact on the behavior of the pile while loading it as the bending stiffness is related to it. Using classical beam theory, the stresses due to

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bending can be represented by:

$$\sigma = \frac{M}{Z} \quad (7.5)$$

$$Z = \frac{\pi}{2} \frac{t}{D} \left(\frac{D}{2} - t \right)^3 [mm^3] \quad (7.6)$$

The API gives us a relationship between the pile steel thickness and its diameter [72] :

$$t = 6.35 + \frac{D}{100} [mm] \quad (7.7)$$

With t and D given in mm, which scaled gives :

$$t = 0.0635 + \frac{D}{100} [mm] \quad (7.8)$$

Which for the small pile would give a thickness of 0.6635 mm. Since this thickness was not available on the market for such an experimental setup, we have chosen the closest possible approximation and are evaluating the theoretical impact of this approximation.

To be able to get a correspondence between the results given by the small scale experiments and the real size events, relationships between forces, moments and stresses in the small and real size had to be determined.

As the horizontal force is counterbalanced by the passive resistance of the soil around the pile, the following relation can be drawn using basic statics :

$$F = \int_0^L K_p \cdot \sigma_v \cdot D \cdot dz \quad (7.9)$$

As K_p , the passive coefficient of earth pressure, is equal to $\frac{1+\sin(\phi)}{1-\sin(\phi)}$ and $\sigma_v = \gamma z$ and γ and ϕ are not scaled, the relationship gives :

$$F = K_P \cdot D \cdot \gamma \cdot \frac{L^2}{2} \quad (7.10)$$

So the force scales as DL^2 . Since D and L scale with a factor $\lambda = 100$, F scales with a factor $\lambda_F = 1000000$. That means that a result of 1N in the model test corresponds to 1MN of loading on real life monopiles.

As the moment is the product of that force and a distance scaled like the diameter, the moment scaling factor is the following :

$$\lambda_M = 100000000 \quad (7.11)$$

Finally, the stresses in the pile can be computed using the moment :

$$\sigma = \frac{M}{Z} \quad (7.12)$$

Using the length and moment relationships, the pile stresses scaling law can be computed as:

$$\lambda_\sigma = 100$$

The model piles are composed of the same materials as used in (Jafari [44]). Thus they haven't been tested again. The properties of the steel used are directly coming from his thesis :

Description	Outer Diameter (D_0) [mm]	Wall Thickness (t) [mm]	D_0/t [/]	Specifications	Tensile Strength (R_m) [MPa]	Elongation After Fracture (A) [%]	Upper yield Strength (ReH) [MPa]
60 mm model piles	60	1.5	40	EN 10305-3 E235 +CR1S2: Steel tubes for precision applications	390	7	$\geq 0.7 \cdot R_m = 273$
110 mm model piles	110	1.5	73.3	EN 10305-2 E235+C: Steel tubes for precision applications	490	6	$\geq 0.8 \cdot R_m = 392$

Table 7.1: Steel tube characteristics as given by M. Jafari [44]

Thus, as the steel material keeps the same properties between the model and prototype, the strains are scaling the same way as the stresses since the stresses stay in the elastic domain and the relationship between stresses and strains is :

$$\sigma = E_p \cdot \epsilon \quad (7.13)$$

$$\lambda_\epsilon = \lambda_\sigma = 100 \quad (7.14)$$

7.2 Discussion of choices and similitude

The characteristics of the different experiments conducted can be found further in table 8.1. First of all, the length scaling ratio, $\lambda = 100$, has been kept in the diameter.

Concerning the embedded length, ratios $L/D = 5.5$ and 10 have been chosen and are in the range 3-10 and on both sides of the line $L = 8 D-5$.

On the other side, for the loading height, the following ratios are kept :

Experiment	Loading Height	Ratio H/D	Ratio H/L
1	910	15.17	2.758
2	970	16.17	2.939
3	960	16	2.909
4	695	11.58	1.158
5	950	18.83	2.879

Table 7.2: Loading height versus diameter and embedded length ratios

It can be seen that except for the fourth experiment, where the loading height was conditioned by the pile length due to the high embedded length, these ratios are kept in the range recommended : H/D between 5 and 18.3 and H/L around 2.5. That means that we can expect the fourth experiment to resist to slightly higher forces than the other compared to the embedded length.

One of the relationships that could not be kept is the ratio between the diameter and the thickness of the pile. Indeed, while API recommends to keep the ratio around $t = 0.0635 + D/100$, which would mean a 0.6635 mm thick pile wall, we could not find such a thin tube.

This implies that the stresses and strains recorded in the tube are going to be lower than those recorded in real size monopiles. The pile will also react as more rigid than supposed to due to the increased inertia.

Finally, the sand and steel properties have been kept the same as in full scale monopile designs.

7.3 Model pile driving facility

The pile driving facility used in our experiment is based on the Fondeole system developed at UCLouvain by A. Holeyman and S. Hamdi (2017) [38].

7.3.1 Initial pile driving facility - Fondeole

This small scale pile driving facility has been used until now to drive tubular steel piles into weak rock masses by S. Hamdi and M. Jafari. The setup consists of the following parts (fig. 7.2) :

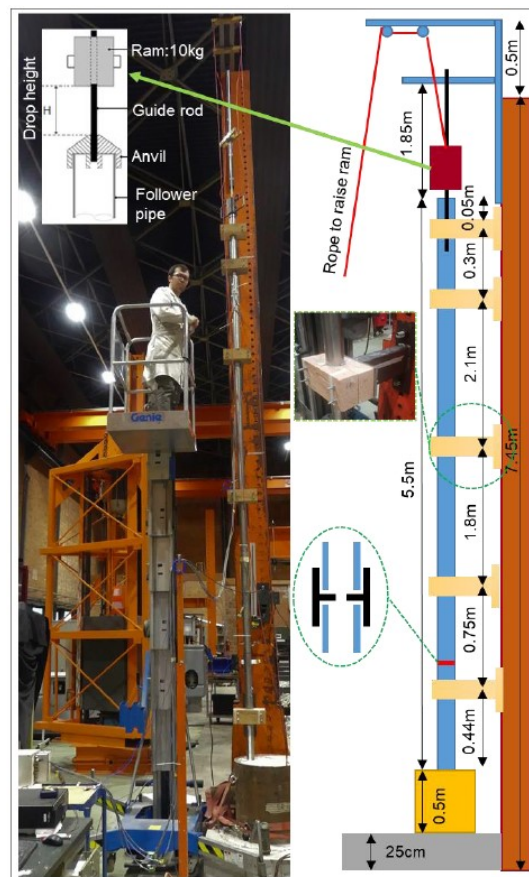


Figure 7.2: Initial Fondeole setup as used by M.Jafari [44]

- A 10 kg weight constitutes the hammer. Two ropes are knotted on rings situated on its head. Pulleys fastened to a frame supported by a rigid H-column fixed on the test floor allow for easy raising of the hammer.
- An anvil that has been machined specifically to fit perfectly the pile and the centering rod. Its role is to damp the hammer blows and to center them on the pile in order to avoid any eccentricities in the hits. The anvils are made of 42CrMo4 steel alloy with a yield strength of 500 MPa (Fig 7.3).
- A centering rod that is fastened to the top of the hammer frame and finishes inside the pile. The anvil has a hole in its center in order to let it pass through. The hammer also has one that allows for it to slide around the centering rod and hit the pile right in its middle, avoiding all eccentricities of the blows.
- Lateral supports to keep the pile vertical on its whole height. They are composed of a metal piece up to a certain distance to the H-column then two wood pieces machined to keep a hole with the same diameter as the pile in their middle can be fastened in order to clamp the pile. The holes are fitted with felt to reduce friction when the pile is being driven.

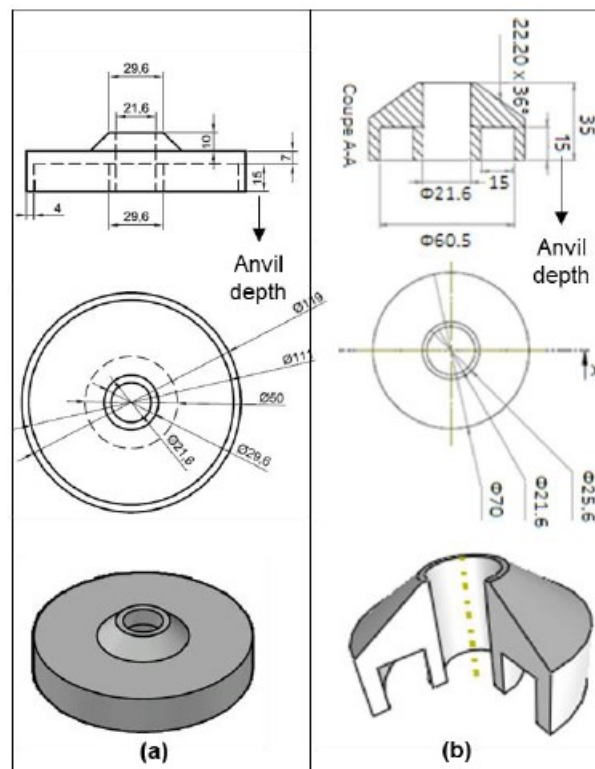


Figure 7.3: Anvils for (a) 110mm model piles and (b) 60mm model piles (Hamdi,2016)[44]

The pile used in their experiment was composed of two parts and was all together over 5 m high.

The pile was placed in the vertical supports and the anvil-hammer-centering rod system was placed on top of it. Then the hammer was raised manually by pulling on the ropes until the desired drop height was reached and then released. This was repeated until the desired driven depth was reached.

7.3.2 Changes added for our project

As the former use of this experimental setup was to drive pipe piles into calcarenite or reconstituted weak rocks, the pile could be driven quite close to the H-column into small samples. The different elements were thus designed in order to have the pile at 30 cm from the side of the column.

In order to keep the border effects as limited as possible in our experiments, a container with a 1 m diameter and 5 cm concrete sides was used. This means that the pulley frame and the lateral supports had to be extended.

As the pulleys are fixed on an aluminium plate clamped to the frame, we could just make a new frame with the extended dimensions, but the lateral supports had to be cut in two and extended thanks to two thick walled square hollow steel bars welded inside the original support.

Also, as shorter piles will be used (1.35 m model piles), only one support is needed in order to keep the pile vertical but limit the friction losses.

Thanks to all these changes, the Fondeole system could be used for our experiments.

7.4 Model pile instrumentation

The pile was equipped with sensors to allow for a post processing analysis of the experiments.

7.4.1 Overview

The sensors added to the pile are the following (Fig 7.4) :

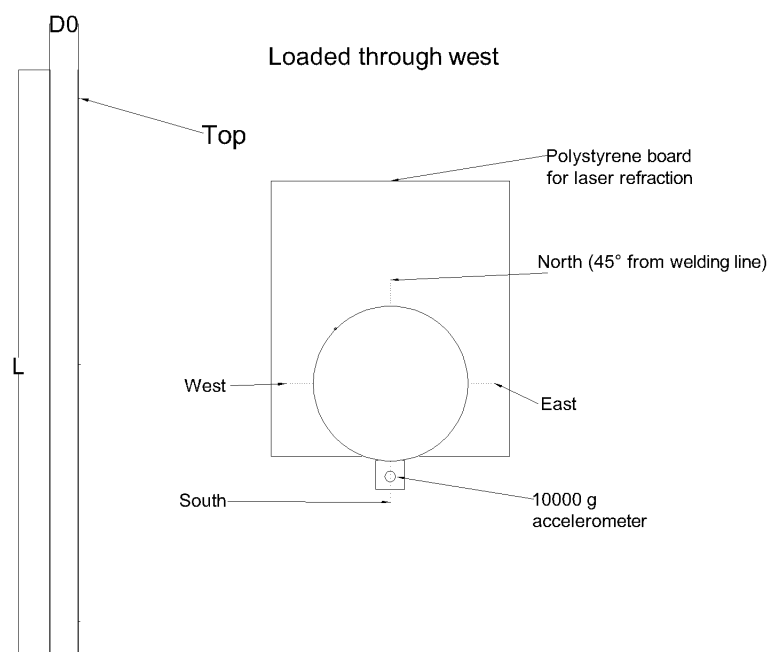


Figure 7.4: Instrumentation top view

On the pile :

- A 10000 g accelerometer in order to record the acceleration at the top of the pile for each blow.
- A 5 m optic fibre thread to record continuously the deformations of the pile while loading it.

On the H-column :

- A laser for recording the vertical setting distance then horizontal deflection of the model pile during respectively the driving and lateral loading phase. A polystyrene board is fastened on the side of the pile as a target for the vertical laser and a white sticker is pasted on the side of the pile for measuring horizontal distances. The laser position is changed between the driving and lateral loading phases of the experiments.

On the jack :

- A 100kN load cell to transcript the force needed while laterally loading the model pile.

On the container :

- A 50g accelerometer in order to record the influence of the boundary conditions on the results.

7.4.2 Optic fibre

A 5m optic fibre was glued on both model piles in a serpentine manner so that data on the strains of the four sides of the pile can be recorded(fig. 7.5).

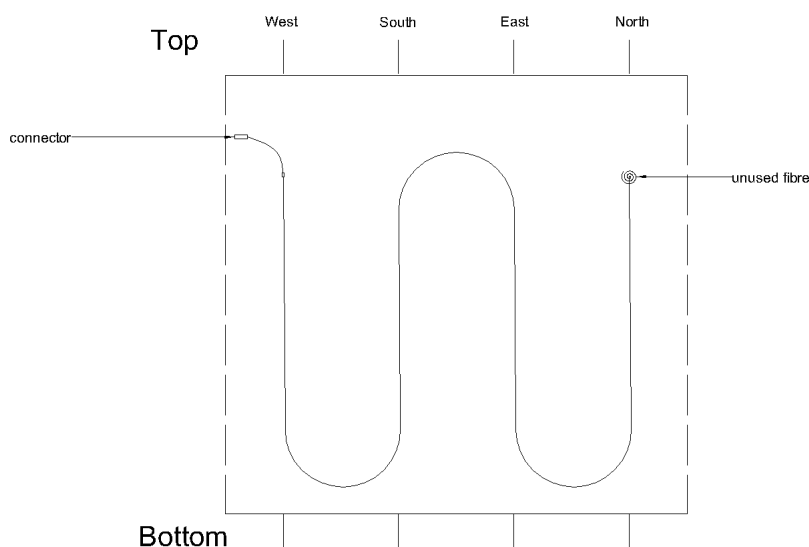


Figure 7.5: Fibre placing on the side of the pile

Working principle of the fibre

The optic fibres allow to measure the strains on the sides of the model pipes in a nearly continuous manner. A system is sending a light signal in the fibre and measures the time difference between the sent signal and the reflected light. The Luna HD-FOS (High Definition Fibre Optic Sensing) is using an Optical Frequency Domain Reflectometer (OFDR) to do just that and determine the exact place and amplitude of the deformation. Their fiber can also calculate temperature differences (fig. 7.6).

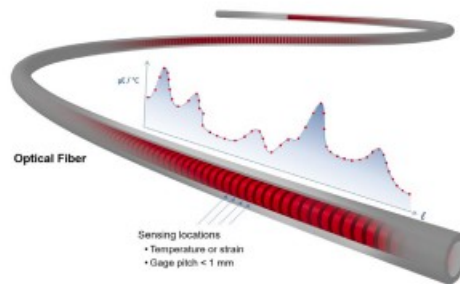


Figure 7.6: Working of the optic fibre - source Luna

The Luna HD fiber optic strain sensors are composed of standard optic fibers connected to a plug-in connector and a glass termination (fig. 7.7). The complete specifications can be found in chapter H.

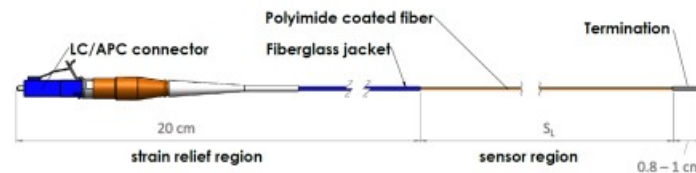


Figure 7.7: Composition of an optic fibre as given by Luna

Advantages of the fiber

Compared to more standard strain gage, the optic fibre has a lot of advantages ([50]):

- The fibre is flexible which permits to measure the deformation in more complicated places. It is also very small and lightweight which limits the interference with the experiences.
- The fibre is not conductor and is thus not influenced by electromagnetic phenomenons happening around it.
- It can be purchased in long length which allows to have nearly continuous measures all along the element using only one measuring device.
- Thanks to the Odisi data measurement system and the key-ID unique for each fiber given when bought it can be used straight after the installation without losing too much preparation and calibration time.

Installation and protection protocol

In order for the fiber to record the same strains as the pile, it needs to totally adhere to it. To get this perfect adhesion, the following protocol¹ has been followed.

Material needed for the installation

The following materials were used while preparing the pile, gluing and protecting the fibre (table 7.3) :

Material	References
High Definition Fiber Optic Strain Sensors	HD6S05LC300P
Optical Distributed Sensor Interrogator	ODISI 6000 SERIES
Compressed air duster	HG nettoyant à air comprimé
Degreaser	CSM-3
Conditioner A mild phosphoric acid	M-Prep Conditioner A (MCA-2)
Neutralizer 5A Ammonia-based liquid	M-Prep Neutralizer 5A (MN5A-2)
adhesive glue	Loctite 495
Epoxy glue	Würth ESK-50
Adhesive Activator	Loctite 7471
Brushes	different types
Adhesive tape	Würth Crepe Masking Tape
Wooden stick	small round stick

Table 7.3: Materials needed for optic fibre installation

Pile preparation

The first step is to delimit the four sides of the pile (N-W-S-E). In order to do that, a first line was traced at N. Then, a paper sheet was rolled around the pile and marks were drawn at the places where the end recovered the beginning. Then the sheet was cut at those marks to roll it around the pile with both sides of the sheet perfectly next to each other. Finally, lines were drawn on the sheet as to have four equal parts. Fixing this to the pile allowed to find the three other directions. All four lines have been marked thanks to a marking gauge.

A 2 cm space around the lines and around the bends has been delimited by placing three masking tape layers at 1 cm on each side of the pile (fig. 7.8). Finally, the space in between the tapes has been sanded to have a perfect surface state (fig. 7.9).

¹given by S. Awad and A. Bertholet

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Figure 7.8: Pile taped around the future fibre course



Figure 7.9: Sanded pile

Afterwards, the tape has been removed and the surface has been chemically prepared through the following steps (fig. 7.10) :

1. Using the degreaser and a small brush to clean and remove the big particles.
2. Drying with compressed air.
3. Cleaning using the M-Prep conditioner A acting as an etchant.

7 – Experimental setup

4. Follow by using the M-Prep Neutralizer 5A that stops the chemical reaction due to the acid.
5. Finish by drying using compressed air.



Figure 7.10: Chemicals used for surface preparation

Once the pile is clean where the fibre will be glued, it can be unrolled to the length of the pile, then kept tight between the beginning and the end of the intended course. The beginning is taped around the pile (fig. 7.11), for other lengths, one drop of glue is placed at the beginning and the end of the course, and the polymerisation is directly started thanks to a small activator spray (fig. 7.12). This way, the fibre stays tight while the glue can be applied to the rest of the course.



Figure 7.11: Taped beginning of fibre

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Figure 7.12: Glue and activator

A small triangular notch is cut in the wooden stick, then glue is dropped on the top of this notch and by sliding the stick over the fibre, it is applied on and around the fiber. Afterwards, the fibre is rolled around a big arc of a circle so the fiber is not too bent until the next side of the pile and the former steps are done again until all sides are glued (fig. 7.13).



Figure 7.13: Fibre glued on the pile

Finally, after the glue has dried, an epoxy coating is added around the fibre to protect it against abrasion while driving and against other hazards.

7.4.3 Accelerometer

In order to measure the pile head acceleration during driving, a 10000g accelerometer is fastened on one of the supports welded at the pile head.

This accelerometer is a PCB 350C03 shock accelerometer with a sensitivity of 0.5 mv/g.

Another accelerometer was fastened to the container to investigate the shock wave arriving to it and its influence on the final result.

This accelerometer is a PCB 353B33 shear accelerometer with a measurement range of $\pm 50g$ and a sensitivity of 100 mV/g. It was shown afterwards that a smaller measurement range would have been sufficient due to the very low values of the container acceleration levels.

7.4.4 Laser

Finally, a laser has been used to measure vertical and horizontal displacements of the pile during driving and lateral loading respectively.

The laser used was a Keyence LK-502.

Vertical measurement A polystyrene board was attached to the side of the pile as a target for the laser beam using tape. The laser was attached to a wooden board itself bolted to a vertical support.

Horizontal measurement The laser was fastened to an aluminium profile so that it would hit the pile on a sticker used as a target. This allowed to calculate the tipping angle of the pile while laterally loaded using Pythagoras theorem.

7.5 Horizontal loading

Once the pile has been driven to the desired depth, a lateral loading is applied on the pile.

7.5.1 Loading device and lateral loading procedure

The lateral loading device is constituted by :

- A jack Enerpac RC1010, with a capacity of 101 kN and a stroke of 257 mm.
- A force cell.
- A manual pump.
- A transition piece between the force cell and the pile.

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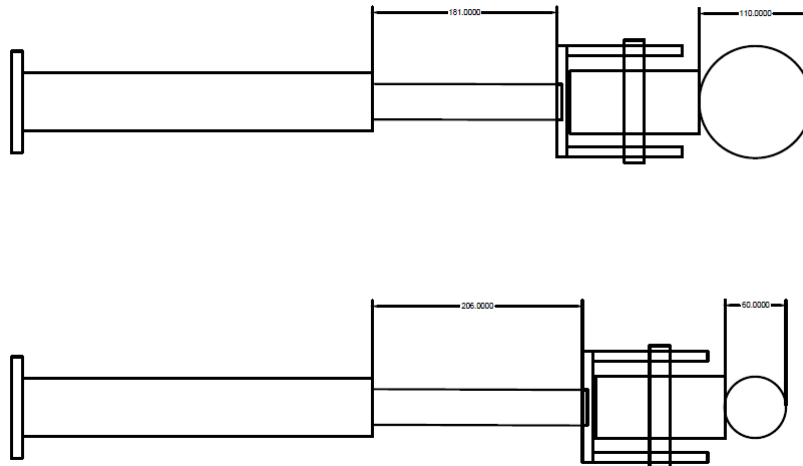


Figure 7.14: Dimension of lateral loading device



Figure 7.15: Real lateral loading device

In order to analyse the lateral loading influence on the pile and fibers behaviour, the jack pushes the pile until some distance and returns to its initial position. Then, the jack goes back at the level of the pile and pushes the pile until the end of its course.

7.6 Experimental procedure

1. Using the sand raining system, fill the tank with a certain constant drop height throughout the filling process and set to achieve a desired relative density. The sand is dried and sieved with a 2.36 mm sieve to avoid obstructing the openings of the sand raining system.



Figure 7.16: Sand sieved by a 2.36 mm sieve

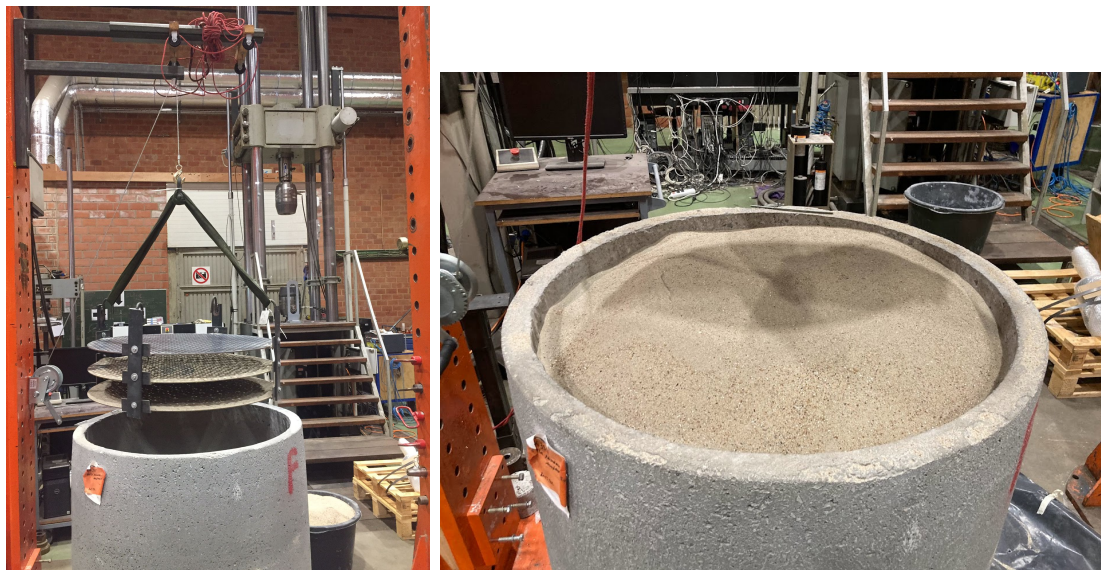


Figure 7.17: Tank filling process with sand raining system

2. Once the tank has been filled, place the lateral pile guide supports at different heights on the H-column fixed to the floor to ensure the verticality of the pile during impact driving. Connect the hammer to the pulley system at the top of the column.



Figure 7.18: Position of lateral guide supports and hammer on the H-column

3. Place and centre the pile previously instrumented with an accelerometer and optical fibres in the sand-filled tank.



Figure 7.19: Position of pile in the tank

4. Place the guide rod at the top of the column and the hammer in it. The movement of the hammer during impact driving is ensured by this 21 mm diameter steel rod. The hammer slides along this rod until it hits the anvil. Place the anvil on top of the pile. The anvil acts as a centering device to ensure an impact along the pile axis.



Figure 7.20: Position of rod guide, hammer and anvil

5. Place the laser on the H-column to measure the pile vertical displacement during the impact driving.

When all the above steps have been completed, the impact driving process can begin.

6. Launch the acquisition program and lifts the hammer until the predefined drop height is reached. Drop the hammer and wait a few minutes for the sensors to become stable after the blow.
7. Measure the new pile height after the blow. The height is the distance between the sand at the pile base and below the anvil. This allows to check that the laser measurement is correct.
8. Repeat the previous two steps until the desired driving depth is reached. Stop the acquisition program.



Figure 7.21: Position of the pile at the desired driving depth

9. Remove the hammer, the guide rod, the laser and the lateral supports.
10. Place the lateral loading device and the laser on the H-column to measure the horizontal displacement. Place the acquisition device for optical fibers on the pile.



Figure 7.22: Position of lateral loading device, optical fiber device and laser

11. With the jack pump, push the jack until a contact between the transition piece and the pile has been achieved.
12. Launch the acquisition program for the sensors and for the optical fibers.
13. With the jack, push the pile until some distance. Stop the acquisition programs. Then the jack comes back to its initial distance to analyse the loading influence on the pile.



Figure 7.23: Position of the pile after the first lateral loading

7 – Experimental setup

14. With the jack pump, push the jack until a contact between the transition piece and the pile has been achieved and launch the acquisition programs.
15. With the jack pump, push the pile until the end of jack course.
16. Stop the acquisition programs. Measure the pile driving and the sand displacement created during the lateral loading.



Figure 7.24: Position of the pile after the second lateral loading



Figure 7.25: Measurement of the sand displacement during the lateral loading

17. Remove the lateral loading device and the pile. Empty the tank by removing the screws and the board under the tank.



Figure 7.26: Emptying of the container

CHAPTER 8

Interpretation of Results

Five different experiments were conducted, four of them in dense sand and the last one in loose sand conditions.

They are denominated in the following way :

essai-D-density-H (-hc5)

with x the test identification number, from 1 to 5, D the pile diameter in mm, here 60 for all tests, Dense or Loose for the sand density and H for the embedded length to diameter ratio. The suffix -hc5 is added when the drop height is changed from the reference drop height.

The reference properties are the following : a 60 mm diameter, dense sand, a 5.5 ratio between the embedded length and the diameter and a drop height of 10 cm. This corresponds to test 1 and 2. Each other experiment has one characteristic changed from the reference in order to investigate its influence on the pile behavior.

A recapitulating table of the experiments can be found in table 8.1.

Experience Number	Sand Relative Density [%]	Pile Diameter [mm]	Drop Height [mm]	Embedded Length [mm]	Ratio $\frac{L}{D}$ [/]	Loading Height [mm]
1	76.4	60	100	330	5.5	910
2	76.4	60	100	330	5.5	970
3	76.4	60	50	330	5.5	960
4	76.4	60	100	600	10	695
5	57.5	60	100	330	5.5	950

Table 8.1: Experiments characteristics

Each experiment is composed of three phases :

1. Container filling
2. Pile driving
3. Lateral loading

The results for each phase are the following :

8.1 Container filling

In order to manage to do all the experiments in a short time frame, the dense sand container has not been emptied and refilled between experiments and the container has been used as is. This means that the container was only filled twice:

Density type	Sand fall height [mm]	Sand weight [kg]	Container volume [m^3]	Sand Density [$\frac{Mg}{m^3}$]	Sand Relative Density [%]
Dense	600	913.838	0.52	1.756	76.4
Loose	300	888.143	0.52	1.706	57.5

Table 8.2: Container filling data

The container filling curves are the following (fig. 8.1 and fig. 8.2) :

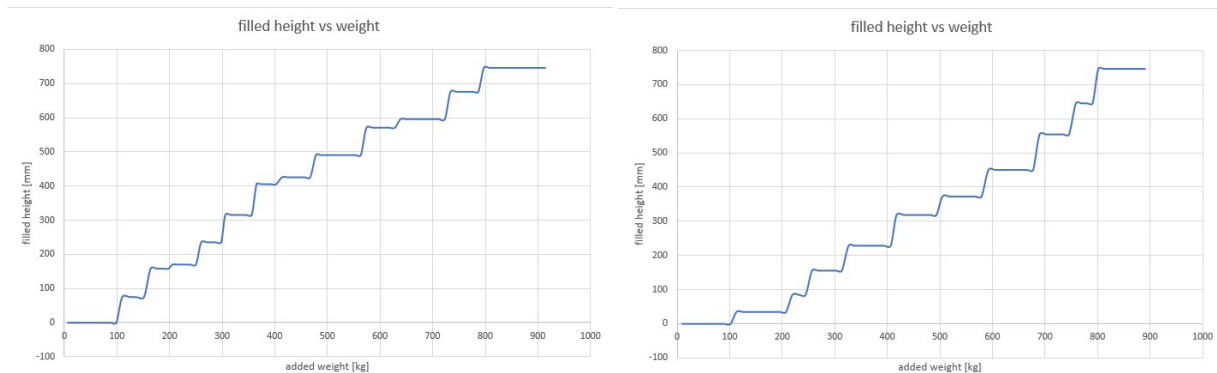


Figure 8.1: Filling curve for the dense sand

Figure 8.2: Filling curve for the loose sand

It is to be remarked that the height has been computed from the sand fall height, which explains the steps look of it and the fact it does not go up until the full height, 800 mm, but it is a good way to see the trend. The linear average between filled height and weight means that the relative density can be considered close to continuous over the total filled height.

8.2 Impact driving

During the driving phase of the experiments, each blows were recorded with its dropped height, set length, acceleration, container acceleration and impact duration. Using the acceleration, the set speed could also be computed. For the integration, a 5th degree Newton-Cotes formula was used, Boole's rule:

$$\int_{x_1}^{x_5} f(x)dx = \frac{2h}{45}(7f(x_1)+32f(x_2)+12f(x_3)+32f(x_4)+7f(x_5)) + \left(-\frac{8}{945}h^7 f^{(6)}\right)(\xi) \quad (8.1)$$

with x_{1-5} being following recordings, the last term being the error term and h defined as the time difference between two recordings.

