

École polytechnique de Louvain

Influence of vibratory pile driving on offshore wind turbine mechanical behaviour

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Abstract

Master thesis :

Influence of vibratory pile driving on offshore wind turbine mechanical behaviour

by Shahrour Mohamed

As global energy demands continue to rise, the transition to renewable energy sources, such as offshore wind power, has become increasingly critical. This thesis investigates the comparative effectiveness of two installation methods for offshore wind turbine monopile foundations: impact hammering and vibratory driving. While vibratory driving is often favored for its reduced noise levels, quicker installation times, and potentially lower costs, its impact on the soil's mechanical behavior, particularly in terms of lateral resistance, remains less understood. The primary focus of this research is to evaluate how these driving methods affect the p-y curves, which describe the lateral load-displacement behavior of the monopiles in Northern Sea sand. The study delves into the differences in soil resistance and the implications of grain displacement and rearrangement under different vibratory frequencies.

To achieve accurate and meaningful results, the research employs scaling laws and an optimized laboratory setup, including a controlled sand pluviation system that ensures consistent sand density in both dry and saturated conditions. The 1g laboratory environment is carefully calibrated to simulate real-life conditions, making the findings relevant to actual offshore scenarios.

By comparing the performance of impact hammering and vibratory driving in these controlled settings, this thesis aims to provide deeper insights into the suitability of vibratory driving not only in terms of installation efficiency but also in terms of long-term soil resistance and monopile stability. The outcomes of this research are expected to contribute valuable knowledge to the design and installation of offshore wind turbine foundations, particularly in the context of the European Union's renewable energy goals.

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Nomenclature

$\gamma_{d,\max}$	Dry unit weight of sand at its densest state
$\gamma_{d,\min}$	Dry unit weight of sand at its loosest state
γ_{sat}	Saturated unit weight of the soil
κ	Curvature of the beam
μ	Viscosity of the pore fluid
ν	Pile displacement at the mudline level
ϕ	Peak friction angle of the soil
ϕ_{peak}	Peak friction angle of the sand
ϕ_{pile}	Pile diameter of scaled model
ϕ_{res}	Residual friction angle of the sand
ρ	Radius of the arc of circle of the beam segment
ρ_d	Dry density of the material
ρ_p	Pile material density
ρ_s	Sand density
σ_θ	Circumferential stress in the container wall
σ_h	Horizontal stress in the soil
σ_p	Stress in the pile
σ_s	Stress in the soil
σ_v	Vertical stress in the soil
σ'_v	Effective vertical stress in the soil
ε_p	Volumetric strain
ε_q	Deviatoric strain
ε_y	Axial strain of a pile segment

D	Pile diameter
D_R	Relative density of sand sample
d_X	Particle size for which X percent is smaller than that particle size
E	Elastic modulus
E_p	Elastic modulus of the pile
E_s	Elastic modulus of the sand
E_{kin}	Energy of impact
EI	Flexural stiffness of the pile
F_f	Frictional forces on the pile shaft
F_h	Force on the pile head
G_s	General shear modulus of the soil
$G_{s,e}$	Elastic shear modulus of the soil
H	Horizontal force applied on the pile
h^P	Height above mudline level at which the horizontal load is applied on the pile
H_{drop}	Drop height of ram in scaled model
h_{drop}	General drop height of the ram
$H_{sat,av}$	Average height of sand in the container
I	Second moment of area of the beam's cross section
I_p	Second moment of inertia of the pile
K_a	Active lateral earth pressure coefficient
L	Pile embedded depth
L_c	Critical embedded length (Poulos and Hull, 1989)
M	Bending moment
m_s	Mass of sand
m_r	Mass of ram in scaled model
N	Scaling reduction factor
n_X	Scale factor of the quantity X , equal to $\frac{X_{modtul}}{X_{protetyu}}$
$p_{sat,soil}$	Pressure exerted by saturated sand on the container wall
R_{pile}	Radius of the pile
t	Thickness of the container

t_f	Time for dissipation of pore pressure
t_i	Time between each impact (for impact driving)
t_p	Time for wave propagation in the pile
t_s	Time for shear wave propagation in the soil
$t_{cushion}$	Thickness of wooden cushion
u	Pore water pressure
U_c	Pile deflection resulting from rigid body movement
U_d	Pile deflection resulting from flexural deformation
V_c	Volume of a metal can
v_f	Velocity of the pore fluid
v_p	Wave velocity in the pile
v_r	Velocity of the ram
V_s	Volume of the sand in the container
v_s	Shear wave velocity in the soil
w	Total deflection of the pile
y	Height taken from the mudline
Z	Impedance
z	Height of soil above the considered depth

Chapter 1

Context

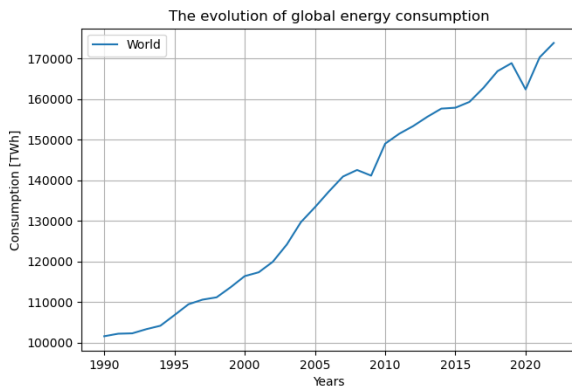
1.1 Energy and the Global Situation

Energy is the foundation of modern life, powering everything from homes and industries to transportation and communication systems. But what exactly is energy?

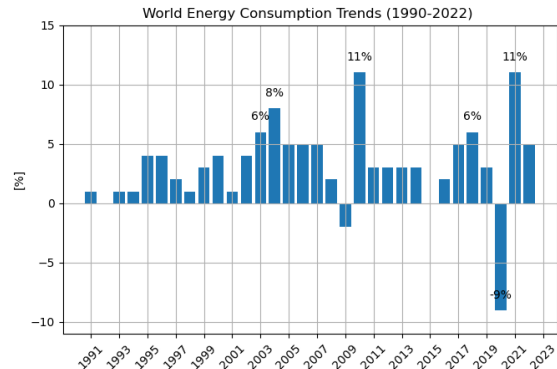
Energy is defined as the capacity to do work or create change. It exists in various forms mechanical, electrical, thermal, and chemical and can transform from one form to another. For instance, when fuel burns in an engine, chemical energy is converted into mechanical energy, enabling the vehicle to move. Similarly, when we switch on a light, electrical energy is converted into both light and heat.

In recent decades, global energy consumption has grown substantially. From 1990 to 2022, energy usage increased by an impressive 72,304 terawatt-hours (TWh), with nearly 28% of that growth occurring within just the last decade. By 2022, total energy consumption reached 173,880 TWh, as illustrated in (Figure : 1.1a).

The first chart demonstrates the steady increase in total global energy consumption over the years, while the second chart (Figure : 1.1b) provides insight into year-over-year percentage changes. It highlights significant spikes and dips, reflecting major global economic events like the 2008 financial crisis and the COVID-19 pandemic in 2020, along with other trends.



(a) Global energy consumption growth



(b) Energy consumption trends, 1990-2022

Figure 1.1: Comparison of global energy consumption [15]

The energy landscape in 2022 was marked by a diverse mix of energy sources. Fossil fuels, including oil, coal, and

gas, still dominated global energy production, accounting for a significant portion of the overall supply. However, the share of renewable like wind, solar which make electricity, and biomass has been steadily increasing, reflecting the global shift toward more sustainable energy solutions. Nuclear power continues to play a stable role, offering a reliable alternative to fossil fuels.

The left chart in Figure 1.2 illustrates the breakdown of energy sources in 2022. Oil remains the largest contributor at 30%, followed by coal (28%) and gas (23%). The right chart shows the regional distribution of energy consumption over time, highlighting the growth in Asia, which now accounts for the largest share, driven by rapid industrialization and population growth.

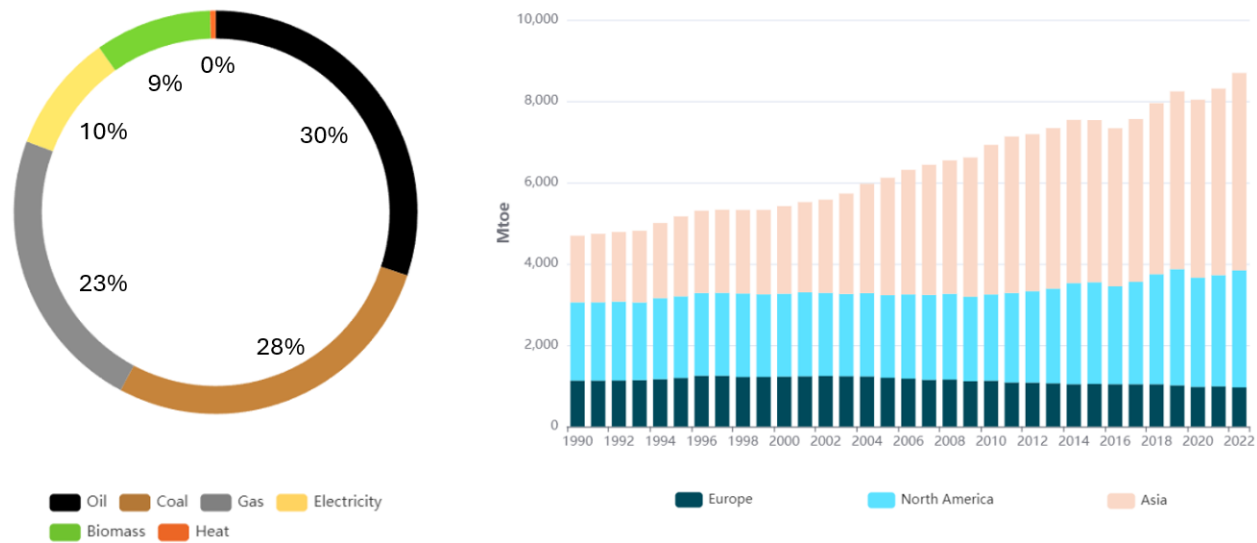


Figure 1.2: Breakdown by energy source in 2022 and trends from 1990-2022 (1 Mtoe = 11.63 TWh).

While both energy consumption and production continue to rise, it is crucial to consider how they align globally. This figure provides a snapshot of energy production and consumption by region and country in 2022.

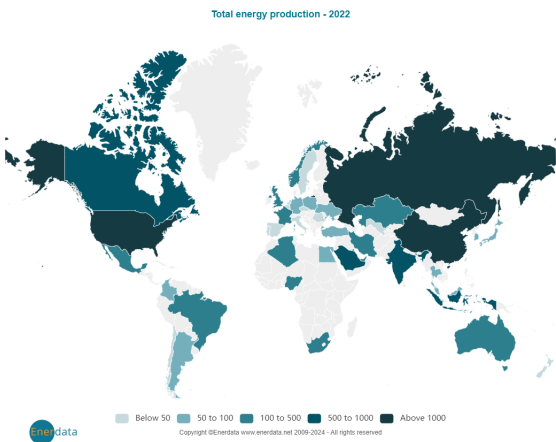


Figure 1.3: Energy production (2022)

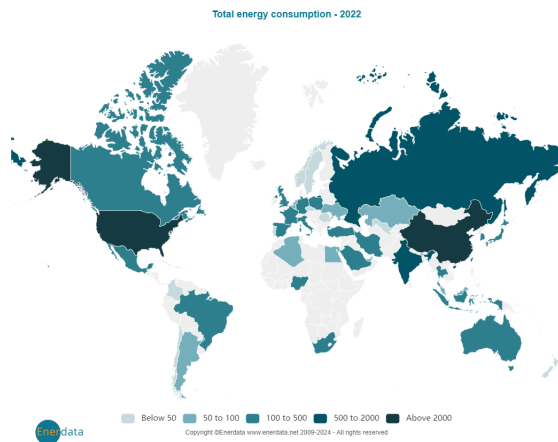


Figure 1.4: Energy consumption (2022)

Figure 1.5: Energy production and consumption by region in 2022 (in Mtoe, 1 Mtoe = 11.63 TWh) [15].

Country/Region	Production (TWh)	Consumption (TWh)
World	175,880	173,880
Asia	56,406	76,525
Europe	11,200	19,922
EU	6,408	15,270
China	36,193	44,206
United States	27,000	25,377
Russia	16,922	9,560
Saudi Arabia	8,141	2,954
India	7,594	11,688
Belgium	186	593

Table 1.1: Global energy production and consumption in 2022 [15]

The data reveals significant differences in energy production and consumption across regions. Some countries, like China and the United States, lead in both production and consumption due to their industrial size and population. In contrast, others, particularly in regions with smaller economies, show a disparity between their energy production and consumption. These differences highlight the varied levels of industrialization, economic activity, and energy policy priorities worldwide.

In global electricity generation, which accounts for around 10% of the total energy sources, we can typically categorized it into two main groups:

- **Non-renewable energy:** This category includes fossil fuels such as coal, oil, and natural gas, as well as nuclear power. Despite global efforts to transition to cleaner energy sources, non-renewables continue to dominate the energy landscape, providing the majority of global electricity.
- **Renewable energy:** This group consists of solar, wind, hydroelectric, biomass, and geothermal energy. Renewables have seen a significant increase in adoption, driven by the need to reduce greenhouse gas emissions and mitigate the effects of climate change.

By 2022, these electricity sources collectively contributed to a global energy production of 29,029 TWh [15]. However, the global energy transition remains uneven. In some areas, particularly in Europe and parts of Asia, renewables are being rapidly integrated into national grids, while in other regions, they still dependent on fossil fuels.

The left chart (in Figure : 1.6) highlights the speed of this transition, showing how Europe has outpaced Asia and North America in adopting renewable energy. This growth is largely driven by advances in wind and solar technologies, supported by favorable policies and substantial investments.

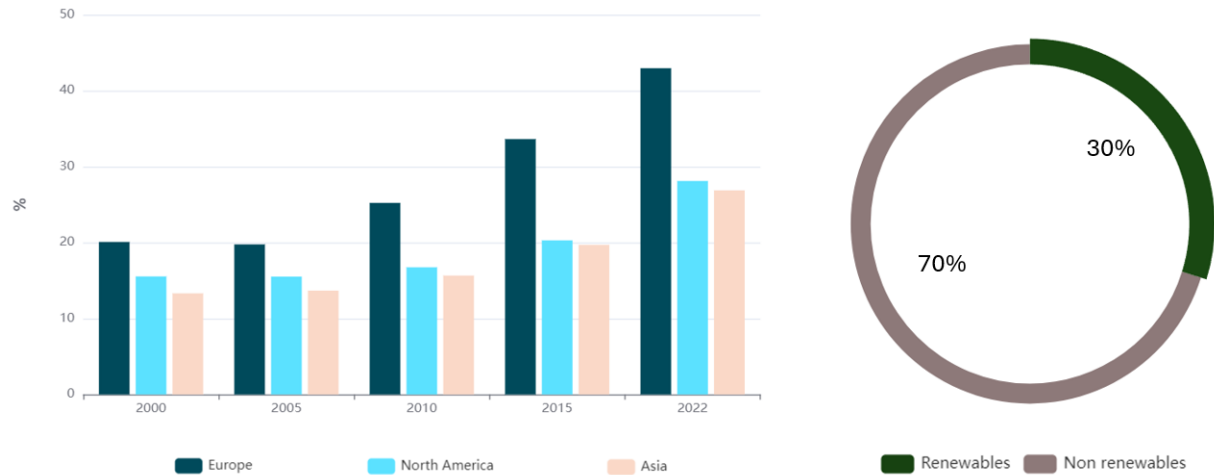


Figure 1.6: Energy production trends, 1990-2022 [15]

1.2 Europe objective

In 2021, energy consumption within the European Union (EU) experienced a significant 6% rise compared to the previous year, reflecting a strong rebound from the reduced activity levels during the COVID-19 pandemic [12]. This growth was primarily driven by increased demand across various sectors as economies recovered. Notably, a substantial portion of this consumption was attributed to energy transformation processes, underscoring the EU's ongoing focus on transitioning towards more sustainable energy sources. The increased share of energy transformation highlights the region's commitment to integrating renewable energy technologies and improving energy efficiency in alignment with long-term climate goals.

The regulation (EU) 2021/1119 of the European Parliament and of the Council, established within the framework of the European Green Deal, sets the objective of the Union achieving climate neutrality by 2050, with a reduction of at least 55% in net greenhouse gas emissions by 2030 compared to 1990 levels. This transition to climate neutrality requires a significant increase in the use of renewable energies, given their potential to reduce greenhouse gas emissions and address environmental challenges such as biodiversity loss and pollution [30].

Directive (EU) 2018/2001 establishes a binding overall target for the Union, aiming for the share of energy produced from renewable sources in the Union's gross final energy consumption to be at least 32% by 2030, with a recent proposal by the Commission to increase this share to 40% by 2030. Recent geopolitical events, with the COVID-19 pandemic, underscore the urgent need to accelerate the transition to renewable energies to ensure the Union's energy independence, reduce dependence on fossil fuels, and promote fair and affordable energy prices [30].

The REPowerEU plan, outlined in the Commission's communication of May 18, 2022, aims to make the Union independent of Russian fossil fuels before 2030 by accelerating the deployment of wind and solar energies and increasing renewable energy capacities. This plan proposes raising the Union's overall renewable energy target to 42.5% to accelerate the transition to renewable energies and gradually eliminate dependence on Russian fossil fuels [30].

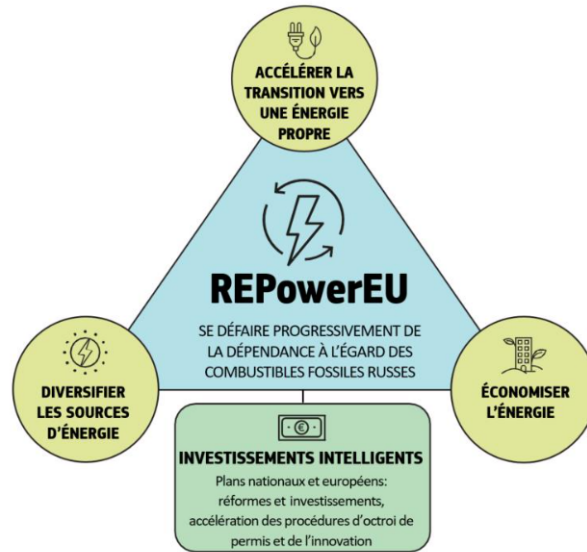


Figure 1.7: Plan REPowerEU [9]

1.3 Solar and Wind Energy: A Cross-Section of Europe

Both wind and solar are important in the sector's transition to renewable energy in Europe. Recent data shows that wind and solar are more efficient than fossil fuels when it comes to electricity generation [6]. In May 2023, wind and solar both accounted for almost a third of EU electricity, more than all fossil fuels combined. Solar alone supplied a record 14% of EU electricity in May 2023, an all-time high, while wind contributed 17% [14]. This growth in renewable energy is driven by factors such as new renewable sources, stronger performance from wind and falling demand for electricity.

The planned deployment of wind and solar is far from evenly spread. The leading countries (e.g. the Netherlands, Spain & Denmark) plan major transformations of their electricity systems by the end of this decade, replacing about half (or more) of their current electricity supply with wind and solar, while other countries (e.g. Czechia, Slovakia & Bulgaria) plan very limited wind and solar additions at all [52].

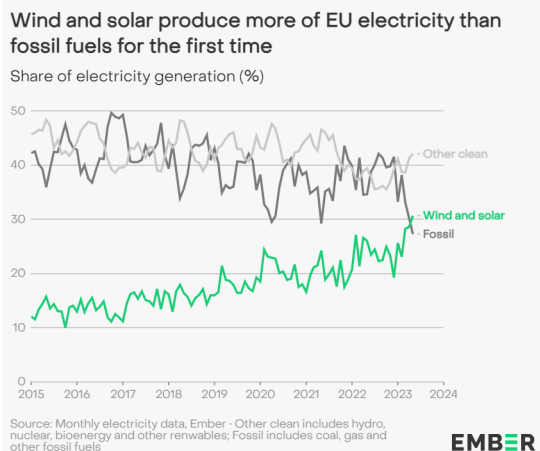


Figure 1.8: Monthly electricity generation in Europe [14]

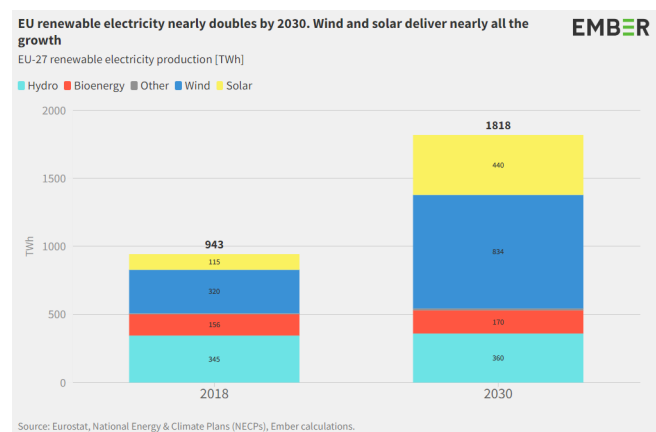


Figure 1.9: EU renewable electricity progression between 2018-2030 [13]

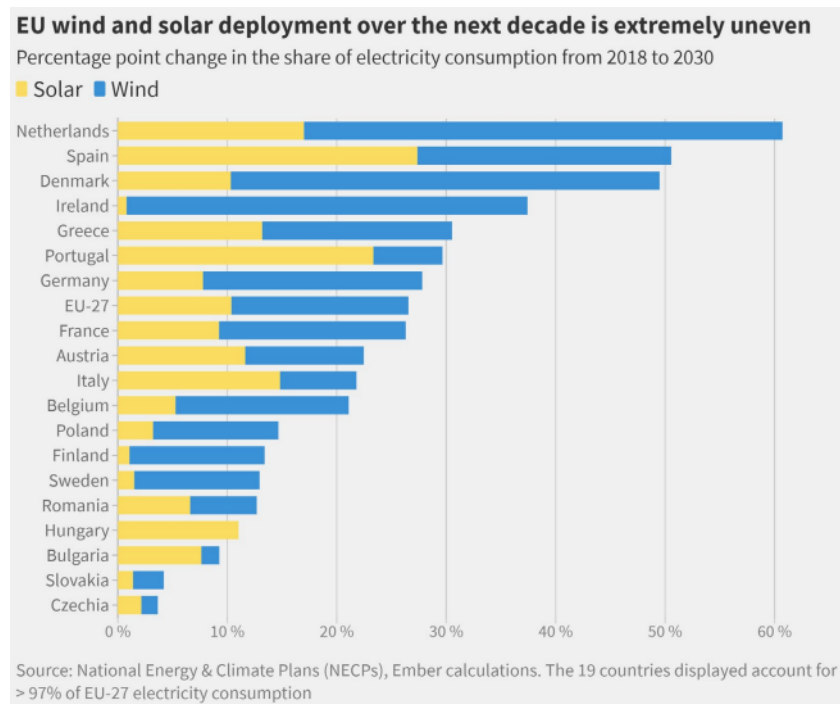


Figure 1.10: Percentage point change in the share of electricity consumption from 2018 to 2030 [52]

1.4 Exploring Onshore and Offshore Wind Energy Generation

Within the context of wind energy, which is central to this study, two main types of systems stand out: onshore and offshore wind turbines. Both technologies have distinct characteristics that impact their efficiency, cost, and associated technical and environmental challenges.

Onshore wind turbines are typically installed in rural or sparsely populated areas. They benefit from proximity to electricity transmission networks, reducing infrastructure costs and simplifying maintenance. However, their energy production is often constrained by less consistent and lower-intensity winds compared to those encountered offshore. The average capacity factor for onshore wind ranges between 25% and 40%, with some variations depending on location and turbine technology [39].

In contrast, offshore wind turbines are located at sea, often far from the coast, where winds are stronger and more consistent. This leads to a more stable and efficient energy output. Offshore wind farms, therefore, play a critical role in meeting the growing demand for green electricity. The capacity factor for offshore installations generally exceeds 50%, and advancements in technology are pushing this figure closer to 60% [50]. However, installing these structures requires substantial initial investments due to the need for specialized infrastructure and the challenges posed by extreme environmental conditions. Maintenance costs are also significantly higher at sea, compounded by the need for technologies that protect the foundations against erosion and waves.

The choice between onshore and offshore wind depends on a thorough analysis of local conditions, operational costs, and environmental impact, while aiming for optimal energy production in the context of the energy transition.

In 2022, Europe witnessed a surge in wind energy installations, with a total of **19.1 GW** added to its capacity. This included **16.7 GW** of onshore installations and **2.5 GW** offshore. Despite economic challenges, installations grew by **4%**, marking a record year. However, these fell short of targets, hindering Europe's climate goals.

To reach a **45%** renewable energy target by **2030**, Europe needs an average annual installation rate of **31 GW** for wind energy. Germany led onshore installations, while countries like Sweden, Finland, and France saw record additions. Yet, Poland’s restrictive regulations pose a threat to future projects. Offshore wind accounted for **13%** of installations, with the UK leading. However, challenges remain, notably in Poland. The **EU-27** represented **84%** of all installations, underscoring the region’s commitment to renewable energy[54].

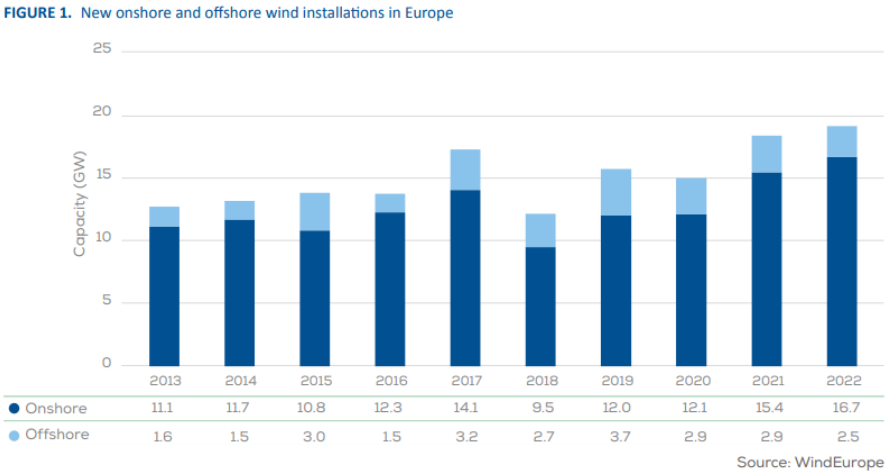


Figure 1.11: New onshore and offshore wind installations in Europe [54]

1.5 Future Prospects for Offshore Wind Turbines

As of 2023, Europe’s offshore wind capacity reached approximately 30 GW. However, to meet the ambitious renewable energy targets set for 2030, the European Union aims to scale this capacity up to 160 GW. This represents a significant leap, underscoring the need for accelerated deployment, substantial investments in infrastructure, and advancements in technology. The development of offshore grid connections, onshore grid reinforcements, and enhanced port facilities will be crucial in supporting the integration of this increased capacity into Europe’s electricity grid [5].

The chart below highlights the growth trajectory of wind power capacity in Europe, where a total of 272 GW is currently installed. Of this capacity, 87% (238 GW) is installed onshore, while 13% (34 GW) is offshore.

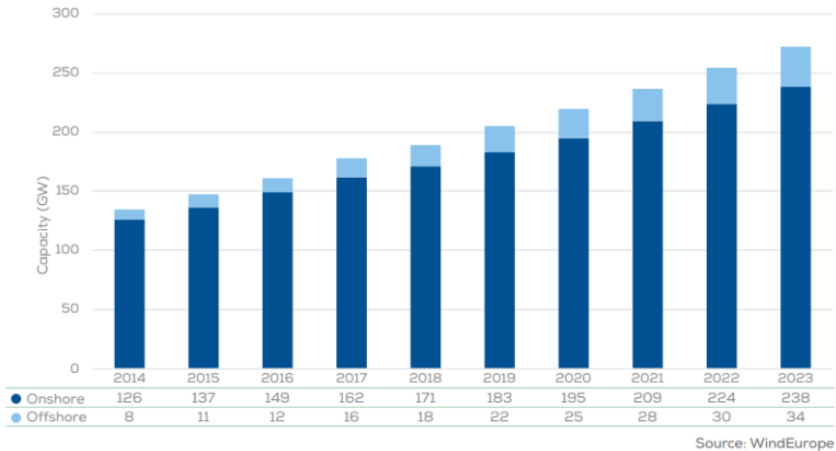


Figure 1.12: The growth of wind power capacity in Europe, 2014-2023 [54]

Chapter 2

Offshore wind turbines and their installation

2.1 Types of Offshore Wind Turbine Support Systems

Offshore wind turbines rely on two primary types of support structures: floating systems and grounded (fixed-bottom) systems. The choice between these systems depends on factors such as water depth, installation cost, and environmental conditions.

2.1.1 Floating Support Systems

Floating platforms are designed for deep waters typically greater than 60 meters. These structures are essential in areas where the seabed is too deep for conventional fixed-bottom systems, such as in Japan, the U.S. West Coast, and parts of Europe like Scotland. Common types include:

- **Spar Buoy:** Spar buoys are long cylindrical structures that provide stability by extending deep underwater. An example is the Hywind Scotland project, where each turbine is mounted on a spar buoy with a draft of over 100 meters. While effective in deep waters, spar buoys have high material and installation costs.
- **Semi-Submersible Platforms:** These platforms feature multiple columns connected by pontoons. They offer more flexibility in water depths ranging from 60 to 200 meters. The WindFloat Atlantic project in Portugal is an example, utilizing a semi-submersible platform to reduce installation complexity. Semi-submersibles are costlier to manufacture but offer easier installation than spar buoys.
- **Tension-Leg Platforms (TLP):** TLPs use vertical tendons anchored to the seabed. They are highly stable and suited for deep waters. However, their high installation cost and complex design make them less favorable in many commercial projects.

While floating platforms enable offshore wind farms in deep waters, their costs are still significantly higher than grounded systems. A floating offshore wind project typically costs between 4,000 € and 6,000 € per installed kilowatt, compared to 2,000 € to 3,500 € per kilowatt for fixed-bottom structures [24]. However, ongoing research and innovation are expected to lower these costs over the next decade.



Figure 2.1: Types of floating offshore wind turbine platforms [27].

2.1.2 Grounded (Fixed-Bottom) Support Systems

Grounded systems are currently the dominant technology for offshore wind farms, especially in European waters, where water depths typically range up to 60 meters. These systems are cost-effective and well-suited to the relatively shallow continental shelves found in the North Sea and Baltic Sea. Grounded support structures include:

1. **Monopiles:** Monopiles are the most common fixed-bottom foundation type, used in about 80% of offshore wind projects [17]. These cylindrical steel tubes, driven deep into the seabed, are relatively simple and quick to install. Their typical diameters range from 5-6m (regular), 6-8m (XL) and 8-13m (XXL) [40], with installation depths of up to 50 meters. Monopiles are generally less expensive than other support types, making them a preferred choice for shallow to medium depths.
2. **Gravity Base:** Gravity base structures are heavy concrete or steel platforms that rest on the seabed, relying on their weight for stability. They are less common due to the difficulty in transporting and installing such large structures. However, they offer excellent stability and are often used in regions with challenging seabed conditions.
3. **Jacket Structures:** These are steel lattice frameworks typically used in deeper waters (30 to 60 meters). While more complex and expensive than monopiles, jacket structures provide greater stability and are suitable for sites with higher wind speeds and larger turbines.

Grounded systems generally offer lower installation costs compared to floating systems. A typical fixed-bottom offshore wind project ranges from €2,000 to €3,500 per installed kilowatt, depending on water depth and foundation type. The larger scale and more mature technology of grounded systems also contribute to their cost-effectiveness.

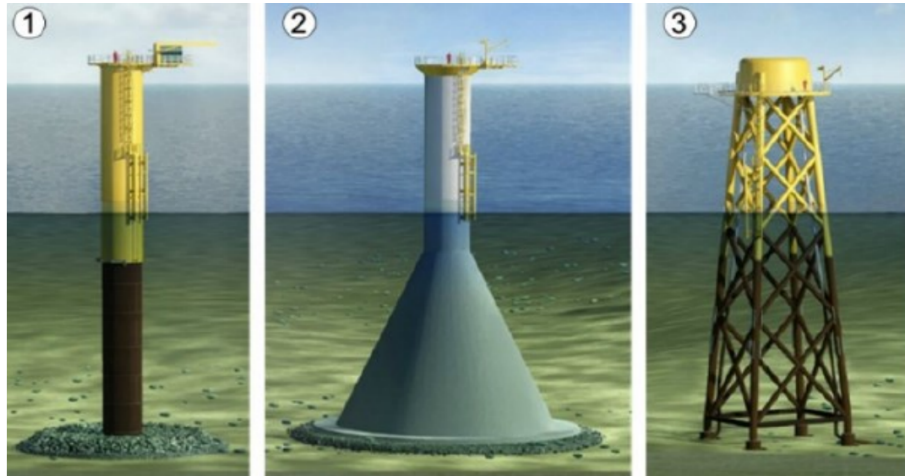


Figure 2.2: Types of grounded offshore wind turbine platforms.

2.2 Focus on Monopiles

2.2.1 Advantages of Monopiles

Monopiles offer several benefits:

- **Cost-Effective:** Lower material and manufacturing costs compared to other structures.
- **Simple Installation:** Fewer installation steps make it suitable for mass deployment.
- **Scalability:** Effective in water depths up to 50 meters [26].

2.2.2 Relevance to EU Projects

The European Union (EU) has heavily invested in monopile-based offshore wind farms as part of its renewable energy targets. The EU's focus is on scaling up capacity while ensuring recyclability and reducing environmental impacts [8].

2.2.3 Monopile Dimensions and Recyclability

Modern monopiles can reach up to 10 meters in diameter and 100 meters in length, with weights exceeding 1,500 tons. Increasing attention is being given to the recyclability of steel used in monopiles, aligning with circular economy principles [16].

2.3 Additional Considerations for Monopiles

To enhance performance, monopiles are often optimized through advanced modeling techniques, which consider factors like scour protection and dynamic loading [22]. Furthermore, research is ongoing to improve their environmental impact by optimizing the material usage and exploring more sustainable decommissioning options [23].



Figure 2.3: Monopiles in at Taiwan's first commercial offshore wind farm

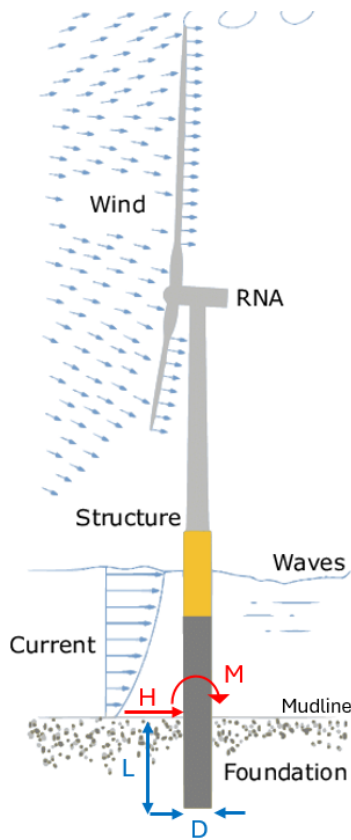


Figure 2.4: Environmental actions on a monopile-based offshore wind turbine.

Loading Types and Key Parameters:

The performance and stability of offshore monopiles are primarily determined by their response to lateral and vertical loads. These include:

- **Wind Load :** Exerted on the rotor and nacelle assembly (RNA), influencing the lateral forces acting at the pile's top.
- **Wave and Current Loads:** Contribute significantly to the bending moment along the monopile, especially near the mudline, it is also considered cyclic.

Key Design Parameters:

Two critical parameters often discussed in research are the L/D ratio and the wall thickness:

- **L/D Ratio:** The embedded depth (L) on the diameter (D) ratio is crucial in determining the pile's lateral resistance. Studies suggest that an optimal range of L/D between 4 and 8 provides a balance between stability and material efficiency [34].
- **Wall Thickness:** The monopile's thickness plays a key role in its stiffness and resistance to buckling. Industry standards often recommend a thickness that is a function of the pile diameter (D) and the expected loads. In offshore applications, monopiles with diameters ranging from 4 to 10 meters generally feature wall thicknesses between 40 and 150 mm.[45].

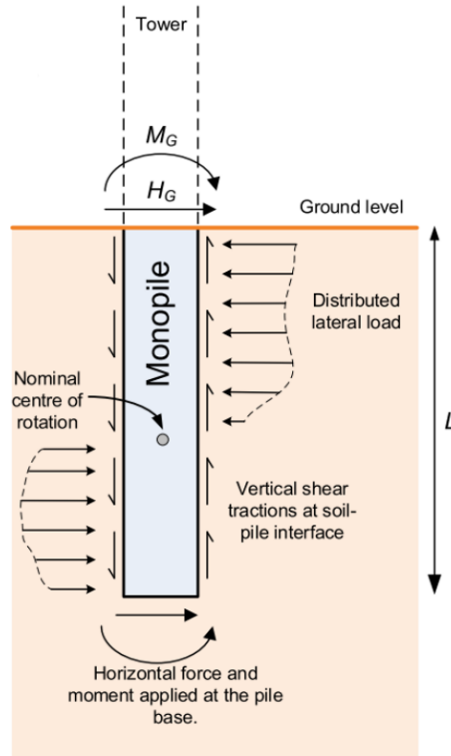


Figure 2.5: schematic of the soil reaction components acting on a rigid monopile

After a monopile is installed, external forces from waves, currents, and wind act on it, creating lateral loads that need to be resisted. The resistance is mainly governed by four critical components: distributed lateral soil pressure, vertical shear traction at the soil-pile interface, and horizontal forces and moments at the monopile base (Figure 2.5). While flexible monopiles have been widely studied, the design methodologies for rigid monopiles remain less developed. These rigid monopiles experience distinct soil-structure interactions that are often overlooked in traditional designs.

(Byrne et al. 2017 [7]) emphasized that in rigid monopile designs, certain soil reactions that are typically neglected in flexible monopile analysis become crucial. Ignoring these factors can lead to significant inaccuracies in predicting monopile behavior, especially under complex loading conditions. This research gap presents challenges in accurately forecasting the performance of monopiles under lateral loading.

In this thesis, special emphasis is placed on the horizontal forces and bending moments along the pile, which are essential parameters for evaluating monopile stability.

In this thesis, particular attention will be given to the horizontal forces and bending moments at the mudline, which are key parameters for evaluating monopile behavior.

Building on previous work by Maxence Deckers and Cédric Defer [37], the complexity of load combinations acting on monopiles under cyclic conditions is underscored. Offshore monopiles are subject to dynamic forces from wind, waves, and currents, which apply loads in varying directions. These forces result in a combination of bending moments, lateral forces, and rotational effects that challenge conventional design approaches.

The cyclic nature of these loads, characterized by changes in amplitude, direction, and frequency over time, adds further complexity. The resulting non-linear response of the monopile foundation under such conditions makes it difficult to accurately predict behavior. Understanding the influence of these loads, particularly in terms of bending moments and lateral forces, is essential for optimizing monopile design with respect to both stability and material efficiency. This thesis will simplify by focusing on the application of a monotonic load at a specific

height, representative of wind and wave-induced forces, as a controlled basis for analysis.