Typical results

The typical results for an experiment look like the following, where, in order to have all the distinctive blows on one graph, their beginning time was incremented by a fraction of a second:

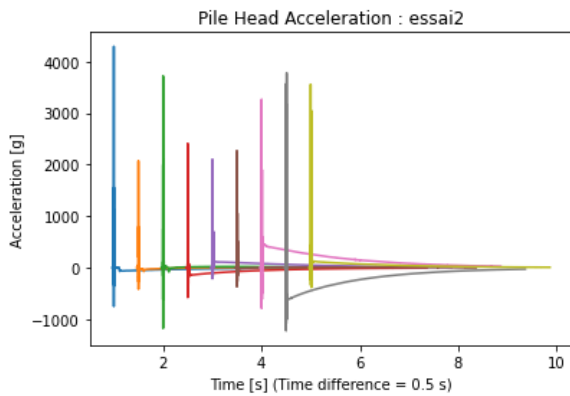


Figure 8.3: Pile acceleration measures for the second experiment

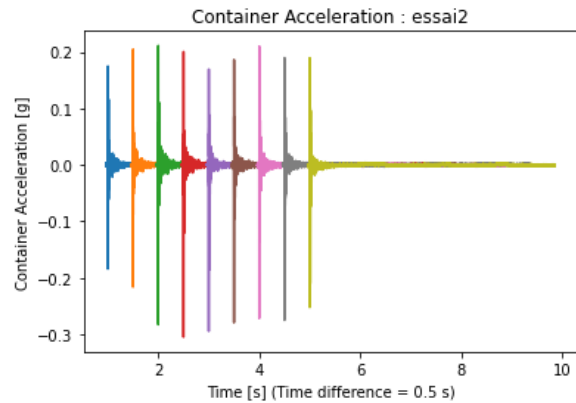


Figure 8.4: Container acceleration measures for the second experiment

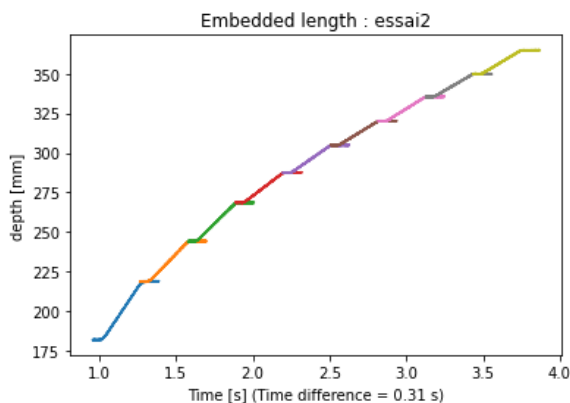


Figure 8.5: Displacement measures for the second experiment

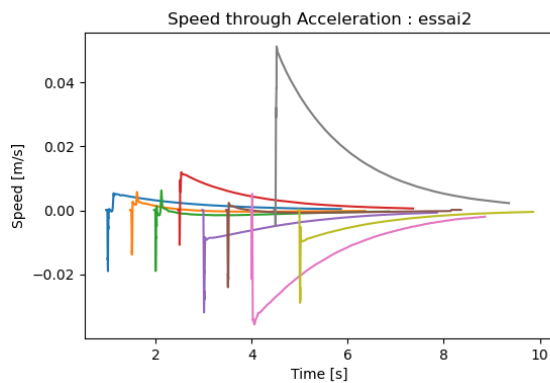


Figure 8.6: Set speed calculated for the second experiment

From these graphs already a few conclusions can be drawn :

The blows have a very short actual impact time.

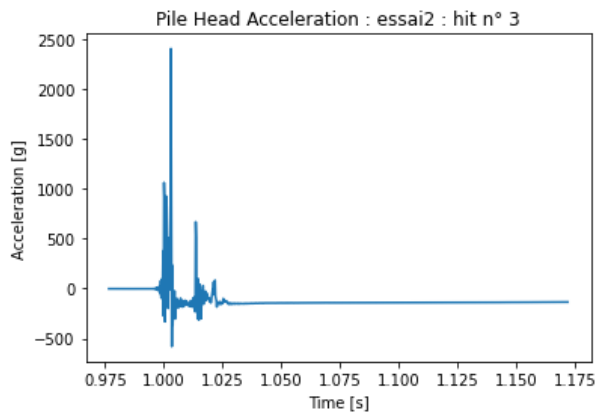


Figure 8.7: Small timescale pile acceleration

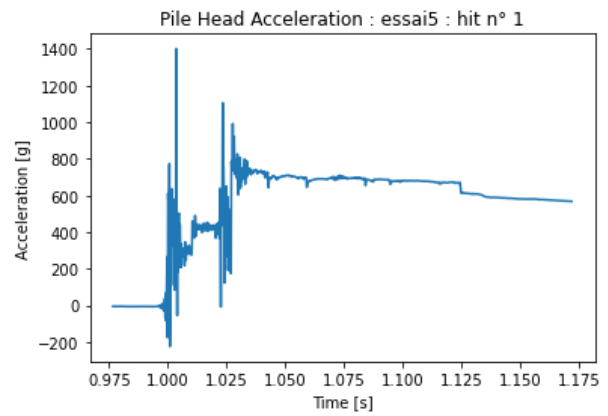


Figure 8.8: Small timescale pile acceleration

It can be seen here that there is a wave reflection after around 0.15 s for the second experiment and 0.2 s for the fifth. Due to the short length of the pile and the measure noise, the exact reflection time is not always easy to quantify but in most blows, it is laying between 0.008 s and 0.02 s. It is interesting to remark that the longer time have been recorded in the experiment with the looser sand (essai5).

In the graphs (fig. 8.3) it can also be remarked that for example for the blows n°7 and 8, the accelerometer continues to record an acceleration after the blow, while the displacement (fig. 8.5) has already reached its final value. This is quite rare and could be due to the fact that the accelerometer is fastened to a welded support outside the pile that takes time to get back to its original place and takes more time to settle than the pile. The fact that the laser target is located below the accelerometer on the pile and that the signal is stopping very clearly, without oscillations, would suggest that conclusion.

Furthermore, the amplitude and shape of the container acceleration values (fig. 8.4) are quite uniform and low compared to the pile head acceleration (fig. 8.3).

Finally, two conclusions can be drawn concerning the displacement (fig. 8.5) : it looks like the time taken for the pile to settle is uniform for all blows at around 0.31 s. On the other side, the total set per blow is decreasing with an increasing current embedded length.

Comparison of experimental results

In order to investigate these phenomenons further for each experiment, these values have been plotted against the embedded length for each experiment :

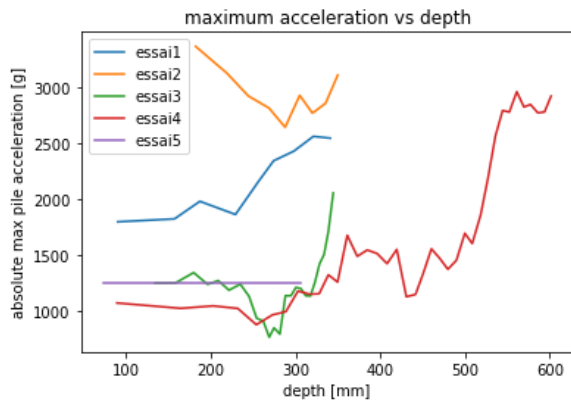


Figure 8.9: Pile acceleration measures against depth

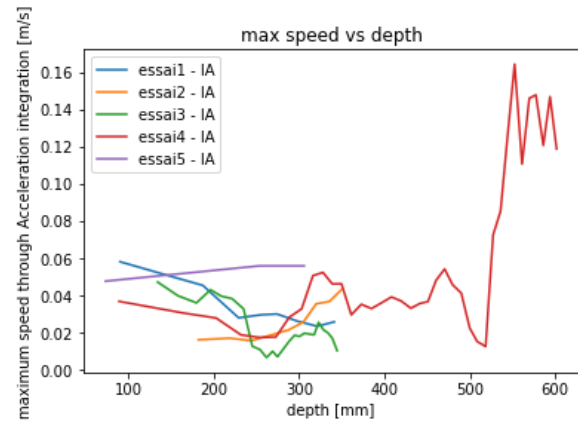


Figure 8.10: Set speed peak values against depth from acceleration integration

It is important to note that for fig. 8.9, a moving average of three points has been implemented to get the trend as actual values are oscillating and that it represents the maximal absolute value of each blow as some blows have a negative acceleration peak and no positive acceleration values. For fig. 8.10, the same treatment has been applied with a moving average of two points.

It can be seen in fig. 8.9 that the values of acceleration are higher for essai1 and essai2 and lower for essai3 and essai5. This observation could be expected as essai3 has a lower impact energy and essai5 has a lower sand density which leads to a decreased soil resistance and thus reduced force and acceleration.

On the other hand, the fact that essai4 has lower peak values when in the same domain is concerning. It might show that after three tests in the same sand, this layer is disturbed. Another explanation might come from the fact that the lateral support had to be set lower in order to drive the pile to deeper depths. It might thus just show the influence of the support height on the pile accelerations. It can be seen that those values get higher when the pile is driven further than 330 mm.

Furthermore, the set speeds (fig. 8.10) and set per blows (fig. 8.11) of the three experiments conducted in dense sand with a drop height of 10 cm are similar. That shows that as the the amplitude of the pile acceleration is lower, its duration must be longer, giving comparable results.

Finally, it can be remarked that at a depth of around 520 mm, 280 mm from the container bottom, both the maximum acceleration and speed are increasing quite rapidly. It might be a sign that from this point, the influence of the bottom of the container on the driving resistance of the pile is not negligible. It must be noted that the lateral support was removed after 455 mm of embedded length was reached. This might also cause some change in the recorded acceleration as less friction was applied to the pile outside the sand friction.

8 – Interpretation of Results

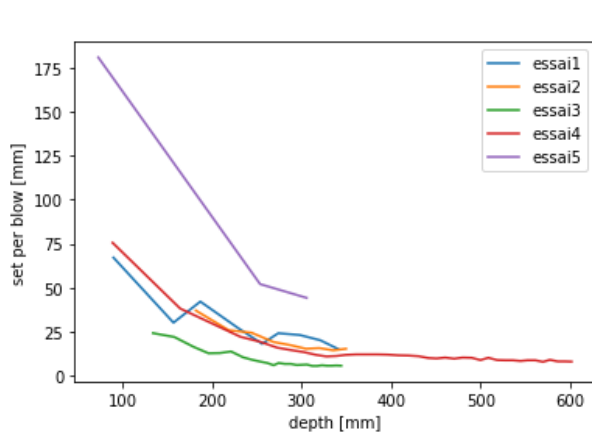


Figure 8.11: Set per blow against depth

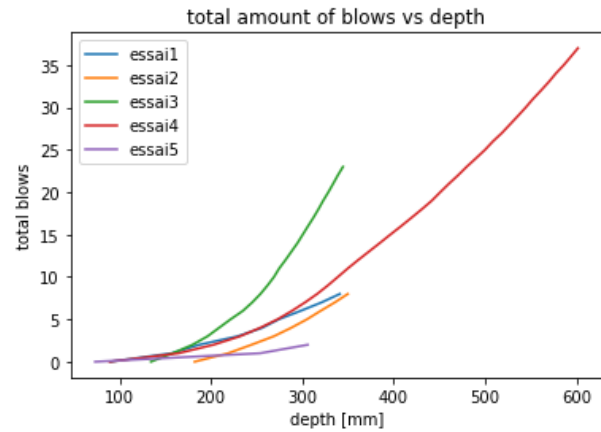


Figure 8.12: Total blows needed to reach a given depth

In both graphs, three groups can easily be distinguished: dense sand and drop height of 10 cm, dense sand and drop height of 5 cm and loose sand. As could be expected, it takes less blows to drive to the same depth a pile in loose sand than in dense sand. An increased drop height also means a higher set per blow. On the other side, it is interesting to notice that as the depth grows, the difference in set per blow closes in. That implies that, all driving characteristics kept, the refusal depth of looser sand will be way lower than that of denser sand, but it can be changed by increasing the drop height.

Extrapolation prediction equations can be drawn (fig. 8.13) :

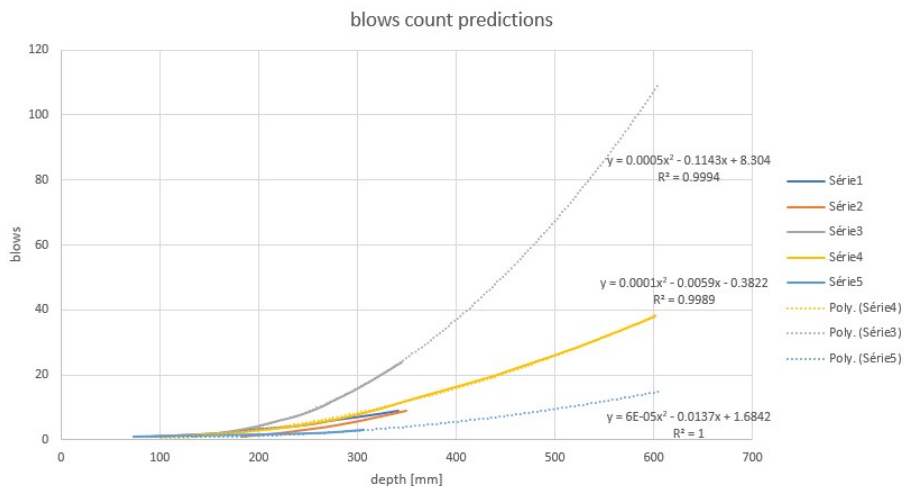


Figure 8.13: Blow count predictions

The x and x^2 coefficients of the predictive equations are nearly double for essai3-60-dense-5.5-hc5 compared to the reference experiments and nearly ten times those of essai5-60-loose-5.5. We can expect nearly 110 blows to get to a 600 mm embedded length for essai3-60-dense-5.5-hc5 while essai5-60-loose-5.5 would not need more than 20 blows to get to the same embedded length. It needs to be pointed out though that essai5-60-loose-5.5 got to the desired depth in only three blows which means that even though the curve fits perfectly the data, it is not very accurate for a large amount of blows.

Comparing these values with the values computed with PDI Wave (fig. 6.5), some substantial differences can be seen. Indeed while PDI did not assume any difference between the loose and dense sand, the experiments showed some substantial difference, needing three times less blows to get to the desired driving depth in essai5-60-loose-5.5 than for essai1-60-dense-5.5. Also, the software overestimated the amount of blows needed for each of the experiments, needing more than 1.5 more blows to get to the desired depth. This might also be one of the effects of the fact that the software stopped the simulation while the blow effects were still significant. But this fact is a good thing as it shows the software has some safety compared to the real results.

The fact that all three experiments with the same driving parameters have the same range of values for those two graphs comforts us in the fact that the reusing of the sand container for multiple experiments does not have a substantial impact on the driving.

Finally, it is interesting to look into the impact of the boundary conditions materialised by the container on the experimental results. To quantify this impact, the acceleration due to the shock wave was used :

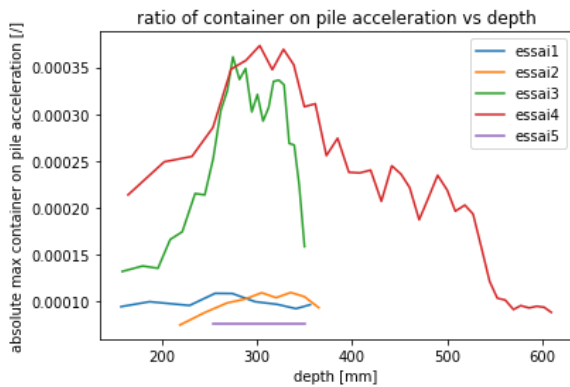


Figure 8.14: Container to pile acceleration ratio against depth

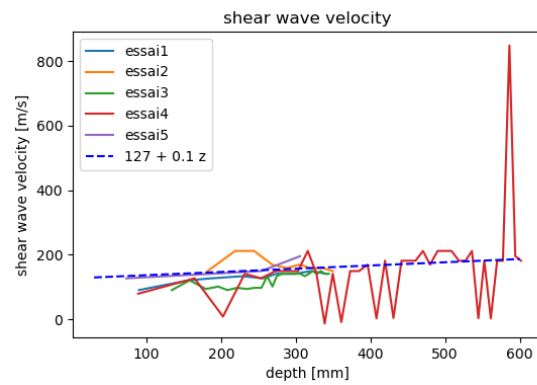


Figure 8.15: Velocity between pile and container acceleration peaks against depth

As the accelerometer was disposed on the top of the container's side, this is only a way of looking into the horizontal waves reflections on the sides. The vertical impact on the driving resistance has already been discussed about (fig. 8.9 and fig. 8.10).

The two experiments with a higher container to pile acceleration ratio are the two experiments with lower acceleration amplitude and that the ratio keeps close. It means that the value of the ratio is more dependant on the value of the pile acceleration than on the depth (fig. 8.16) :

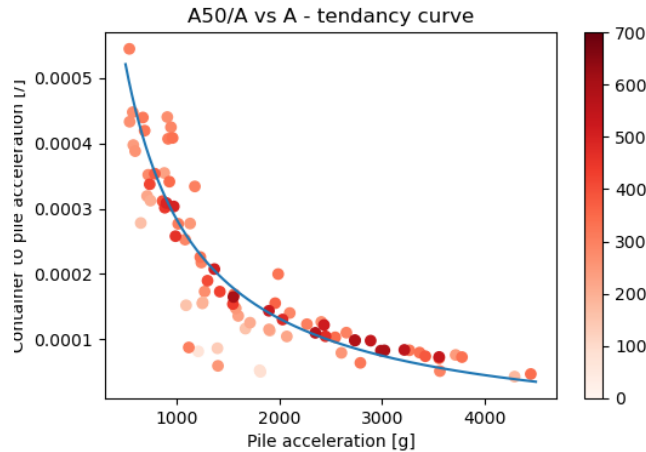


Figure 8.16: Prediction of the relation between the ratio and the pile acceleration

From this graph, it can be seen that the ratio is not dependant on the depth represented by the colormap. The relation is nearly linear though as the tendency line has the following equation :

$$Acc50/Acc = \frac{0.4}{Acc + 200} - 0.00005 \quad (8.2)$$

And fits quite well the data. The low values of the ratio and the fact that it is not dependant on the depth shows the low influence of the horizontal boundary conditions on the results.

Finally, from the time difference between peaks, an approximation of the wave velocity as a function of the depth can be computed:

$$v_{wave} = 127 + 0.1z \quad (8.3)$$

Indeed it is only an estimation as the pile acceleration spike time has been computed from an accelerometer located on the top of the pile. The wave travel time in the pile itself has not been taken into account and might be responsible for part of the speed difference with depth.

On the wave speed though two things can be pointed out :

First, there does not look like being a difference between the loose or dense sand. Maybe a bigger density difference is needed to be able to find the real change, but it means that for our experiments, the assumption that the wave velocity is a constant value is right. Secondly, this velocity can be put into perspective compared to the shear wave velocity computed thanks to the bender elements (table 8.3) :

Measures	Acceleration data (300 mm)	Saturation phase	Consolidation phase	Cyclic phase
wave velocity [m/s]	157	95.63	138.37	52.76
G [MPa]	43.3 / 42	17.22	36.05	5.24

Table 8.3: Comparison of shear wave velocities and shear modulus in the experiments and using the bender elements

The wave speed value computed using the accelerometer data is quite close to that measured after the consolidation phase. Considering this, these values could be used further to investigate the pile behavior during the loading phase.

8.3 Loading

Once the pile was driven, it was laterally loaded in two phases separated by an unloading. These will be denominated by *mec1* and *mec2* in the following results.

The force, horizontal displacement and axial strains were recorded during each phase of each test. These allowed to compute the internal forces and tipping angle as a function of the time.

Internal forces and angle calculations

The internal forces were computed using classical beam theory and the strains recorded on four locations at each 90° along the pile : W-S-E-N. The loading direction is the axis W-E, sometimes mentioned as x in the following graphs, while the transverse direction is N-S.

The averaging of the strains multiplied by the elastic modulus of steel, 210000 MPa, and the transverse area of the pile, $A_p = \pi(r^2 - (r - t)^2) = 275.675mm^2$ gives the axial stresses in the pile :

$$N = \epsilon \cdot E_p \cdot A_p \quad (8.4)$$

It was computed at each location along the pile for each timestep for each experiment.

The moment was computed through the sum of the contribution of 6000 small forces computed along the height of the transversal section of the pile considering a linear relation between the opposite strains :

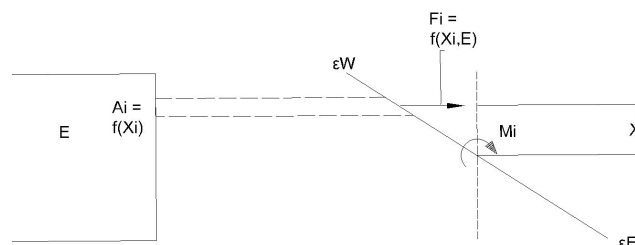


Figure 8.17: moments calculations from strain recordings

$$M_i = F_i X_i \quad (8.5)$$

$$M = \Sigma M_i \quad (8.6)$$

Finally, the angles are calculated from the displacement considering the pile rotates rigidly around its toe. this hypothesis is credible due to the pile's high bending stiffness, but might be less accurate for larger embedded length.

$$angle = atan\left(\frac{displacement_v}{h_{laser} + L}\right) \quad (8.7)$$

Typical results

Typical results for a test are the following :

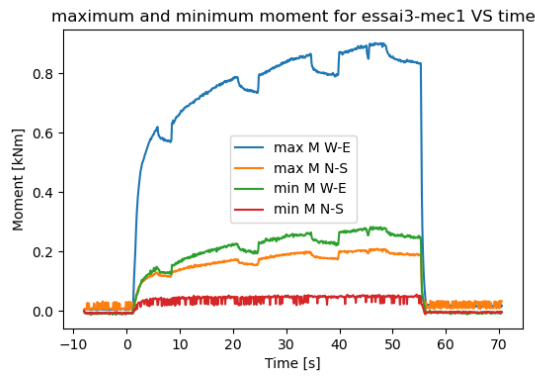


Figure 8.18: Moment evolution with time for essai3-mec1

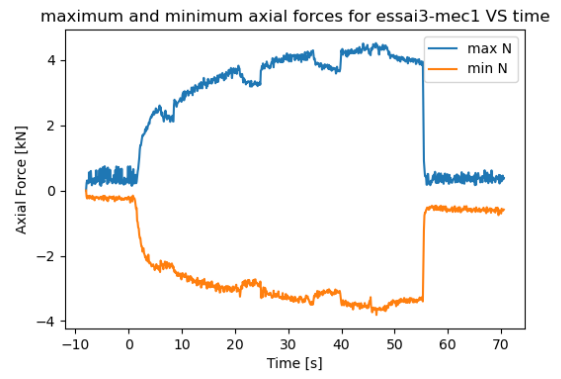


Figure 8.19: Axial forces evolution with time for essai3-mec1

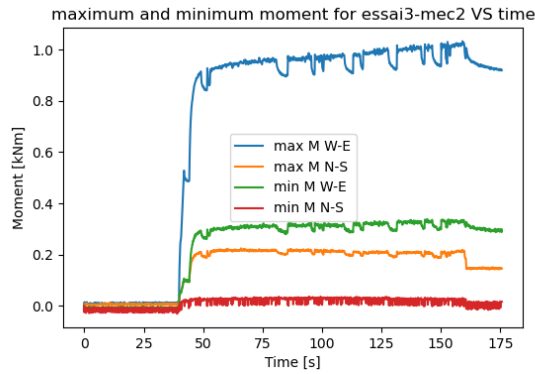


Figure 8.20: Moment evolution with time for essai3-mec2

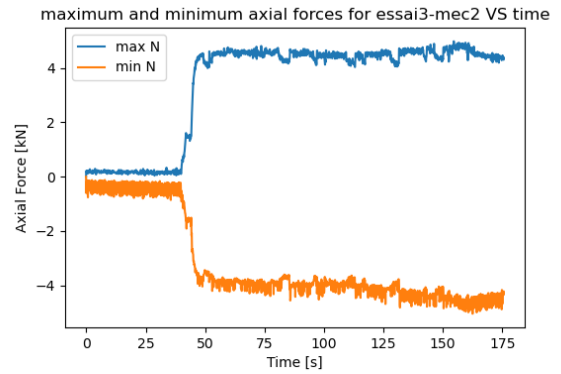


Figure 8.21: Axial forces evolution with time for essai3-mec2

Already here, some key features can be shown :

First of all, as can be seen in the graphs of mec1(fig. 8.19 and fig. 8.18), once the contact is totally released, there is no residual moment nor axial forces and the strains go back to the initial state. That means that the pile is still in its elastic phase, which is not the case of the soil.

Secondly, as the jack used to load the pile is connected to a hand pump, the pump strokes and reloading phases can clearly be identified. It highlights something quite interesting : once in place and with no loading increment, the internal forces will decrease with time. But as soon as the pile is loaded again, the forces necessary to increase the tipping angle are back on the original loading curve as can be seen in the following graph (fig. 8.22) :

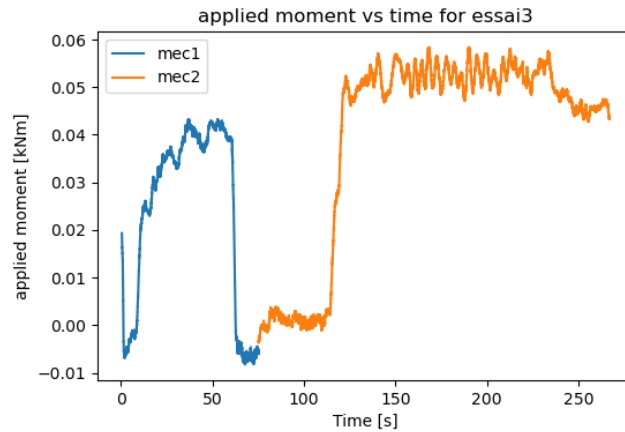


Figure 8.22: Evolution of the applied moment in time for essai3

Finally, another interesting feature is the shape of the loading curve : as could be expected, there is a linear relationship between the applied moment and the internal forces even though it is not easy to distinguish the exact relationship :

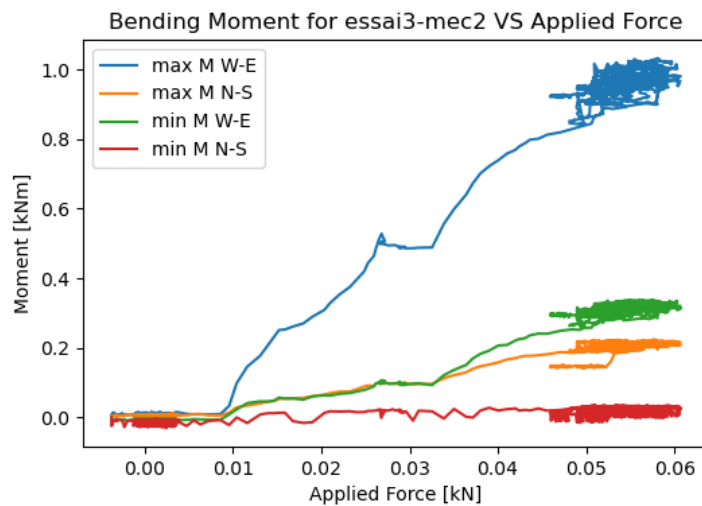


Figure 8.23: Relationship between the applied force and maximum moment for essai3-mec2

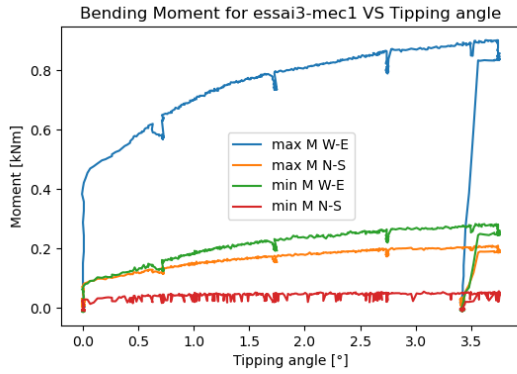


Figure 8.24: Moment evolution with tipping angle for essai3-mec1

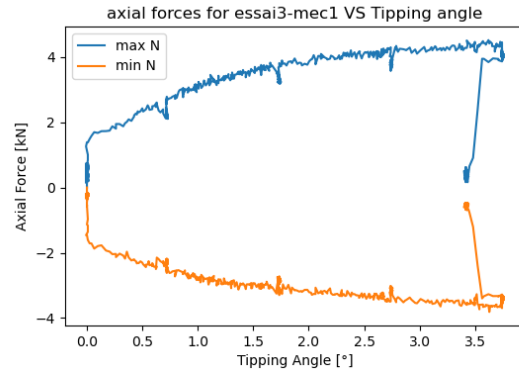


Figure 8.25: Axial forces evolution with tipping angle for essai3-mec1

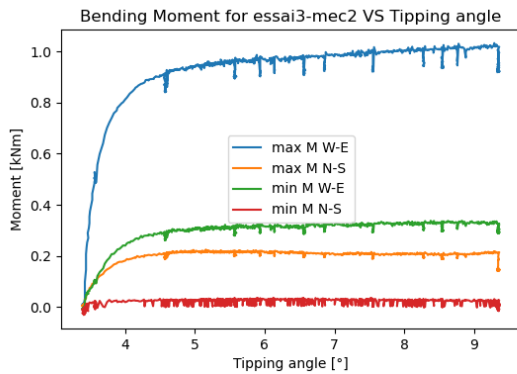


Figure 8.26: Moment evolution with tipping angle for essai3-mec2

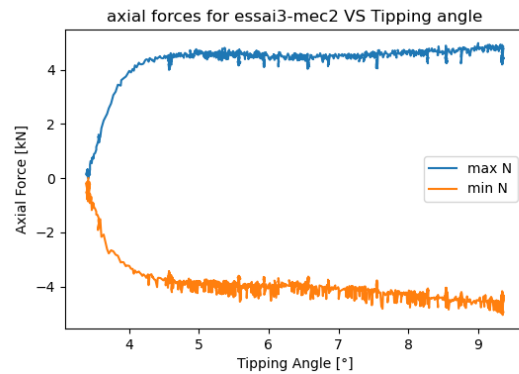


Figure 8.27: Axial forces evolution with tipping angle for essai3-mec2

These graphs (fig. 8.24, fig. 8.25, fig. 8.26 and fig. 8.27) show the evolution of the same internal forces as before but in relationship with the tipping angle. The moments vs angle graphs have a recognisable stress strain relationship form with two phases : an elastic phase until a fragment of a degree and a plastic phase afterwards.

It is interesting to note that when the pile got unloaded, its tipping angle recovers with the same modulus as the initial elastic modulus after having an instant angle drop. This instant drop is a particularity of this test and has not happened in other experiments. As the particularity of this experiment is the hammer energy while driving the probability of both being related is low but it might be interesting to investigate it more in detail with other drop heights.

8 – Interpretation of Results

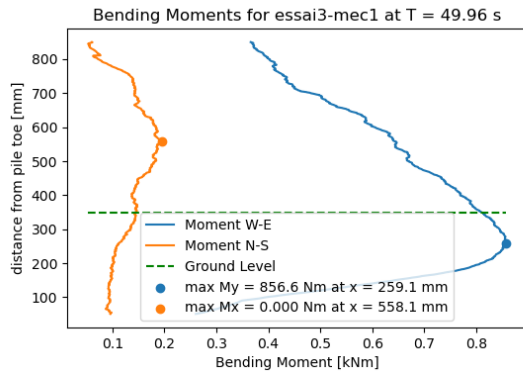


Figure 8.28: Moment evolution in the pile for essai3-mec2

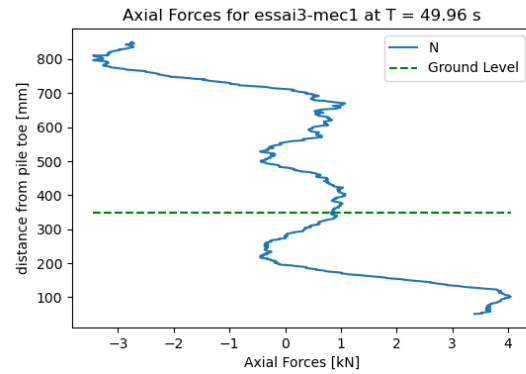


Figure 8.29: Axial forces evolution in the pile for essai3-mec1

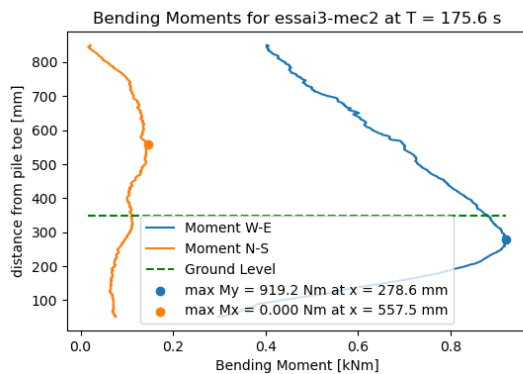


Figure 8.30: Moment evolution in the pile for essai3-mec2

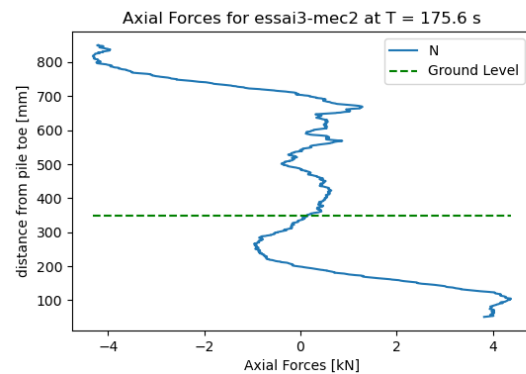


Figure 8.31: Axial forces evolution in the pile for essai3-mec2

Finally, these graphs show the evolution of the internal forces along the pile.

Concerning the moments, as could be expected, the maximal moment in the direction of loading is located just below the ground level and confirms that the internal forces respect the theory of active and passive earth pressure. It is interesting to note that the moment is very similar to the isostatic case of walls without anchor at head and clamped at toe of Blum's theory for embedded walls (fig. 8.32) :

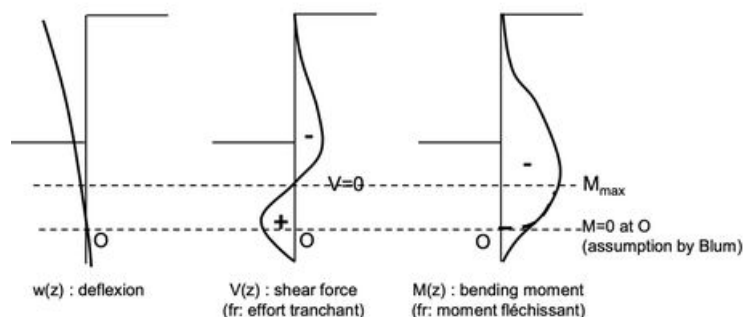


Figure 8.32: Internal forces of Blum's method for embedded walls clamped at toe (source : H.Rattez Design of Embedded retaining walls, LGCIV2071 Geotechnics,2020)

8 – Interpretation of Results

On the other side, the transversal moment is, as expected, quite low compared to the former. It is mostly uniform on the height of the pile and just slightly lowering when getting closer to the pile's ends. Its maximal value, as opposed to the former moment, is situated nearly at the middle of the pile length, above the ground level.

It would be expected that this kind of moment would be due to the ovalisation of the pile due to the load application. That would not create moment when the pile is taken as a single beam, but would create independent moments on the sidewall itself as can be seen from the strain records taken at the same timestamps (fig. 8.33 and fig. 8.34) :

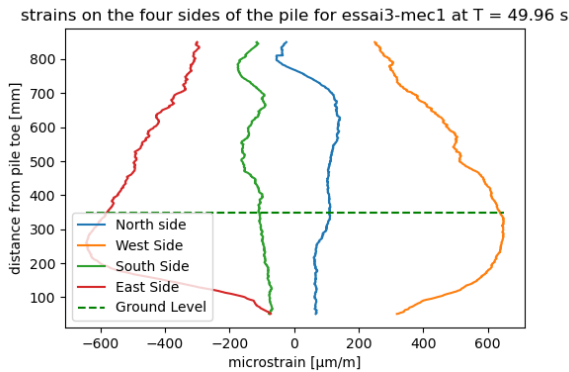


Figure 8.33: Strains evolution in the pile for essai3-mec1

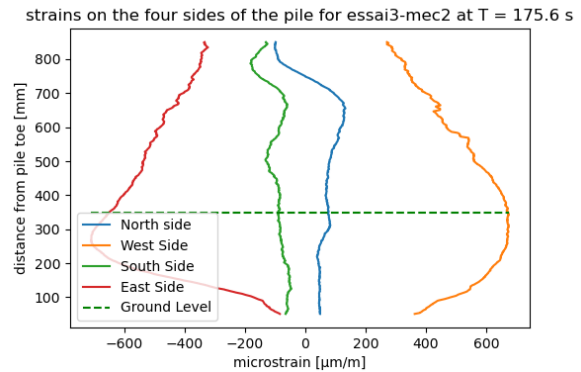


Figure 8.34: Strains evolution in the pile for essai3-mec2

Both lines are definitely nearly parallel and it can be seen that, at the top, some wave is happening showing the effect of the load on the transverse part of the tube.

The transversal and lateral moment values are uncoupled, showing no direct link from one to the other.

The interesting shape of axial forces graphs (fig. 8.29 and fig. 8.31) can be explained by these waves and by the shape of the moment values. The high level of axial forces close to the ends of the pile were not expected and are interesting to notice. They happen mostly when the strains in the N,S and E tend towards 0 and W still has a higher deformation. It can thus be expected that most of the effect of this normal force to happen close to the west side of the pile.

This is one of the disadvantages of considering the pile as a beam as it is difficult to understand the exact local impacts of the internal forces. This shows that as well as being checked for global failure, monopiles are subject to important local stresses disparities and should thus be checked for local stresses and buckling conditions imperatively.

Comparison of the different experiments

As moments, axial forces and tipping angle are important design characteristics for monopiles, these have been put together for each experiment in order to investigate the parameters impact on the response of the monopile (fig. 8.35, fig. 8.36 and fig. 8.37).

8 – Interpretation of Results

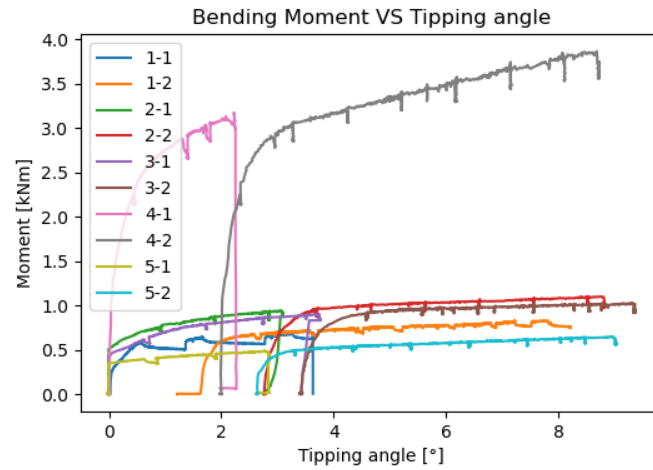


Figure 8.35: Maximal moment compared with tipping angle

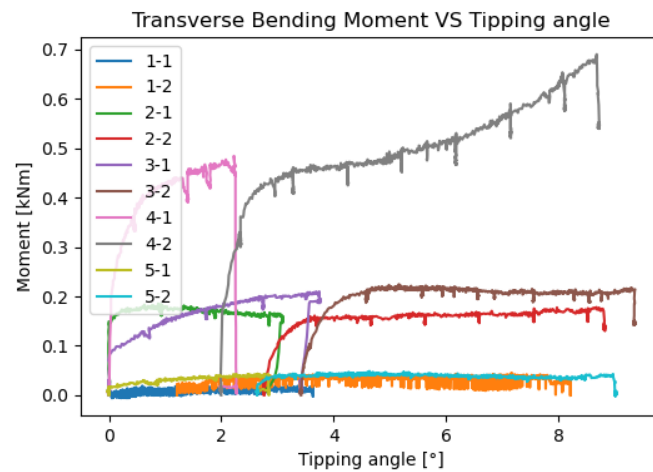


Figure 8.36: Maximal transverse moment compared with tipping angle

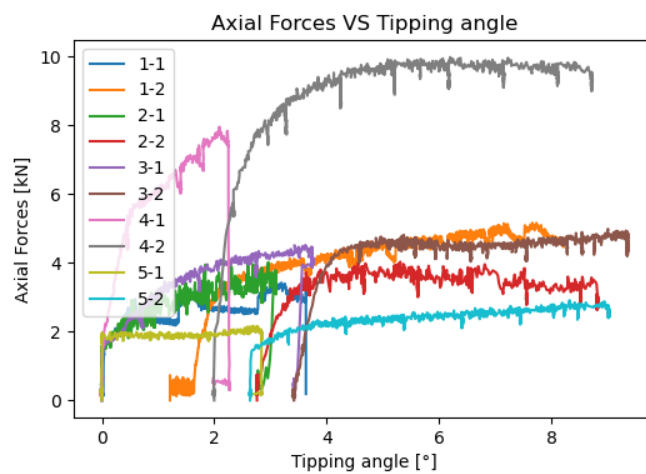


Figure 8.37: Maximal axial forces compared with tipping angle

As expected, three categories can be distinguished : the dense and loose sand experiments and the normal or longer embedded lengths.

Indeed, the longer embedded length induced a better resistance, but what is interesting to point out is the fact that while the embedded length was not even doubled, the resistance is nearly quadrupled. This again corresponds to what would be expected if the soil resistance was modelled using Rankine's theory.

Considering this, it also validates the relationships between model and real size resistance and stresses given in section 7.1, based on the same theory :

model ratio	value
λ	100
λ_σ	100
λ_F	1000000
λ_M	100000000

Table 8.4: Similitude ratios for force, moments and stresses

Using these relationships, and applying them to the values of the different parameter when the tipping angle reaches 2° , considered close to failure for a monopile, the following graphs representing the full scale values for the different experiments can be drawn:

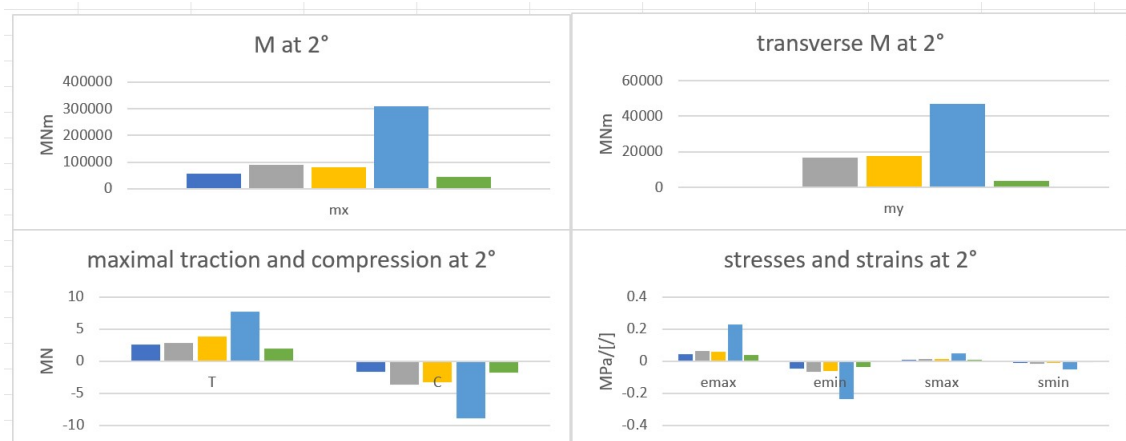


Figure 8.38: Full scale values at failure (2°)

In these values, as well as in the former graphs, the similar values of the three first experiments can be remarked, which confirms the fact that the use of the same filled container for multiple experiments does not change the results significantly. It also shows that the fact of using a lower drop height, thus a lower driving energy, does not change the lateral capacity of a pile significantly.

It can be noticed in fig. 8.38 that the strains and stresses values are very low while the force and moments compare with what could be the resistance of an offshore monopile. This is due to the fact that the pile used for the experiments has a lower d/t ratio than recommended in the literature. It was thus expected that the pile would show a more rigid behavior than full scale monopiles.

Furthermore, the influence of having a longer embedded length against the influence of having a less dense sand on the capacity can be investigated. While the looser sand gives a lower resistance, about having the admissible load here, the impact of a longer embedded length is more sensible, as it is a function of the squared length difference.

It could thus be more interesting to do more tests in order to confirm this relationship and to find the terminal embedded depth, where an increment in depth does not change significantly the increment in resistance.

Finally, coming back to the fig. 8.36, fig. 8.35 and fig. 8.37, it can be remarked that from a certain moment in time, the internal forces get to a point where the force increment needed to tip the pile a little bit further becomes very small and the curve gets nearly to a horizontal line. At that point, the pile does not have any resistance anymore, and can be considered past failure.

As a conclusion, the pile parameter which has the highest impact on the lateral resistance of piles is the embedded length where the relationship between the resistance and the length is quadratic. The state of the sand has an impact but which can be moderated using deeper foundations. Its impact is less important for laterally loaded piles than for axially loaded piles as can be seen when comparing the driving and loading values. A decreased driving energy does not create a substantial change in pile lateral capacity.

The pile behavior when laterally loaded follows Rankine theory of soil states. This theory might thus be used as a first rough approximation of the pile capacity.

Finally, it is very important to investigate the local stresses in the pile as some big disparities have been observed from looking at the pile using classical beam theory and the local strains and stresses recorded on four locations around the pile.

CHAPTER 9

Conclusion

To respond to the climate emergency and low reliability of fossil energies supply, European countries are changing their energy mix in order to incorporate more renewable energy sources such as wind turbines, hydroelectric dams and solar panels. Their goal is to become carbon neutral by 2050, in less than 30 years. There is still a lot of progress to be made from a technical and economic point of view in order to be completely independent of fossil fuels, which are more pollutant. Offshore wind turbines are a great opportunity to meet these challenges and reduce the environmental impact of energy production despite their high installation and maintenance costs and the intermittent nature of their production.

Thanks to initiatives like the SIMOX consortium, new installation techniques by impact and vibratory driving are being researched, developed and tested for the installation of future XXL monopiles. These piles increase energy efficiency and allow offshore wind energy to be a competitive option in the energy markets. This new generation of wind turbines has a capacity of 15 to 20 MW, a monopile with a diameter between 7.5 and 11 m and rotors with a diameter of up to 230 m. These turbines are also installed in deeper waters up to 65 m and subject to more severe environmental situations. To meet these new requirements, monopile transport companies have also needed to adapt and reinvent themselves to increase their wind turbines transport capacities .

Although monopiles are by far the most commonly used foundation for offshore wind turbines due to their simplicity, manufacture cost and installation ease, this type of foundation has technical and economic limitations. They are mainly used in shallow waters. New types of foundations are being researched to install wind turbines in deeper waters, notably floating foundations. In addition, in order to meet the growing demand for electricity, turbines are becoming larger and heavier, which requires increasingly robust foundations. Research is being carried out to reduce the weight by using stronger steels or by increasing the slenderness ratio.

Currently, most offshore wind turbines are installed in shallow waters down to 40-50 m. In order to install the monopiles, two main installation techniques are possible: impact and vibratory driving, which has emerged in recent years to overcome the negative effects of impact driving.

Impact driving involves hitting the pile top with a hammer to drive it into the seabed. The hammer's impact on the pile head generates a wave that propagates along the pile to the tip. The energy generated at the pile head overcomes the lateral friction along the pile and causes plastic deformation of the soil at the tip. This causes the pile to be driven in.

Impact driving is widely used for its high load-bearing capacity and its application to a variety of soil conditions. However, the vibrations induced by impact driving cause significant disturbance to marine life or buildings near wind farms.

To counteract the negative environmental impacts, the economic costs and the slow rate of impact driving, vibratory driving has been developed in recent years. This technique takes advantage of the fact that the soil resistance decreases under cyclic loading. The degradation of the soil strength makes it easier to drive the pile. Vibratory piling is based on the emission of longitudinal vibrations which generate only a vertical force that allows the pile to be driven.

This thesis contributes to a better understanding of the influence of impact driving, embedded length and relative density of dry sand on the lateral behaviour of the pile through a small scale model of a foundation pile for an offshore wind turbine.

Through this thesis, the properties of North sea sand were determined through different laboratory tests. These data can be used for simulations and other experimental tests, such as vibratory driving or impact driving in saturated soil.

From the various laboratory tests, it appears that the majority of the sand grain size is between 0.25 and 4 mm. Knowing the grain size of the sand is useful when sizing the openings of the three plates that make up the sand raining system. In order to be able to perform triaxial and cyclic triaxial tests correctly and to obtain different relative densities through the sand raining system, it is important to know the minimum and maximum density of the sand. In the case of North Sea sand, these are 1.821 Mg/m^3 and 1.572 Mg/m^3 respectively. Similarly, the optimum water content obtained using the Proctor test is equal to 7.63% and is necessary when carrying out saturated cyclic triaxial tests in order to achieve liquefaction and deduce all the properties that result from it.

Triaxial testing involves applying a deviatoric stress to a cylindrical specimen while subjecting it to an isotropic lateral stress to simulate stress conditions and shear failure. By applying three different cell pressures, Mohr's circles can be obtained, the cohesion and friction angle can be derived and the Young's modulus can be calculated in the elastic phase. For North Sea sand, it has a cohesion close to zero and an angle of friction between 30 and 35° depending on whether peak or residual data are used and whether the sand is loose or dense.

Cyclic triaxial tests measure the evolution of the soil's properties while cyclically loaded. At the end of the test, from the measurements of axial and lateral stresses and vertical strains, the shear modulus and damping ratio can be calculated. For saturated tests, the degradation and transition to liquefied soil can be studied. In this thesis, strain controlled cyclic tests have been undertaken with a strain amplitude of 0.15%. they were conducted on saturated or dry, loose or dense sand for a frequency varying between 1 Hz which is the wind frequency and 5 Hz which is the maximum frequency of the cyclic triaxial cell. The bender method also provides

the wave velocity in the sample and the shear modulus and was carried out in conjunction with a cyclic triaxial test.

In order to test the influence of sand drop height on relative density, a sand raining system was set up to reproduce a sandy soil that reflects the properties of natural sand deposits as closely as possible. This method allows to control the relative density of the sand in the container and to obtain relatively homogeneous sand samples repeatedly. The raining system consists of three plates with scattered openings with diameters of 13 mm, 8 mm and 6 mm respectively. For a constant drop height throughout the filling process, loose and dense sand are obtained for drop heights of 30 and 60 cm respectively. Above 60 cm, the influence of the drop height on the relative density is minimal.

In order to obtain a correspondence between the small-scale experiments and the full-scale effects, some non-dimensional groups must be respected in order to have a correlation between the results. For the purpose of this thesis, a factor of 100 was taken for the length dimension ratios and ratios $L/D = 5.5$ and 10 for the embedded length. The loading height is conditioned by the embedded length and the range H/D between 5 and 18.3 and H/L around 2.5 are respected. However, the scaled down pile is stiffer than it should be, which increases the inertia of the pile. The properties of the sand and steel are respected as in reality.

The pile driving system used in our experiments is based on the Fondeole system developed at UCLouvain by A.Holeyman and S.Hamdi. The pile-driving system consists of the free fall of a 10kg hammer on an anvil placed at the top of a 6 cm diameter, 1.35 m long and 1.5 mm thick pile. The hammer is lifted manually by pulling on ropes to the desired height and then released. This operation is repeated until the desired length is reached. A centering rod ensures the verticality of the hammer and prevents eccentricity of the blows. Lateral supports ensure that the pile is vertical over its entire height. The pile is equipped with an accelerometer, an optic fibre and a laser displacement sensor. A detailed description of the instrumentation and data acquisition system is also provided in section 7.4.

Once the pile has been driven to the desired depth, lateral loading is applied to the pile using a jack actuated by a hand pump. The lateral loading is done in two parts in order to see the influence of an unloading and then a full loading on the lateral behaviour of the pile.

In total, five experiments were carried out, four in dense sand and one in loose sand. A summary of the experiments carried out can be found in table 9.1 :

Experience Number	Sand Relative Density [%]	Pile Diameter [mm]	Drop Height [mm]	Embedded Length [mm]	Ratio $\frac{L}{D}$ [/]	Loading Height [mm]
1	76.4	60	100	330	5.5	910
2	76.4	60	100	330	5.5	970
3	76.4	60	50	330	5.5	960
4	76.4	60	100	600	10	695
5	57.5	60	100	330	5.5	950

Table 9.1: Experiments characteristics

During the pile driving phase of the experiments, each blow is recorded with its drop height, set length, the pile and container acceleration and the impact time. Using the acceleration, the velocity of the whole set could be calculated. Once the pile was driven, it was laterally loaded in two phases separated by unloading. The force, horizontal displacement and local strains on four sides of the pile were recorded during each phase of each test. These allowed the calculation of internal forces and tilt angle as a function of time.

As a result of the experiments, three categories can be distinguished: dense and loose sand experiments and normal or longer embedded lengths.

Indeed, a longer embedded length induces a better resistance. For an almost doubled embedded length, this results in an almost fourfold increase in strength. This resistance corresponds to the soil modelled by Rankine's theory. When the tilt angle reaches 2° , the monopile is close to failure.

From the graphs obtained, it appears that using the same filled container for several experiments does not significantly change the results. Furthermore, the use of a lower head and thus a lower driving energy does not change the pile capacity significantly.

The deformations and stresses obtained are very small, while the force and moments are comparable to the strength of an offshore monopile. This is due to the fact that the pile is stiffer than desired in the similarity laws. High axial forces levels have also been recorded at the pile toe and head, and might be an important design parameter. Finally, high stresses disparities compared to classical beam theory were recorded in the pile, highlighting the importance of taking local stresses into account while designing monopiles. In terms of internal forces, at a certain point, the increase in force required to tilt the pile a little further becomes small. At this point, the pile no longer offers any resistance and can be considered to have failed.

As a result of all these experiments, we can conclude that the parameter with the greatest impact on the lateral resistance is the embedded length, whose relationship between resistance and length is quadratic. The relative density of the sand has an impact but this can be moderated by using deeper foundations. Its impact has less consequence on laterally loaded piles than on axially loaded ones. Decreasing the hammer drop height has little effect on the lateral capacity of the pile but increases the number of blows during pile driving. The behaviour of piles under lateral loading follows Rankine's theory and this can be used as a first approximation. It is important to study the local stresses in the pile further through simulations and experiments.

9.1 Future perspectives

As has been said in the beginning of this thesis, this work imprints itself in a series of works that will happen around the same theme in the next few years. Indeed, as this thesis focused on scale piles installed by impact driving, it would be interesting to do the same experiments with piles installed through vibro-driving or jacking and observe the changes in the loading and displacement curves. That would allow to provide clearer guidelines on the impact of the installation process on the pile behavior.

Furthermore, it is interesting to do more experiments with model piles driven in dry sand. These experiments can have the same characteristics as the ones studied but with other ratios (e.g. different embedded length ratios in the spectrum 5.5-10, other sand densities or driving energies). It is also interesting to study the influence of these characteristics on a larger pile diameter. A 110 mm-diameter pile was prepared but no tests were undertaken with it due to a lack of time. As the average diameter of monopiles is fast increasing nowadays, it is important to be able to understand the scale effects in more detail.

Finally, all the tests have been undertaken in dry sand, which gives a good first approximation of monopiles behavior. However, the sand properties are strongly dependent on the water content (cfr. the cyclic loading results section 4.5.5). Since actual offshore monopiles are installed in saturated soils, it is interesting to study the effect of water content on the pile behaviour.

Concerning the sand properties tests, a few other tests might be interesting to undertake with this sand in order to characterise its properties more precisely.

First, classical and cyclic direct shear tests might be interesting to validate the results obtained by the triaxial tests, but also the disparities. Indeed, while the triaxial tests are recreating the soil conditions at the pile end while driving or vibratory driving, the direct shear tests recreate the conditions at the interface between the pile wall and the surrounding soil, and might thus tell more about the friction forces to be taken into account for a driveability analysis.

On another hand, classical triaxial tests with saturated or partially saturated specimens might be interesting to understand the effect of the soil's water content on its subsequent strength and behavior.

It could also be interesting to soil data computed in this thesis with soil data computed using correlations from CPT results in future offshore windturbines parks in the North sea .

On another note, a few of the tests undertaken have given interesting, but confusing results that merit to investigate further in order to understand if it is just a one off due to inaccurate results, or if some interesting mechanism is hiding behind it.

- The possible sand healing properties (fig. 4.33) might be interesting to look further into with longer and recorded loading stopping times as it could be an interesting property for the long term soil degradation around cyclically loaded structures.
- The instant tip angle recovery when the load is removed for the pile installed with the lower driving energy (fig. 8.24) is also interesting, but probably due to a problem with the laser recording saved. It might still be interesting to investigate other piles driven with another driving energy in order to confirm this hypothesis.

Finally, other foundations types could be investigated as all foundation types will be part of the mix that will allow us to lower our dependency on fossil fuels thanks to offshore windturbines.

Of those, a specific mention should be given to floating windturbines design as they will allow to extend the possible windturbine area outside the close to shore continental shelf and create a bigger possibility for new windturbines development.

Bibliography

- [1] Standard NBN EN/ISO 17892-4. Reconnaissance et essais géotechniques – Essais de laboratoire sur les sols – Partie 4 : Détermination de la distribution granulométrique des particules. *International Organization for Standardization*, pages 1–37, 2004.
- [2] Standard NBN EN/ISO 17892-8. Reconnaissance et essais géotechniques - Essais de laboratoire sur les sols - Partie 8: Essai triaxial non consolidé non drainé. *International Organization for Standardization*, pages 1–27, 2018.
- [3] 4Coffshore. Wind turbine information database. <https://map.4coffshore.com/offshorewind/>. Accessed: 2022-06-5.
- [4] Standard NBN EN 933-1. Essais pour déterminer les caractéristiques géométriques des granulats – Partie 1 : Détermination de la granularité – Analyse granulométrique par tamisage. *European Committee for Standardization*, pages 1–19, 2012.
- [5] Standard NF P 94-059. Sols:Reconnaissance et essais:Détermination des masses volumiques minimales et maximales des sols non cohérents. *Afnor Edition*, pages 1–21, 2000.
- [6] NF P 94-093. Sols : Reconnaissance et Essais – Determination des références de compactage d'un materiau – Essai Proctor normal – Essai Proctor modifié. *Edition Afnor*, pages 1–28, 2014.
- [7] J. Abdollahi and J. Bolouri Bazaz. Reconstitution of sand specimens using a rainer system. *International Journal of Engineering*, pages 1451–1463, 2017.
- [8] Gordillo Africa. COP26 : les objectifs climatiques de l'Union européenne, un écheveau ambitieux à 27 États. <https://www.rtbef.be/article/>. Accessed: 2022-05-17.
- [9] Muhammad Arshad and Brendan C. O'Kelly. Offshore wind turbine structures: A review. *ICE Publishing*, pages 139–152, 2013.
- [10] Fatiha Bahda. Etude du comportement du sable à l'appareil triaxial: Expérience et modélisation. *Ecole nationale des ponts et chaussées*, pages 1–271, 1997.
- [11] BOP. Belgian Offshore Platform - Accueil. <https://www.belgianoffshoreplatform.be/fr/>. Accessed: 2022-05-17.

- [12] K.T. Brodbaek, M. Moller, S.P.H Sorensen, and Anders Hust Augustesen. Review of p-y relationships in cohesionless soil. *Aalborg University*, pages 1–33, 2009.
- [13] Bruce. Hydraulic vibro hammer. <https://www.powerquip.co.kr/vibratory-hammers/>. Accessed: 2022-06-11.
- [14] Byron W. Byrne, Ross Mcadam, Harvey Burd, and Guy Timmouth Houlsby. PISA: New design methods for offshore wind turbine monopiles. *Society Underwater Technology*, pages 142–161, 2017.
- [15] Choisir.com. L'éolienne offshore: quel avenir pour la filière de l'éolien en mer? <https://www.choisir.com/energie/articles/160796/leolienne-offshore-quel-avenir-pour-la-filiere-de-leolien-en-mer>. Accessed: 2022-08-10.
- [16] Paul Da Mota. Fiche technique FT 7.10.4 - L'essai Proctor. Accessed: 2022-05-17.
- [17] Huynh Dat Vu Khoa, Tang Anh Minh, Doan Dinh Hong, and Phil Watson. *Proceedings of the 2nd Vietnam Symposium on Advances in Offshore Engineering: Sustainable energy and marine planning*. Springer, 2022.
- [18] L. de Neef, P. Middendorp, and J. Bakker. Installation of monopiles by vibrohammers for the Riffgat project. pages 1–14, 2013.
- [19] Frick Dennis and Achmus Martin. An experimental study on the parameters affecting the cyclic lateral response of monopiles for offshore wind turbines in sand. *Soils and Foundations* 60, pages 1570–1587, 2020.
- [20] Connaissances des énergies. Éolien offshore : les Pays-Bas en tête des installations européennes en 2020. <https://www.connaissancedesenergies.org/eolien-offshore-les-pays-bas-en-tete-des-installations-europeennes-en-2020-220218>. Accessed: 2022-05-17.
- [21] Paul Doherty and Kenneth Gavin. Laterally loaded monopile design for offshore wind farms. *ICE publishing*, pages 7–17, 2011.
- [22] Eoliennes en mer: Iles d'Yeu et de Noirmoutier. Fiche thématique d - les différents types de foncations existants. pages 1–4, 2015.
- [23] Topsector energie. Sustainable installation of XXL monopiles. <https://projecten.topsectorenergie.nl/projecten/sustainable-installation-of-xxl-monopiles-35257>. Accessed: 2022-06-12.
- [24] M.D. Esteban and al. Gravity based support structures for offshore wind turbine generators: Review of the installation process. *ScienceDirect*, pages 281–291, 2015.
- [25] Euronews. L'industrie européenne des éoliennes offshore a le vent en poupe. <https://fr.euronews.com/green/2021/06/09/l-industrie-europeenne-des-eoliennes-offshore-a-le-vent-en-poupe>. Accessed: 2022-05-17.
- [26] Wind Europe. Daily wind power numbers. <https://windeurope.org/about-wind/daily-wind-archive/2022-05-31/>. Accessed: 2022-06-5.