2.4 Installation process

The installation of monopiles for offshore wind turbines can be accomplished using various methods, with impact hammering and vibratory driving. Each method has distinct advantages and limitations that influence their applicability and performance in different scenarios.

2.4.1 Impact hammering

Impact hammering is the most widely adopted method for driving monopiles into the seabed. It involves a hydraulic hammer delivering a series of high-energy blows at the pile head, pushing it progressively deeper into the seabed. This method is favored for its effectiveness in installing monopiles across various soil conditions and water depths, making it suitable for large-scale offshore wind projects. The efficiency and speed of impact hammering are well-established, which is crucial for minimizing installation time, particularly in challenging offshore environments.



Figure 2.6: IQIP's Hydraulic Impact Hammer

2.4.1.1 Limitations and Environmental Concerns and Installation Challenges

The use of impact hammering in the installation of large monopiles remains a technically reliable approach, yet it brings about significant environmental and operational challenges. A primary concern is the underwater noise pollution generated during pile driving, which has been shown to cause substantial disruptions to marine ecosystems. High decibel levels can lead to serious consequences for marine life, particularly for species such as marine mammals and fish. Research indicates that exposure to intense underwater noise can result in hearing impairments, as well as physical injuries including hemorrhages and ruptured swim bladders, necessitating strict regulatory measures (by MDPI [41] and from SLR Consulting [10]). In response, noise mitigation strategies like bubble curtains and acoustic barriers have become essential to minimizing these harmful impacts on marine environments.

To address these concerns, ongoing research is focused on innovations aimed at reducing noise at the source. For example, recent developments in hammer design seek to control the energy transfer process, resulting in more

moderated impact forces and consequently lower noise levels. Despite the promise of these emerging technologies, they are still in experimental stages and have yet to see widespread application (See Appendix B).

The installation of increasingly large monopiles necessitates the use of more powerful hammers, which amplifies both the complexity of the operation and its environmental impacts. The intensified noise pollution not only exacerbates the aforementioned risks to marine life but also presents engineering challenges during installation. The mechanical stress imposed by high-impact hammering can compromise the structural integrity of the monopile, particularly at its upper sections, which are especially vulnerable to damage.

Moreover, the new generation of the wind turbine, has a capacity of 10 MW which require bigger foundation and so bigger ram adapted to it which will end up by an increased scale and force for such installations lead to longer project timelines and higher costs due to the added labor and time-intensive processes. The evolving demands of these installations emphasize the urgent need for advancements in both hammer technology and monopile design, aiming to enhance structural durability while also streamlining the installation process. Thus, balancing environmental mitigation and engineering efficiency remains a key challenge in the development of offshore wind infrastructure.

2.4.1.2 Dimensions and Numbers

The impact hammering method for monopile installation involves precise engineering parameters that ensure optimal pile driving efficiency. Among these parameters, the mass of the ram, blow frequency, energy per blow, and hammer dimensions play crucial roles.

The ram mass typically ranges between 100 to 320 tons, depending on the size of the monopile being driven. Larger monopiles, often exceeding diameters of 8 meters, require heavier rams to generate the necessary impact force [42].

Blow frequencies generally vary from 20 to 30 blows per minute (bpm), with modern hammers capable of delivering from 1,000 to 6,000 kJ per blow. The height of the ram drop can range from 1 to 2 meters, providing control over the energy delivered with each impact [42].

The installation process itself typically takes 3 to 5 days for a monopile, depending on the seabed conditions, pile diameter, and penetration depth. During this time, adjustments are made in real-time to the blow energy and frequency to optimize penetration rates [35].

These dimensions and numbers are essential for designing efficient monopile installation strategies and understanding the complexities of the offshore environment.

In the figure below we have a representation of the setup that will represent the impact driving process and some fixed and changing parameters.



Figure 2.8: Vibro-driving system [46]

2.4.2.2 Technical Parameters

The most commonly used vibration frequency for offshore applications is around 23 Hz.

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

Figure 2.9: Vibrotor parameters [21]

2.4.2.3 Limitations and Challenges

- **Uncertainty in Bearing Capacity:** Unlike impact hammering, vibro-driving does not provide direct measurements of bearing capacity during installation, leading to potential uncertainties in pile performance. This issue often requires supplementary testing, especially in critical projects as mentioned by [53].
- **Certification and Standards:** Vibro-driving is not fully accepted by all certification bodies, and in some cases, piles driven with this method must be impact-driven for the final stages to ensure compliance with standards such as those from DNV or API as it was highlighted by (Garance and Nathan [19]).
- **Soil Conditions:** While vibro-driving is effective in soft and granular soils, its performance diminishes in cohesive soils like clays, where cyclic degradation of soil strength is less pronounced.
- **Operational Complexity:** Managing multiple vibrators, especially in large-scale offshore operations, can be challenging, with issues arising in aligning the clamping mechanisms and controlling the vibrational forces.

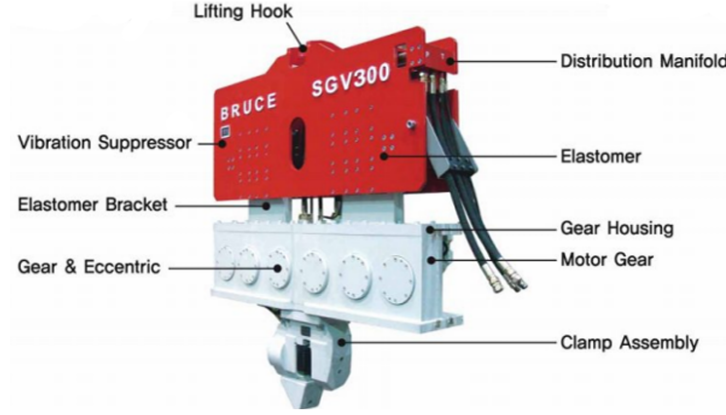


Figure 2.10: Detailed scheme of vibrator [19]

2.4.2.4 Related formulas

2.4.2.4.1 Vertical Force Generated by Vibratory Action The vertical force applied by the vibrator can be expressed as:

$$F_v(t) = m_e \omega^2 \sin(\omega t) = F_e \sin(\omega t)$$

Explanation:

- $F_v(t)$: The sinusoidal vertical force generated by the vibrator at time t .
- m_e : Eccentric moment of the vibrator [kg·m].
- ω : Angular frequency of the vibrator [rad/s] = $2\pi f$.
- F_e : Maximum centrifugal force of the vibrator [N].

This equation models the periodic vertical force exerted by rotating eccentric masses, which is crucial for analyzing the vibratory motion during pile driving.

2.4.2.4.2 The Total Load Applied During Vibratory Driving The total load acting on the pile during vibratory driving is given by:

$$F_s = (M_{eb} + M_{ib} + M_{clamp}) \cdot g - T$$

Explanation:

- F_s : Net stationary load on the pile [N].
- M_{eb} : Mass of the exciter block [kg]. The exciter block contains the rotating eccentric masses responsible for generating the vibratory motion and the vertical force.
- M_{ib} : Mass of the isolator block [kg]. The isolator block is designed to dampen vibrations and stabilize the system.
- M_{clamp} : Mass of the clamping device [kg], which secures the pile to the vibrator.
- T : Suspension force [N].

This formula accounts for the weight of the vibrating system and the clamping device, adjusted for the suspension force, which determines the net stationary load.

2.4.2.4.3 The Free Hanging Double Amplitude S_{sp} The double amplitude for a free-hanging vibrator is expressed as:

$$S_{sp} = 2S_0 = \frac{2m_e}{M_{eb} + M_{clamp}}$$

Explanation:

- S_{sp} : Specified maximum amplitude [mm].
- S_0 : Actual single amplitude [mm].
- M_{eb} : Mass of the exciter block [kg].
- M_{clamp} : Mass of the clamping device [kg].
- m_e : Eccentric moment of the vibrator [kg·m].

This formula (by Wilson, 2018 [53]) indicates that the double amplitude, often specified in vibrator designs, depends on the effective dynamic mass. It highlights how the system's mass affects the movement amplitude during operation.

2.4.2.4.4 Static Surcharge M_s The static surcharge load applied by the vibrator can be modeled as:

$$M_s = \frac{(M_s + M_{vib} + M_p)g \cdot T}{a \cdot R \cdot T_c}$$

Explanation:

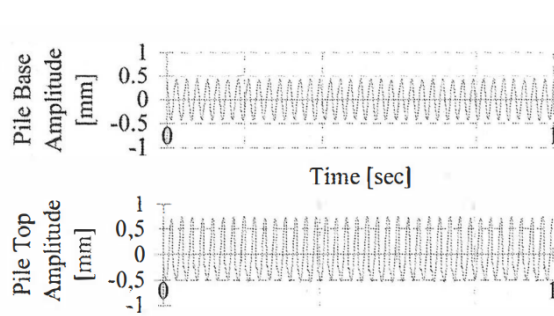
- M_s : Static surcharge mass [kg].
- M_p : Pile mass [kg].
- T_c : Contact time between pile toe and soil within a cycle [s].
- a : Acceleration ratio [/>].
- R : Soil resistance at the pile toe [N].

This formula is used for momentum conservation analysis during pile driving, considering the mass of the system and soil resistance.

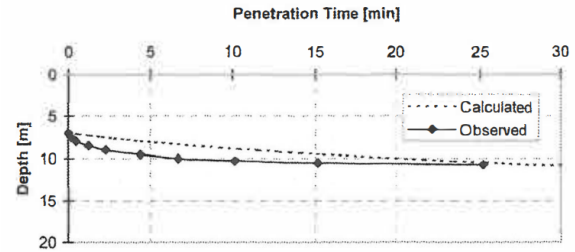
2.4.2.5 Experimental in real-life

The Kortrijk site provided significant insights into the behavior of piles during vibratory driving. The experiment involved the installation of a tubular steel pile with a diameter of 1 m and a length of 20.6 m. The pile was driven using a PTC 30HFV vibratory hammer. The site was characterized by very soft river deposits in the upper layers, followed by a stiff tertiary clay layer below, where the pile experienced refusal at a depth of 11 meters instead of the expected 20 meters. This discrepancy was attributed to a combination of factors, including the vibration amplitude, energy transfer inefficiencies, and soil conditions.

The first figure shows the recorded vibration amplitude upon refusal, highlighting the reduced amplitude at both the pile head and toe, which contributed to the unexpected refusal, it also show the frequency which is 38 Hz. The second figure compares the predicted and observed penetration logs, demonstrating how applying the actual vibratory amplitude improved correlation between calculated and observed penetration times



(a) Record from the vibration amplitude upon refusal



(b) Predicted and Observed penetration log at Kortrijk site compared with predicted log using actual vibratory amplitude [53]

2.5 Objectives of the Study

The primary goal of this thesis is to understand and compare the mechanical behavior of offshore wind turbine monopiles installed using two methods: impact hammering and vibratory driving. The research focuses on how different installation techniques influence the soil-structure interaction, especially under varying conditions such as changes in load, sand density, and pile dimensions.

2.5.1 Key Research Steps

1. **Development of an Advanced Sand Pluviation System:** A critical first step is the design and implementation of a sand pluviation system, capable of depositing 1.1 m^3 of sand uniformly within the container. This system must be stable, able to withstand the considerable volume of sand, and equipped with interchangeable plates featuring different hole sizes. By varying the hole size and spacing, the sand density can be controlled precisely, simulating realistic soil conditions. This system ensures consistent sand deposition, allowing for repeatable, quicker and reliable experimental results, particularly in tests involving impact hammering and vibratory driving.
2. **Impact Hammering Tests:** The objective here is to optimize the existing impact hammering setup to better reflect real-world conditions and to repeat some interesting experiments that were done by Deckers Maxence and Defer Cédric. The study will involve varying key parameters such as:
 - L/D ratio
 - The mass of the ram,
 - The drop height,
 - Sand conditions (dry vs. saturated).

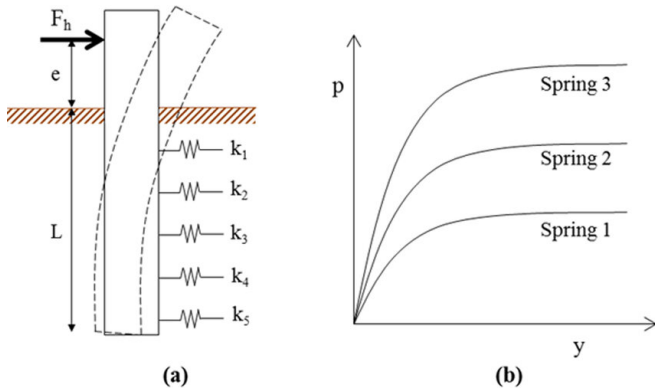
These tests will explore the relationship between these factors and the mechanical behavior of the monopile, such as lateral displacement, bending moments, and soil resistance.

3. **Vibratory Driving Setup Development:** Working on Luc Simonin's device, a robust vibro-driving setup will be developed with his device to investigate the impact of varying parameters, including:

- **Vibration frequency** [10 - 60] krpm
- **Eccentric mass** [4 ; 8]mm.g
- **L/D ratio**
- **Sand conditions** (dry vs. saturated)

The study will assess how these variables affect monopile penetration, soil displacement, and long-term stability in both dry and saturated sand conditions. Vibro-driving is increasingly popular due to its efficiency and reduced environmental impact, but understanding its limitations and optimizing its application is crucial for advancing offshore wind installations.

4. **Lateral Loading Tests and Data Acquisition:** Both installation methods (impact hammering and vibro-driving) will undergo lateral load tests simulating wind and wave forces. High-resolution strain measurements will be captured using an advanced optical fiber system along the length of the pile. These measurements are critical for determining bending moments and lateral deflections.
5. **Development of a Python code for p-y Curve Analysis:** A Python code will be developed to generate *p-y curves*, a fundamental tool in offshore geotechnical engineering for modeling the lateral load-displacement behavior of piles (as in Figure 2.12). These curves will be derived from experimental data to provide a comparative analysis of the lateral performance of monopiles installed via impact hammering and vibro-driving. The findings will highlight differences in soil reaction and pile deflection under cyclic loads, offering practical insights for optimizing installation techniques in offshore wind turbine projects.



This study aims to contribute significantly to the understanding of offshore monopile installation methods, focusing on the interplay between installation techniques and soil behavior, which is critical for designing more efficient, durable, and sustainable offshore foundations.

Figure 2.12: (a) Schematic Illustration of Applied Loads on an Offshore Wind Turbine; (b) Non-Linear p-y Curve Spring Foundation

Chapter 3

Methodology

3.1 Established Methods and Theoretical Concepts

In this section, i will explain the methods that were used from the previous experiments and research of Cedric and Maxence, it will be divided into theoretical research and experimental apparatus.

3.1.1 Scaling laws

The monopiles, crucial components in offshore wind farms, require meticulous design. While large-scale tests provide valuable insights, they are often resource-intensive and logistically complex. As an alternative, small-scale laboratory tests offer a cost-effective approach.

However, to address this issue, we relied on scaling laws found in the literature [55], validated by renowned researchers. These laws, mathematical relationships, describe how physical properties can change as materials, applied forces, and dimensions vary.

In examining the behavior of monopiles, we come across three essential types of similarity:

- **Geometric** : which ensures the corresponding physical dimensions of the system.
- **Kinematic** : which relates to the movement of piles, sand, and fluid.
- **Dynamic** : which ensures the appropriate distribution of forces between the prototype and the real scale.

When using scaling laws, it is crucial to consider scale effects and decide which laws to follow based on the test's objectives. Practical limitations in the lab must also be taken into account. Consequently, small-scale testing, while valuable, cannot fully replace large-scale testing. Laboratory constraints can reduce the representativeness of the results, highlighting the inherent limitations of small-scale testing and scaling laws.

The upcoming sections will show the developed rules for how various aspects of the tests setup change in relation to each other. It's important to remember that these rules are written and applied by (Deckers Maxence and Defer Cédric [37]) on this context and for particular conditions of the research, so as they mentioned they might not work the same way in other situations.

3.1.1.1 Length, area and volume

The scaling laws for satisfying the geometric similarity which means lengths and displacements are done by a reducing factor $N = 100$ as the piles are 6cm and 11cm of diameter:

$$n_L = \frac{(Length)_{\text{model}}}{(Length)_{\text{prototype}}} = \frac{1}{N} \quad (3.1)$$

$$n_A = \frac{(Area)_{\text{model}}}{(Area)_{\text{prototype}}} = \frac{1}{N^2} \quad (3.2)$$

$$n_V = \frac{(Volume)_{\text{model}}}{(Volume)_{\text{prototype}}} = \frac{1}{N^3} \quad (3.3)$$

These relations could be observed in Figure 3.1

3.1.1.2 Density

We want to keep the density of the sand and the pile equal to 1 knowing that theoretically it have also to be changed, so the relations are :

$$n_{\rho_s} = \frac{(Density_{sand})_{\text{model}}}{(Density_{sand})_{\text{prototype}}} = 1 \quad (3.4)$$

$$n_{\rho_p} = \frac{(Density_{pile})_{\text{model}}}{(Density_{pile})_{\text{prototype}}} = 1 \quad (3.5)$$

3.1.1.3 Mass

Since the density of the material remains constant but the volume is scaled by a factor of N^3 , the mass will also scale by the same factor, as illustrated in Figure 3.1:

$$n_m = n_{\rho} \cdot n_V = \frac{1}{N^3} \quad (3.6)$$

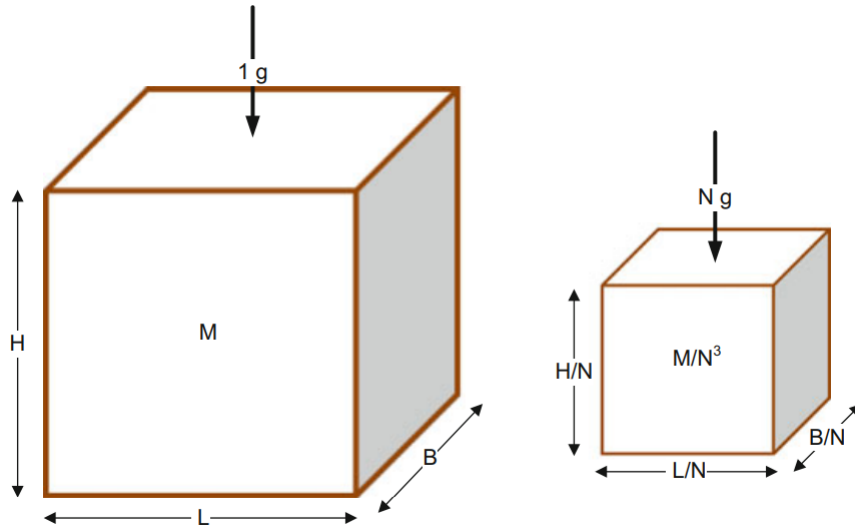


Figure 3.1: The scaling principal of mass by (Madabhushi, 2014 [36])

3.1.1.4 Viscosity of the pore fluid

In this thesis, water was primarily used to saturate the soil in several experiments, with the viscosity of the pore fluid (μ) considered as a constant **scaling factor of 1**. However, in geotechnical modeling, especially when studying dynamic loading conditions, the choice of pore fluid can be critical. Higher viscosity fluids are often used to better simulate the behavior of fluids in large-scale scenarios [49].

This is because, in such tests, it's common to keep the grain-size distribution unchanged, ensuring that the permeability remains consistent between the model and the full-scale prototype [32].

In practical terms, this means that in both monotonic and cyclic loading tests, especially those conducted using centrifuges, researchers often replace water with more viscous alternatives. These include silicone oils, water-glycerol mixtures, or hydroxypropyl methylcellulose (HPMC) in water. These substitutions help ensure that the fluid behavior in the model accurately reflects the conditions expected in real-world applications .

3.1.1.5 Acceleration

As the research focuses on one-gravity (1g) reduced model tests in the laboratory the scale factor for gravitational acceleration is 1, however it exist other laboratory-scale testing at elevated stress-level using a centrifuge as shown in Figure 3.2 which involves subjecting a geotechnical model to accelerations (ng) to simulate effective unit weights (Richards 2019)



Figure 3.2: 5 m radius beam centrifuge at the University of Western Australia.

However in our case we will still on the model (1g) and the relationship is as follows :

$$n_a = \frac{a_{model}}{a_{prototype}} = 1 \quad (3.7)$$

3.1.1.6 Stresses and forces

3.1.1.6.1 Stress in the soil, σ_s

- As the vertical stress in the soil σ_v is lineary related to the depth z with the relation : $\sigma_v = \rho \cdot g \cdot z$ the scaling law will be the same of the length
- And as the horizontal stress is expressed by the vertical stress, both of them will have the same scaling.

So in total we can express the stress in soil by a unique notation σ_s and where the corresponding scaling law is :

$$n_{\sigma_s} = n_\rho \cdot n_g \cdot n_L = \frac{1}{N} \quad (3.8)$$

3.1.1.6.2 Frictional forces on the pile shaft, F_h As the friction forces is $F \propto \sigma_s \cdot A_{shaft}$ then the scaling law follows:

$$n_{F_f} = n_A \cdot n_{\sigma_s} = \frac{1}{N^3} \quad (3.9)$$

3.1.1.6.3 Force on the pile head, F_h This force is related to the ram masses and falling heights but as a force is an acceleration times a mass the scaling law is :

$$n_{F_h} = n_a \cdot n_m = \frac{1}{N^3} \quad (3.10)$$

in reality this hitting force as described by (Deeks 1992 [11]) is expressed as : $F_h = Z \cdot v_r$ where Z the impedance is constant for a chosen material and v_r is the velocity of the ram just before the impact.

This impedance is expressed by $Z = \frac{E_p \cdot A_p}{v_p}$ where E_p represents the elastic modulus of the pile and, A_p stands for the cross sectional area of the pile, and v_p indicates the axial wave velocity within the pile, consequently, the impedance Z and the ram velocity scales as follows :

$$n_Z = \frac{n_{E_p} \cdot n_{A_p}}{n_{v_p}} = \frac{1}{N^2} \quad (3.11)$$

$$n_{v_r} = \frac{n_{F_h}}{n_Z} = \frac{\frac{1}{N^3}}{\frac{1}{N^2}} = \frac{1}{N} \quad (3.12)$$

And with principle of energy conservation $mgH_{drop} = \frac{1}{2}mv^2$ the H_{drop} represents the drop height of the ram, scales as follows :

$$n_{H_{drop}} = \frac{n_{v_r}^2}{n_g} = \frac{1}{N^2} \quad (3.13)$$

We observe that this scaling law is not the same as the scaling of geometrical lengths as discussed in Section [3.1.1.1](#)

3.1.1.6.4 Force on the cushion Previously, we discussed the force on the pile head, which is directly influenced by the impact of the ram and the resulting stress distribution along the pile. However, the dynamics of this process are further complicated by the inclusion of a cushion between the hammer and the pile head.

"The cushion plays a crucial role in absorbing and distributing the impact force. It significantly affects the stress distribution along the pile and the overall efficiency of the driving process. When the hammer strikes the cushion, the force is transmitted to both the anvil and the pile head. The material properties of the cushion, such as stiffness and damping, dictate how much of the impact force is absorbed versus how much is transmitted to the pile. A stiffer cushion will transmit more force directly to the pile, potentially increasing stress concentrations at the pile head. Conversely, a softer cushion absorbs more energy, reducing the peak force but increasing the duration of the impact.

This cushioning effect alters the dynamics of the system. As discussed by (Deeks 1992) [11], the inclusion of a cushion can improve the accuracy of the model by acting like a linear spring. However, this also introduces damping effects, making the motion equation more complex. Therefore, while the model with a cushion (similar to model d in Figure 3.3) provides a more realistic representation, we simplify our calculations by using a model without damping (similar to model c).”¹

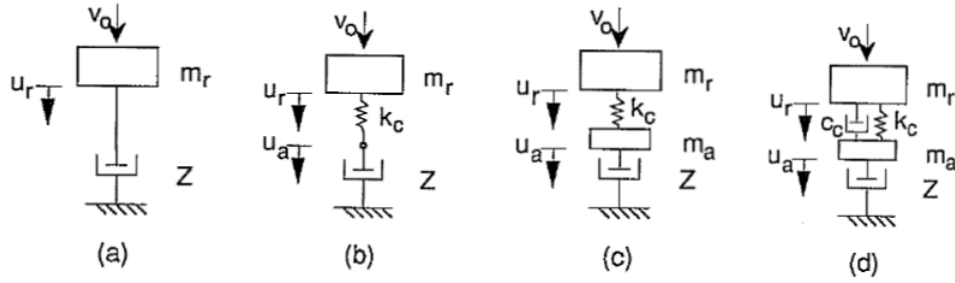


Figure 3.3: Analytical pile hammer models

To further enhance stability, a small metallic cable was added to keep the anvil securely on the pile head, preventing it from bouncing during the impact (see Figure 3.4). The cushion also helps in controlling the ram’s movement, reducing bounce and maintaining better control over both energy absorption and transmission. However, these aspects, while important, were not the primary focus of this thesis and therefore were not studied in detail.

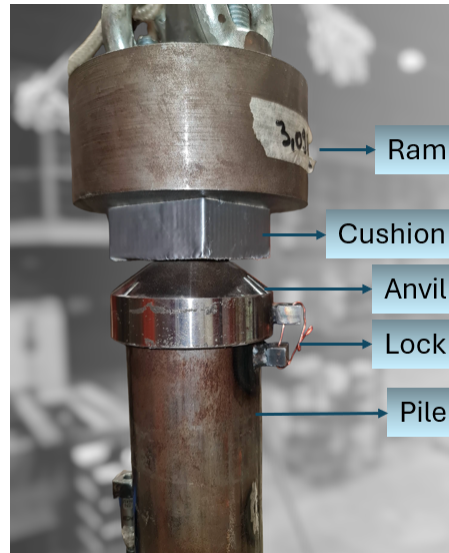


Figure 3.4: Metallic cable used to lock the anvil with the pile

3.1.1.6.5 Stress in the pile, σ_p The stress measured in the pile (σ_p), due to the impact of the hammer, for example, can be scaled using the definition of stress induced by a force in a solid material. With the scale factors for the force of the hammer on the pile head F_h and for an area already known, the following expression can be obtained:

¹Text generated by ChatGPT, June 14, 2024, OpenAI, <https://chat.openai.com> with the source : (Deeks 1992) [11]

$$\sigma_p = \frac{F_h}{A_{\text{pile section}}} \implies n_{\sigma_p} = \frac{\frac{1}{N^3}}{\frac{1}{N^2}} = \frac{1}{N} \quad (3.14)$$

Thus, the stress in the pile (σ_p) is scaled by a factor $1/N$, fortunately ensuring a sought coherence with the stress in the soil (σ_s).

3.1.1.7 Stiffness

3.1.1.7.1 Elastic shear modulus of the soil, $G_{s,e}$ The soil's elastic shear modulus, $G_{s,e}$, typically depends on the effective stress level σ' or the mean effective stress level p' , following the relationship $G_{s,e} \propto p'^m$. For sand, m is often around 0.5 (Iwasaki 1978 [28], Santos 2000 [44], Wood 2003 [55]). However, higher values such as $m = 0.65$ for Hostun Sand are found in the literature (Adamidis 2022 [3]). Considering that stress scales as $1/N$, the elastic stiffness scales as:

$$n_{G_{s,e}} = \left(\frac{1}{N}\right)^m \quad (3.15)$$

3.1.1.7.2 General shear modulus of the soil, G_s The general shear modulus, G_s , taking plasticity into account, can reasonably be approximated as being linearly proportional to the effective stress, p' . Therefore, $G_s \propto p'$, instead of p'^m in the plastic case. Hence, the scaling is identical to that described in Section 3.1.1.6.5.

$$n_{G_s} = \frac{1}{N} \quad (3.16)$$

3.1.1.7.3 Elastic modulus of the pile, E_p As the model pile was chosen to be of the same material as the prototype pile (steel in this case), the stiffness of the material scales as 1, so:

$$n_{E_p} = 1 \quad (3.17)$$

3.1.1.7.4 Flexural stiffness of the pile The consideration of flexural stiffness EI of the pile, distinct from material stiffness, holds particular significance as it becomes relevant during lateral loading. This flexural stiffness corresponds to the elastic modulus of the pile, which scales as 1 as previously mentioned, multiplied by the inertia or "second moment of area", I . The latter is directly proportional to the length raised to the power of four (Madabhushi, 2014), resulting in the following scaling factor for the flexural stiffness:

$$n_{EI} = \frac{(EI)_{\text{model}}}{(EI)_{\text{prototype}}} = \frac{1}{N^4} \quad (3.18)$$

However, in the context of this experimental research, examining the relative stiffness of the pile, in comparison to the soil, holds greater relevance. This relative stiffness plays a crucial role in achieving comparable behavior between the model and the prototype. The model monopiles should have an intermediate and rigid behavior to be representative of offshore monopiles.

3.1.1.8 Time

In the reduced-model experiment, various times come into play, including the time of the fall of the ram on the pile head, the time for stress waves to travel in the pile or in the soil, and the time for the dissipation of pore pressure in the soil. The experiment's nature implies that some scale factors are fixed (inherent to the chosen

set-up) and others deduced from these fixed scale factors, leading to different scalings depending on the specific time being considered. The derivation of these scaling factors is presented hereafter.

3.1.1.8.1 Time for wave propagation in the pile, t_p The shear wave velocity $v_{p,S}$ in the pile is given by $v_{p,S} = \frac{L}{t_{p,S}}$. With both the shear modulus G_p and material density ρ scaled as 1, $v_{p,S}$ also scales as 1. The time for shear wave propagation $t_{p,S}$ then scales as:

$$n_{t_{p,S}} = \frac{1}{N}$$

For compressional (P) waves, applying the same principles gives the same scaling factor for the time of propagation:

$$n_{t_{p,P}} = n_{t_{p,S}} = \frac{1}{N}$$

3.1.1.8.2 Time for shear wave propagation in the soil, t_s P- and S-waves are referred to as elastic waves, causing elastic deformations to the soil as they propagate through it, unlike shock waves (Gelius, 2007). Similarly to the previous deduction for t_p (time for wave propagation in the pile), the scaling of the time for shear wave propagation in the soil, denoted as t_s , can be deduced. The shear wave velocity in the soil, v_s , is given by $v_s = bG_{s,e}/\rho$ (Slawinski, 2002). Considering that the soil's elastic shear modulus $G_{s,e}$ scales as $1/N^m$, and assuming an elastic behavior, the scaling factor is determined as follows:

$$n_{t_s} = \frac{n_L}{n_{v_s}} = \frac{1}{N^{1-m/2}} \quad (3.19)$$

3.1.1.8.3 Time for dissipation of pore pressure, t_f The time required for pore pressure dissipation depends on the fluid velocity through the soil, described by Darcy's law:

$$v_f = k \cdot i$$

Given that permeability k and fluid properties scale as 1, and pore pressure Δp scales as $\frac{1}{N}$ (since $F = p \cdot A$), the velocity v_f scales as:

$$n_{v_f} = 1$$

Thus, the scaling factor for the time of pore pressure dissipation is:

$$n_{t_f} = \frac{n_L}{n_{v_f}} = \frac{1}{N}$$

3.1.1.8.4 Time Between Each Impact The time for wave propagation in the pile and the time for dissipation of pore pressure are critical quantities that need to be correctly scaled in the analysis of the current experimental setup. Similarly, the time between each impact during impact driving must be scaled to maintain similitude with these previously established time scales.

Thus, the time between each impact should be scaled as:

$$n_{t_i} = \frac{1}{N}$$

This scaling indicates that the impacts should ideally occur at a higher frequency—100 times faster in this specific research context. However, practical constraints and available technology limited the achievable impact

frequency to a rate similar to or even lower than industry practice. Typically, this resulted in about 27 to 30 blows per minute, compared to the industry standard of 30 to 45 blows per minute, depending on the maximum blow energy applied to the pile [25, 2].

3.1.1.9 Frequency

To scale the frequencies associated with the system, one simply needs to take the inverse of the previously discussed time-scales.

3.1.1.10 Summary

This chapter builds upon the work originally conducted by Deckers Maxence and Defer Cédric in their previous thesis. They laid the groundwork by applying scaling laws to predict the behavior of monopiles without resorting to costly full-scale prototype tests. Their study focused on ensuring physical similarity between experimental models and real-world prototypes, guided by geometric, kinematic, and dynamic similarities.

The revised summary of the scaling factors determined in their study, as clarified and simplified here, is presented in Table 3.1.

Quantity	Scale factor notation	1g scale factor
Length	n_L	$\frac{1}{N}$
Area	n_A	$\frac{1}{N^2}$
Volume	n_V	$\frac{1}{N^3}$
Sand density	n_{ρ_s}	1
Pile material density	n_{ρ_p}	1
Ram mass	n_{m_r}	$\frac{1}{N^3}$
Acceleration	n_a	1
Stress in the soil	n_{σ_s}	$\frac{1}{N}$
Stress in the pile	n_{σ_p}	$\frac{1}{N}$
Frictional force on the pile shaft	n_{F_f}	$\frac{1}{N^3}$
Force on the pile head	n_{F_h}	$\frac{1}{N^3}$
Impedance	n_Z	$\frac{1}{N^2}$
Ram drop height	n_h	$\frac{1}{N^2}$
Elastic modulus of the pile	n_{E_p}	1
Elastic shear modulus of the soil	$n_{G_{s,e}}$	$\frac{1}{N^m}$
General shear modulus of the soil	n_{G_s}	$\frac{1}{N}$
Flexural stiffness of the pile	n_{EI}	$\frac{1}{N^4}$
Deviatoric strain	n_{ϵ_q}	1
Volumetric strain	n_{ϵ_p}	1
Viscosity of the pore fluid	n_{μ}	1
Time for wave propagation in the pile	n_{t_p}	$\frac{1}{N}$
Time for shear wave propagation in the soil	n_{t_s}	$\frac{1}{N^{1-m/2}}$
Time for dissipation of pore pressure	n_{t_f}	$\frac{1}{N}$
Time between each impact	n_{t_i}	$\frac{1}{N}$
Pore pressure	n_u	$\frac{1}{N}$
Velocity of the pore fluid	n_{v_f}	1
Velocity of the ram	n_{v_r}	$\frac{1}{N}$
Shear wave velocity in the soil	n_{v_s}	$\frac{1}{\sqrt{N^m}}$
Wave velocity in the pile	n_{v_p}	1

Table 3.1: Scale factors applicable for present work's 1g laboratory set-up with N being the scaling reduction factor [37].

3.1.2 Existing Experimental Apparatus

3.1.2.1 Container

To conduct scaled monopile testing effectively, the same container that was designed previously, was used to replicate in-situ soil conditions under both dry and saturated states.

Based on recommendations found in the literature, the diameter of the container was carefully chosen to optimize the experimental setup. Various sources suggest using a container diameter that is several times larger than the pile diameter to minimize boundary effects:

Container diameter	Literature
7 D	(LeBlanc et al., 2010 [33]; Abadie, 2015)
10 D	(Al-biker et al., 2017 [4]; Richards, 2019 [43])
12 D	(Achmus et al., 2007 [1])

Table 3.2: Recommended tank diameters from the literature.

The container was designed with a circular shape to reduce material waste in the corners, resulting in a diameter of 110 cm. This size was specifically chosen to accommodate both the 11-cm and 6-cm diameter piles. Although the 110 cm diameter may seem oversized for the smaller 6-cm pile, it allows for strategic positioning, as shown in Figure 3.5. This design enables the installation of the 6-cm pile in up to three different locations within the same container without encountering significant boundary effects or interference from previous tests. This approach not only maximizes the number of experiments with the 6-cm pile but also avoids the need for a larger soil volume when testing the 11-cm pile.

Furthermore, the container's height was set at 100 cm, which corresponds to the typical recommendation of having a sand layer beneath the embedded pile tip that is 2 to 3 times the pile diameter as mentioned by (Richards, 2019) [43].

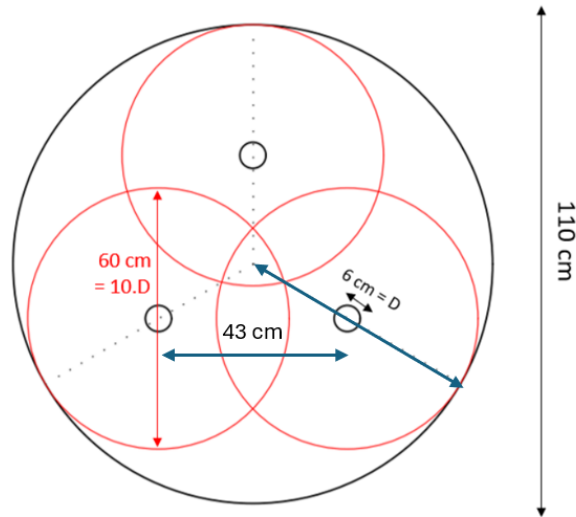


Figure 3.5: Locations for 6cm diameter pile minimizing boundary effects (red circles) in a container with a diameter of 110 cm.

3.1.2.1.1 Material and Construction Constructed from a 2 mm thick steel plate, ensuring it can support the pressure from wet sand, with a maximum calculated horizontal pressure of 15.4 kPa and a Circumferential Stress of 4.3 MPa as calculated in Appendix A. Reinforced with steel beams and lifting blocks to facilitate lifting and emptying without deformation.

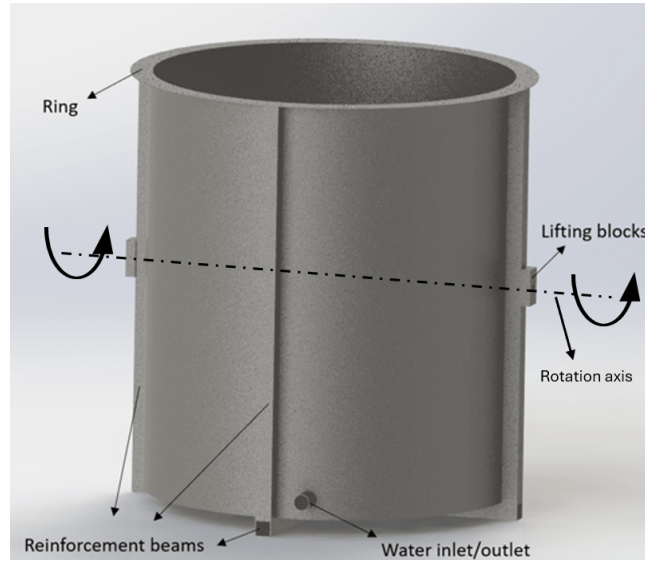


Figure 3.6: The steel container with its reinforcement frame of beams and lifting blocks.

Container dimensions	Values
Height of container wall	1250 mm
Inner diameter of container	1110 mm
Thickness of container wall	2 mm
Cross-section of reinforcement beams	50 × 50 × 3 mm
Ring - Outer diameter	1224 mm
Ring - Inner diameter	1088 mm
Ring - Thickness	2 mm
Height of rotation axis (from ground level)	700 mm
Weight	218 kg

Table 3.3: Container dimensions and corresponding values.