- [27] Wind Europe. Offshore wind in Europe. pages 1–40, 2019.
- [28] Commission européenne. Développer les Énergies renouvelables en mer pour une Europe climatiquement neutre. *Commission européenne*, pages 1–32, 2020.
- [29] Jan Fisher, Hauke Sychla, Joost Bakker, and Joachim Stahlmann. A comparison between impact driven and vibratory driven steel piles in the German North Sea. 2013.
- [30] Comité français de mécanique des sols et géotechnique. Recommendations for planning and designing foundations of offshore wind turbines. *CFMS*, pages 1–172, 2020.
- [31] Kenneth Gavin, S.D. Block, and G.R. Eiksund. The impact of installation method on monotonic loading of monopiles in sand. *TU Delft*, pages 1–11, 2020.
- [32] Factor Geotechnical. What does friction angle of soil mean? <https://factorgeo.com/what-does-friction-angle-of-soil-mean/>. Accessed: 2022-06-11.
- [33] Grow. Sustainable installation of XXL Monopiles (SIMOX). <https://www.grow-offshorewind.nl/project/simox>. Accessed: 2022-08-17.
- [34] M. Hakhamaneshi, J.A. Black, A. Cargill, C.M. Cox, and T. Elmrom. Development and calibration of a sand pluviation device for preparation of model sand bed for centrifuge tests. *International society for soil mechanics and geotechnical engineering*, pages 2–7, 2017.
- [35] Chennarapu Hariprasad, Mekala Rajeshkhar, and Balunaini Umashankar. Preparation of uniform sand specimens using stationary pluviation and vibratory methods. *Geotechnical and Geological engineering*, pages 1–15, 2016.
- [36] A. Holeyman. Contribution à l'étude du comportement dynamique non-linéaire des pieux lors de leur battage. *Université catholique de Louvain*, pages 1–600, 1984.
- [37] A. Holeyman. Vibratory driving analysis. Sixth international conference on the application of stress-wave theory to piles. pages 479–494, 2000.
- [38] A. Holeyman. Battage de pieux métalliques dans la roche. *Revue Francaise de Géotechnique*, pages 153–174, 2017.
- [39] IHC. IHC Hydrohammer - Pile driving equipment. pages 1–13, 2010.
- [40] Deep Foundations institute and GDG. Comparaison of impact versus vibratory driven piles: With a focus on soil-structure interaction. pages 1–76, 2015.
- [41] GDS Instrument. GDS Bender Element system. https://www.gdsinstruments.com/___assets___/Products/00025/GDSBES_Datasheet.pdf. Accessed: 2022-08-1.
- [42] GDS Instrument. GDS entreprise level dynamic triaxial testing system. <https://www.gdsinstruments.com/gds-products/gds-enterprise-level-dynamic-triaxial-testing-system>. Accessed: 2022-07-31.
- [43] BYRON W. BYRNE IONA A. RICHARDS and GUY T. HOULSBY. Monopile rotation under complex cyclic lateral loading in sand. *Géotechnique 70*, pages 916–930, 2020.

- [44] Mustafa Jafari. Numerical and experimental investigation of monopile driving resistance in carbonate rocks. *Université Catholique de Louvain*, pages 1–245, 2020.
- [45] Vinil Kumar Gade and Murty Dasaka. Development of a mechanized traveling pluviator to prepare reconstituted uniform sand specimens. *ASCE*, 2016.
- [46] Arany Laszlo, S. Bhattacharya, John Macdonald, and S.J. Hogan. Design of monopiles for offshore wind turbines in 10 steps. *ScienceDirect*, pages 126–152, 2017.
- [47] Toute l'Europe. L'énergie dans l'Union Européenne. <https://www.touteleurope.eu/environnement/1-energie-dans-l-union-europeenne/>. Accessed: 2022-05-17.
- [48] D. Levacher, J. Garnier, and P. Chambon. Reconstitution d'éprouvettes de sable - Appareils de pluviation. *Revue française de géotechnique*, pages 49–56, 1994.
- [49] Diego C.F Lo Presti, Sergio Pedroni, and Virginio Crippa. Maximum dry density of cohesionless soils by pluviation and by ASTM D 4253-83: A comparative study. *Geotechnical Testing Journal*, pages 180–189, 1992.
- [50] Luna. HD6S High-Definition Fiber Optic Strain Sensors. [https://lunainc.com/product/hd6s#:~:text=Learn%20More-,High%2DDefinition%20Distributed%20Fiber%20Optic%20Sensing,sensor%20\(up%20to%2050m\)](https://lunainc.com/product/hd6s#:~:text=Learn%20More-,High%2DDefinition%20Distributed%20Fiber%20Optic%20Sensing,sensor%20(up%20to%2050m).). Accessed: 2022-07-20.
- [51] Rainer Matuschek and Klaus Betke. Measurements of construction noise during pile driving of offshore research platforms and wind farms. *Institut für technische und angewandte physik*, pages 262–265, 2009.
- [52] Sols mesures. Mécanique des sols : Mémo essai triaxial. <http://test.sols-mesures.com/wp-content/uploads/2015/11/memo-essai-triaxial.pdf>. Accessed: 2022-05-17.
- [53] Kelle Moreau. L'impact environnemental des parcs éoliens offshore: résultats à propos de la plus grande zone éolienne opérationnelle du monde. [https://www.naturalsciences.be/fr/news/item/19981#:~:text=Les%20parcs%20%C3%A9oliens%20offshore%20influencent,intensivement%20de%20la%20faune%20colonisatrice](https://www.naturalsciences.be/fr/news/item/19981#:~:text=Les%20parcs%20%C3%A9oliens%20offshore%20influencent,intensivement%20de%20la%20faune%20colonisatrice.). Accessed: 2022-08-10.
- [54] Jens Niclaes. Recueil de procédures d'essai. *Ecole Polytechnique de Louvain*, pages 1–33, 2021.
- [55] L'usine nouvelle. Ces sept pays plombent la décarbonation de l'électricité européenne. *Geotechnical Testing Journal*, 2020.
- [56] Duan Nuo and Chenf Yi Pik. Mechanical characteristics of monopile foundation in sand for offshore wind turbine. pages 1–7, 2015.
- [57] offshoreWIND.biz. Offshoretronic presents installation solution for ultra long XXL monopiles. <https://www.offshorewind.biz/2021/07/30/offshoretronic-presents-installation-solution-for-ultra-long-xxl-monopiles/>. Accessed: 2022-06-12.

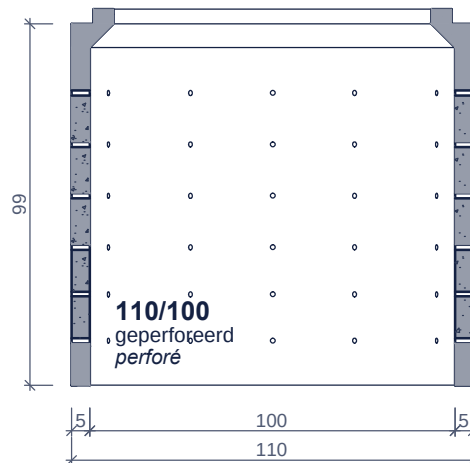
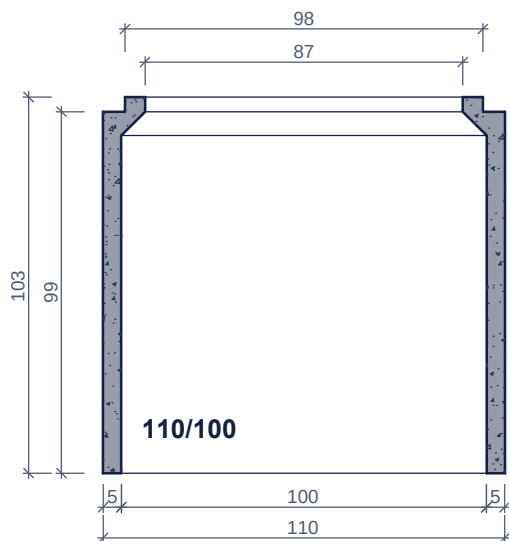
- [58] offshoreWINND.biz. Beyond XXL-Slim monopiles for deep water wind farms. <https://www.offshorewind.biz/2020/05/11/beyond-xxl-slim-monopiles-for-deep-water-wind-farms/>. Accessed: 2022-06-12.
- [59] American petroleum institute. Recommended practice for planning, designing and constructing fixed offshore platforms - working stress design. *API energy*, pages 1–288, 2000.
- [60] Sean Rees. Part one: Introduction to triaxial testing. *GDSinstruments*, pages 1–4, 2013.
- [61] Fabian Remspecher, Hung Le Viet, Frank Rackwitz, Volker Herwig, and Benjamin Matlock. Vibratory driven installation of monopiles-an experimental investigation of the soil-pile interaction. pages 1–7, 2019.
- [62] A. Rodger and G. Littlejohn. A study of vibratory driving in granular soils. In *Geotechnique*, volume 30, pages 269–293, 1980.
- [63] Feia Sadok. Essais triaxiaux drainés et non drainés sur sables saturés avec mesure locale des déplacements. *Cermes Procédure d’essai*, pages 1–17, 2008.
- [64] P. Staubach, R. Machacek, J. abd Sharif, and T. Wichtmann. Back-analysis of model tests on piles in sand subjected to long-term laterak cyclic loading: Impact of the pile installation and application of the HCA model. *ScienceDirect*, pages 1–17, 2021.
- [65] VJ TECH. Introduction to Bender Element testing. <https://www.vjtech.co.uk/blog/introduction-to-bender-element-testing>. Accessed: 2022-08-7.
- [66] VJ TECH. Triaxial cell-sand sample preparation. <https://vjtech.co.uk/blog/triaxial-cell-sand-sample-preparation>. Accessed: 2022-08-7.
- [67] K. Sien Ti, Bujang B.K. Huat, J. Noorzaie, M.S. Jaafar, and G. See Sew. A review of basic constitutive models for geotechnical application. *EJGE*, pages 1–18, 2009.
- [68] Dave Trudeep N. and Murty Dasaka. Assessment of portable traveling pluviator to prepare reconstituted sand specimens. *Geomechanics and engineering*, pages 79–90, 2012.
- [69] Carbon trust. Floating wind joint industry project: Phase III: Summary report. pages 1–68, 2021.
- [70] Athanasios Tsetas, Apostolos Tsouvalas, and Andrei V. Metrikine. Installation of large diameter monopiles: Introducing wave dispersion and non-local soil reaction. *Marine Science and Engineering*, pages 1–17, 2021.
- [71] Jean-François Vanden Berghe. Sand strength degradation within the framework of vibratory pile driving. *Université Catholique de Louvain*, pages 1–359, 2001.
- [72] Negro Vicente, López-Gutiérrez José-Santos, Esteban[U+204E] M. Dolores, Alberdi Pablo, Imaz Mario, and Serracarla José-María. Monopiles in offshore wind: Preliminary estimate of main dimensions. *Ocean Engineering v.133*, pages 253–261, 2017.
- [73] Saleem Zohaib. Alternatives and modifications of monopile foundation or its installation technique for noise mitigation. *TUDeft*, pages 1–68, 2011.

APPENDIX A

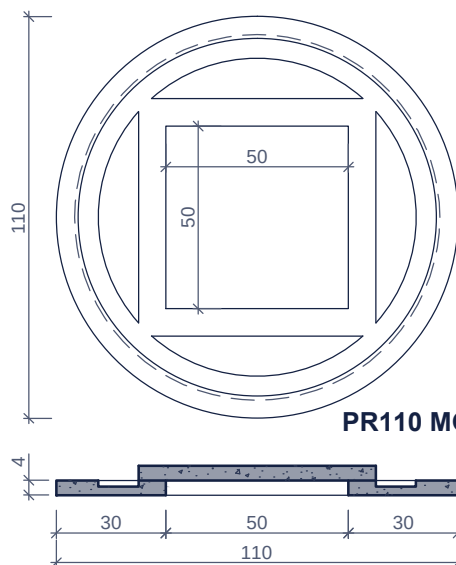
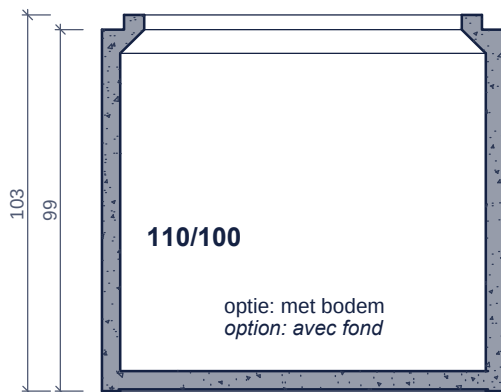
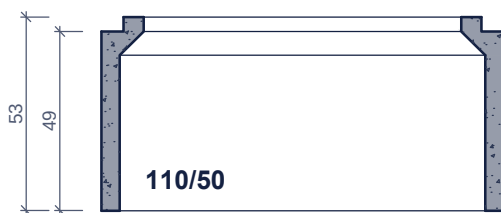
Technical sheet of concrete tank

PUTRINGEN Ø 100 ANNEAUX DE PUIITS Ø 100

Toepassing:
infiltratie van water
Application:
infiltration de l'eau

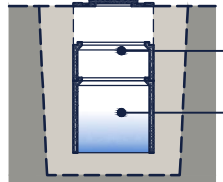


doorsnede van een geboorde betonnen ring
coupe anneau en béton perforé



betonnen deksel couvercle en béton

gewicht / poids = 132kg
hijslussen / points d'ancrage = 3

Kenmerken	PR100	Caractéristiques	Principe
gewicht 110/50	300kg	poids 110/50	
gewicht 110/100	440kg	poids 110/100	
buitenafmetingen	Ø 110cm / H 50-100cm	dimensions extérieures	Plaatsingsvoorschriften op aanvraag Prescriptions de pose sur demande
plaatsingsdiepte Dmax	4,00m	profondeur de pose Dmax	
verkeersbelastingsklasse	A15 (NBN EN 124)	classe de charge de trafic	
omgevingsklasse	EE3 (NBN B 15-001)	classe d'environnement	
milieuklasse	XC4, XF1 (NBN EN 206-1)	classe d'exposition	
druksterkteklasse	C35/45	classe de résistance	

afmetingen in cm - wijzigingen voorbehouden
dimensions en cm - sous réserve de modifications

APPENDIX B

Impact driving data

A Essai1-60-Dense-5.5

- Initial data :
 - Initial pile driving : 9.5 cm
 - Pile height : 126 cm
- Data during impact driving process :

Number of blows	Drop height [cm]	Pile height [cm]	Laser measurement [mm]
1	10	119.3	67
2	10	116.3	104.63
3	10	112.1	137.56
4	10	109.4	-
5	10	107.6	-
6	10	105.2	-
7	10	102.9	-
8	10	100.9	-
9	10	99.4	-

Table B.1: Data during impact driving

B Essai2-60-Dense-5.5

- Initial data :
 - Initial pile driving : 8.6 cm
 - Pile height : 126.4 cm
- Data during impact driving :

Number of blows	Drop height [cm]	Pile height [cm]	Laser measurement [mm]
1	10	116.8	2.78
2	10	113.1	-34.17
3	10	110.6	-59.49
4	10	108.1	-89.73
5	10	106.6	-102.97
6	10	104.9	-120.08
7	10	102.7	-135.31
8	10	102.1	-150.7
9	10	100.1	-165.01
10	10	98.9	-179.99

Table B.2: Data during impact driving

- Data during lateral loading :
 - Laser-support distance : 31.7 cm
 - Laser height : 50.9 cm
 - Rope height : 45 cm
 - Sand height in the pile : 30.3 cm

C Essai3-60-dense-5.5-hc5

- Initial data :
 - Initial pile driving : 9,4 cm
 - Pile height : 125.6 cm
 - Laser initial height : 98.6 mm
- Data during impact driving :

Number of blows	Drop height [cm]	Pile height [cm]	Laser measurement [mm]
1	5	121.2	58.17
2	5	120.3	34.07
3	5	117.2	12.59
4	5	115.2	-3.58
5	5	114.2	-16.49
6	5	113.6	-29.117
7	5	111.9	-42.69
8	5	110.8	-52.9
9	5	110	-61.87
10	5	108.9	-69.66
11	5	108.3	-76.65
12	5	107.5	-82.32
13	5	106.9	-89.36
14	5	106	-95.96
15	5	106	-102.38
16	5	105.6	-108.26
17	5	104.8	-114.08
18	5	104	-120.09
19	5	103.8	-125.26
20	5	103	-130.7
21	5	102.4	-136.18
22	5	102	-141.68
23	5	101.5	-146.87
24	5	100.4	-152.14
25	5	100.2	-157.71

Table B.3: Data during impact driving

- Size of the weathered surface after lateral loading :
 - Left of the pile : Length=18 cm and width=11 cm
 - Right of the pile : Length=18 cm and width=10 cm

D Essai4-60-dense-10

- Initial data :
 - Initial pile driving : 8.9 cm
 - Pile height : 126.1 cm
- Data during impact driving :

Number of blows	Drop height [cm]	Pile height [cm]	Laser measurement [mm]
1	10	119.3	-75.5
2	10	114.5	-113.68
3	10	111.9	-142.58
4	10	109.7	-164.81
5	10	107.8	-183.81
6	10	106.6	-199.5
7	10	105.1	-214
8	10	103.9	-227.14
9	10	102.8	-238.84
10	10	101.4	-249.8
11	10	100.8	-260.8
12	10	99.6	-272.5 (laser reset)
13	10	98.2	-11.41
14	10	97.6	-23.1
15	10	95.6	-34.89
16	10	94.4	-46.71
17	10	93.5	-58.23
18	10	92.9	-69.68
19	10	91	-81
20	10	90	-91.85
21	10	89.5	-101.72 (Remove support)
22	10	88.4	-110.64
23	10	87.4	-120.74
24	10	86.5	-130.04
25	10	85	-140.3
26	10	84	-150.15
27	10	83	-158.92
28	10	82	-168.71
29	10	81.2	-177.34
30	10	80.4	-186.14
31	10	79.5	-194.73
32	10	79	-202.74
33	10	78.3	-211.44
34	10	77.5	-219.85
35	10	76.7	-227.61
36	10	75.8	-236.52
37	10	74.9	-244.16
38	10	74.1	-252.08
39	10	73.1	-259.7

■ Data during lateral loading :

- Lateral loading height : 69.5 cm
- Laser height : 20.5 cm

E Essai5-60-Loose-5.5

- Initial data :
 - Pile height : 127.7 cm
 - Initial pile driving : 7.3 cm
 - Laser height : 39.4 mm
- Data during impact driving :

Number of blows	Drop height [cm]	Pile height [cm]	Laser measurement [mm]
1	10	109.3	-74.96
2	10	103.9	-126.82
3	10	100.1	-170.93

Table B.4: Data during impact driving

- Lateral loading data :
 - Lateral loading height : 95 cm
 - Laser height : 524 mm
 - Read value on laser : 43.89 mm
 - Sand height in the pile : 26.9 cm