3.1.2.1.2 Adaptation for Saturated Tests The container was modified to allow water saturation by adding a water inlet at the bottom, a gravel layer for even water distribution, and a geotextile barrier to prevent sand loss. This setup ensures minimal disruption during the saturation process and allows controlled flooding and drainage of water.

3.1.2.1.3 Adaptation for Saturated Tests "The Steel allows easy perforation and welding, which was essential to introduce water into the container for achieving sand saturation. The most effective method was to flood the container from the bottom, ensuring uniform water distribution.

Maintaining sand homogeneity during saturation is critical, as noted by (Hoffmann et al. 2020) [20]. Therefore, water was introduced uniformly from the bottom to cover the entire sand volume. (Klinkvort et al. 2018) [31] recommended flooding the sand with CO₂ before saturation, as CO₂ dissolves in water better than air, helping to prevent air bubbles between sand grains. However, CO₂ pre-saturation was not applied in this study." ²

The container was modified by perforating the bottom wall to install a water valve just above ground level. After filling the container with approximately 85 cm of dry sand at a density of about 1700 kg/m³, water was

²Text rewritten by ChatGPT, June 15, 2024, OpenAI, <https://chat.openai.com>

introduced. The water passed through a 7 cm thick gravel layer, which slowed and evenly distributed the flow. Above the gravel, a geotextile barrier was placed to allow water to rise while preventing sand from escaping. To stabilize the gravel layer, a sturdy steel mesh, complemented by a plastic mesh to prevent gravel loss, was added. This setup allowed the container to be filled with water up to 100 cm in about 35 minutes. Draining the water before emptying the container took approximately 90 minutes under these conditions.

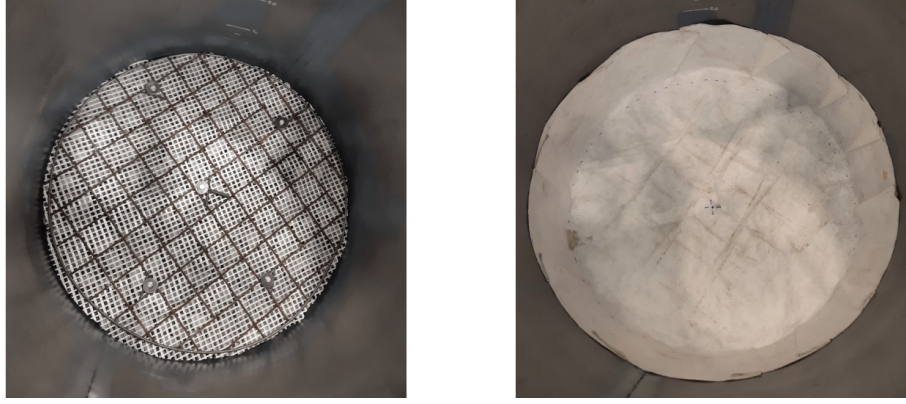


Figure 3.7: Container bottom adaptation for water flooding with retaining of the gravel layer (by plastic and steel mesh) and retaining of the sand particles (by geotextile).

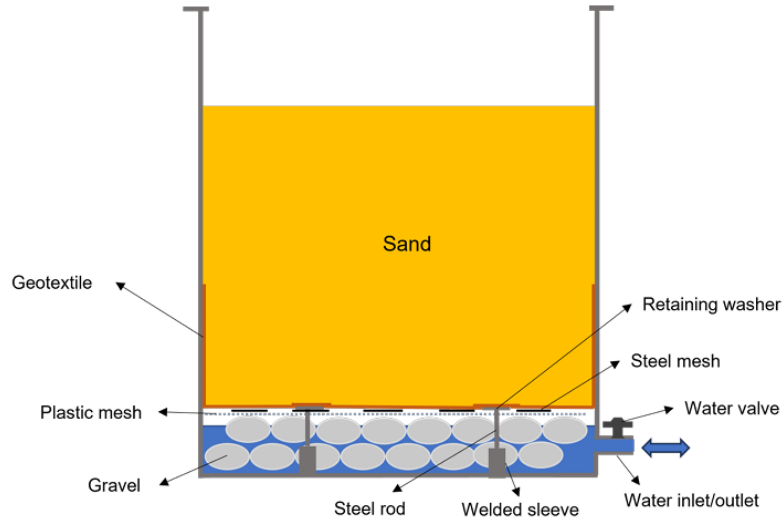


Figure 3.8: Container set-up for experiments in saturated conditions.

3.1.2.1.4 Determining the Exact Volume of the Container To accurately determine the container's volume, a series of water-filling tests were conducted. The water mass was measured at different fill heights, enabling the calculation of the corresponding volumes. The relationship between the water depth and the container's volume was then modeled using linear regression, which allowed us to predict the container's total volume when fully filled.



Figure 3.9: Water mass measurements at different heights.



Figure 3.10: Container and water weight.

The measurements involved determining the volume V at different depths h using the formula:

$$V(h) = \frac{W(h)}{\rho_{\text{water}}} \quad (3.20)$$

where $W(h)$ is the weight of the water at height h and ρ_{water} is the density of water, 997 kg/m^3 . The experimental data was processed using linear regression, resulting in a mathematical relationship between the water depth and the corresponding volume:

$$V(h) = m \cdot h + b \quad (3.21)$$

where m is the slope and b is the intercept. The regression yielded a slope of $m = 0.98$ and an intercept $b = 0$, indicating a nearly linear relationship between depth and volume. The experimental maximum volume was determined to be $V_{\text{exp}} = 1.18 \text{ m}^3$, closely matching the theoretical volume $V_{\text{theo}} = 1.16 \text{ m}^3$ calculated using:

$$V_{\text{theo}} = \pi \cdot R_{\text{inner}}^2 \cdot H_{\text{res}} \quad (3.22)$$

with $R_{\text{inner}} = 0.555 \text{ m}$ and $H_{\text{res}} = 1.2 \text{ m}$.

Height from the Top (m)	Weight (W - Cont) (kg)
0.25	928.0
0.5	685.5
0.85	347.5
1.2	0.0

Table 3.4: Measured Depths and Corresponding Volumes

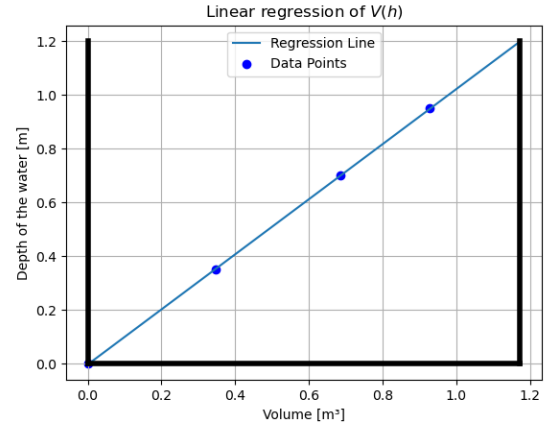


Figure 3.11: Relationship between container volume and water height.

These tests provide a precise understanding of the container's capacity, essential for experiments requiring accurate sand or water loading. The linear regression model effectively simulates the volume based on measured data, ensuring reliability in experiments involving varying fill heights.

After completing the tests, the water was drained into a secondary reservoir to be reused for other purposes.



Figure 3.12: Water storage for reuse.

3.2 Evolution and Advanced Theoretical Concepts

3.2.1 Sand Raining System Upgrade: Introduction of the New System

3.2.1.1 Introduction

The sand raining system utilized in previous experiments presented several limitations that necessitated its redesign and enhancement. These limitations impacted various aspects of the testing process, including:

- Inconsistent sand coverage of the container area.
- Limited control over sand release.
- Complexity in operation.
- Possible blockages and interruptions in sand flow.
- Stability concerns during lifting and positioning operations.
- Limited accuracy for the sand density.
- Lack of uniformity of the sand.

Addressing these limitations was imperative to ensure the reliability, efficiency, and accuracy of the experiments conducted in the laboratory setting.

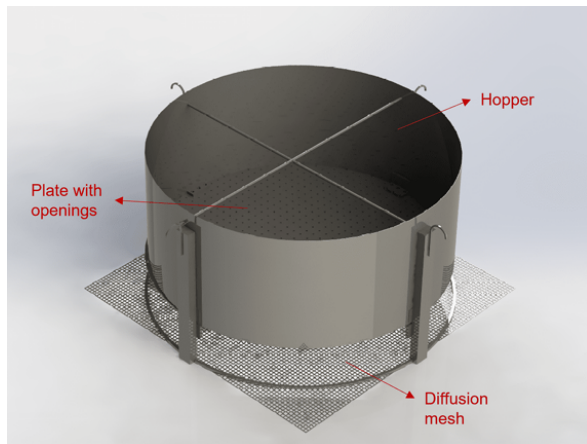


Figure 3.13: The old pluviometer system realised by Deckers Maxence and Defer Cédric [37].

3.2.1.2 Design Overview

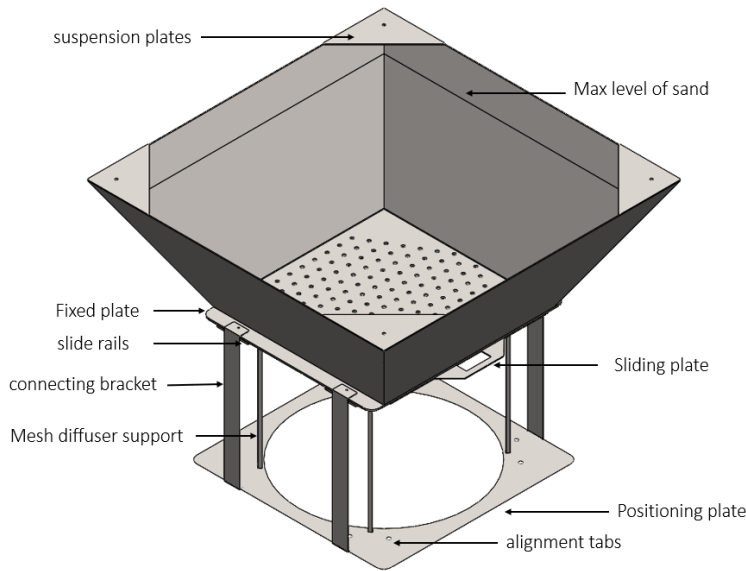


Figure 3.14: 3D design of the new pluviation system



Figure 3.15: Fully suspended PV-system with protective cover in place

In response to the identified limitations, a new sand raining system has been developed to address key issues and enhance efficiency. The upgraded system is designed to contain 1.1 m^3 of sand which is the entire quantity of sand required to fill the container in a single operation. It features plates with different hole sizes (20mm and 10mm) strategically placed to control the density of the sand.

The new system consists of four inclined plates, each 5mm thick and inclined at a 30-degree angle from the vertical. These plates are welded onto a horizontal plate, 10mm thick, featuring holes of 20mm in diameter. The holes are arranged in a triangular pattern with a distance of 80mm between them (See plan in Appendix).

SolidWorks was used for the design of the system, and a finite element study was conducted to simulate its adequacy, as shown in Figures 3.20 and 3.21.

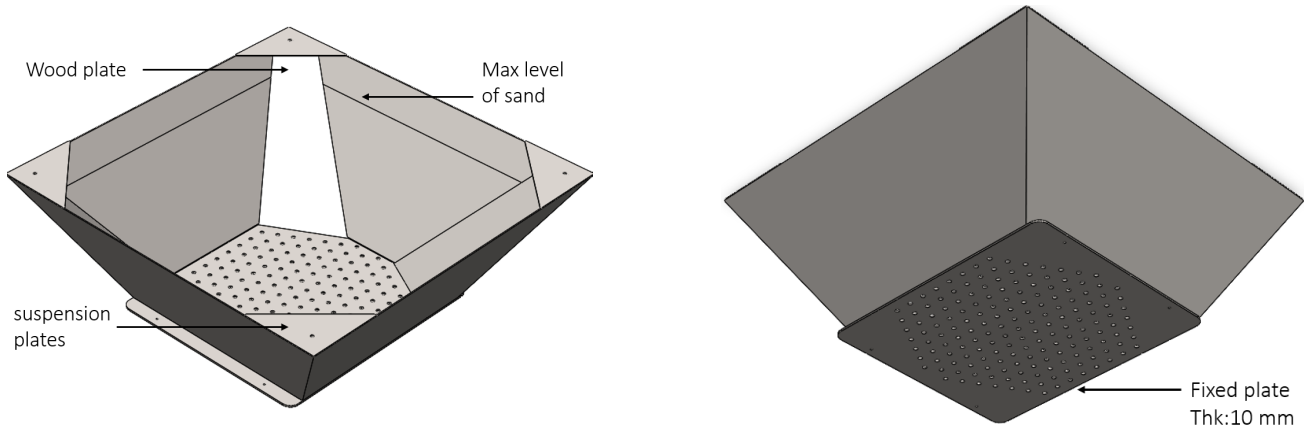


Figure 3.16: Model of the newly designed pluviation system (modelled in *Solidworks*).

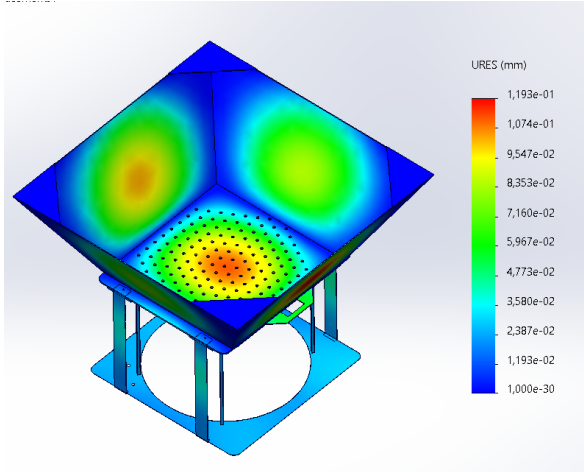


Figure 3.17: FEM analysis when the PV-sys is suspended from the 4 corner with a security factor of 1.33

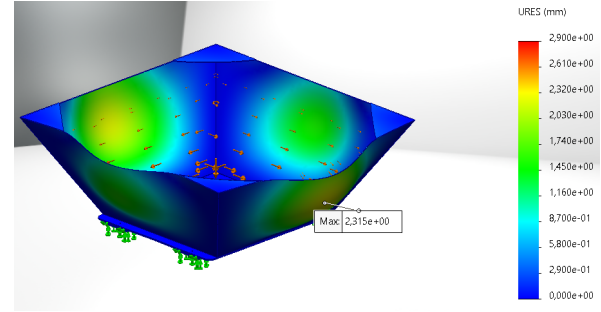


Figure 3.18: FEM analysis when the PV-sys is deposited on the 4 rail-way.

Figure 3.19: Finite element analysis for 1.26 m^3 of sand (1630 kg/m^3)

The thicknesses and design parameters were optimized through iterative Finite Element Method (FEM) simulations using SolidWorks. By running multiple analyses, the configurations that provided the most favorable results in terms of Von Mises stress distribution and overall displacement were selected. This iterative approach allowed for refining the design to ensure structural integrity while minimizing material usage.

In the case of the suspended structure, a safety factor of 1.33 was incorporated to account for the laboratory conditions. This adjustment reflects the practical scenario where, in a suspension setup, only three support points are typically necessary to stabilize an object. The fourth point serves primarily for balancing and distributing the load more evenly. By including this safety factor, the design better mirrors real-world conditions (as advised by Benoît Herman).

Furthermore, the sand level was conservatively assumed to reach a height of 552.7 mm, corresponding to a volume of 1.26 m^3 . This margin provides additional security, considering that the pluviation system may be repurposed for other projects or in cases where the sand density is not uniformly distributed throughout the container. This consideration ensures that the system remains robust even under varying experimental conditions.

To simulate the structural behavior in SolidWorks, a distributed pressure was applied across the five plates, with a maximum value calculated as $1630 \times 9.81 = 15,990 \text{ N/m}^2$. A safety factor of 1.33 was included in the analysis when the structure is suspended to account for uncertainties, resulting in a maximum displacement of 2.3 mm over a span of 1900 mm. This displacement is well within the acceptable limit according to the 1/500 criterion typically applied to metallic plates. According to this standard, the maximum allowable deflection would be 3.8 mm, so the observed 2.3 mm is safely within this range.

3.2.1.3 Construction and Installation

The system is constructed in two main pieces: the steel basin and the positioning plate. Initially, the steel basin is positioned on the ground to facilitate filling. Once filled, it is lifted using a crane and secured via the triangular attachments to suspend it in the air.

The positioning plate is then installed to maintain the diffuser mesh at a height of 150mm below the lower plate of the basin. This plate is affixed using manual fasteners onto the lower plate of the basin, with pins inserted into the provided holes to prevent lateral movement.

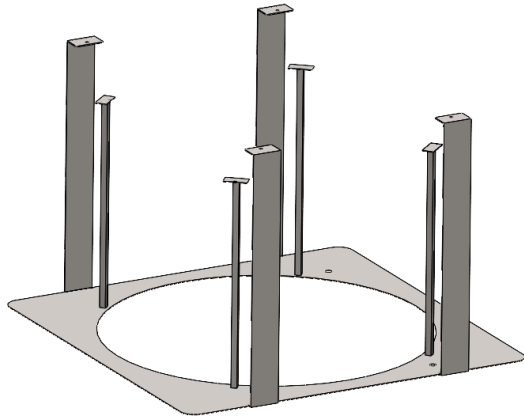


Figure 3.20: The positioning plat modeled with *solidworks*



Figure 3.21: The positioning plat with a plastic cover around it

The positioning plate will be reinforced later with square tube 40x40x2 mm along its parameter for more rigidity as the lower sheet is only 2mm of thickness.

The system is securely suspended using welded tabs located on the bottom of the positioning plate, ensuring precise placement above the container. Once the system is in place, the positioning plate is manually adjusted to align the holes correctly, ensuring an even distribution.

The sliding plates (*plaque glissante*) are guided on rails with a 1 mm clearance to minimize friction during operation, allowing for smooth adjustments (See Figure 3.22).

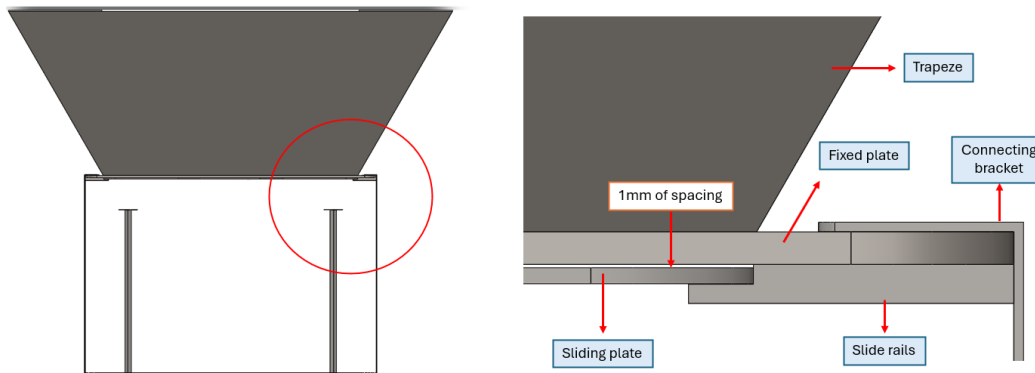


Figure 3.22: 2D schema of the diffents plates.

The structure is constructed using S355 LAC steel, chosen for its durability and stability, ensuring the system can handle the required load and maintain its integrity over repeated uses.

Further content will be added to discuss the advantages, disadvantages, and a comparison with the previous system design.

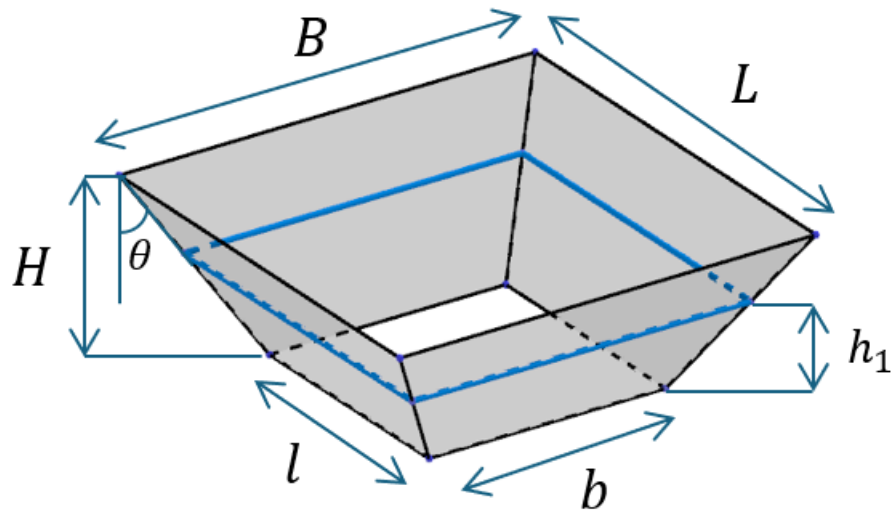


Figure 3.23: Illustration of the internal dimensions for calculating sand volume.

B [mm]	L [mm]	b [mm]	l [mm]	H [mm]	h_1 [mm]	θ [°]
1890	1910	1080	1085	695	545	30

Table 3.5: Internal dimensions of the sand pluviation system.

3.2.1.4 Operation and Volume Calculation of the PV-System

To accurately estimate the volume of the PV-system based on the sand height from the bottom, a decomposition of the system's geometry was conducted. The decomposition approach, as illustrated in Figure 3.24, provides a realistic and straightforward method to calculate the total volume at varying heights h_1 .

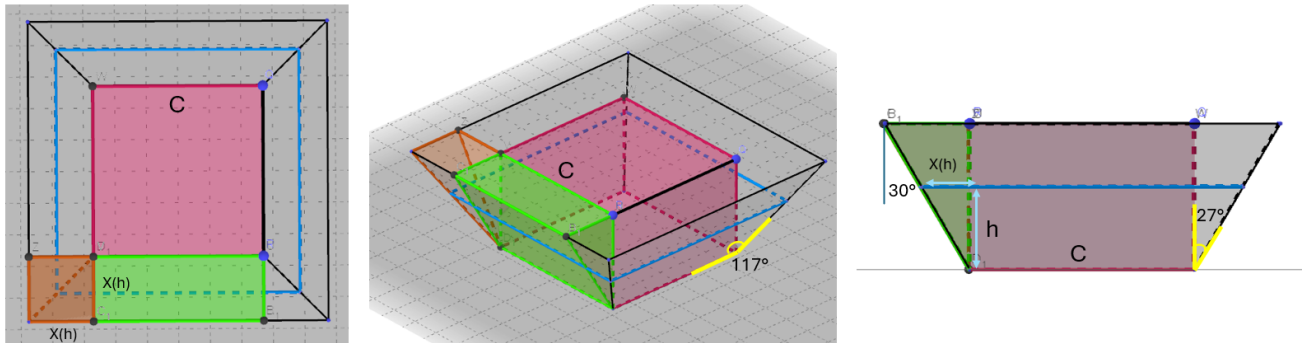


Figure 3.24: PV-System: Volume decomposition

The exact volume $V_{exact}(h_1)$ can be expressed as:

$$V_{exact}(h_1) = 4 \cdot (V_{pyramid} + V_{par}) + V_{cube}, \quad (3.23)$$

$$x(h) = \frac{h \cdot \tan(27^\circ)}{\cos(30^\circ)}, \quad (3.24)$$

$$V_{cube} = c^2 \cdot h \quad \text{where } c = 1.088 \text{ m}, \quad (3.25)$$

$$V_{pyramid} = \frac{x^2(h) \cdot h}{3} = \frac{h^3 \cdot \tan^2(27^\circ)}{3 \cdot \cos^2(30^\circ)}, \quad (3.26)$$

$$V_{par} = \frac{c \cdot x(h) \cdot h}{2} = \frac{c \cdot h^2 \cdot \tan(27^\circ)}{2 \cdot \cos(30^\circ)}. \quad (3.27)$$

This decomposition method is preferred because it allows for more accurate volume estimations. Alternatively, the volume could be estimated using the truncated pyramid formula:

$$V_{tr-pyr}(h_1) = \frac{B \cdot L + b \cdot l + \sqrt{B \cdot L \cdot b \cdot l}}{3} \cdot H \quad (3.28)$$

$$= 2.28 \cdot H = 1.5846 \text{ m}^3. \quad (3.29)$$

However, the total estimated volume of 1.5846 m^3 is larger than the targeted sand capacity of approximately 1.1 m^3 . Two key factors must be taken into account to address this discrepancy:

1. **Inclined Wooden Plates:** Four inclined wooden plates are introduced to the PV-system's design, creating a 30° slope around the corners (Figure 3.26) to prevent sand accumulation so the volume will be reduced by the zone behind them. The decision lies between either filling the space behind the plates with sand, adding unnecessary weight, or increasing the thickness of the plates without filling behind them.
2. **Safety Margin:** It is crucial to avoid filling the container to the very top. Leaving a safety margin ensures stability and avoids potential issues with sand density and distribution.

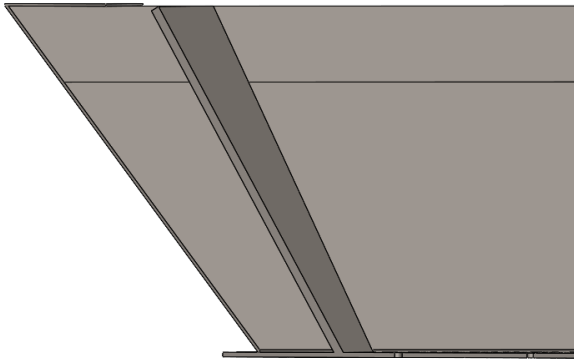


Figure 3.25: Space behind the inclined wooden plates

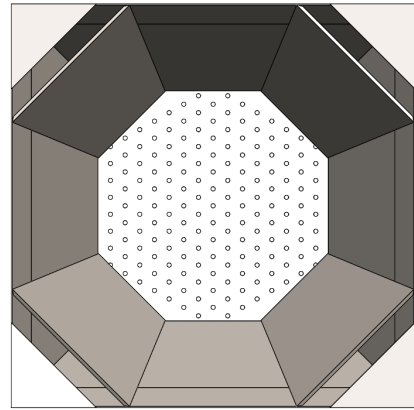


Figure 3.26: *SolidWorks* design with inclined wooden plates. $V_{sand} = 1.1 \text{ m}^3$ if $h_1 = 0.62 \text{ m}$

After adding the wooden plates, measurements were taken for different heights using *SolidWorks*, with corresponding volumes calculated and a polynomial fit was applied to the data, after that, the resulting plot is shown in Figure 3.27, providing a comparison of the PV-system's volume as a function of height, both with and without the wooden plates.

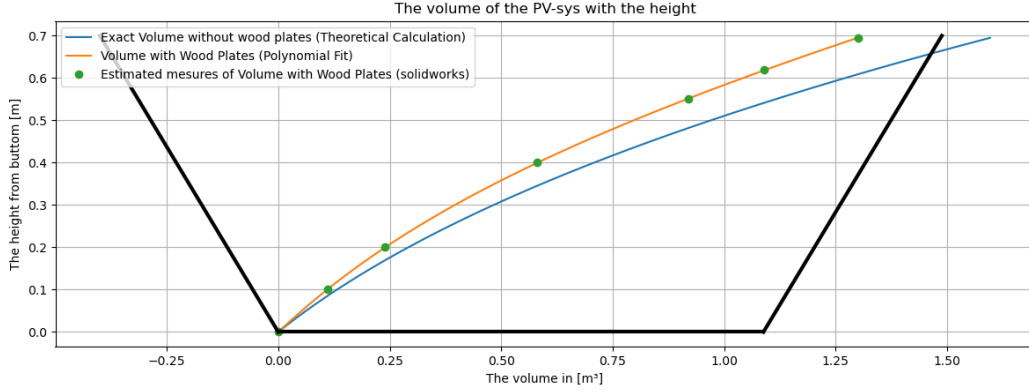


Figure 3.27: Volume of the PV-system as a function of height

The weight of the PV-system is theoretically 390 kg, although it is advisable to remeasure it once it is empty to confirm.

During one of the filling stages, when $h_1 = 315$ mm, the total weight was measured at 1211 kg, corresponding to a volume $V_{exact}(h_1) = 0.541 \text{ m}^3$. Hence:

$$W_{\text{PV-sys}} + \rho_{\text{sand}} \cdot V_{\text{exact}} = 1211 \text{ kg} \quad (3.30)$$

Assuming a sand density of $\rho_{\text{sand}} = 1630 \text{ kg/m}^3$, the calculated weight of the PV-system would be $W_{\text{PV-sys}} = 329 \text{ kg}$. However, if the actual weight is closer to 390 kg, the sand density would be $\rho_{\text{sand}} = 1517 \text{ kg/m}^3$. Therefore, it is crucial to measure the sand density during each filling and accurately determine the weight of the system to confirm these data.

3.2.1.5 Mesh Diffuser Optimization

During the initial trials, two mesh diffusers with square openings of 1 mm and 2 mm were tested. However, it was observed that the high velocity of the falling sand caused significant accumulation on the mesh surfaces. This resulted in a dense packing of sand at the bottom of the container, reaching a density of approximately 1667.67 kg/m^3 .

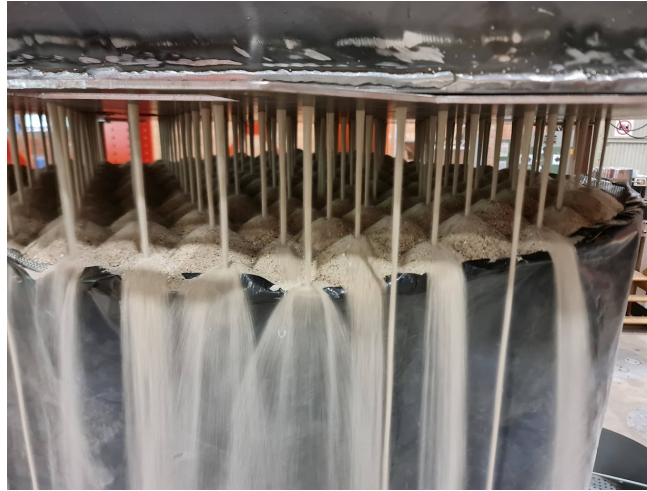


Figure 3.28: Sand accumulation on the 2 mm mesh diffuser

To address this issue, a new mesh diffuser with larger 6 mm square openings was introduced. This adjustment successfully alleviated the sand buildup, enabling a more uniform flow of sand into the container. The resulting sand density was slightly reduced to 1661.85 kg/m^3 , which is expected given that larger grains can pass through the mesh, creating a looser packing arrangement.



Figure 3.29: Mesh diffuser with 6 mm x 6 mm openings



Figure 3.30: Pluviation process using the 6 mm mesh diffuser

Although the sand density achieved with the 6 mm mesh diffuser is slightly lower, this outcome is consistent with expectations due to the larger grain sizes introduced into the tank. Additionally, the sliding plate with 10 mm openings was employed to get a small flow rate during the pluviation process.

The complete pluviation system, shown below, includes a plastic cover around the diffuser area to minimize the dispersion of sand grains beyond the target area and maintain a controlled environment during the sand deposition.

3.2.2 Additional Assemblies Designed

3.2.2.1 Pile Support During Vibro-Driving

To ensure stability during the vibro-driving of the 6 cm pile, a specialized support was designed to maintain the pile in a perfectly vertical position (See Figure 3.31). This support system is designed to be easily detachable, making it practical for experimental setups. The key feature is a removable ring that can be easily (See Figure 3.32) disengaged once the pile reaches its target embedded length. After the ring is detached, an actuator is engaged to apply a controlled push on the pile. Once the experiment is complete, the pile can be manually removed, and the tank can be rotated to its next position for subsequent pile insertions (See to the detailed design plan in the Appendix).

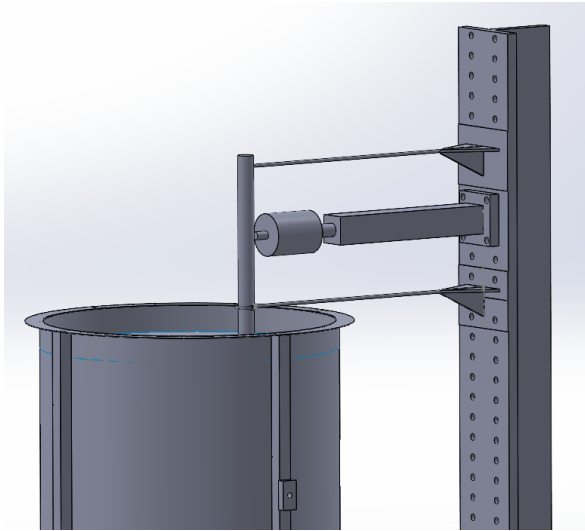


Figure 3.31: SolidWorks assembly for the pile support during vibro-driving

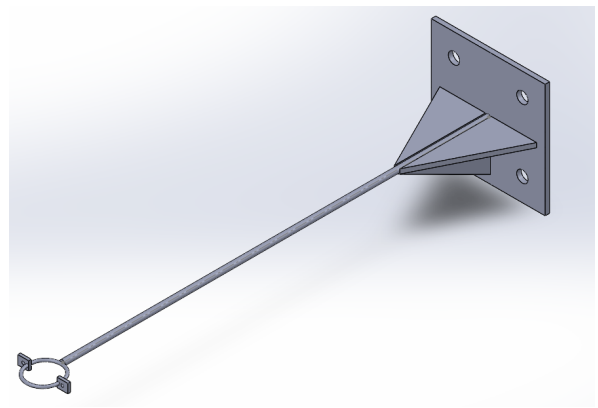


Figure 3.32: Design of the pile support structure

3.2.3 P-Y curve

3.2.3.1 Introduction and theoretical background

The behavior of laterally loaded piles is a critical aspect of geotechnical engineering, especially in the design of foundations for offshore structures. One widely used method to model lateral soil-pile interaction is the p-y curve approach, which conceptualizes the soil resistance (P) against pile deflection (y) as a series of nonlinear springs (See Figure 3.33). These p-y curves are essential for predicting the lateral performance of piles, and their accuracy directly impacts the reliability of foundation designs.

The p-y curves can be developed through experimental data, field testing, or numerical modeling. However, experimental and field approaches often require back calculation from bending moment profiles, typically obtained via strain gauges distributed along the pile in each direction (see Figure 3.34).

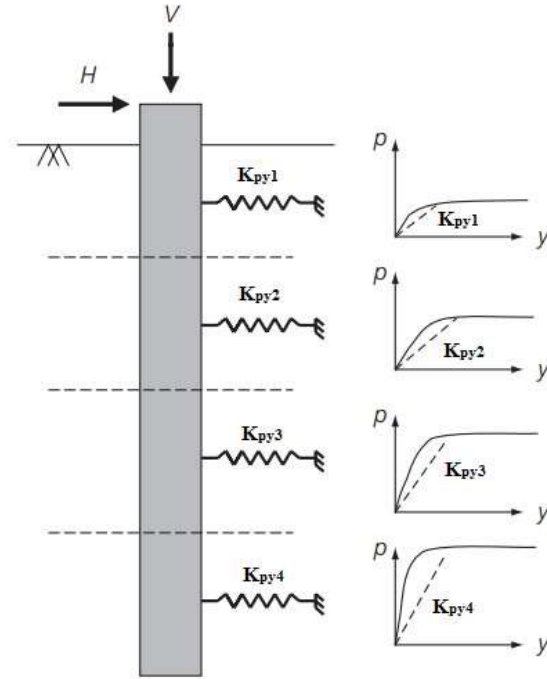


Figure 3.33: Winkler model approach [56]

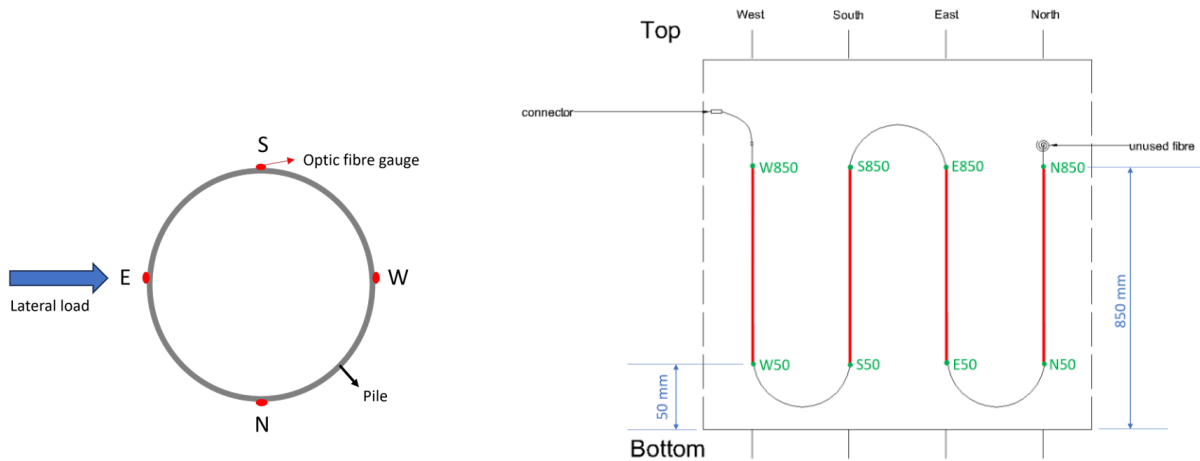


Figure 3.34: Illustration of the pile cross section with optical fibre gauges during lateral loading [37]

Figure 3.35: Positioning of the optical fibre along the pile, considering an "unrolled" pile view [37]

In this study, we propose a novel method that leverages optical fiber sensors to measure bending moments along the pile. Optical fibers offer high-resolution and almost continuous data along the pile, enabling more accurate and detailed bending moment profiles.

Data is typically gathered from optical fibers placed along the pile (See Figure 3.35). These readings reflect the longitudinal deformation along the pile that is subjected to lateral forces. The fibers are glued along the pile

circumference (e.g., West, South, East, North) to capture responses along the different axes of the pile. Key parameters to take in consideration:

- **Pile geometry:** Diameter, embedment depth, etc.
- **Material properties:** Young's modulus (E), moment of inertia (I).
- **Loading conditions:** Applied force, load height.

The raw data from the fibers is cleaned, processed, and interpolated. Noise is filtered out and the data is smoothed to ensure accuracy. Common steps include:

- **Moving average smoothing:** Applied to reduce fluctuations in strain readings.
- **Removing NaN values:** Ensures data continuity for calculations.

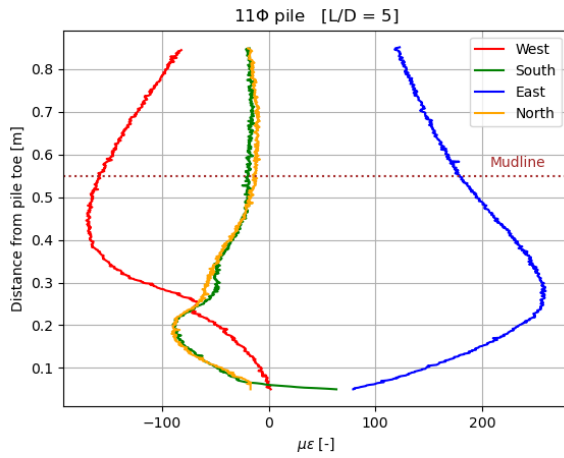


Figure 3.36: Strain on the 4 gauges.