A Driving results for essai1-60-dense-5.5

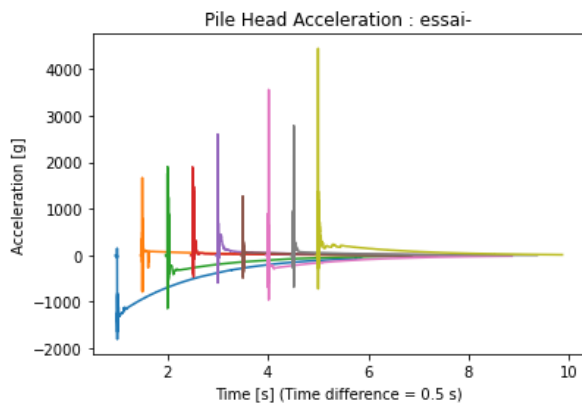


Figure C.1: Acceleration data

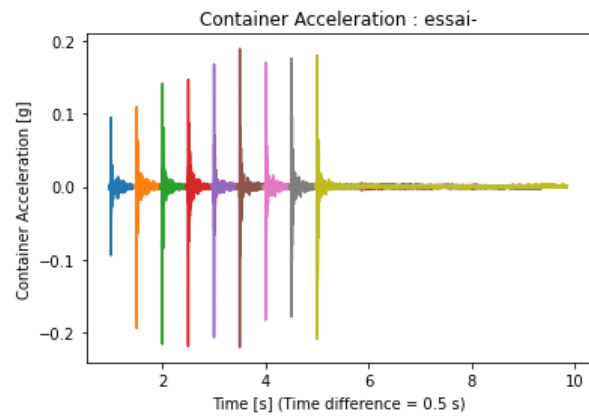


Figure C.2: Container acceleration data

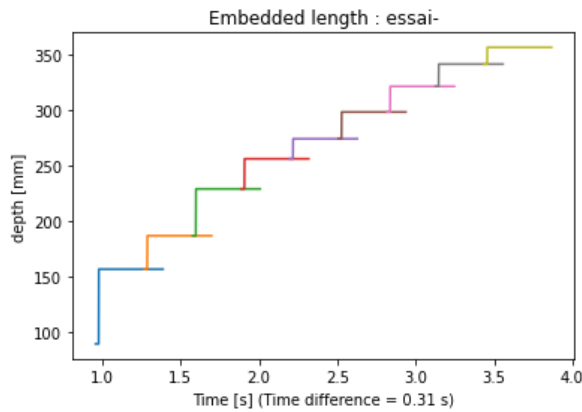


Figure C.3: Displacement data

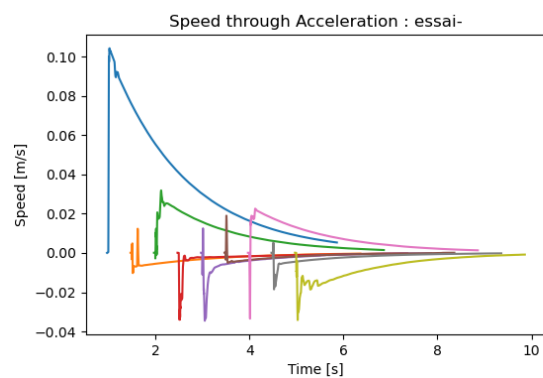


Figure C.4: Speed data

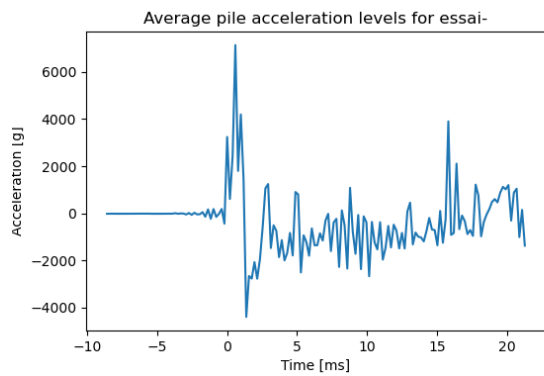


Figure C.5: Average Acceleration data

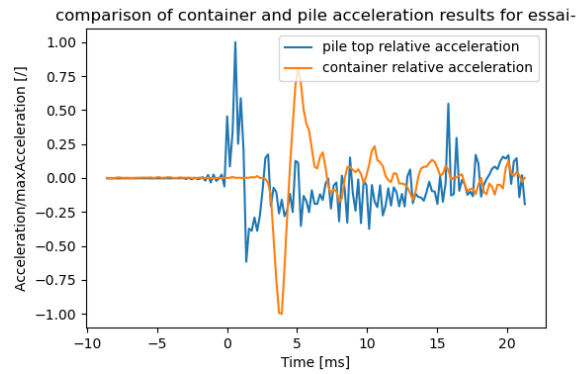


Figure C.6: Comparison of relative acceleration data

B Driving results for essai2-60-dense-5.5

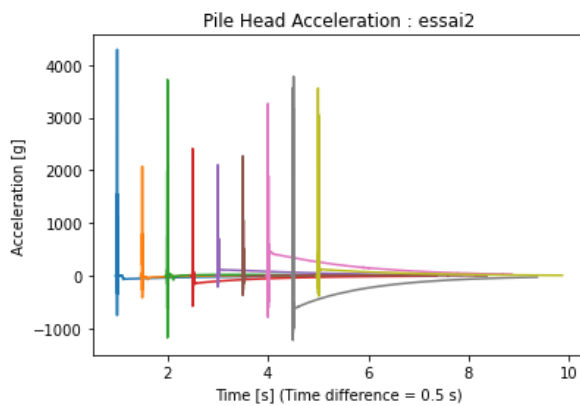


Figure C.7: Acceleration data

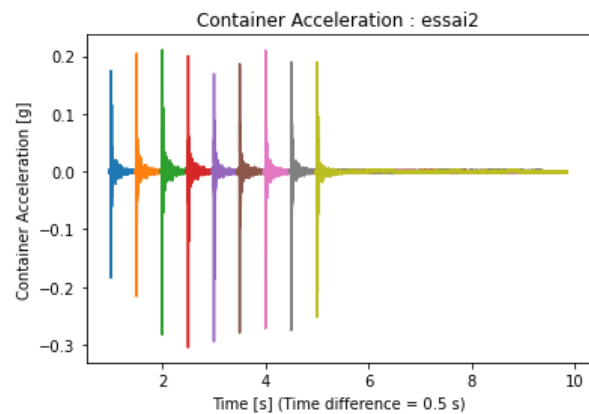


Figure C.8: Container acceleration data

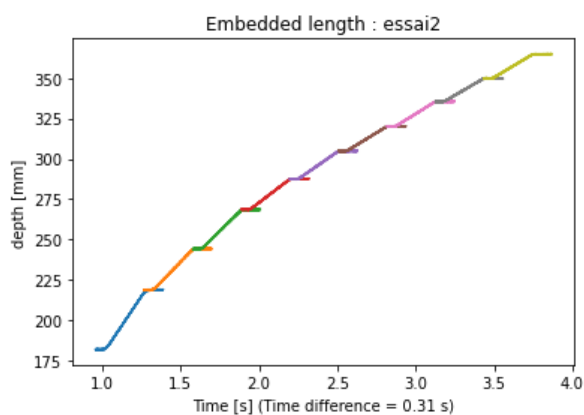


Figure C.9: Displacement data

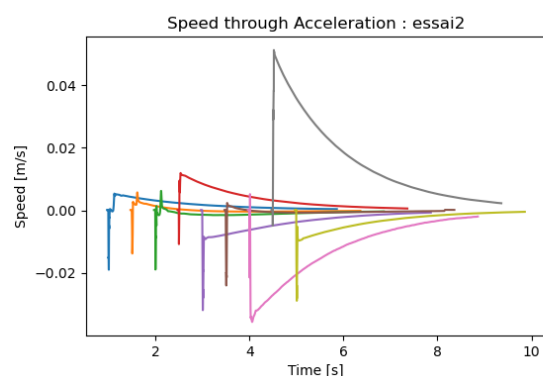


Figure C.10: Speed data

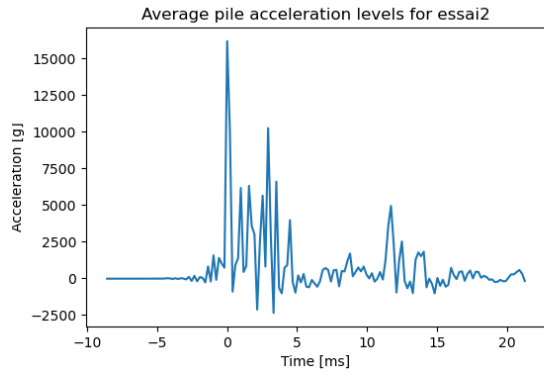


Figure C.11: Average Acceleration data

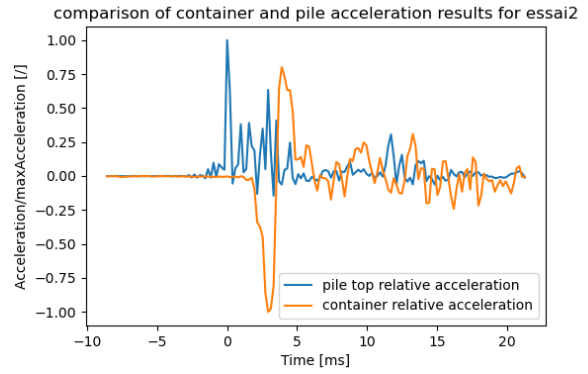


Figure C.12: Comparison of relative acceleration data

C Driving results for essai3-60-dense-5.5-hec5

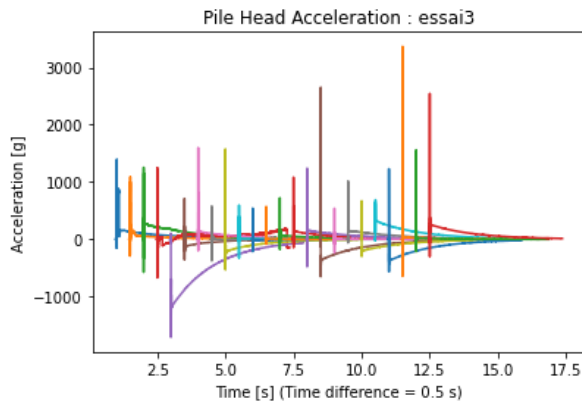


Figure C.13: Acceleration data

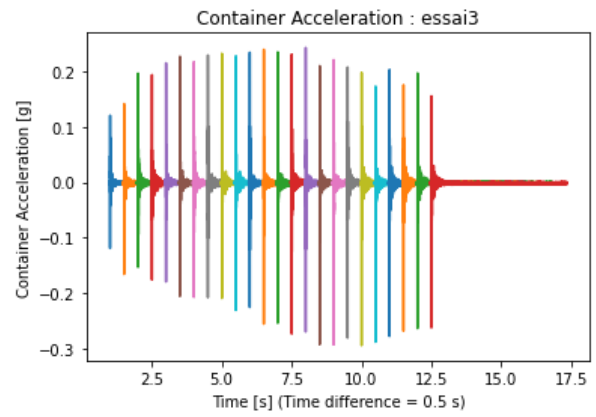


Figure C.14: Container acceleration data

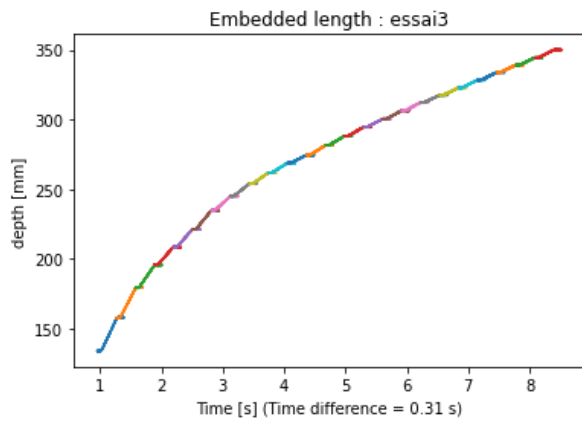


Figure C.15: Displacement data

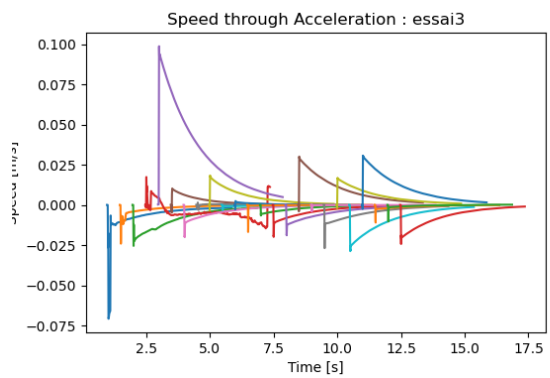


Figure C.16: Speed data

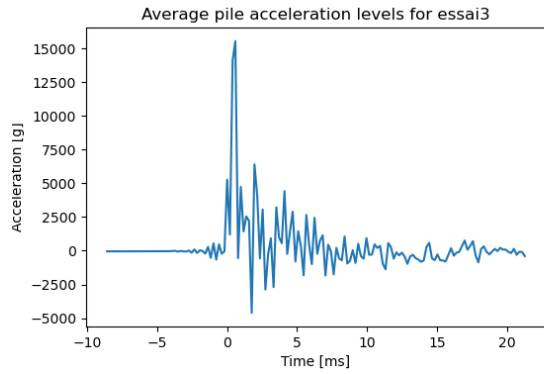


Figure C.17: Average Acceleration data

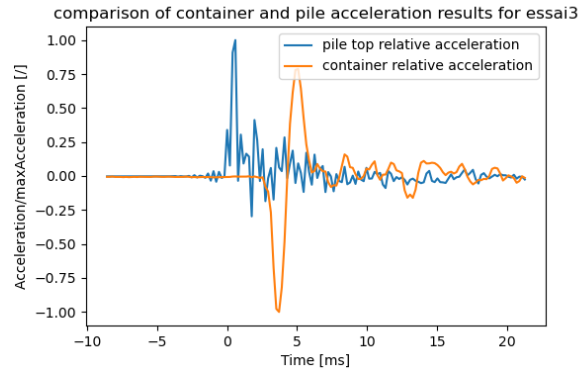


Figure C.18: Comparison of relative acceleration data

D Driving results for essai4-60-dense-10

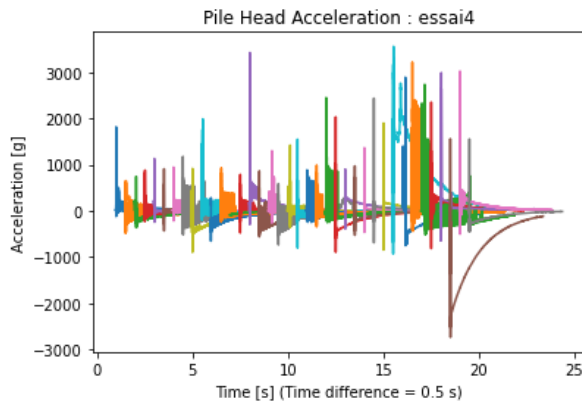


Figure C.19: Acceleration data

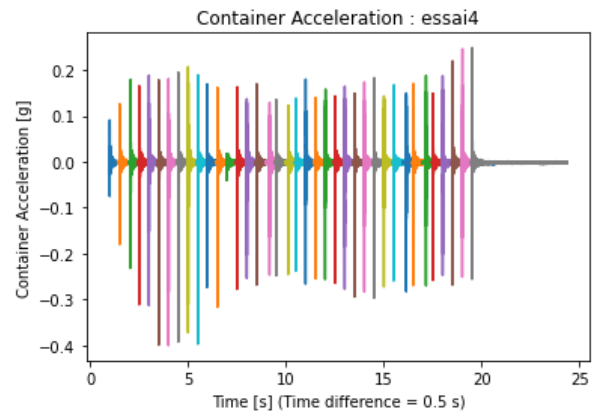


Figure C.20: Container acceleration data

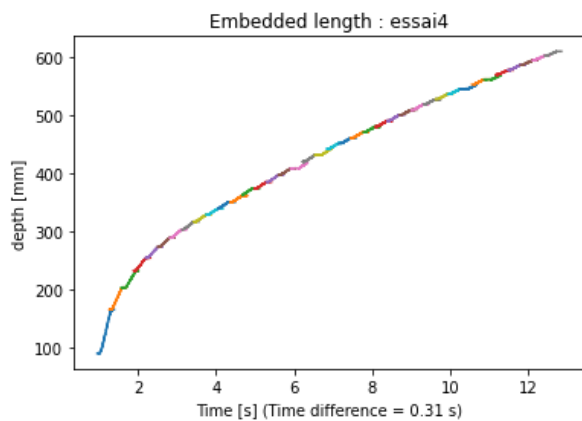


Figure C.21: Displacement data

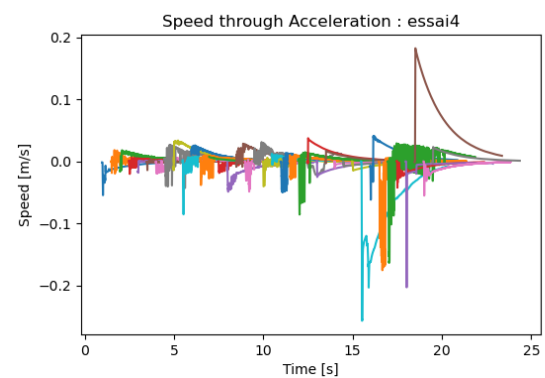


Figure C.22: Speed data

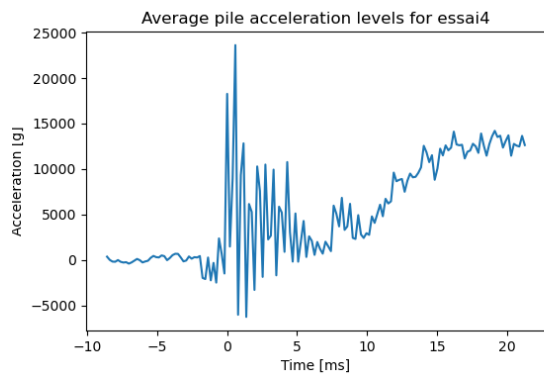


Figure C.23: Average Acceleration data

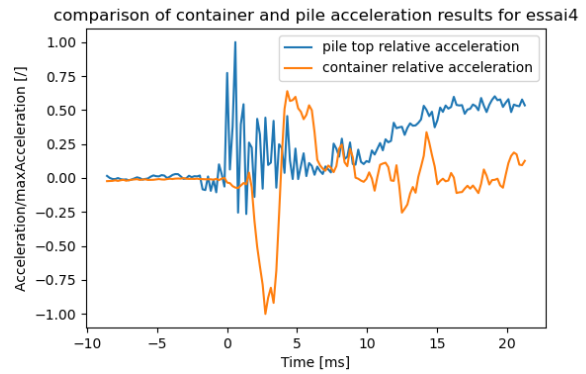


Figure C.24: Comparison of relative acceleration data

E Driving results for essai5-60-loose-5.5

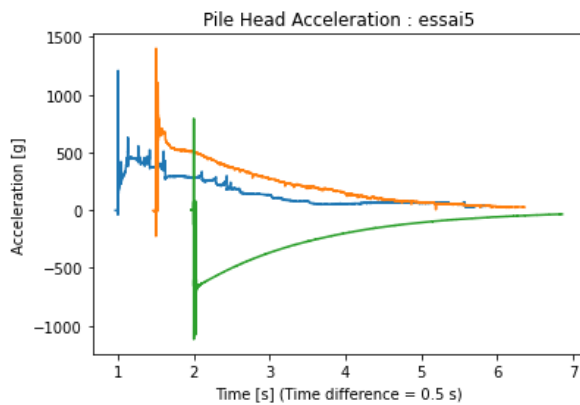


Figure C.25: Acceleration data

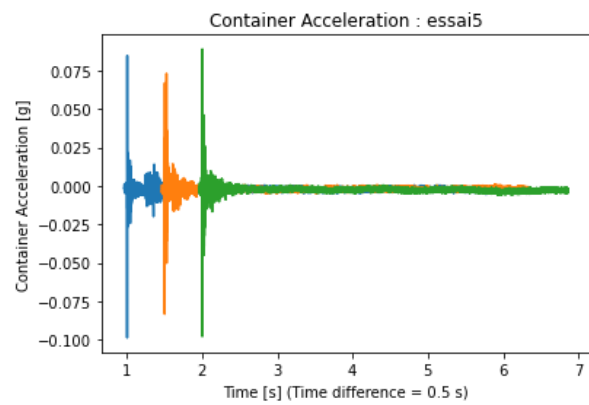


Figure C.26: Container acceleration data

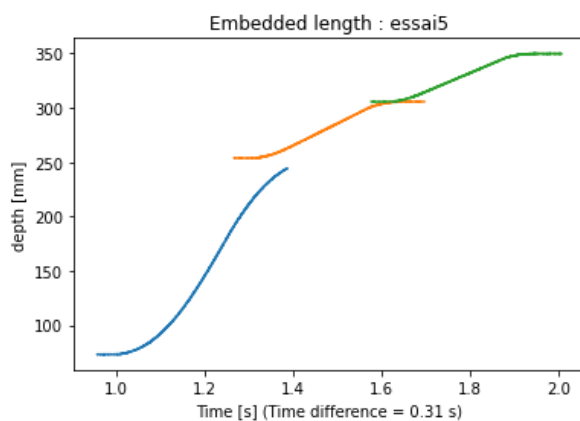


Figure C.27: displacement data

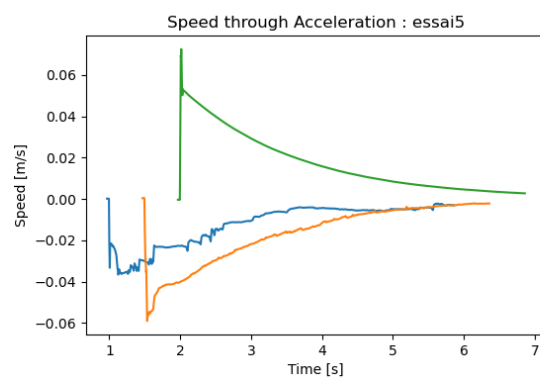


Figure C.28: speed data

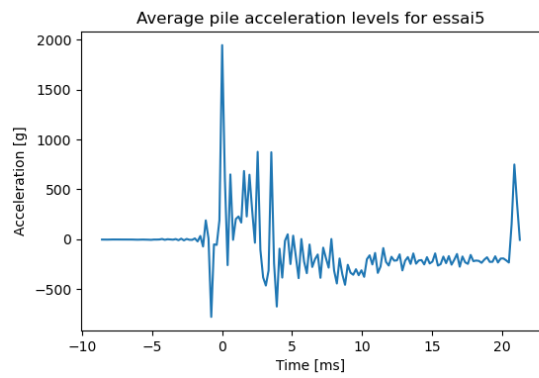


Figure C.29: Average Acceleration data

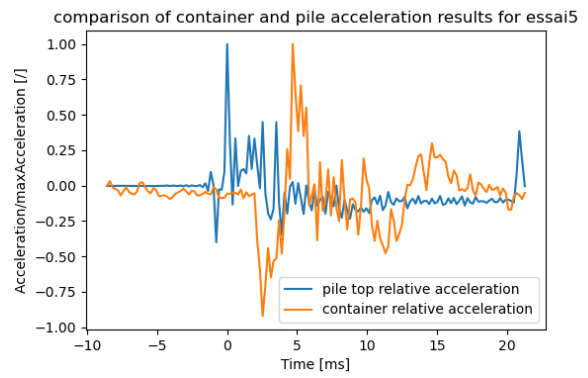


Figure C.30: Comparison of relative acceleration data

APPENDIX D

Loading results

A Results for essai1-60-dense-5.5-mec1

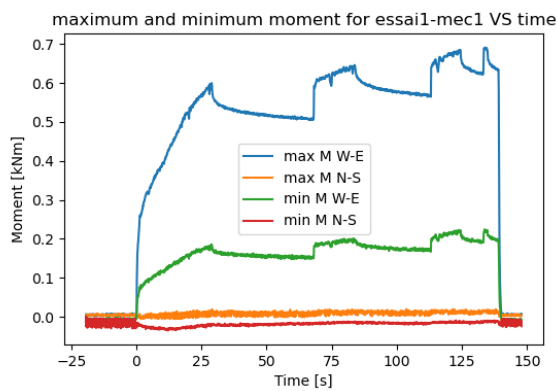


Figure D.1: Moment evolution with time

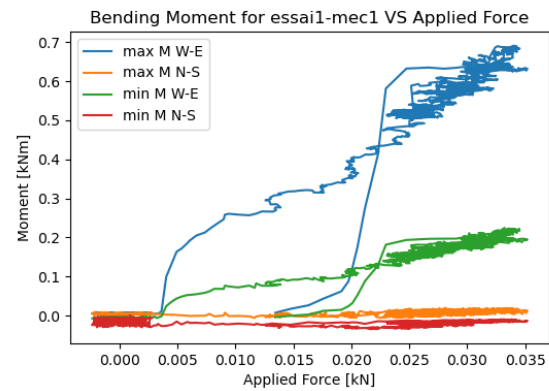


Figure D.2: Maximal bending moment evolution with applied moment

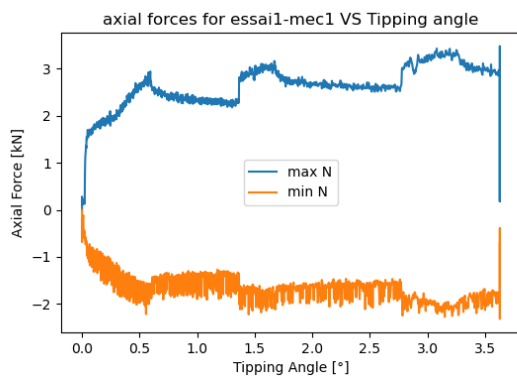


Figure D.3: Axial forces evolution with tipping angle

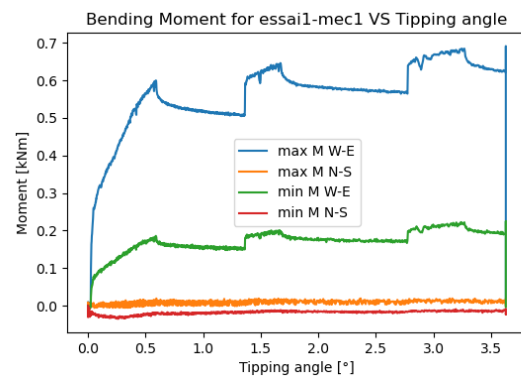


Figure D.4: Moment evolution with tipping angle

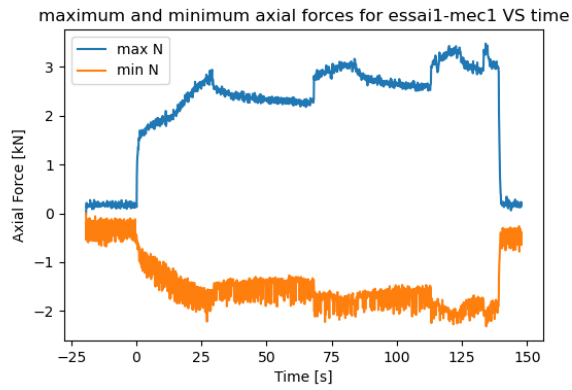


Figure D.5: Axial forces evolution with time

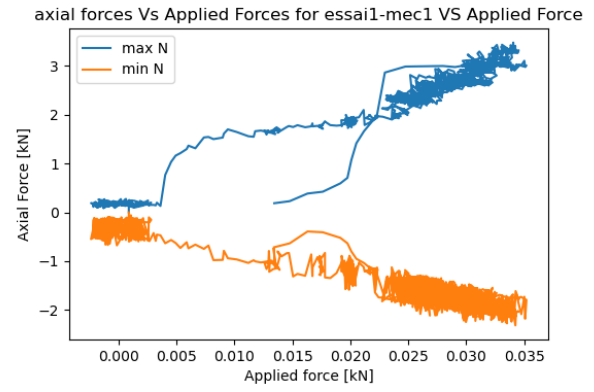


Figure D.6: Axial forces evolution with applied force

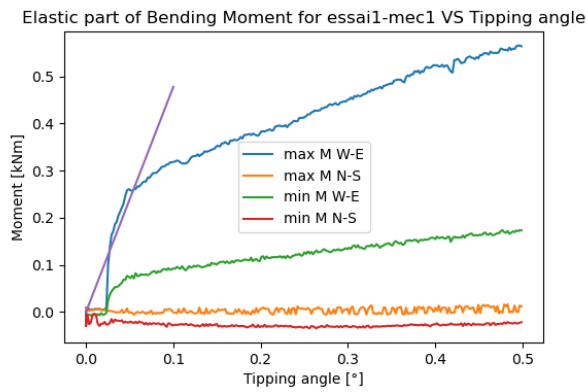


Figure D.7: Elastic modulus approximation

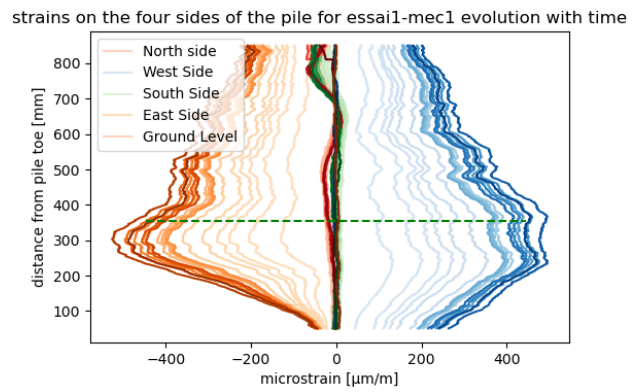


Figure D.8: Strain evolution with time

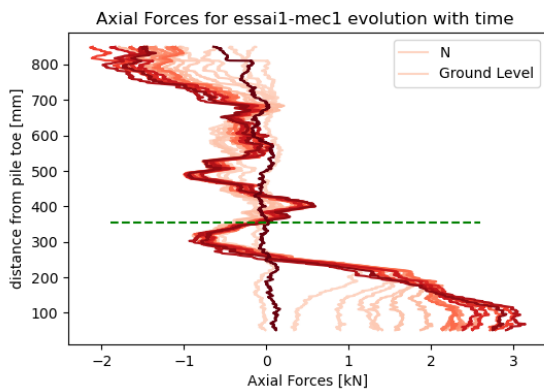


Figure D.9: Axial forces evolution with time

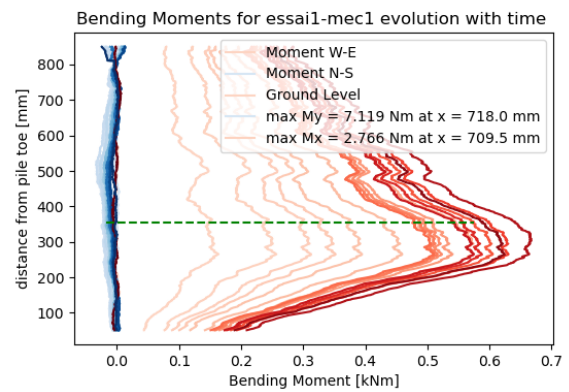


Figure D.10: Moments evolution with time

B Results for essai1-60-dense-5.5-mec2

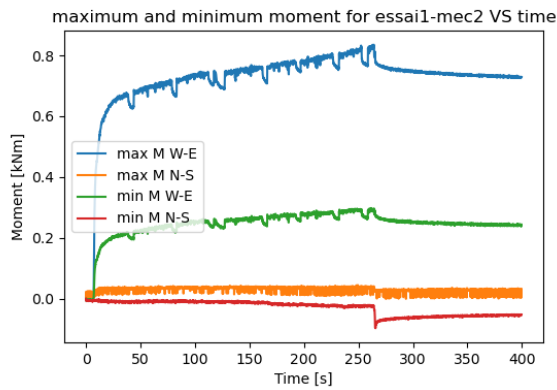


Figure D.11: Moment evolution with time

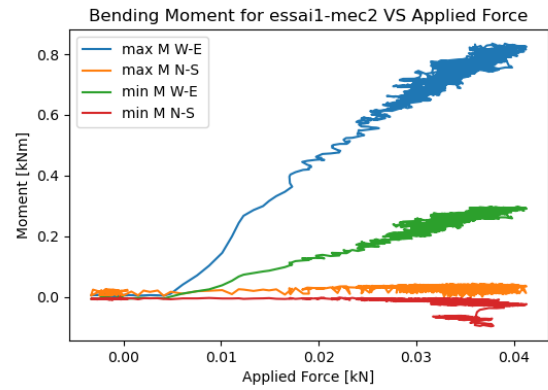


Figure D.12: Maximal bending moment evolution with applied moment

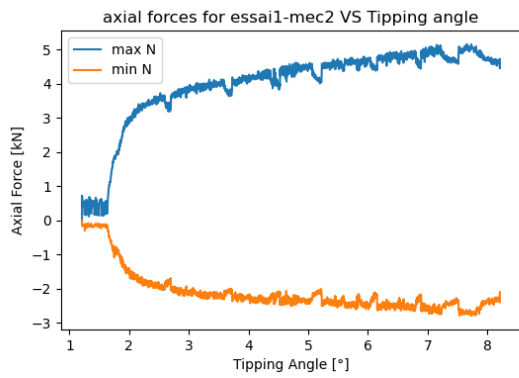


Figure D.13: Axial forces evolution with tipping angle

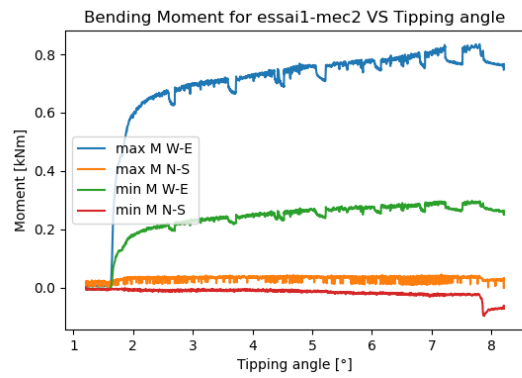


Figure D.14: Moment evolution with tipping angle

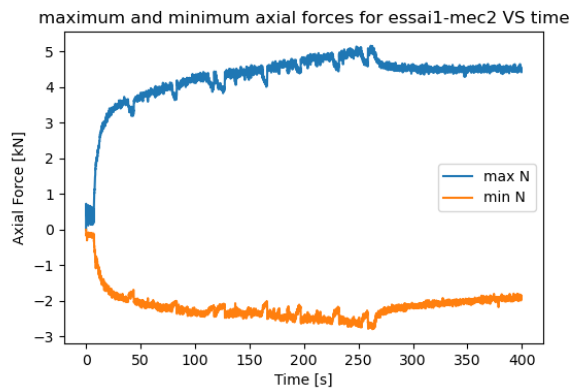


Figure D.15: Axial forces evolution with time

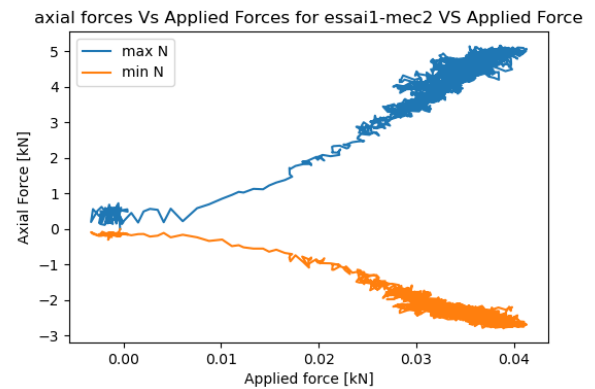


Figure D.16: Axial forces evolution with applied force

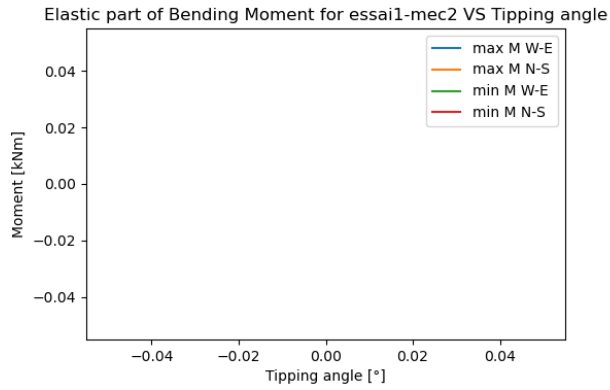


Figure D.17: Elastic modulus approximation

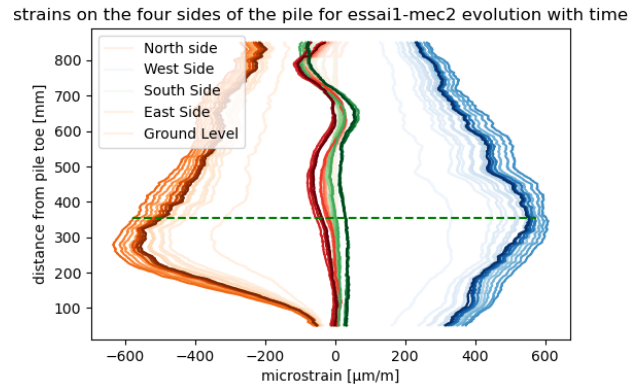


Figure D.18: Strain evolution with time

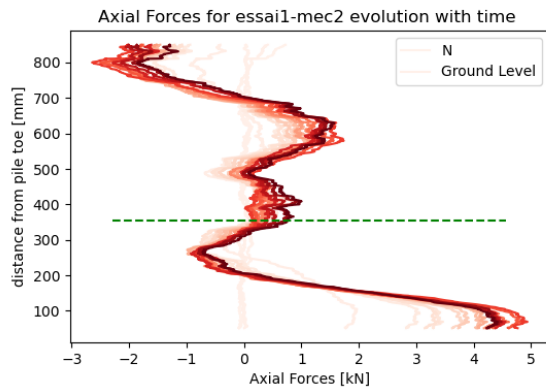


Figure D.19: Axial forces evolution with time

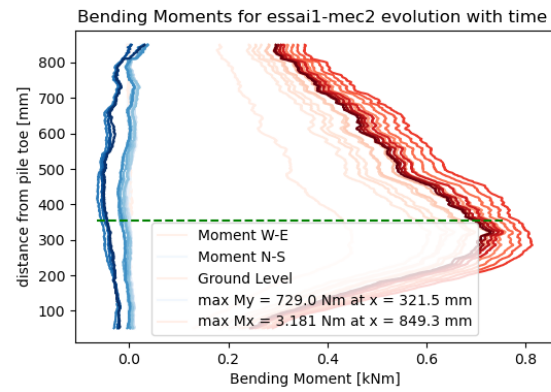


Figure D.20: Moments evolution with time

C Results for essai2-60-dense-5.5-mec1

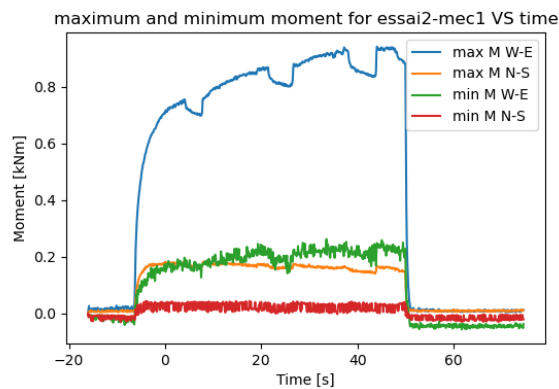


Figure D.21: Moment evolution with time

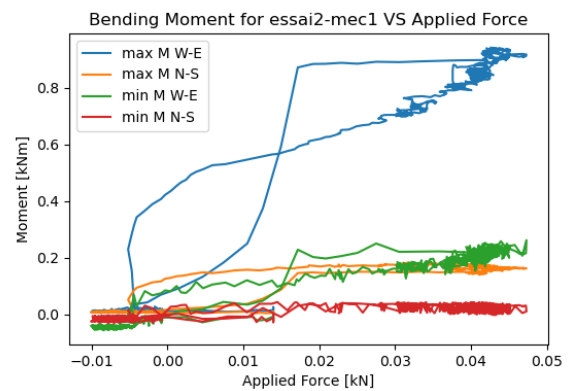


Figure D.22: Maximal bending moment evolution with applied moment

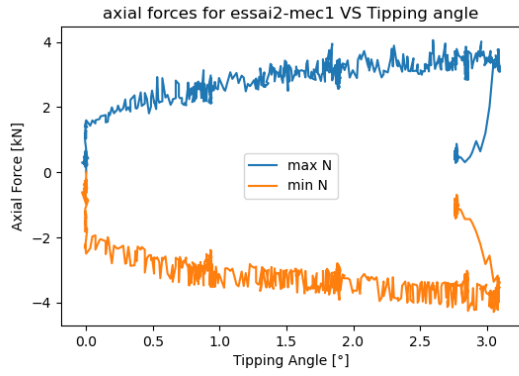


Figure D.23: Axial forces evolution with tipping angle

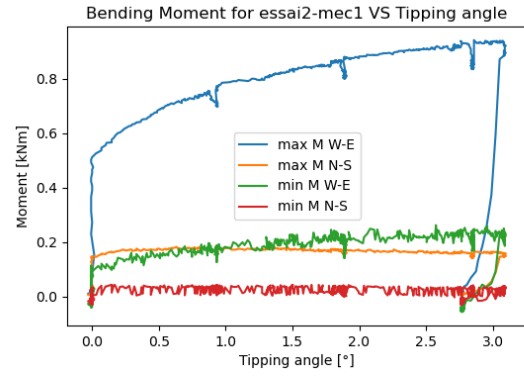


Figure D.24: Moment evolution with tipping angle

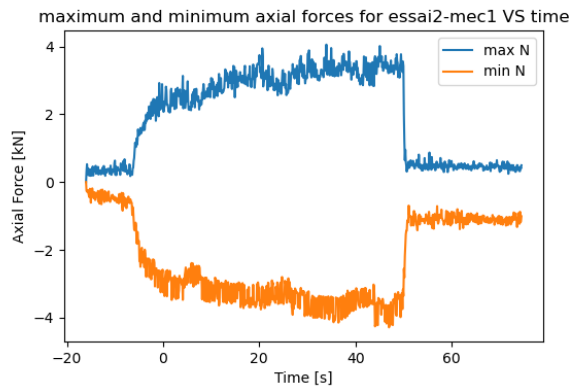


Figure D.25: Axial forces evolution with time

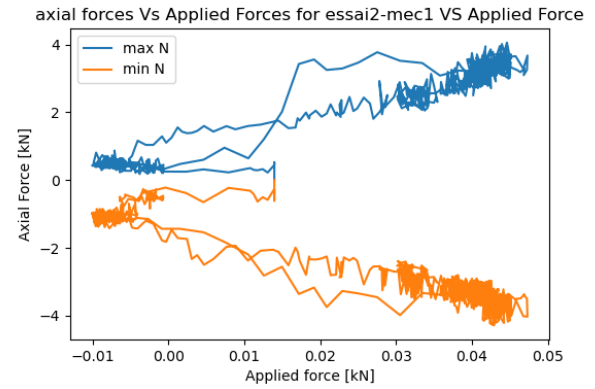


Figure D.26: Axial forces evolution with applied force

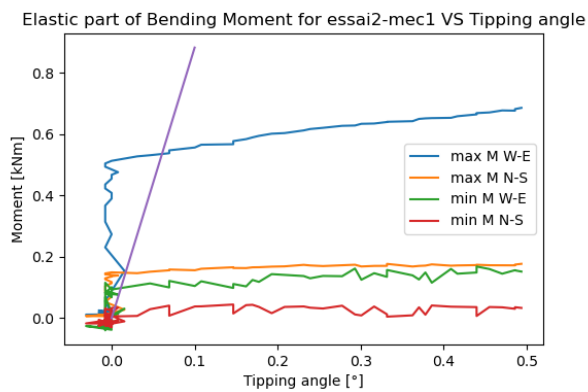


Figure D.27: Elastic modulus approximation

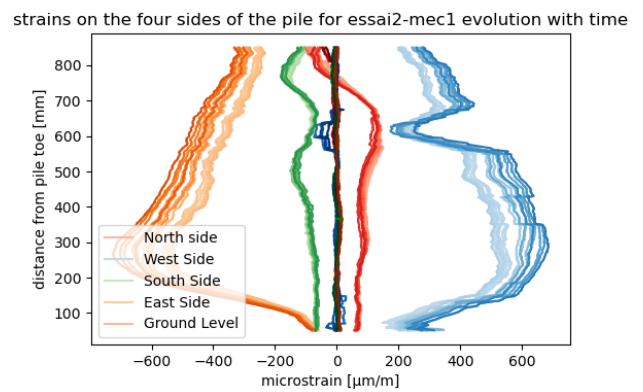


Figure D.28: Strain evolution with time

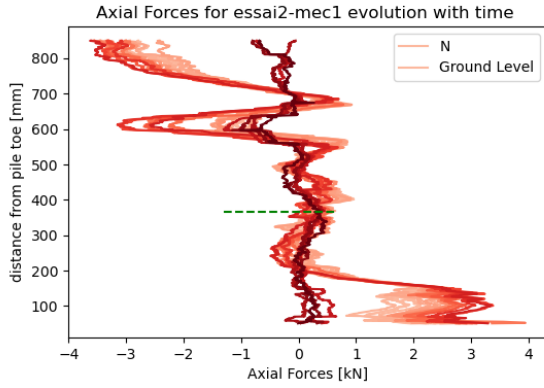


Figure D.29: Axial forces evolution with time

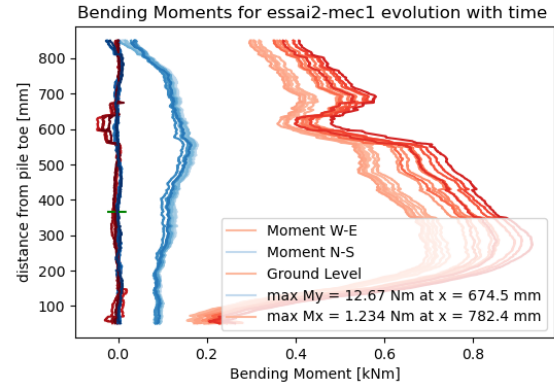


Figure D.30: Moments evolution with time

D Results for essai2-60-dense-5.5-mec2

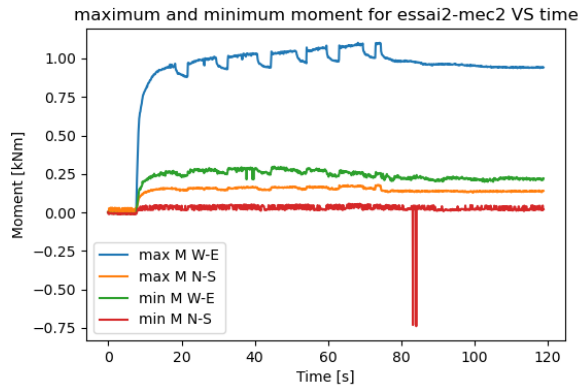


Figure D.31: Moment evolution with time

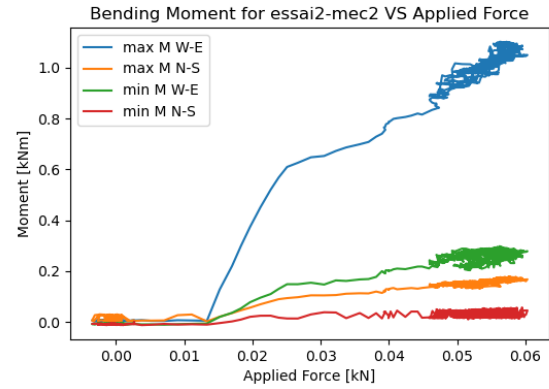


Figure D.32: Maximal bending moment evolution with applied moment

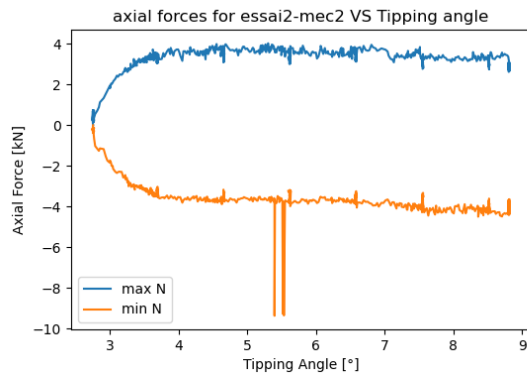


Figure D.33: Axial forces evolution with tipping angle

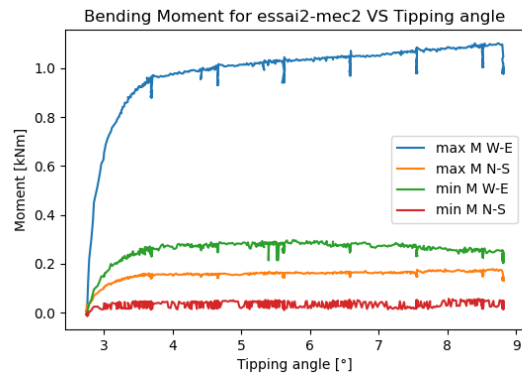


Figure D.34: Moment evolution with tipping angle

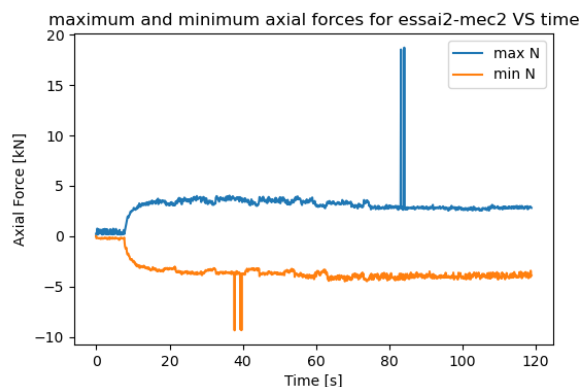


Figure D.35: Axial forces evolution with time

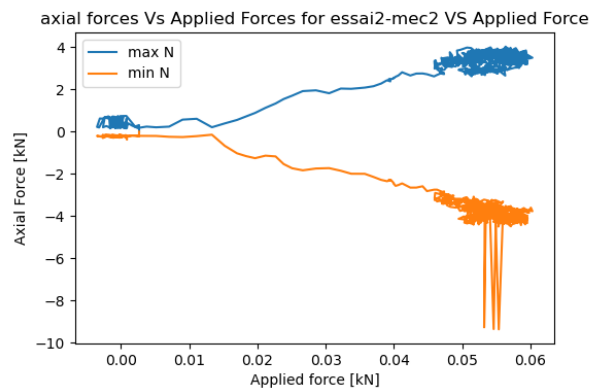


Figure D.36: Axial forces evolution with applied force

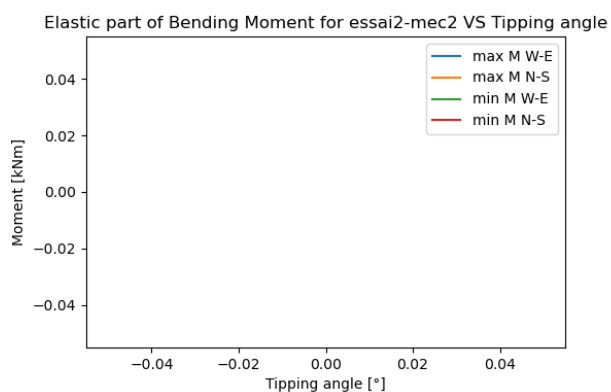


Figure D.37: Elastic modulus approximation

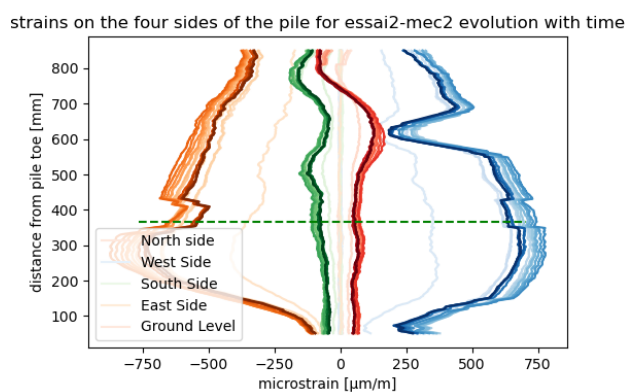


Figure D.38: Strain evolution with time

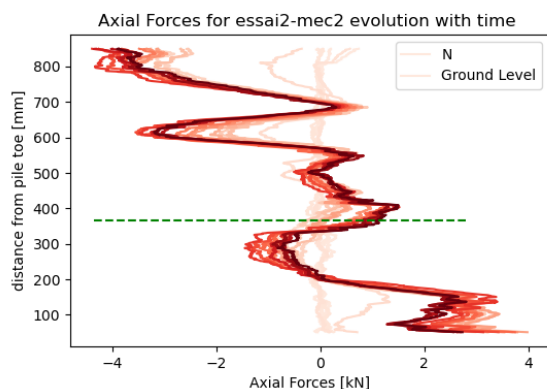


Figure D.39: Axial forces evolution with time

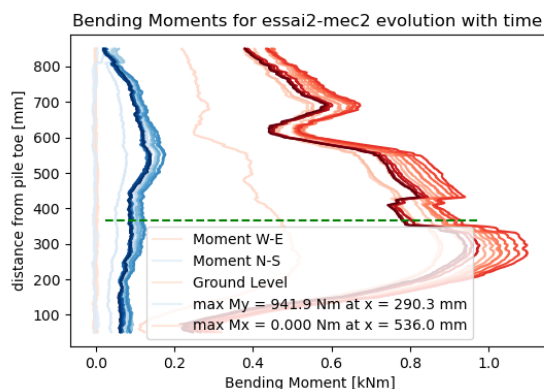


Figure D.40: Moments evolution with time

E Results for essai3-60-dense-5.5-hc5-mec1

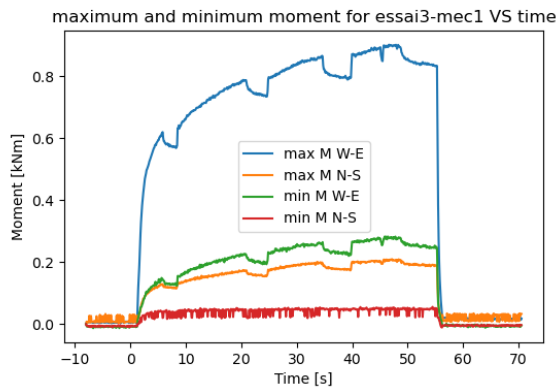


Figure D.41: Moment evolution with time

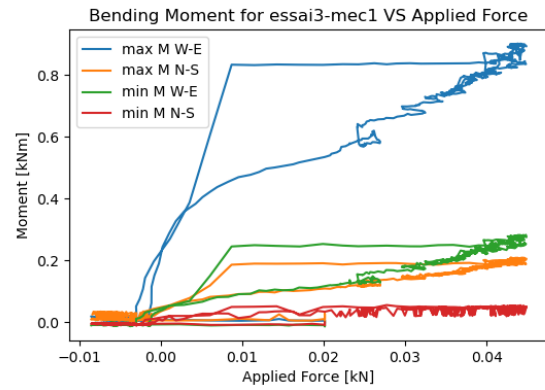


Figure D.42: Maximal bending moment evolution with applied moment

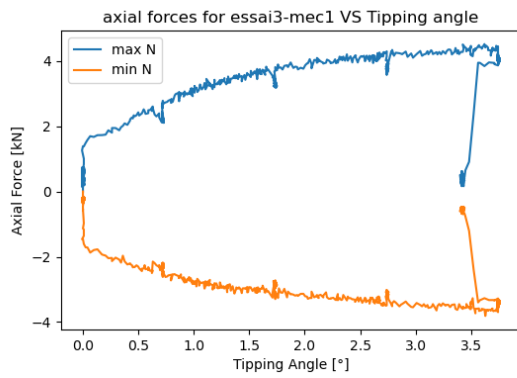


Figure D.43: Axial forces evolution with tipping angle

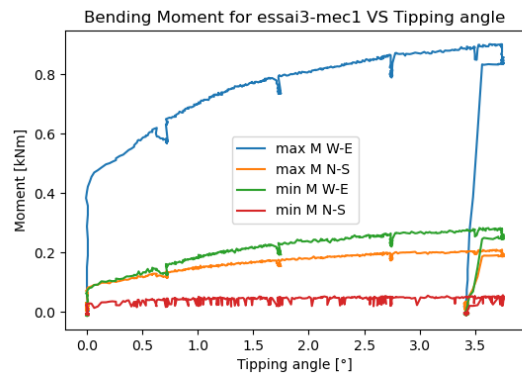


Figure D.44: Moment evolution with tipping angle

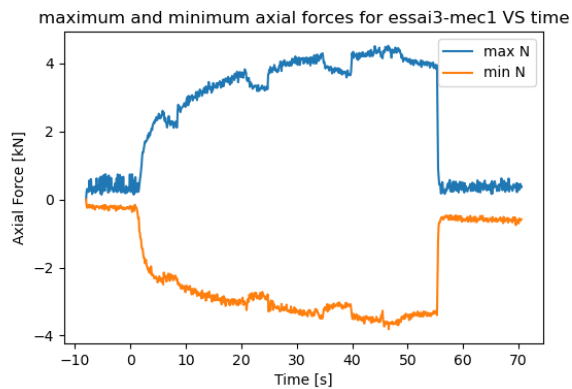


Figure D.45: Axial forces evolution with time

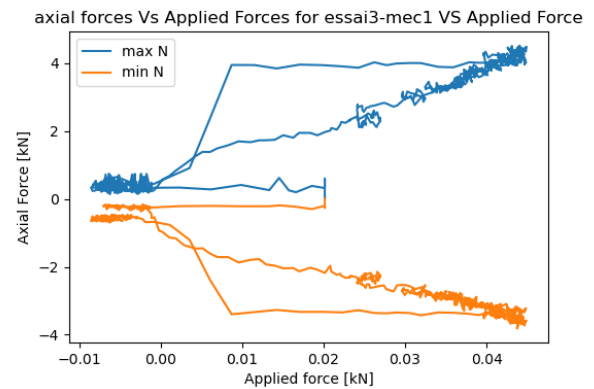


Figure D.46: Axial forces evolution with applied force

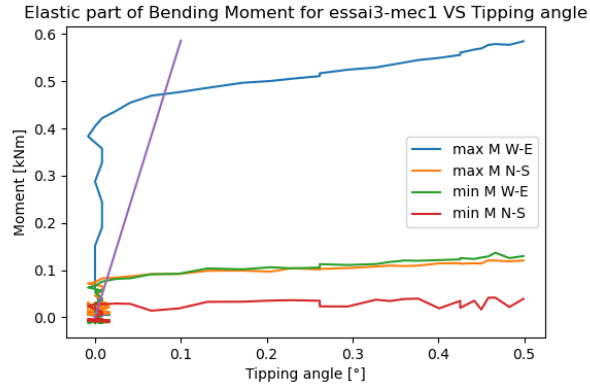


Figure D.47: Elastic modulus approximation

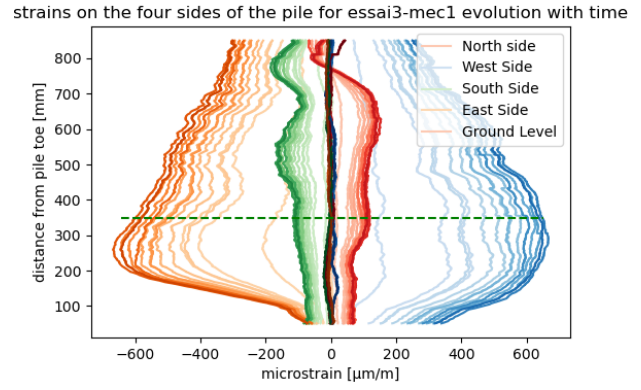


Figure D.48: Strain evolution with time

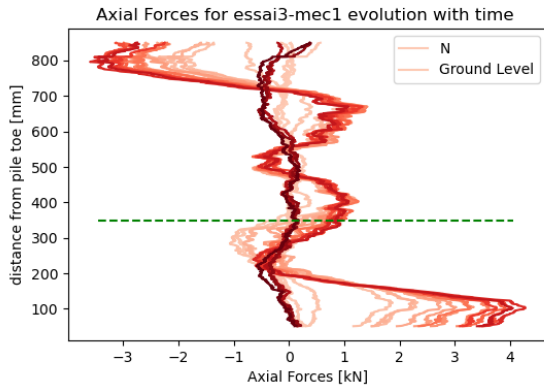


Figure D.49: Axial forces evolution with time

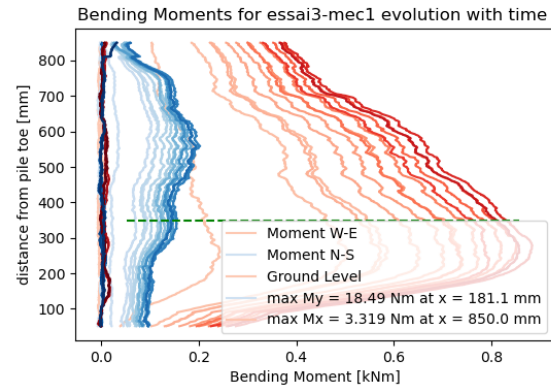


Figure D.50: Moments evolution with time

F Results for essai3-60-dense-5.5-hc5-mec2

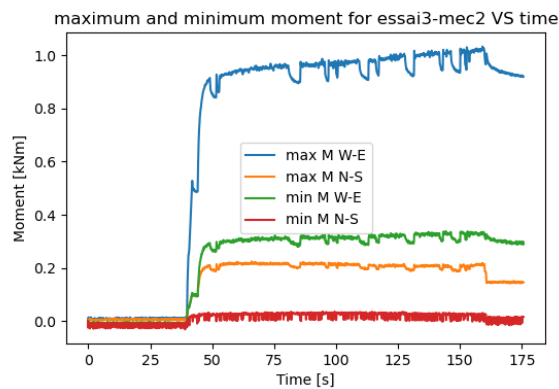


Figure D.51: Moment evolution with time

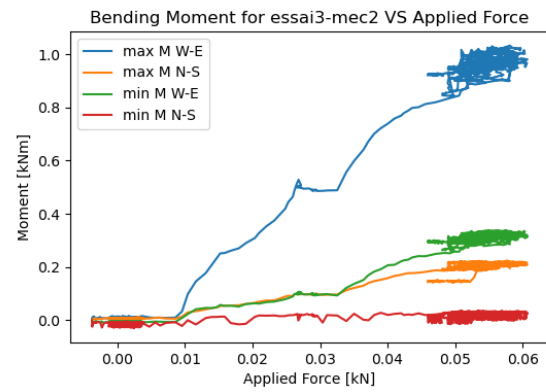


Figure D.52: Maximal bending moment evolution with applied moment

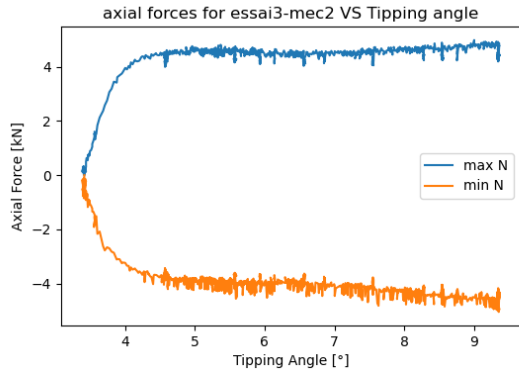


Figure D.53: Axial forces evolution with tipping angle

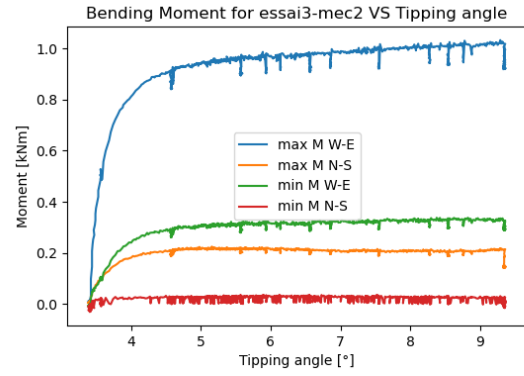


Figure D.54: Moment evolution with tipping angle

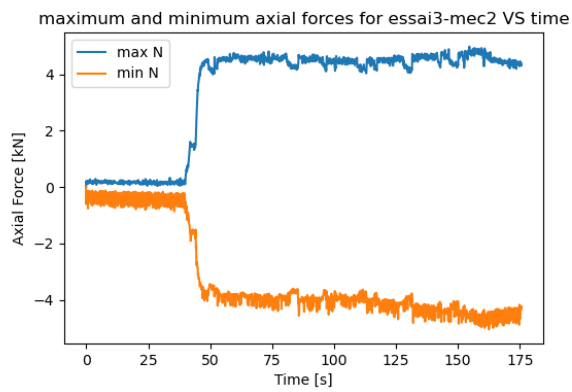


Figure D.55: Axial forces evolution with time

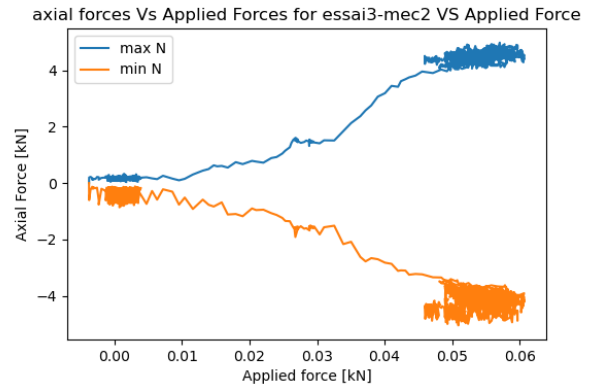


Figure D.56: Axial forces evolution with applied force

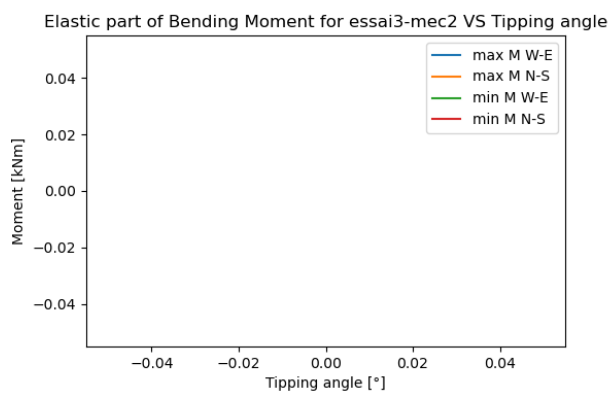


Figure D.57: Elastic modulus approximation

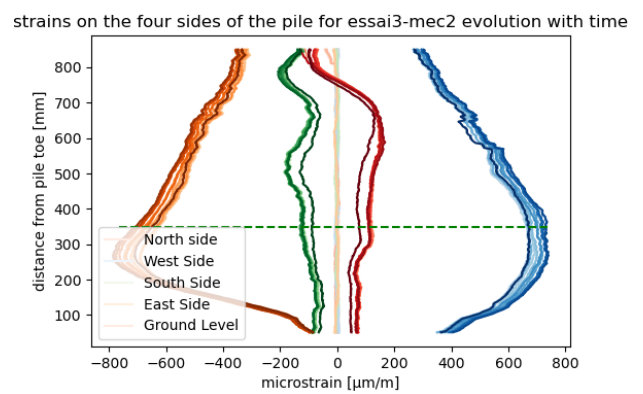


Figure D.58: Strain evolution with time

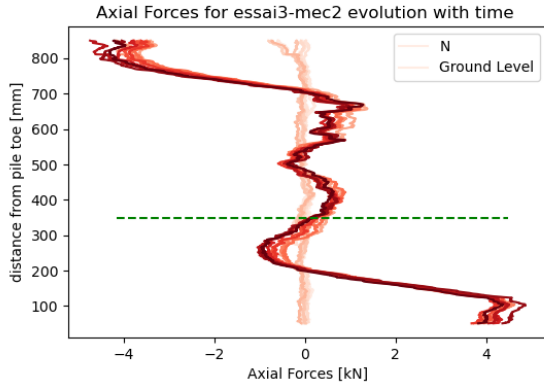


Figure D.59: Axial forces evolution with time

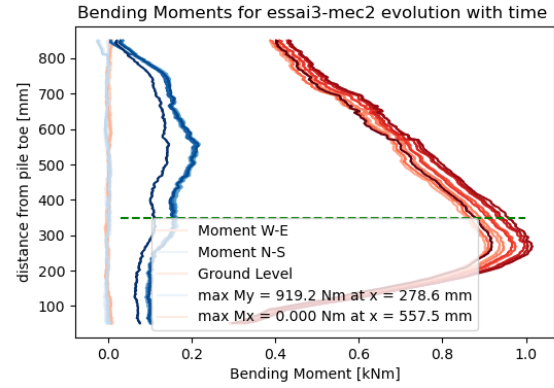


Figure D.60: Moments evolution with time

G Results for essai4-60-dense-10-mec1

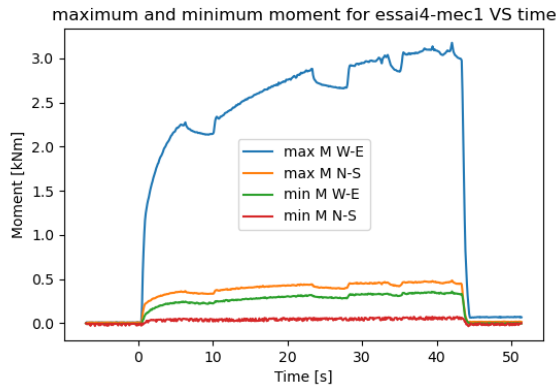


Figure D.61: Moment evolution with time

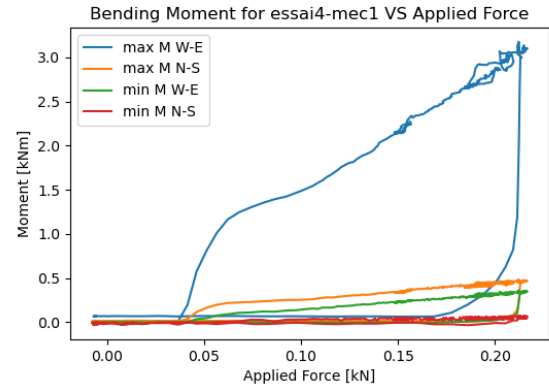


Figure D.62: Maximal bending moment evolution with applied moment

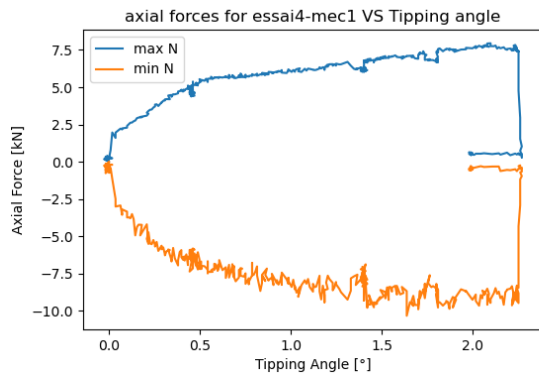


Figure D.63: Axial forces evolution with tipping angle

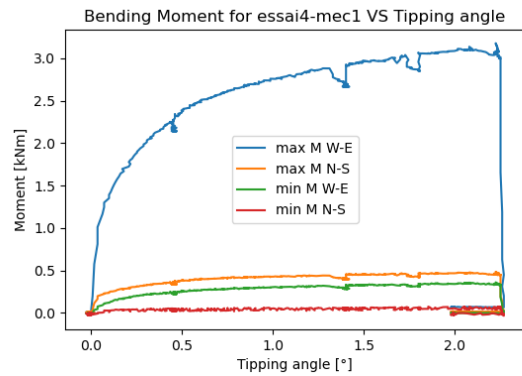


Figure D.64: Moment evolution with tipping angle

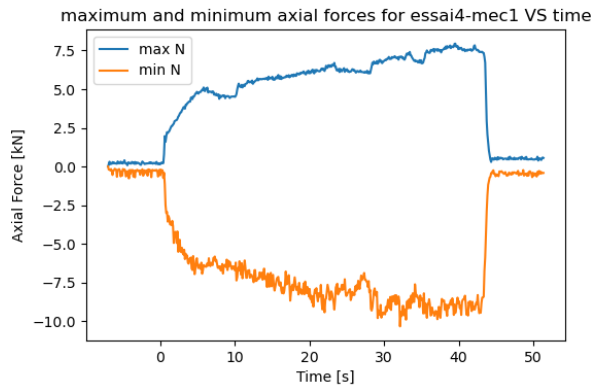


Figure D.65: Axial forces evolution with time

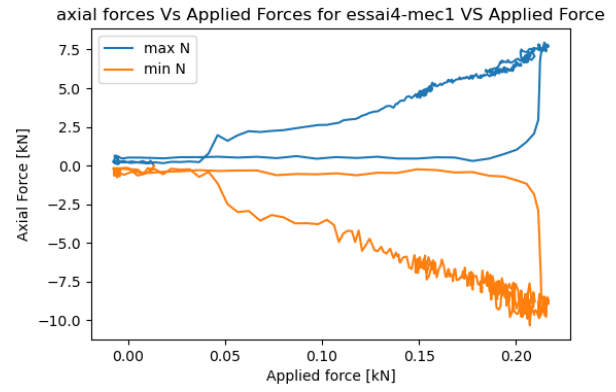


Figure D.66: Axial forces evolution with applied force

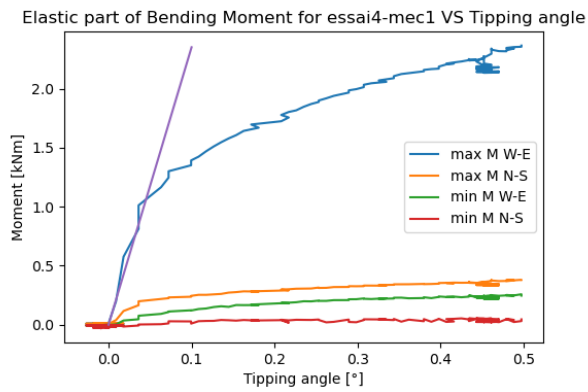


Figure D.67: Elastic modulus approximation

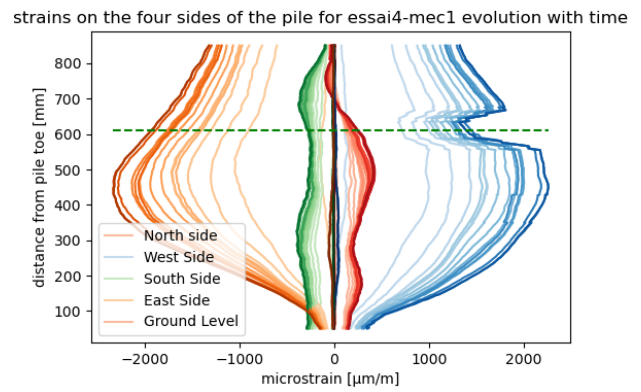


Figure D.68: Strain evolution with time

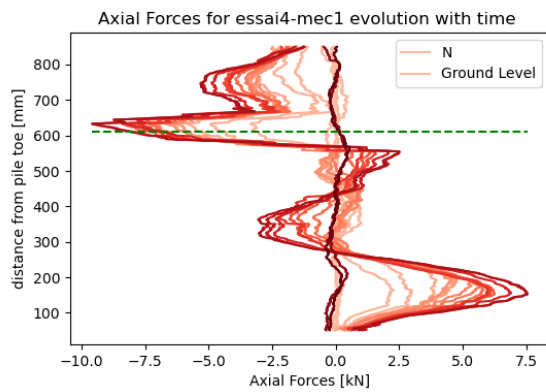


Figure D.69: Axial forces evolution with time

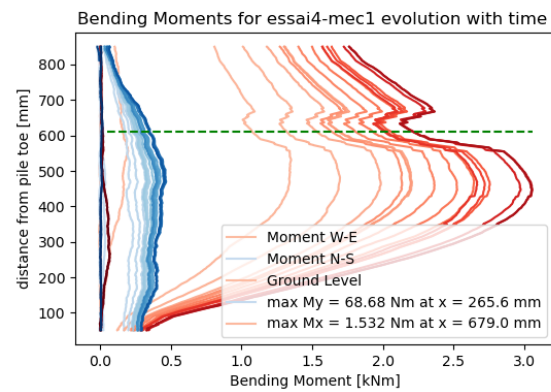


Figure D.70: Moments evolution with time

H Results for essai4-60-dense-10-mec2

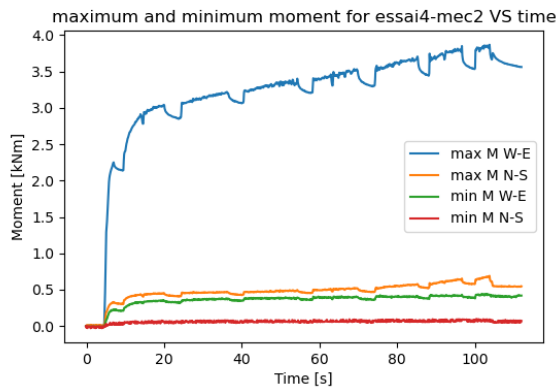


Figure D.71: Moment evolution with time

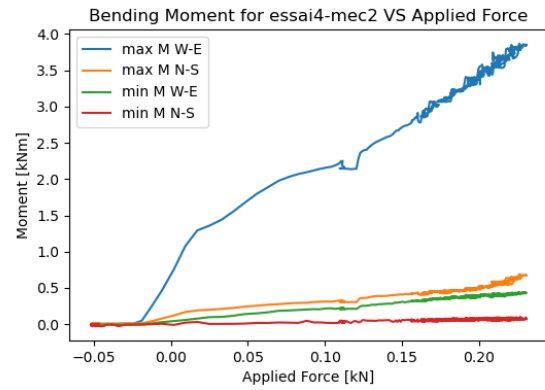


Figure D.72: maximal bending moment evolution with applied moment

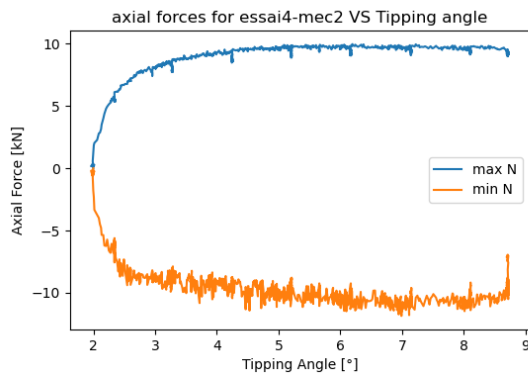


Figure D.73: Axial forces evolution with tipping angle

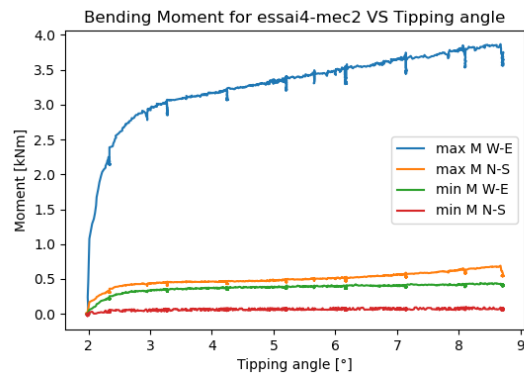


Figure D.74: Moment evolution with tipping angle

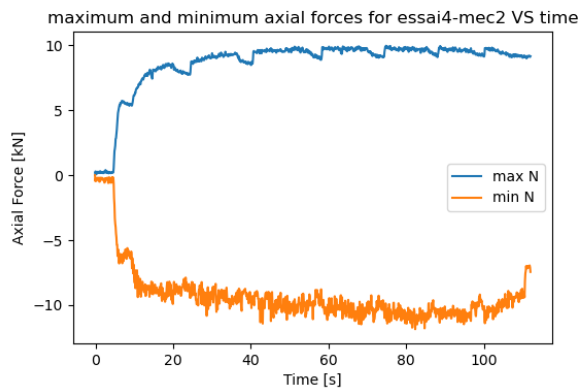


Figure D.75: Axial forces evolution with time

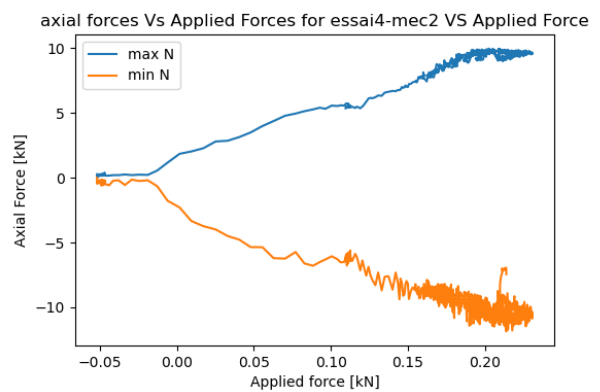


Figure D.76: Axial forces evolution with applied force

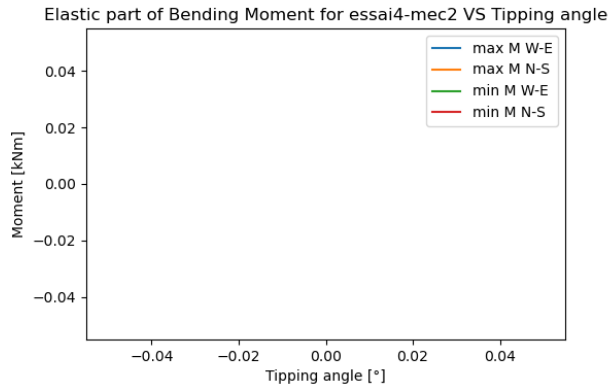


Figure D.77: Elastic modulus approximation

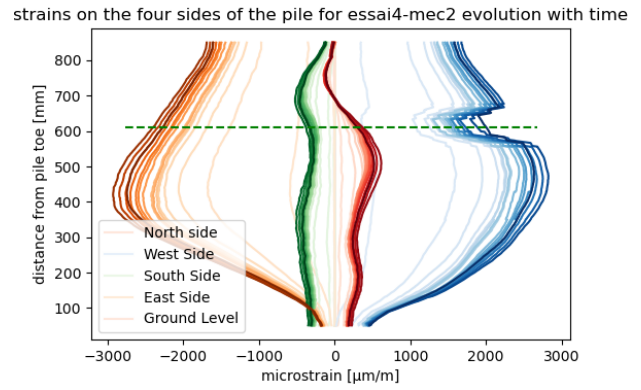


Figure D.78: Strain evolution with time

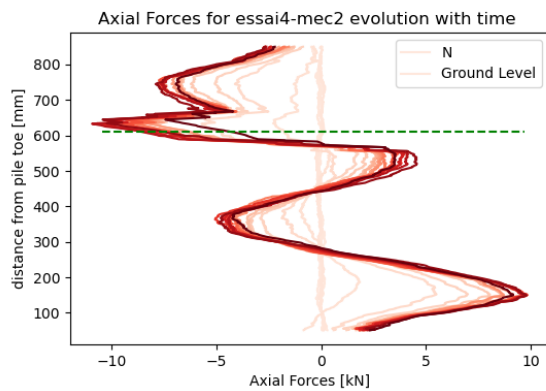


Figure D.79: Axial forces evolution with time

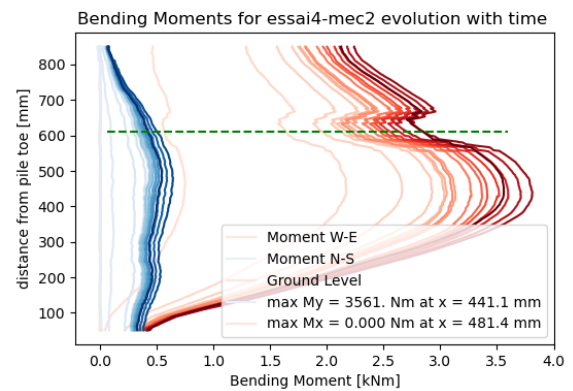


Figure D.80: Moments evolution with time

I Results for essai5-60-loose-5.5-mec1

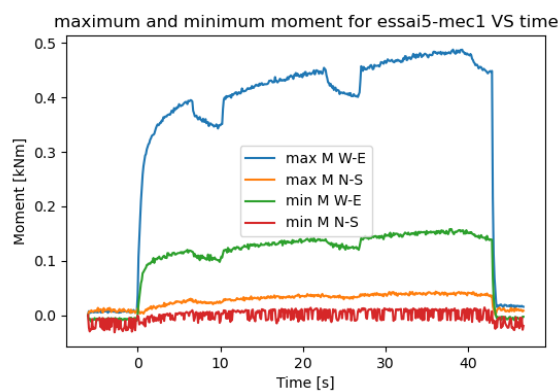


Figure D.81: Moment evolution with time

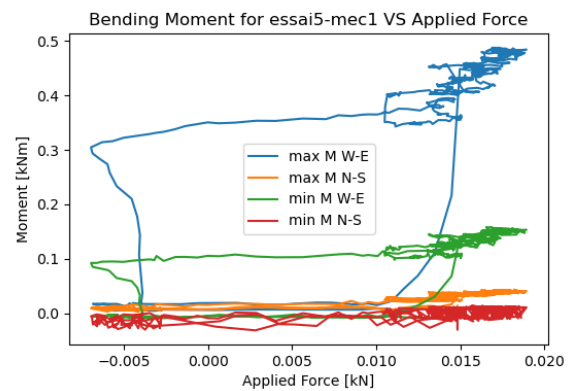


Figure D.82: Maximal bending moment evolution with applied moment

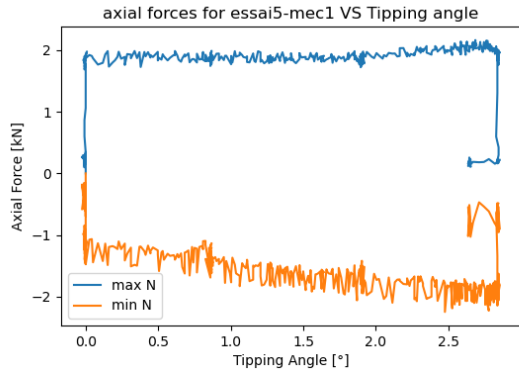


Figure D.83: Axial forces evolution with tipping angle

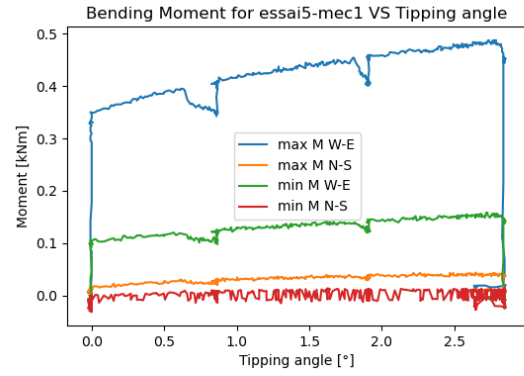


Figure D.84: Moment evolution with tipping angle

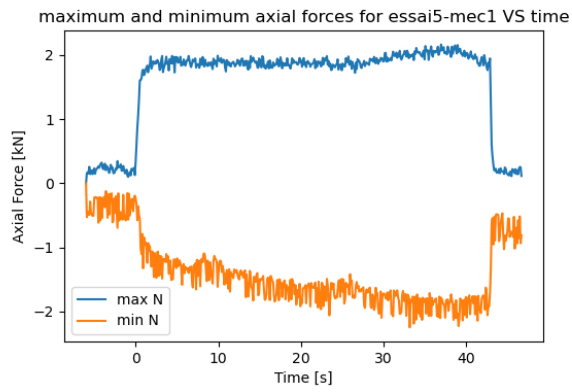


Figure D.85: Axial forces evolution with time

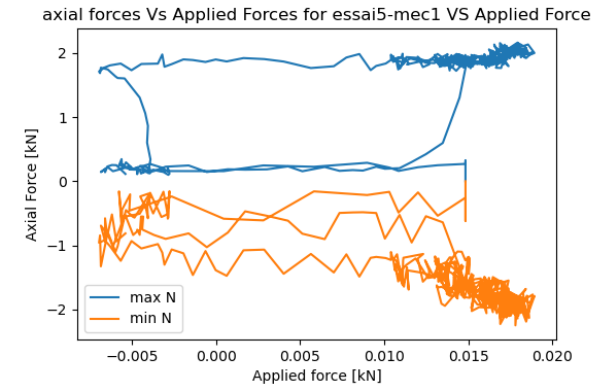


Figure D.86: Axial forces evolution with applied force

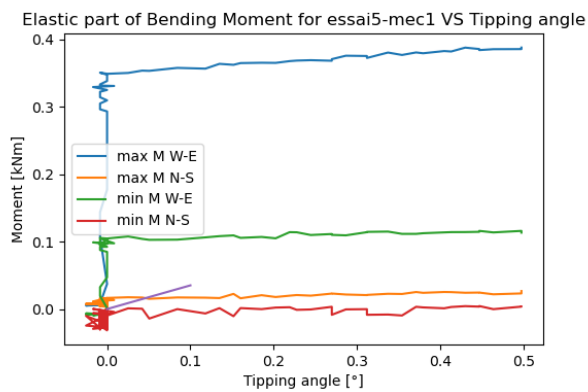


Figure D.87: Elastic modulus approximation

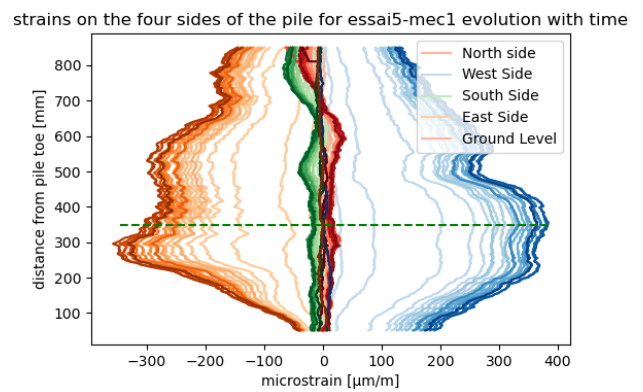


Figure D.88: Strain evolution with time

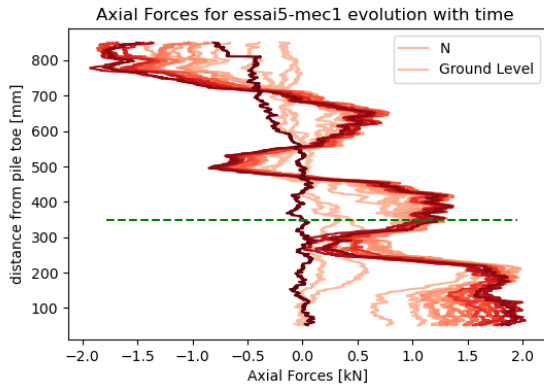


Figure D.89: Axial forces evolution with time

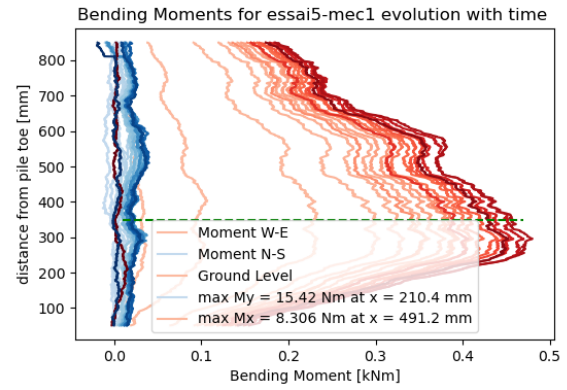


Figure D.90: Moments evolution with time

J Results for essai5-60-loose-5.5-mec2

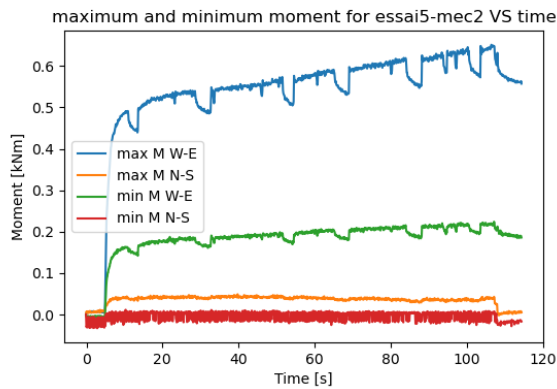


Figure D.91: Moment evolution with time

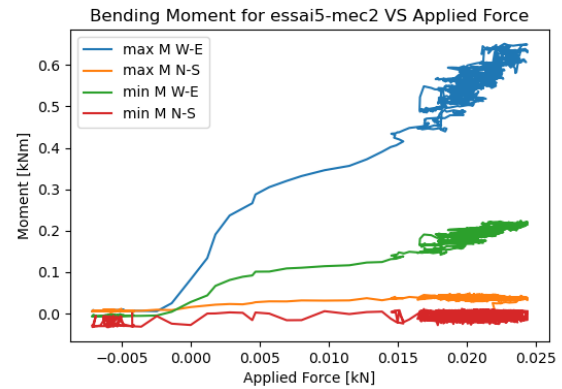


Figure D.92: Maximal bending moment evolution with applied moment

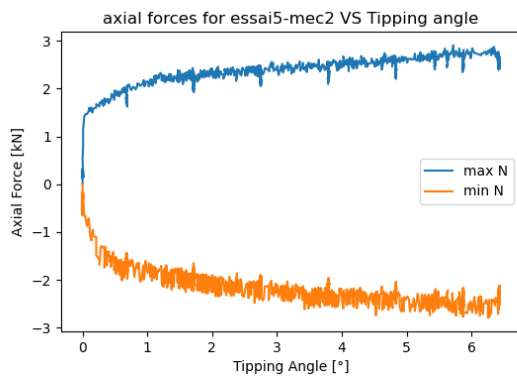


Figure D.93: Axial forces evolution with tipping angle

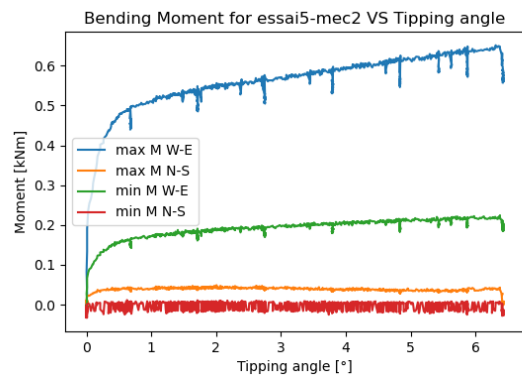


Figure D.94: Moment evolution with tipping angle

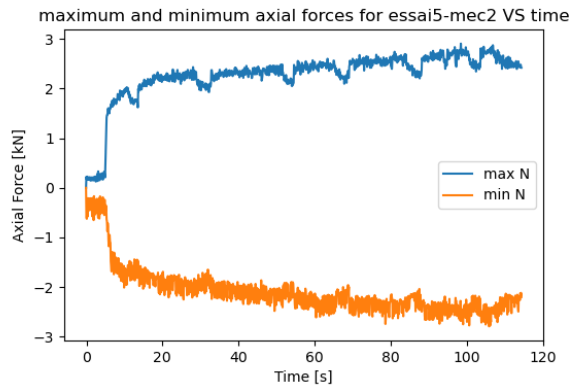


Figure D.95: Axial forces evolution with time

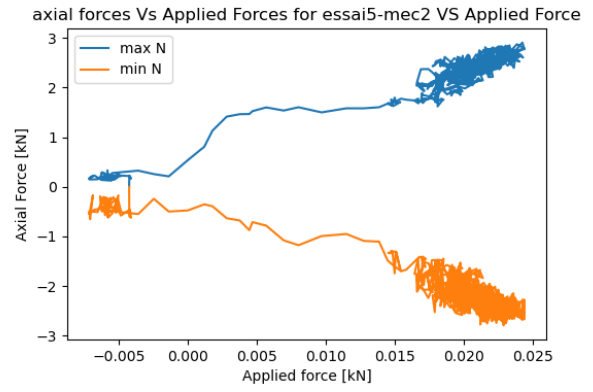


Figure D.96: Axial forces evolution with applied force

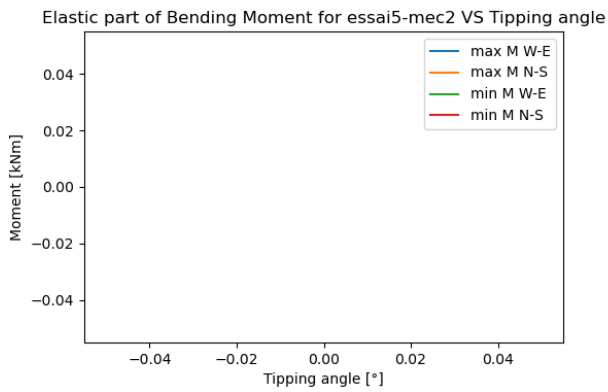


Figure D.97: Elastic modulus approximation

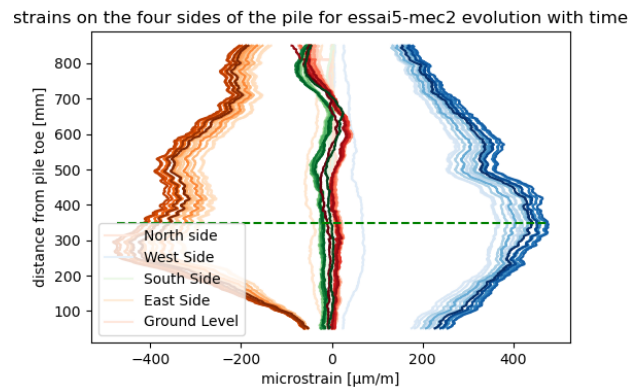


Figure D.98: Strain evolution with time

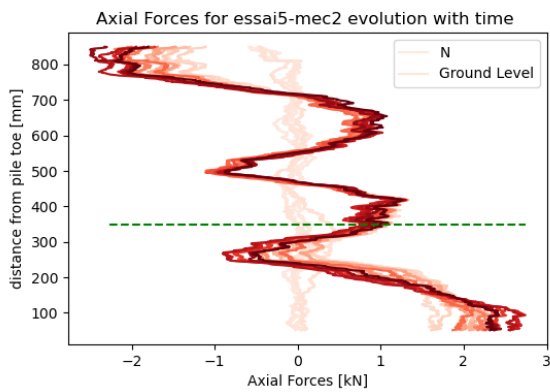


Figure D.99: Axial forces evolution with time

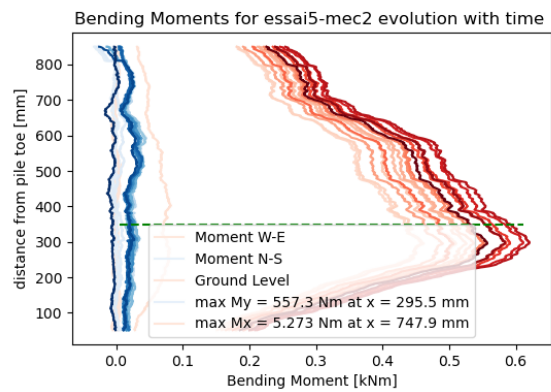


Figure D.100: Moments evolution with time

APPENDIX E

Initial cyclic data

Soil core 3-24

Client			Borehole			Test no		
Site			Sample			System		
Job no			Sample depth (m)	-		Specimen depth (m)	-	

Ladd undercompaction preparation sheet - Triaxial tests

Test Type	CIUc		Instrumentation				
Min dry density (Mg/m ³)		Max density (Mg/m ³)		Relative density, D _r (%)			No data
Suggested U _{ni}	15,0	Membrane Thickness (mm)		Estimated density, D _r (%)			
Total mass (gr)	945,05						

REQUIRED SPECIMEN CONDITIONS

Dry Density (Mg/m ³)	Specimen Diameter (mm)	Number of layers (n)	Specimen height (mm)	Initial Moisture Content (%)	Particle density Assmd. (Mg/m ³)	Mould height (mm)	Under compaction of first layer U _{ni} (%)