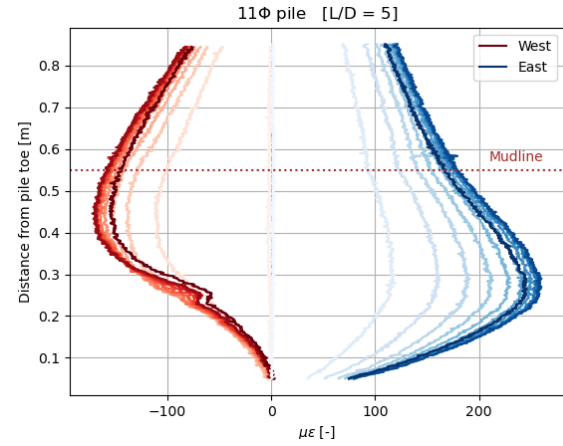


Figure 3.37: Strain on the gauges where the load is applied

Figure 3.38: Strains on the four gauges 11 Φ cm pile during Test 13 (done by Deckers Maxence and Defer Cédric).

The bending moments, $M(y)$, at different locations along the pile are derived using strain data. The relationship is given by:

$$M(y) = -E \cdot I \cdot \frac{\text{deformation}(y)}{\text{pile radius}} \quad (3.31)$$

Where:

- E is the Young's modulus of the material.
- I is the moment of inertia of the pile cross-section.
- $\text{deformation}(y)$ is the strain at location y along the pile.

The bending moment is calculated for each gauge position and plotted as a function of depth from the pile's mudline (reference point). Important features like the maximum bending moment and its location provide insights into where the pile experiences the most stress.

To generate p-y curves from the bending moment profiles, techniques such as cubic/quintic spline smoothing and polynomial fitting are commonly employed as cited by (Guevara , 2022 [18]) . These methods provide smooth and continuous representations of the pile's bending moment, which are essential for accurate calculations of soil resistance and deflection.

The key difference between cubic spline smoothing and polynomial fitting lies in their approach and suitability for different types of data. Cubic splines ensure that the curve passes through all data points, making them ideal when precision at each measurement is critical, such as in cases where the data has minimal noise and high resolution. For instance, spline fitting is often preferred in applications like interpolating exact measurements in geotechnical surveys or aligning gauge readings in laboratory experiments.

In contrast, polynomial fitting provides an overall approximation by creating a polynomial curve of a chosen degree. This method is particularly useful when the data is noisy or exhibits trends over a broad range, where strict adherence to each point would introduce unnecessary oscillations. For example, in soil mechanics, where experimental data often contains measurement inconsistencies, polynomial fitting of a moderate order (e.g., 3rd or 4th) can smooth out the noise while capturing the overall behavior, making it more robust for deriving p-y curves.

First, it is crucial to understand how p-y curves are derived. Soil resistance, denoted as $p(x)$, is obtained from the second derivative of the moment profile:

$$p(x) = \frac{d}{dx} \left(\frac{dM}{dx} \right) \quad (3.32)$$

Pile deflection y_p is determined using the Euler-Bernoulli beam equation:

$$EI \frac{d^4 y_p}{dx^4} = p(x) \quad (3.33)$$

$$y_p = \frac{1}{E_p I_p} \int \int M(x) dx \quad (3.34)$$

Where: - $M(x)$ is the bending moment, - E_p is the pile's Young's modulus, - I_p is the moment of inertia of the pile cross-section.

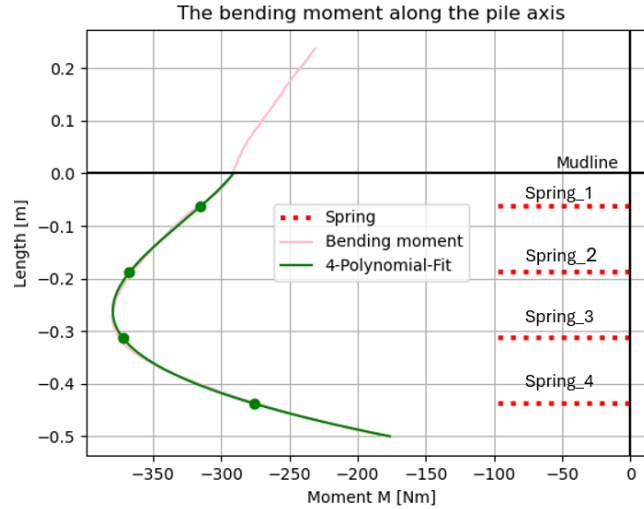


Figure 3.39: Bending moment profile under lateral loading

One challenge in deriving p-y curves is that raw bending moment data, are typically noisy and fluctuate considerably. Directly applying a double differentiation to such data can produce unrealistic soil resistance profiles due to the amplified effect of noise.

Various methods, such as smoothing, filtering, and polynomial fitting, were evaluated to enhance the reliability of derived p-y curves. Among these, polynomial fitting of 3rd or 4th order provided the most accurate results (Figures 3.40 and 3.41).

The smoothing technique, while effective in certain scenarios, introduces excessive curvature due to its requirement that the curve passes through all data points. This characteristic results in significant oscillations when performing double differentiation, leading to unrealistic zigzag patterns in the derived soil resistance profiles. Additionally, as the smoothing factor s increases, the fitted curve becomes less responsive to localized variations in the data, which further compounds the issue when calculating the second derivative.

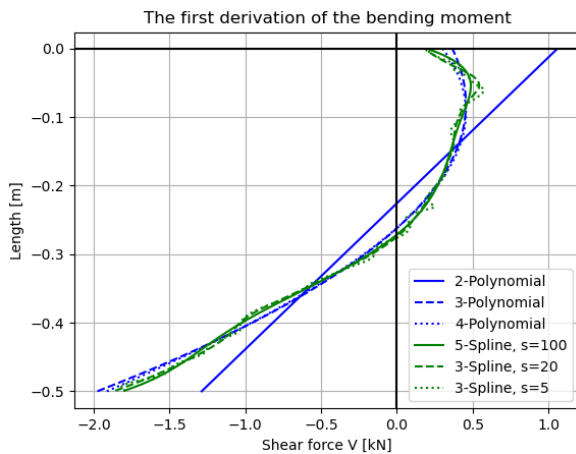


Figure 3.40: First derivative of the bending moment: comparison between methods.

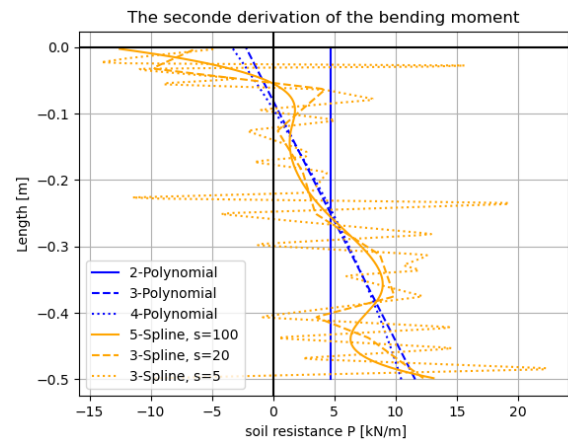


Figure 3.41: Second derivative of the bending moment: comparison between methods.

The second derivative of the moment for 3rd and 4th-order polynomial fits yielded smoother and more realistic soil resistance profiles. For the pile deflection, the double integration of the bending moments directly yields the

terms of order superior to two of the polynomial deflection. However, boundary conditions should be applied to derive the constant and the first order term. This is done by taking advantage of the laser measurements of the pile shaft above mudline during the pile lateral loading (this is further explained in the next section). The building of p-y curves was here simplified by neglecting the terms of order superior to two for the deflection. This is acceptable as (Figures 3.42 and 3.44) can attest: the more than quadratic part of the deflection is more than an order of magnitude inferior to the rigid rotation of the pile. Repeating this process over 10 time steps (spanning measurements from 109 to 3150, with a sampling frequency of 10 Hz, so a time step of 0.1s, and a loading event of 5min) enables the generation of p-y curves for each spring, as shown in the subsequent analysis. These are pictured (in Figure 3.43)

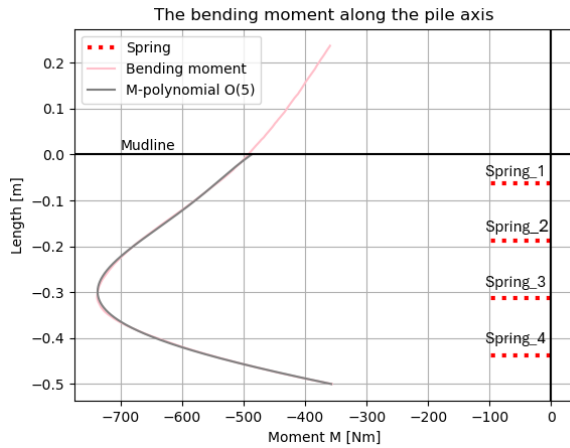


Figure 3.42: Bending moment with a polynomial fit $O(5)$.

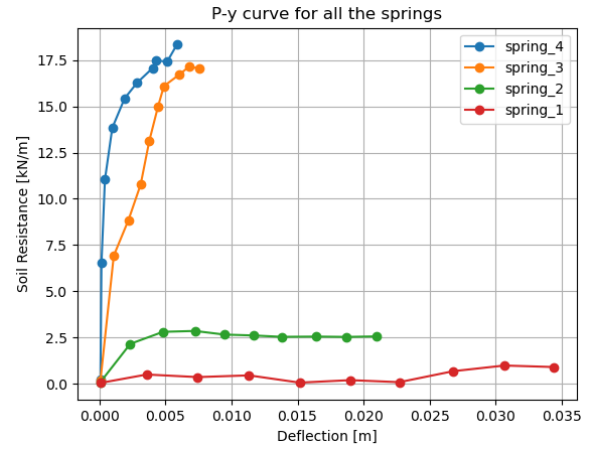


Figure 3.43: P-Y curves for each spring along the pile height and during the loading

It was observed that as the depth increases, the soil resistance also increases, which is consistent with theoretical expectations. However, to further refine these results, additional tests under varied conditions would be beneficial. For future improvements, enhancing the code by increasing the number of springs along the pile could provide finer resolution in capturing soil-pile interactions. Caution is advised, as this could introduce non-linear soil reactions, leading to added complexities in the analysis (*T13_Optical_fiber_process_V26.py*).

Moreover, to obtain the p-y curves shown in Figure 3.43, the analysis assumed rigid pile rotation. This assumption arose after the deflection derived from double-integrating the bending moment (from optical fiber data) was found to be negligible. Therefore, the rigid rotation assumption was deemed appropriate, simplifying the analysis and enabling the generation of reliable p-y curves.

3.2.4 Deflection Analysis and Simplifications

In this study, the deflection analysis was primarily based on the bending moments extracted from the optical fiber data. Initially, the deflections were determined by double integrating the bending moment profile along the pile, as expressed by equation 3.34

However, after performing the calculations, it became evident that the deflections were negligible (as illustrated in Figure 3.44) comparing to (Figure 3.45), leading to the assumption that the pile behaved rigidly under the applied loads. Given this rigid behavior, the decision was made to rely on the laser displacement measurements (*L500* and *L40*) positioned at 300 mm and 20 mm from the mudline, respectively (See Figure 3.46).

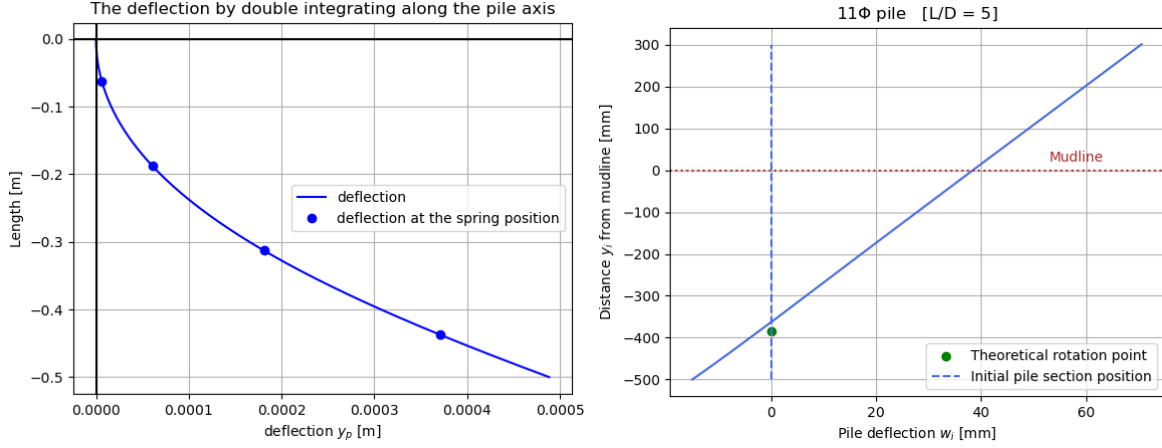


Figure 3.44: Deflection of the pile at the end of the loading. Figure 3.45: Deflection obtained by the laser measurement at the end of the loading.

The deflection profile $y(x)$ was modeled with a linear term, which dominates in the case of a rigid pile:

$$y(x) = Ax + B \quad (3.35)$$

where: - A and B are constants determined from the laser measurements.

Calculation of Constants A and B

The constant A was determined based on the difference in deflections between the two laser positions ($L500$ and $L40$):

$$A = \frac{y(L500) - y(L40)}{L500 - L40} \quad (3.36)$$

The constant B was then derived from the deflection at the reference point (20 mm from the mudline):

$$\begin{cases} y(L500) = A \cdot 300 + B \\ y(L40) = A \cdot 20 + B \end{cases} \quad (3.37)$$

By substituting A in this system of equation we obtain :

$$B = \frac{15 \cdot y(L40) - y(L500)}{14} \quad (3.38)$$

This approach provided a more accurate representation of the pile's deflection, considering its rigid nature and the dominant linear behavior observed in the experimental setup.

Conclusion

The deflection analysis underlines the importance of considering both theoretical models and practical measurements when evaluating pile behavior. The double integration of the bending moment requires boundary conditions that are readily given by the laser measurements. In this study, this analysis allowed to simplify the deflection to a rigid rotation.

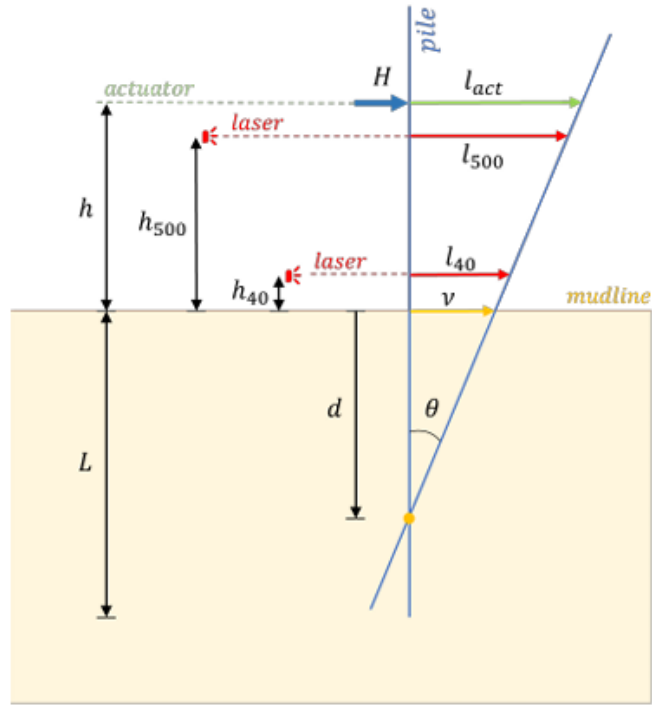


Figure 3.46: Schematic of the laser measurement setup used for the deflection calculations.

Chapter 4

Risk Management and Safety Protocols

This section outlines the safety measures and risk management procedures implemented during the experimental phase of this thesis. A risk registry was developed in collaboration with Mr. Jean-François Vanden Berghe to identify, evaluate, and mitigate risks associated with the project.

4.1 Risk Assessment Overview

The risk assessment was systematically conducted by categorizing potential hazards based on their likelihood and severity. This allowed for targeted safety measures, ensuring that both personnel safety and project integrity were maintained. The risk matrix (shown below) was established to quantify the risks, followed by corresponding control measures to reduce them to an acceptable level.

			Probability				
Consequences			A	B	C	D	E
			Very Unlikely	Unlikely	Possible	Likely	very Likely
	1	Insignificant	A1	B1	C1	D1	E1
	2	Minor	A2	B2	C2	D2	E2
	3	Moderate	A3	B3	C3	D3	E3
	4	Major	A4	B4	C4	D4	E4
	5	Severe	A5	B5	C5	D5	E5

Figure 4.1: Risk matrix

4.2 Severity and Probability Definitions

To fully understand the risk matrix, it is essential to define the severity and probability levels applied to this project:

4.2.1 Severity Levels

- A. **Insignificant:** Consequences do not affect health or thesis results.
- B. **Minor:** Physical integrity is not compromised beyond minor injuries that heal quickly. Financial losses are limited to an amount I can personally cover (e.g., 1000 €).

- C. **Moderate:** May cause temporary disability, financial losses over 1000 €, or a significant impact on grading.
- D. **Major:** Can cause permanent injury or financial losses exceeding 10,000 €.
- E. **Severe:** Can be fatal or lead to catastrophic financial or lab consequences.

4.2.2 Probability Levels

1. **Very Unlikely:** Occurs once in every 1000 theses.
2. **Unlikely:** Occurs once in every 50 theses.
3. **Possible:** Occurs once in every 10 theses.
4. **Likely:** Occurs in 3 out of every 10 theses.
5. **Very Likely:** Occurs in 2 out of every 5 theses.

4.3 Risk Identification and Mitigation

4.3.1 Identified Risks and Mitigations

Safety Module Risk Assessment The first table categorizes risks associated with safety during the experimental process. These include potential hazards such as falling objects, electrical faults, and handling of hazardous materials. Each risk is assessed based on its initial severity and probability, and then mitigation measures are proposed to reduce the likelihood or impact of these risks.

Safety Module	Before	Mitigation Measures	After
Chain fragment from the crane	A5	Regularly check the safety cable and ensure the area is clear during operation	B3
Cracking of the container	B2	Inspect the container before each use	A2
Falling of a heavy object during handling	B4	Use safety shoes and keep the workspace clear	B3
Explosion of a machine due to electrical fault	A5	Regular maintenance and inspections, avoid using machines with explosion risks	A4
Laser in the eyes	C3	Use protective goggles and secure the area	B2
Falling of the sieve due to lifting equipment or hooks	B5	Verify lifting capacities and maintain a secure zone during operation	A3
Transportation of sand, unloading from the truck, and transport to the lab using a pallet truck	C4	Qualified personnel operate the pallet truck, use safety gear	B2
Pile rupture: projectile effect	B3	Establish safety zones and use individual protective equipment	B2
Soil rupture: sand projection and pile collapse	B3	Establish safety zones and use individual protective equipment	B2
Projection of fragments during driving	D2	Use protective goggles, maintain a safety zone	C2
Ejection of the wooden cushion	E2	Establish a safety zone and use protective measures	C2
Hammer impact on fingers	D2	Follow the protocol: both hands on the hammer handle	A2

Table 4.1: Safety Module Risk Assessment: Mitigation Measures and Updated Risk Codes

Analysis: In the above table, we see that the severity of risks like crane operation and electrical faults initially range from high (A5) to moderate (B2). By implementing appropriate safety measures, such as regular inspections and the use of protective equipment, these risks are significantly reduced to manageable levels (e.g., B3 or A2). This demonstrates how effective planning and precautionary steps can transform high-risk scenarios into acceptable ones.