1,64	69,99	9	140,00	7,0	2,65	181,59	15,0
Initial void ratio e _i	Degree of saturation S _r (%)	Volume of voids (cm ³)	Saturated bulk density (Mg/m ³)	Specimen Area (mm ²)	Specimen Volume (cm ³)	Total mas (gr)	Difference in mould height (mm)
0,62	30,12	143,43	1,755	3846,80	538,55	945,05	41,59

UNDER COMPACTION CONDITIONS

Layer (n)	U _n	Final layer height (mm)	Accumulative mass (g)	Incremental mass (g)	Required yoke setting (mm)
1	15,0	17,89	105,01	105,01	163,70
2	13,1	33,15	210,01	105,01	148,44
3	11,3	48,42	315,02	105,01	133,18
4	9,4	63,68	420,02	105,01	117,91
5	7,5	78,94	525,03	105,01	102,65
6	5,6	94,21	630,03	105,01	87,38
7	3,8	109,47	735,04	105,01	72,12
8	1,9	124,74	840,05	105,01	56,86
9	0,0	140,00	945,05	105,01	41,59

INITIAL MOISTURE CONTENT		INITIAL SAMPLE MASS		NOTES
Date sample in oven		Date		
Tin number		Mould and specimen (g)	1546,68	
Wet mass and tin (g)		Mould only (g)	1601,55	
Dry mass and tin (g)		Specimen mass (g)	945,14	
Mass of tin only (g)		Bulk density (Mg/m ³)	2,755	
Moisture content (%)		Sample Preparation by		

SAMPLE DESCRIPTION

Set loose = 1 kg

Client #		Borehole	0	Test no	0
Site ##		Sample	0	System	0
Job no #		Sample depth (m)	0,00	Specimen depth (m)	0,00

LLN		ADVANCED LABORATORY TRIAXIAL WORKSHEET					
Client stage no	GDS stage no	Type of stage	sr' 50	sv' 50	Process	U / D?	Checked by
Tested by			sr' 0	sv' 0			
			sr' 0	sv' 0			
	1	Setup	CP 50	BP 40 fermé	☐		
	2	CO ₂ flush	CP 50	BP 40 fermé injecte CO ₂ 40 kPa	☐		
	3	H ₂ O flush	CP 50	BP 40 je plus d'eau qui sort	☐		
	4	H₂O flush	CP 50	BP 40	☐		
	4	B Solution	CP ↑ 600	BP ↑ 590 1 h	☐		
	5	Back	CP ↑ 700	BP ↑ 690	☐		
	6	Iso setup	CP 700	BP = PWP = 690	☐		
	7	Iso unloading	CP ↑ 780	BP = 690	☐		
	8		(190 - 0,106) · 0,0075 = 0,203841		☐		
					☐		
					☐		

Str 264 Range SL₂

Client			Borehole			Test no		
Site			Sample			System		
Job no			Sample depth (m)			Specimen depth (m)	-	

Ladd undercompaction preparation sheet - Triaxial tests

Test Type			Instrumentation				
Min dry density (Mg/m ³)		Max density (Mg/m ³)		Relative density, D _r (%)	No data		
Suggested U _{ni}	15,0	Membrane Thickness (mm)		Estimated density, D _r (%)			
Total mass (gr)	1048,30						

REQUIRED SPECIMEN CONDITIONS

Dry Density (Mg/m ³)	Specimen Diameter (mm)	Number of layers (n)	Specimen height (mm)	Initial Moisture Content (%)	Particle density Assmd. (Mg/m ³)	Mould height (mm)	Under compaction of first layer U _{ni} (%)
1,82	69,95	9	140,00	7,0	2,65	180,86	15,0
Initial void ratio e _i	Degree of saturation S _r (%)	Volume of voids (cm ³)	Saturated bulk density (Mg/m ³)	Specimen Area (mm ²)	Specimen Volume (cm ³)	Total mas (gr)	Difference in mould height (mm)
0,46	40,75	99,73	1,94847	3842,96	538,01	1048,30	40,86

UNDER COMPACTION CONDITIONS

Layer (n)	U _n	Final layer height (mm)	Accumulative mass (g)	Incremental mass (g)	Required yoke setting (mm)
1	15,0	17,89	116,48	116,48	162,97
2	13,1	33,15	232,96	116,48	147,71
3	11,3	48,42	349,43	116,48	132,44
4	9,4	63,68	465,91	116,48	117,18
5	7,5	78,94	582,39	116,48	101,92
6	5,6	94,21	698,87	116,48	86,65
7	3,8	109,47	815,35	116,48	71,39
8	1,9	124,74	931,83	116,48	56,12
9	0,0	140,00	1048,30	116,48	40,86

INITIAL MOISTURE CONTENT	INITIAL SAMPLE MASS	NOTES
Date sample in oven Tin number Wet mass and tin (g) Dry mass and tin (g) Mass of tin only (g) Moisture content (%)	Date Mould and specimen (g) <u>2554,62</u> Mould only (g) <u>1606,08</u> Specimen mass (g) <u>948,54</u> Bulk density (Mg/m ³) <u>1,75</u> Sample Preparation by	

SAMPLE DESCRIPTION

Soil data sheet

Client #		Borehole	0	Test no	0
Site ##		Sample	0	System	0
Job no #		Sample depth (m)	0,00	Specimen depth (m)	0,00

LLN

ADVANCED LABORATORY TRIAXIAL WORKSHEET

Client stage no	GDS stage no	Type of stage	sr'	sv'	Process	U / D?	Chid
			50	50			
Tested by			sr'	sv'			

Measured membrane thickness

/ 4 =

Number of membranes

	1	Setup	radial stress = 50 ho cl pressure = 40 Is loose		☐		
	2	CO ₂ flush	Done CO ₂ attached CP=50		☐		
	3	H ₂ O flush	CP=50 BP=40		☐		
	4	Saturation	CP 1600 BP 1590 60 min		☐		
	5	B check	CP 1700 BP 1690		☐		
	6	Iso setup	CP=700 BP=PWP=693		☐		
	7	Iso consolidation	CP 1743 BP=693 60 min		☐		
	8	Cyclic	$(140 - 0,1555) \times 0,0015$ = 0,209876 mm		☐		
					☐		
					☐		

Dense SATI-1Hz

Client	Internal Project		Borehole	COMPS		Test no	1c	
Site	Laboratory		Sample	A		System		
Job no	221039		Sample depth (m)	-		Specimen depth (m)	-	

Ladd undercompaction preparation sheet - Triaxial tests

Test Type	CIUc		Instrumentation				
Min dry density (Mg/m ³)		Max density (Mg/m ³)		Relative density, D _r (%)	No data		
Suggested U _{ni}	15,0	Membrane Thickness (mm)		Estimated density, D _r (%)			
Total mass (gr)	985,57						

REQUIRED SPECIMEN CONDITIONS

Dry Density (Mg/m ³)	Specimen Diameter (mm)	Number of layers (n)	Specimen height (mm)	Initial Moisture Content (%)	Particle density Assmd. (Mg/m ³)	Mould height (mm)	Under compaction of first layer U _{ni} (%)
1,70	70,20	9	140,00	7,0	2,65	180,75	15,0
Initial void ratio e _i	Degree of saturation S _r (%)	Volume of voids (cm ³)	Saturated bulk density (Mg/m ³)	Specimen Area (mm ²)	Specimen Volume (cm ³)	Total mas (gr)	Difference in mould height (mm)
0,56	33,19	129,76	1,819	3870,14	541,82	985,57	40,75

UNDER COMPACTION CONDITIONS

Layer (n)	U _n	Final layer height (mm)	Accumulative mass (g)	Incremental mass (g)	Required yoke setting (mm)
1	15,0	17,89	109,51	109,51	162,86
2	13,1	33,15	219,02	109,51	147,59
3	11,3	48,42	328,52	109,51	132,33
4	9,4	63,68	438,03	109,51	117,06
5	7,5	78,94	547,54	109,51	101,80
6	5,6	94,21	657,05	109,51	86,54
7	3,8	109,47	766,55	109,51	71,27
8	1,9	124,74	876,06	109,51	56,01
9	0,0	140,00	985,57	109,51	40,75

INITIAL MOISTURE CONTENT		INITIAL SAMPLE MASS		NOTES
Date sample in oven		Date		
Tin number		Mould and specimen (g)	2578,07	
Wet mass and tin (g)		Mould only (g)	7602,56	
Dry mass and tin (g)		Specimen mass (g)	976,62	
Mass of tin only (g)		Bulk density (Mg/m ³)	1,802	
Moisture content (%)		Sample Preparation by		

SAMPLE DESCRIPTION

1	Setup	CP = 50 BP = 40
2	CO ₂ flush	CP = 50 BP = 40
3	Exhaustion H ₂ O flush	CP = 50 BP = 40
4 5	Sedation	CP ↑ 600 BP ↑ 380 60 min
6	B check	CP ↑ 700 → 1h BP ↑ 680 → cst
7	Iso setup	CP = 700 BP = 680
8	Iso unsoluble	CP ↑ 740
9	§	BP = 680
10	$(140 - 0,130) = 0,015$ $= 0,208805$	

max clear skin = 26
 max clear skin = 0,273
 max clear skin = 0,52
 max clear skin = 0,12



Dense Soil B-SH₂ - Bending elements

Client			Borehole			Test no		
Site			Sample			System		
Job no			Sample depth (m)	-	Specimen depth (m)	-		
Ladd undercompaction preparation sheet - Triaxial tests								
Test Type	CIUc		Instrumentation					
Min dry density (Mg/m ³)			Max density (Mg/m ³)		Relative density, D _r (%)		No data	
Suggested U _{ni}	15,0		Membrane Thickness (mm)		Estimated density, D _r (%)			
Total mass (gr)	1012,32							
REQUIRED SPECIMEN CONDITIONS								
Dry Density (Mg/m ³)	Specimen Diameter (mm)	Number of layers (n)	Specimen height (mm)	Initial Moisture Content (%)	Particle density Assmd. (Mg/m ³)	Mould height (mm)	Under compaction of first layer U _{ni} (%)	
1,76	69,92	9	140,00	7,0	2,65	180,98	15,0	
Initial void ratio e _i	Degree of saturation S _r (%)	Volume of voids (cm ³)	Saturated bulk density (Mg/m ³)	Specimen Area (mm ²)	Specimen Volume (cm ³)	Total mas (gr)	Difference in mould height (mm)	
0,51	36,68	114,31	1,883	3839,66	537,55	1012,32	40,98	
UNDER COMPACTION CONDITIONS								
Layer (n)	U _n	Final layer height (mm)	Accumulative mass (g)	Incremental mass (g)	Required yoke setting (mm)			
1	15,0	17,89	112,48	112,48	163,09			
2	13,1	33,15	224,96	112,48	147,83			
3	11,3	48,42	337,44	112,48	132,56			
4	9,4	63,68	449,92	112,48	117,30			
5	7,5	78,94	562,40	112,48	102,04			
6	5,6	94,21	674,88	112,48	86,77			
7	3,8	109,47	787,36	112,48	71,51			
8	1,9	124,74	899,84	112,48	56,24			
9	0,0	140,00	1012,32	112,48	40,98			
INITIAL MOISTURE CONTENT								
Date sample in oven			Date			NOTES		
Tin number			Mould and specimen (g)			B3-70, T-2-30 ↳ Bending elements		
Wet mass and tin (g)			Mould only (g)					
Dry mass and tin (g)			Specimen mass (g)					
Mass of tin only (g)			Bulk density (Mg/m ³)					
Moisture content (%)			Sample Preparation by					
SAMPLE DESCRIPTION								

Name: DENSESATB-SH₃

	Expt	BP	CP	Rem
1	set up	40 peni	50	
2	CO ₂ flush	40 peni CO ₂ 40	50	20 min
3	H ₂ O flush	40	50	ju → plus ab bulle + V ↑ V _{nois}
4	saturations	↑ 590	↑ 600	2 h
5	B check	↑ 680	↑ 700	1 h
6	Iso setup			

Client			Borehole			Test no		
Site			Sample			System		
Job no			Sample depth (m)			Specimen depth (m)	-	

Ladd undercompaction preparation sheet - Triaxial tests							
Test Type			Instrumentation				
Min dry density (Mg/m ³)		Max density (Mg/m ³)		Relative density, D _r (%)	No data		
Suggested U _{ni}	15,0	Membrane Thickness (mm)		Estimated density, D _r (%)			
Total mass (gr)	998,61						

REQUIRED SPECIMEN CONDITIONS							
Dry Density (Mg/m ³)	Specimen Diameter (mm)	Number of layers (n)	Specimen height (mm)	Initial Moisture Content (%)	Particle density Assmd. (Mg/m ³)	Mould height (mm)	Under compaction of first layer U _{ni} (%)
1,71	72,59	9	141,11	0,0	2,65	141,11	15,0
Initial void ratio e _i	Degree of saturation S _r (%)	Volume of voids (cm ³)	Saturated bulk density (Mg/m ³)	Specimen Area (mm ²)	Specimen Volume (cm ³)	Total mas (gr)	Difference in mould height (mm)
0,55	0,00	207,15	1,71	4138,50	583,98	998,61	0,00

UNDER COMPACTION CONDITIONS					
Layer (n)	U _n	Final layer height (mm)	Accumulative mass (g)	Incremental mass (g)	Required yoke setting (mm)
1	15,0	18,03	110,96	110,96	123,08
2	13,1	33,42	221,91	110,96	107,69
3	11,3	48,80	332,87	110,96	92,31
4	9,4	64,19	443,83	110,96	76,92
5	7,5	79,57	554,79	110,96	61,54
6	5,6	94,96	665,74	110,96	46,15
7	3,8	110,34	776,70	110,96	30,77
8	1,9	125,73	887,66	110,96	15,38
9	0,0	141,11	998,61	110,96	0,00

INITIAL MOISTURE CONTENT		INITIAL SAMPLE MASS		NOTES
Date sample in oven		Date		
Tin number		Mould and specimen (g)		
Wet mass and tin (g)		Mould only (g)		
Dry mass and tin (g)		Specimen mass (g)		
Mass of tin only (g)		Bulk density (Mg/m ³)		
Moisture content (%)		Sample Preparation by		
SAMPLE DESCRIPTION				

Client			Borehole			Test no		
Site			Sample			System		
Job no			Sample depth (m)			Specimen depth (m)	-	
Ladd undercompaction preparation sheet - Triaxial tests								
Test Type			Instrumentation					
Min dry density (Mg/m ³)			Max density (Mg/m ³)			Relative density, D _r (%)	No data	
Suggested U _{ni}	15,0		Membrane Thickness (mm)			Estimated density, D _r (%)		
Total mass (gr)	1051,17							
REQUIRED SPECIMEN CONDITIONS								
Dry Density (Mg/m ³)	Specimen Diameter (mm)	Number of layers (n)	Specimen height (mm)	Initial Moisture Content (%)	Particle density Assmd. (Mg/m ³)	Mould height (mm)	Under compaction of first layer U _{ni} (%)	
1,80	72,59	9	141,11	0,0	2,65	141,11	15,0	
Initial void ratio e _i	Degree of saturation S _r (%)	Volume of voids (cm ³)	Saturated bulk density (Mg/m ³)	Specimen Area (mm ²)	Specimen Volume (cm ³)	Total mas (gr)	Difference in mould height (mm)	
0,47	0,00	187,32	1,8	4138,50	583,98	1051,17	0,00	
UNDER COMPACTION CONDITIONS								
Layer (n)	U _n	Final layer height (mm)	Accumulative mass (g)	Incremental mass (g)	Required yoke setting (mm)			
1	15,0	18,03	116,80	116,80	123,08			
2	13,1	33,42	233,59	116,80	107,69			
3	11,3	48,80	350,39	116,80	92,31			
4	9,4	64,19	467,19	116,80	76,92			
5	7,5	79,57	583,98	116,80	61,54			
6	5,6	94,96	700,78	116,80	46,15			
7	3,8	110,34	817,58	116,80	30,77			
8	1,9	125,73	934,38	116,80	15,38			
9	0,0	141,11	1051,17	116,80	0,00			
INITIAL MOISTURE CONTENT		INITIAL SAMPLE MASS			NOTES			
Date sample in oven		Date						
Tin number		Mould and specimen (g)						
Wet mass and tin (g)		Mould only (g)						
Dry mass and tin (g)		Specimen mass (g)						
Mass of tin only (g)		Bulk density (Mg/m ³)						
Moisture content (%)		Sample Preparation by						
SAMPLE DESCRIPTION								

APPENDIX F

Technical sheet of Advanced Dynamic Triaxial Testing System

Systems Elements & Options

The fundamental system hardware elements are shown in Fig. 1 below. The actual hardware used may be chosen to suit your testing and budgetary requirements. The more common arrangements are as follows:

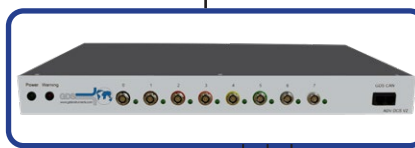
GDSLAB Software

The GDSLAB control and acquisition software is a highly developed, yet extremely flexible software platform. Starting with the Kernel module and the ability to perform data acquisition, additional modules are added for your testing requirements.



ADVDCS v2 Acquisition Pad

The ADVDCS v2 is based around a modern, high speed, 32 bit processing core and has eight simultaneous sampling 24 bit universal analogue input channels, enabling any transducer in the GDS range to be connected. High speed digital bus technology allows real-time streaming of transducer data making it ideal for high speed data acquisition. The ADVDCS v2 supports full digital control of servo motor and hydraulic actuators allowing accurate, precise and noise free control of actuators.



Pore Pressure (kPa)

Axial Strain (mm)

Load Cell (kN)

Load frame control

Cell pressure / volume

Back pressure / volume



Note: Connection via USB

Pressure Volume / Controllers

The cell and back pressures can be controlled with either one of the below:

- The Standard Pressure/Volume Controller (STDDPC) with pressure ranges from 1 to 4MPa, serial PC connectivity and 200cc volumetric capacity.
- The Advanced Pressure / Volume Controller (ADVDPV) with pressure ranges of 2 to 4MPa, USB connectivity and 200cc volumetric capacity, (ADVDPV 2MPa controller can be bought as a 1000cc volumetric capacity item).

Optional Electromechanical dynamic cell pressure actuator: The dynamic actuator allows cycling of cell pressure between two user chosen values. For example between 200kPa- 400kPa cycled at 1Hz, enabling dynamic stress path testing to be performed. The maximum pressure of this actuator is 1MPa.

Frequency Range (Hz)

- 5

Load Range (kN)

- 10, 40 or 60.

Specimen Sizes (mm)

- 38, 39.1, 50, 61.8, 70, 100, 101, 150 or 300

APPENDIX G

Technical sheet of Triaxial Automated System and Bender Elements

Systems Elements & Options

The fundamental system hardware elements are shown in Fig. 1 below. The actual hardware used may be chosen to suit your testing and budgetary requirements.

GDSLAB Software

The GDSLAB control and acquisition software is a highly developed, yet extremely flexible software platform. Starting with the Kernel module and the ability to perform data acquisition, additional modules are added for your testing requirements.



Note: Connection via USB interface to PC.

USB 8 Channel Logger (PAD)

The USB Pad provides eight fully independent channels of simultaneously sampled ultra-high resolution 24-bit data. Each channel provides 22 software selectable gain ranges, precision ratiometric transducer excitation, monitoring and acquisition - an industry standard DIN connection allows the full-range of GDS transducers to be quickly and easily connected and configured.



Pore Pressure (kPa)
Axial Strain (mm)
Load Cell (kN)

Load frame control

Cell pressure / volume

Back pressure / volume



Pressure Volume / Controllers

- Enterprise Triaxial Automated System (ELTAS), which utilises the 1MPa Enterprise Pressure / Volume Controller.
- Standard Triaxial Automated System (STDTAS), which utilises the Standard Pressure / Volume Controller, 1-4MPa.
- Advanced Triaxial Automated System (ADVTAS), which utilises the Advanced Pressure / Volume Controller, 2-8MPa.
- High Pressure Triaxial Automated System (HPTAS), which utilises the High Pressure Controllers (≥ 8 MPa).

Load Frames & Triaxial Cells

- The user can choose the load frame, for their ideal set-up, with frames from 10kN to 1MN.
- 2000kPa, specimens up to 150mm (load frames > 50 kN for 150mm cell due to size).
- 3400kPa, specimens up to 77mm.
- 14MPa, specimens up to 100mm.
- 20MPa, specimens up to 70mm.
- 64MPa, specimens up to 100mm.

Upgrade Options:

Upgrade to Local Strain Measurement

Any GDSTAS system may be upgraded to perform Local Strain measurement using either Hall Effect or LVDT transducers. Both device types enable axial and radial deformation to be measured directly on the test specimen via lightweight aluminium holders. Hall Effect transducers may be used in water up to 1700kPa.

LVDT transducers come in 2 versions:

- Low pressure (up to 3500 kPa) version for use in water.
- High pressure (up to 200 MPa) version for use in nonconducting oil.



Fig. 2 LVDT transducers as shown on sample.

Upgrade to Unsaturated Testing

Any GDSTAS system may be upgraded to perform unsaturated triaxial testing with the addition of the following items:

- Unsaturated pedestal with high air entry porous stone
- 1000cc Advanced Pressure/Volume Controller (for application of pore air pressure and measurement of air volume change).

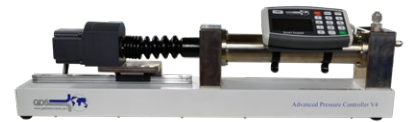


Fig. 3 Advanced Pressure Controller used in Unsaturated Testing

For further information on unsaturated testing methods, please refer to the dedicated unsaturated datasheet.

Upgrade to Bender Element Testing

Any GDSTAS system may be upgraded to perform P and S wave bender element testing with the addition of the following items:

- Bender element pedestal with bender element insert.
- Bender element top-cap with bender element insert.
- High-speed data acquisition card.
- Signal conditioning unit which includes amplification of source and received signals (P and S-wave) with user controlled gain levels (via software).

GDS Bender Element Analysis Tool (GDSBEAT):

The subjectivity and lack of satisfactory standards for interpreting shear wave travel times across the industry from bender element test data, has led GDS to develop a bender elements analysis tool. The tool allows the rapid, automated analysis of bender element tests to objectively estimate the shear wave travel time. The analysis tool is available to download from GDS' website.

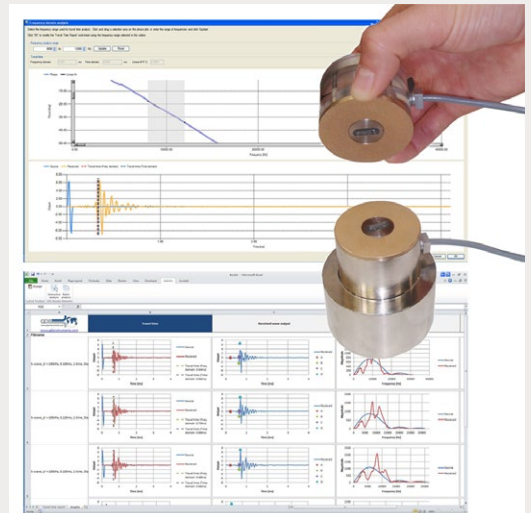


Fig. 4 Shows screenshots of the GDSBEAT software.

Standards:

- ASTM D-4767, ASTM D-5084, ASTM D-2850, ASTM D-2850-03a, ASTM D-7181
- AS 1289.6.4.1, AS 1289.6.4.2, AS 1289.6.6.1
- BS 1377-6, BS 1377-7, BS 1377-8
- CEN ISO/TS 17892-8, CEN ISO/TS 17892-9, CEN ISO/TS 17892-11
- JGS 0412, JGS 0521, JGS 0522, JGS 0523, JGS 0524

Tests that can be Performed:

B-Check, Consolidated Drained (CD) Triaxial, Consolidated Undrained (CU) Triaxial, Consolidation (Triaxial), Constant rate of Loading (CRL) Consolidation, Constant rate of strain (CRS) Consolidation, Slow Cyclic Testing, K0 (K-Zero), Multi-stage Testing, Quasi-Static (low speed/creep) Tests, Stress Paths and Unconsolidation Undrained (UU) Triaxial.

GDSLAB Control Software

GDSLAB is the control and data acquisition software for geotechnical laboratory applications. GDSLAB starts with a core application known as the kernel. The GDSLAB kernel allows for data acquisition from your hardware, but no test control. Simply add the appropriate module or modules to complete the test suite functionality you require. GDSLAB is compatible with all existing GDS equipment and furthermore key hardware from other manufacturers.

GDSLAB has the ability to be configured to your hardware of choice, no matter how unique the arrangement. A text file (*.ini) or initialisation file is created that describes the hardware connectivity to the PC. The hardware layout is available in graphical format via the GDSLAB 'object display'. This makes setting up the devices and checking the connectivity extremely simple.

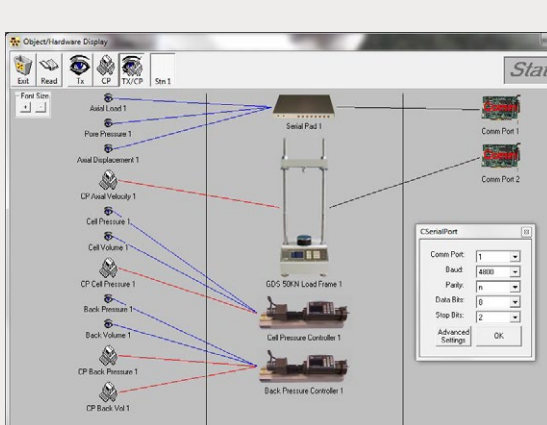


Fig. 5 Show a typical set-up screen in GDSLAB

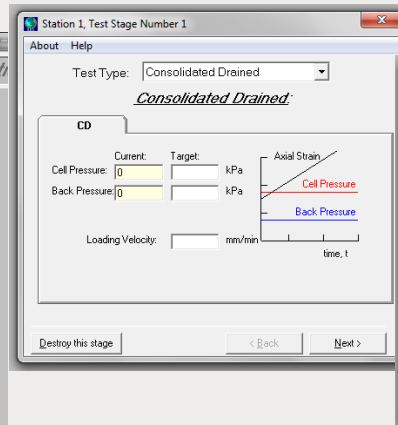


Fig. 6 Show a typical station test stage set-up in GDSLAB

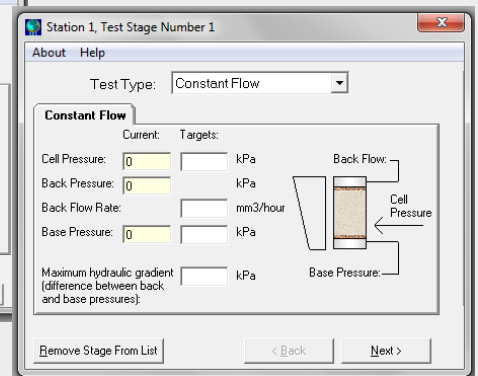


Fig. 7 Show a typical station test stage set-up in GDSLAB

Required Operating System: Windows 7 SP1 or higher (We strongly recommend that Windows is fully up to date and running the latest Service Pack/Version available). Recommended PC Specification: 2GHz processor, 4GB Ram, 64Bit Operating System and USB connectivity. Note: GDS software can run on lower spec PC's however; performance and processing of data may be affected.

GDSLAB REPORTS Presentation Software

GDSLAB REPORTS software presents data obtained by GDSLAB to the National Standard, BS 1377:1990. The program can be used to present data whether obtained from a GDSLAB data file or inputted by hand.

GDSLAB Reports can as be used with other manufacturer's dataloggers as well as all versions of GDS data logger. The results can be exported as a CSV file into Microsoft Excel, allowing the user to customise the layout of results.

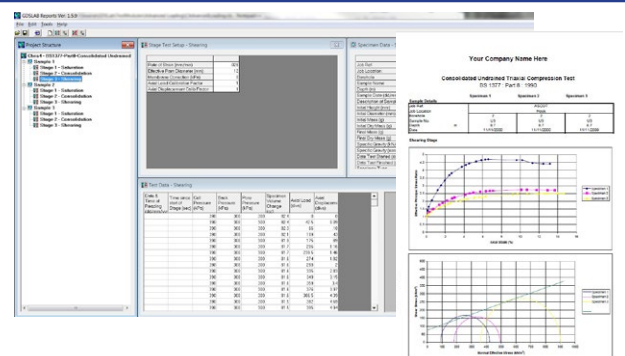
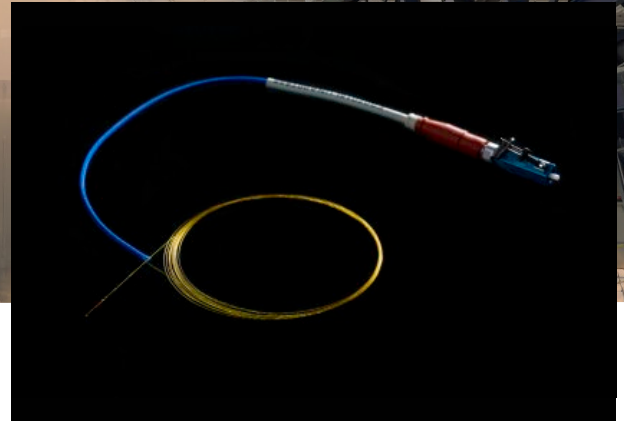


Fig. 8 show a selection of screenshots from the GDSLAB Reports software.

APPENDIX H

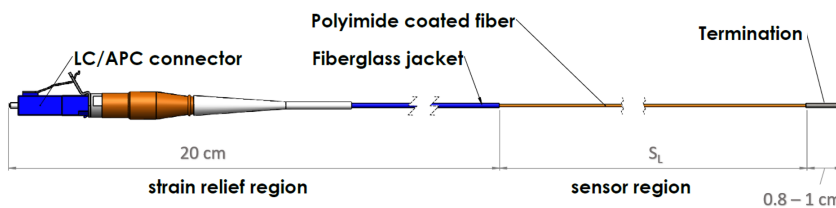
Technical sheet of the optic fibre system



HD Strain Sensors

High-Definition Fiber Optic Strain Sensors

Luna's high-definition (HD) strain sensors are low-profile, flexible sensors for use with the ODiSI measurement system for ultra-high resolution strain measurements with gage pitches down to 0.65 mm. HD sensors are supplied with NIST-traceable calibration coefficients and a unique key ID that enables plug-and-play identification and operation.



KEY FEATURES

- Low profile for embedded or surface applications
- Flexible, polyimide coated fiber
- NIST-traceable calibration
- Included sensor key enables plug-and-play operation with ODiSI system
- Sensing lengths from 1 - 100 m

PARAMETER	SPECIFICATIONS
Fiber type	Polyimide coated low bend loss fiber
Strain Relief	20 cm, fiberglass
Sensor diameter	155 μm
Termination diameter	286 μm
Minimum bend radius	10 mm
Sensor termination length, material	0.8 cm, glass
Operating temperature - sensing region	-40 to 300 $^{\circ}\text{C}$
Operating temperature - connector	-60 to 150 $^{\circ}\text{C}$

ORDERING

Part Number

HD6SxxLC300P

xx

Description

High-definition (HD) strain sensor, with reference calibration key files

Indicates sensor length S_L (m): 01, 02, 03, 05, 10, 15, 20, 30, 40, 50 or 100 m



HDSS Rev 3 01/13/22

+1.866.586.2682
solutions@lunainc.com
www.lunainc.com

*Specifications subject to change without notice.

APPENDIX I

Technical sheets of the accelerometers

Model Number 350C03		ICP® SHOCK ACCELEROMETER		Revision: C ECN #: 52331		
Performance Sensitivity(± 30 %) Measurement Range Frequency Range(± 1 dB) Frequency Range(- 3 dB) Electrical Filter Corner Frequency(- 3 dB) Mechanical Filter Resonant Frequency Resonant Frequency Broadband Resolution(1 to 10,000 Hz) Non-Linearity(per 1000 g (9810 m/s ²)) Transverse Sensitivity Environmental Overload Limit(Shock) Temperature Range(Operating) Temperature Range(Storage) Temperature Response Base Strain Sensitivity Electrical Excitation Voltage Constant Current Excitation Output Impedance Output Bias Voltage Discharge Time Constant Settling Time(within 10% of bias) Physical Sensing Element Sensing Geometry Housing Material Sealing Size (Hex x Height) Weight Electrical Connector Electrical Connection Position Mounting Mounting Thread Mounting Torque	ENGLISH 0.5 mV/g ± 10,000 g pk 0.4 to 10,000 Hz 0.2 to 25,000 Hz 17 kHz 35 kHz ≥ 100 kHz 0.04 g rms ≤ ±2.0 % ≤ 7 % ± 50,000 g pk 0 to +150 °F -40 to +200 °F See Graph 0.002 g/με 20 to 30 VDC 2 to 20 mA ≤ 200 Ohm 8 to 14 VDC 1.0 to 2.0 sec ≤ 10 sec Ceramic Shear Titanium Hermetic 0.375 in x 1.15 in 0.19 oz 10-32 Coaxial Jack Top Stud 1/4-28 Male 25 to 35 in-lb		SI 0.05 mV/(m/s ²) ± 98,000 m/s ² pk 0.4 to 10,000 Hz 0.2 to 25,000 Hz 17 kHz 35 kHz ≥ 100 kHz 0.39 m/s ² rms ≤ ±2.0 % ≤ 7 % ± 490,000 m/s ² pk -18 to +66 °C -40 to +93 °C See Graph 0.02 (m/s ²)/με 20 to 30 VDC 2 to 20 mA ≤ 200 Ohm 8 to 14 VDC 1.0 to 2.0 sec ≤ 10 sec Ceramic Shear Titanium Hermetic 9.5 mm x 29.2 mm 5.4 gm 10-32 Coaxial Jack Top Stud 1/4-28 Male 280 to 390 N-cm			

Model Number 353B33	ICP® ACCELEROMETER			Revision: N ECN #: 26810
Performance	ENGLISH	SI		OPTIONAL VERSIONS
Sensitivity(± 5 %)	100 mV/g	10.19 mV/(m/s²)	[2]	Optional versions have identical specifications and accessories as listed for the standard model except where noted below. More than one option may be used.
Measurement Range	± 50 g pk	± 491 m/s² pk		
Frequency Range(± 5 %)	1 to 4000 Hz	1 to 4000 Hz		
Frequency Range(± 10 %)	0.7 to 6500 Hz	0.7 to 6500 Hz		
Frequency Range(± 3 dB)	0.35 to 12,000 Hz	0.35 to 12,000 Hz		
Resonant Frequency	≥ 22 kHz	≥ 22 kHz		
Broadband Resolution(1 to 10,000 Hz)	0.0005 g rms	0.005 m/s² rms	[1]	
Non-Linearity	≤ 1 %	≤ 1 %	[3]	
Transverse Sensitivity	≤ 5 %	≤ 5 %	[4]	
Environmental				
Overload Limit(Shock)	± 10,000 g pk	± 98,100 m/s² pk		
Temperature Range(Operating)	-65 to +250 °F	-54 to +121 °C		
Temperature Response	See Graph	See Graph	[1]	
Base Strain Sensitivity	0.0002 g/με	0.002 (m/s²)/με	[1]	
Electrical				
Excitation Voltage	18 to 30 VDC	18 to 30 VDC		
Constant Current Excitation	2 to 20 mA	2 to 20 mA		
Output Impedance	≤ 100 ohm	≤ 100 ohm		
Output Bias Voltage	7.5 to 11.5 VDC	7.5 to 11.5 VDC		
Discharge Time Constant	0.5 to 2.0 sec	0.5 to 2.0 sec		
Settling Time(within 10% of bias)	<25 sec	<25 sec		
Spectral Noise(1 Hz)	320 μg/√Hz	3139 (μm/s²)/√Hz	[1]	
Spectral Noise(10 Hz)	70 μg/√Hz	687 (μm/s²)/√Hz	[1]	
Spectral Noise(100 Hz)	18 μg/√Hz	177 (μm/s²)/√Hz	[1]	
Spectral Noise(1 kHz)	6.4 μg/√Hz	63 (μm/s²)/√Hz	[1]	
Physical				
Sensing Element	Quartz	Quartz		
Sensing Geometry	Shear	Shear		
Housing Material	Titanium	Titanium		
Sealing	Welded Hermetic	Welded Hermetic		
Size (Hex x Height)	0.75 in x 0.93 in	19.1 mm x 23.6 mm	[1]	
Weight	0.95 oz	27 gm		
Electrical Connector	10-32 Coaxial Jack	10-32 Coaxial Jack		
Electrical Connection Position	Side	Side		
Mounting Thread	10-32 Female	10-32 Female		
CE [5] Typical Sensitivity Deviation vs Temperature				
NOTES: [1] Typical. [2] B and Q options supplied with a sensitivity tolerance of ± 10 %. [3] Zero-based, least-squares, straight line method. [4] Transverse sensitivity is typically ≤ 3%. [5] See PCB Declaration of Conformance PS023 for details.				
SUPPLIED ACCESSORIES: Model 080A109 Petro Wax (1) Model 080A12 Adhesive Mounting Base (1) Model 081B05 Mounting Stud (10-32 to 10-32) (1) Model ACS-1 NIST traceable frequency response (10 Hz to upper 5% point). (1) Model M081B05 Mounting Stud 10-32 to M6 X 0.75 (1)				
Entered: BLS	Engineer: BAM	Sales: WDC	Approved: EB	Spec Number:
Date: 7-10-07	Date: 7-2-07	Date: 7-2-07	Date: 7-5-07	353-2330-80
PCB PIEZOTRONICS® VIBRATION DIVISION 3425 Walden Avenue, Depew, NY 14043 Phone: 716-684-0001 Fax: 716-685-3886 E-Mail: vibration@pcb.com				

APPENDIX J

Technical sheet of the laser Keyence LK-G502



> Print Page

LK-G Series
CCD Laser Displacement Sensor

CE

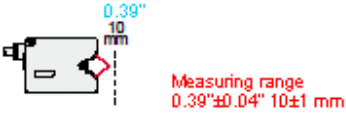
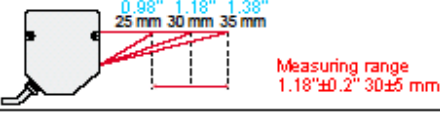
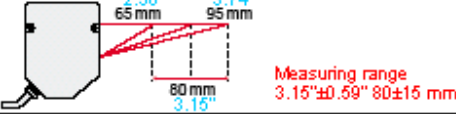
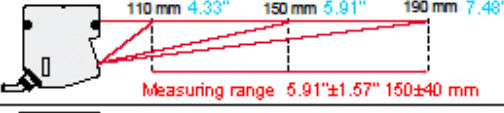
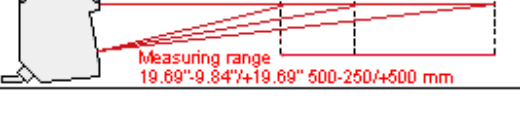
- Affordable and 100 % Reliable Measurement for Steel Manufacturer
- Non-contact Measurement Sensors Reduces Scrap, Provides 100 % Inspection

Specifications

Best in Class Performance with the Specifications to Prove it

For more details, download the [PRODUCT CATALOG](#) by clicking the [CATALOG](#) icon above.

LK-G Series Sensor Head Variations

Type	Model	Measurement Range	Resolution	Spot size
Super Precision	LK-G10		0.0004 Mil 0.01 µm	Ø0.79 Mil Ø20 µm
	Wide beam LK-G15			0.79x19.69 Mil 20x500 µm
High Accuracy	LK-G32		0.002 Mil 0.05 µm	Ø1.18 Mil Ø30 µm
	Wide beam LK-G37			1.18x33.46 Mil 30x850 µm
Multi-purpose	LK-G82		0.008 Mil 0.2 µm	Ø2.76 Mil Ø70 µm
	Wide beam LK-G87			2.76x43.31 Mil 70x1100 µm
Long Distance	LK-G152		0.02 Mil 0.5 µm	Ø4.72 Mil Ø120 µm
	Wide beam LK-G157			4.72x66.93 Mil 120x1700 µm
High-speed, Long Distance	LK-G402		0.08 Mil 2 µm	Ø11.42 Mil Ø290 µm
	Wide beam LK-G407			11.42x326.8 Mil 290x8300 µm
Ultra Long Distance	LK-G502		0.08 Mil 2 µm	Ø11.81 Mil Ø300 µm
	Wide beam LK-G507			11.81x374 Mil 300x9500 µm

LK-G Series Laser Displacement Sensor Head Specifications

UNIVERSITÉ CATHOLIQUE DE LOUVAIN

École polytechnique de Louvain

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