Project Success Risk Assessment The second table focuses on risks that could impact the overall success of the project. These include risks related to data loss, calibration errors, and time management. Addressing these risks is crucial for ensuring that the experimental outcomes are reliable and that the project remains on track.

Project Success	Before	Mitigation Measures	After
Expired or temperature-affected samples	D2	Inventory management and protocols for sample handling	B2
Measurement errors (depth, CPT, etc.)	E2	Verify calibration of measuring instruments, compare expected and measured values	C2
Poor sensor calibration	E3	Regular calibration checks and careful comparison	B2
Data conversion to software	C2	Check unit compatibility and seek expert assistance with software	B3
Loss of measured data or complete loss of the project	C3	Multiple backups using cloud storage and automated protocols	A2
Forgetting references (risk of getting a zero)	C3	Use plagiarism software and systematically encode references	A2
Failed tests requiring repetition	D3	Clear action plan with contingency time included in the schedule	B2
Poor time estimation for the tests	E3	Make high estimates, consult experienced colleagues	B3
Poor estimation of lab availability	C3	Contact lab managers early, keep updated on lab availability	B2
Poor estimation of the number of necessary tests	C2	Adapt the plan and comment on variability where applicable	C2
Poor time estimation for writing/reviewing	D3	Plan ahead, seek advice on realistic time-frames	C3
Loss/theft of the computer	D3	Multiple backups using cloud storage and automated protocols	B2
Inadequate measurement equipment or exceeded precision/maximum range	E3	Research suitable equipment, conduct preliminary tests, ensure a safety margin	B2

Table 4.2: Project Success Risk Assessment: Mitigation Measures and Updated Risk Codes

Analysis: The risks in this table primarily affect the reliability of the experimental results and the timeline of the project. Implementing strategies like regular instrument calibration, maintaining backups, and effective time management are essential in mitigating these risks. For example, risks like data loss (initial code C3) can be reduced to A2 with proper backup strategies. By prioritizing these control measures, the likelihood of project failure is minimized, leading to a smoother workflow and more accurate results.

Finally an acceptable risk is defined as one where the likelihood and severity are minimized to a point where the benefits of conducting the experiment outweigh the potential downsides. Through rigorous assessment and the implementation of targeted mitigation strategies, risks that initially seemed critical can be brought down to an acceptable level. For instance, the initial risk of losing crucial data was identified as C3, but with multiple backup strategies, the risk is brought down to A2, making it acceptable.

In conclusion, the approach used in these tables demonstrates the effectiveness of structured risk management. By categorizing risks into clear modules and systematically applying mitigation measures, the experiment can proceed with a high degree of safety and reliability. This comprehensive risk management plan is a key factor in ensuring both the success of the project and the well-being of all personnel involved.

Chapter 5

Vibro-Driving

5.1 Experimental Setup

This section describes the procedures for setting up, adjusting, and conducting measurements during experiments designed to simulate real-world offshore conditions. By replicating techniques used in offshore environments, the laboratory setup aims to evaluate the effects of vibro-driving on soil resistance, pile stability, and overall system performance like vibro-driving effects. Additionally, alternative configurations are explored to optimize setup efficiency and improve experimental outcomes, providing insights into potential improvements for large-scale applications.

5.1.1 The Vibrodriver Platform

The vibrodriver device, initially developed by Luc Simonin for centrifuge experiments in the TU Delft labs (Netherlands), was adapted for this study:

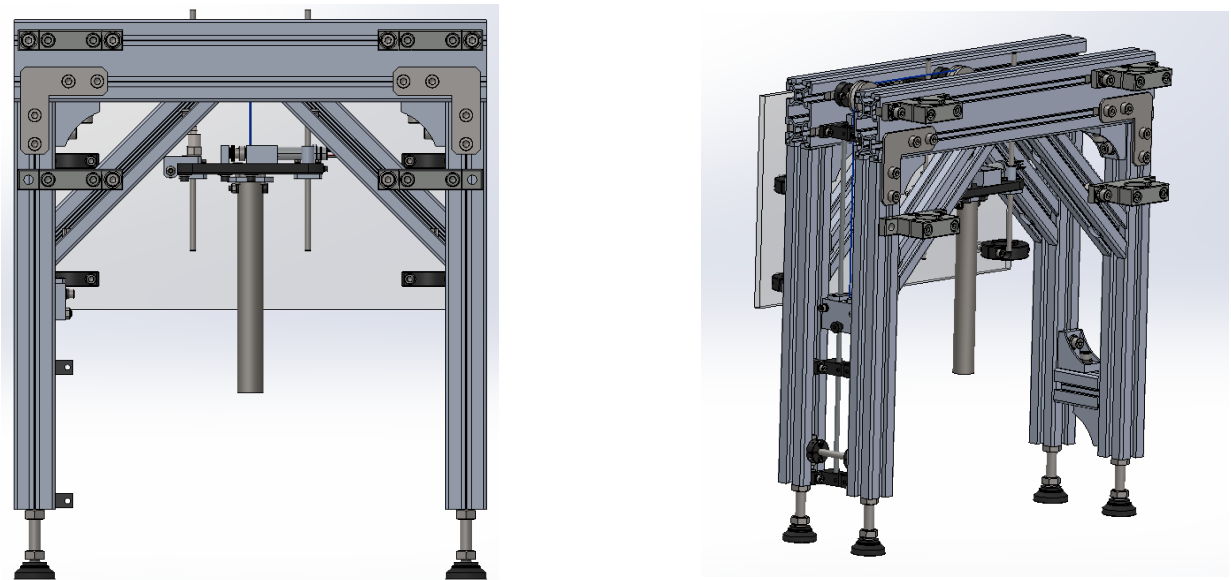


Figure 5.1: Structure of the vibrodriver designed by Luc Simonin.

For this study, only the small platform (14 cm by 6 cm) was used, connected to a motor (*ECX SPEED 13 L: $\phi 13$*

mm, *Moteur BLDC*) capable of reaching a maximum speed of 70,000 rpm without eccentric masses (see motor specifications in Appendix C).

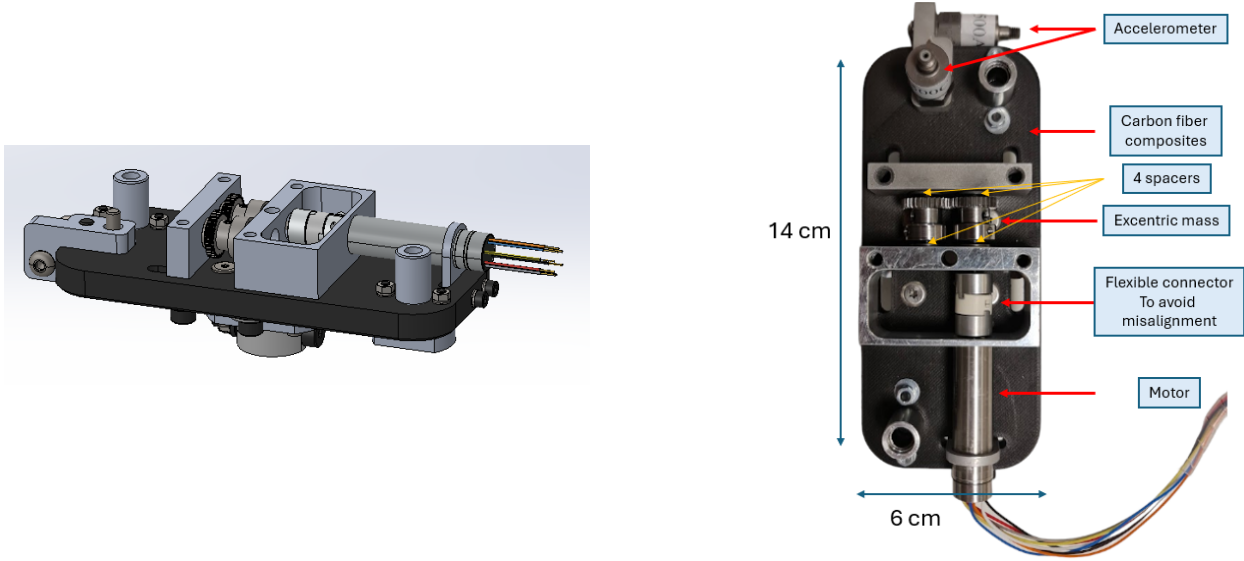


Figure 5.2: 3D design and schematic of the vibrodriver platform used for pile driving.

Once the motor associated to the eccentric masses, the platform's operation is limited to 60,000 rpm for the $4\text{ g} \cdot \text{mm}$ eccentric mass and 35,000 rpm for the $8\text{ g} \cdot \text{mm}$ mass. This limitation directly affects our ability to control the vertical force, which can be expressed with a value of:

$$F_{v,max} = 2 \cdot m_e \cdot (2 \cdot \pi \cdot f)^2 = 315.8N \quad (5.1)$$

Where :

- m_e is the eccentric mass = $4\text{ g} \cdot \text{mm}$
- f = the frequency = 1000 Hz = 60,000 rpm

If the same vertical force is desired with the larger eccentric mass ($8\text{ g} \cdot \text{mm}$), the frequency must be reduced by a factor of $\sqrt{2}$, resulting in 707.1 Hz or 42,426 rpm. However, this speed is not achievable due to the motor's torque limitations.

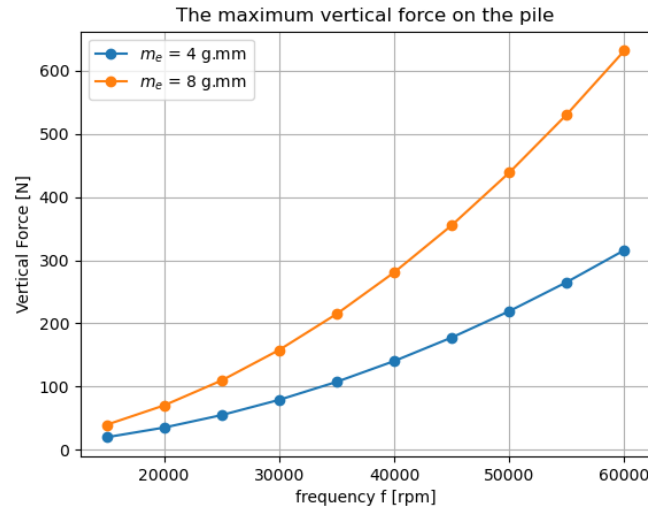


Figure 5.3: Vertical force on the top of the pile vs the frequency.

The next step involved connecting the platform to the pile. With precise measurements provided by Benoît Herman and Simon De Jaeger (CREDEM), a custom 3D-printed plastic connector was designed to secure the pile during driving.

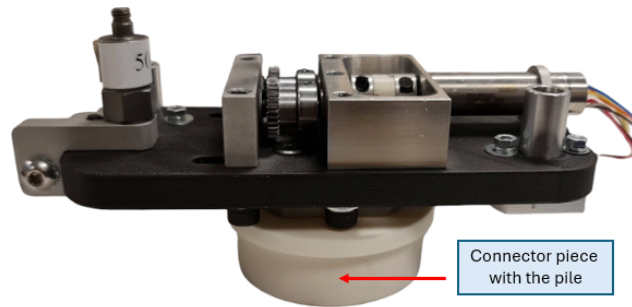


Figure 5.4: 3D-printed connector between the vibrodriver platform and the pile.

Finally, the electrical connections and power supply were set up, allowing motor control via ESCON Studio software (see Appendix C for the download link and configuration settings).

In this part, two key element are distinguished, the Arduino which will control the (on/off) of the system, in our case rotating the plate by 90 degree will run the system. The second key is the controller of the motor (*ESCON 50/5 (409510)*) which is linked to the PC by a USB cable and will control the motor, and both of them are supplied by a generator of 24V and 48V as shown in the figure below.

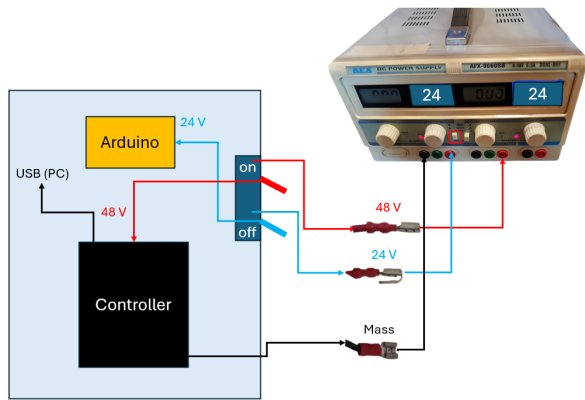


Figure 5.5: Schema for the electronic plate and the supply

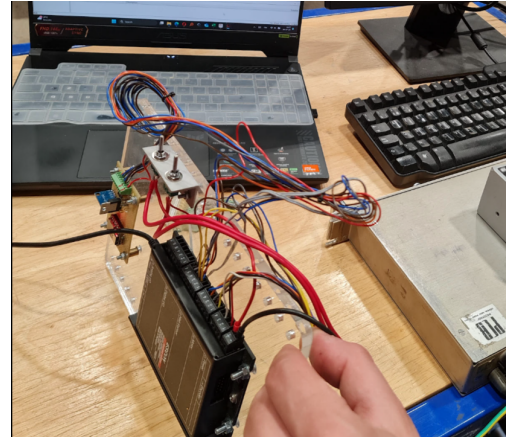


Figure 5.6: Rotating the plate by 90 degree will activate the system

5.1.2 Scaled setup for vibro-driving

This setup takes inspiration from the offshore wind farm project at Calvados (See Figure 5.8), where a combination of pre-drilling and vibro-driving techniques is employed for monopile installation. The project plans to install 64 Offshore Wind Turbines (OWTs) with monopiles weighing between 800 and 1,000 tons and diameters reaching up to 7.75 meters. The installation process begins with drilling a hole and placing an annular ring, followed by partially filling the cavity with compacted sand and gravel. This preparation is key to reducing seabed resistance and enhancing monopile stability during insertion via vibro-driving.



Figure 5.7: Vibro-driving test conducted in a 35 cm sand-filled reservoir, simulating offshore monopile installation.

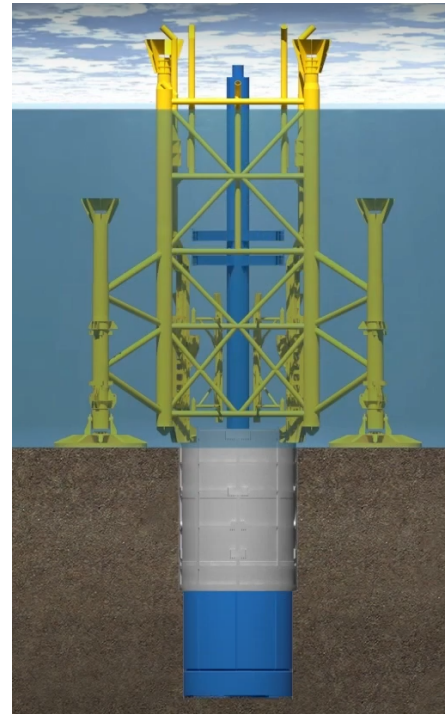


Figure 5.8: The Calvados OWT project : Drilling seabed [38]

The boundary conditions in this setup differ significantly from those in a larger tank. However, the smaller tank was used strategically to accelerate the testing process and evaluate vibro-drivability. This setup enabled to perform 22 embedment tests under dry conditions, varying frequencies, eccentric masses, and three tests under saturated conditions. The detailed results of these experiments are discussed in the next section.

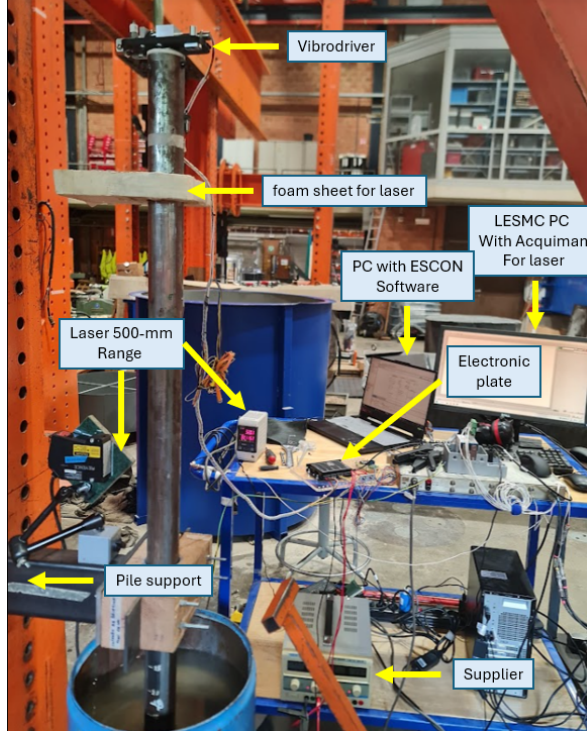


Figure 5.9: All the setup with small tank



Figure 5.10: Cables are fixed with a tap for security

5.2 Numerical analysis

5.2.1 Embedment tests vs time

After preparing the setup and linking the laser to the system, embedment data was recorded over time using the *Acquilman* software and later converted to a *.csv* format. A Python visualization tool (*Embedement_Viewer_vcsv1.py*) was then developed to facilitate easy analysis of the data (See Figures 5.11 and 5.12) :

Figure	eccentric mass	frequency (krpm)						
5.11	$m_e = 4g.mm$	15	20	30	40	50	55	60
5.12	$m_e = 8g.mm$		25	30	30 (sat)			32

Table 5.1: Vibro-driving tests for the $\Phi 6$ cm pile

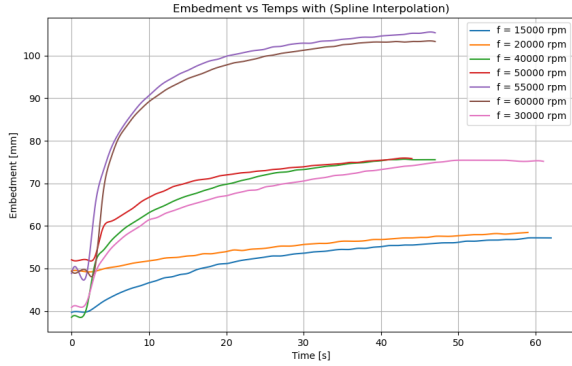


Figure 5.11: $\Phi 6$ cm pile embedment in dry condition with the 4 g.mm eccentric mass.

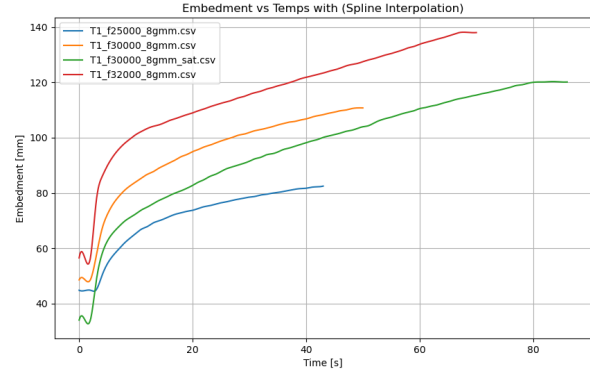


Figure 5.12: $\Phi 6$ cm pile embedment in dry condition with the 8 g.mm eccentric mass and one test in saturation.

5.2.2 Analysis of Embedment Behavior during Vibro-Driving

For both eccentric masses tested, the embedment depth increases rapidly within the first 10 seconds before stabilizing. However, the tests reveal a challenge: to meet the required L/D ratio, an embedment of 180 mm is needed, which would require more than 2 minutes of operation a duration that risks overheating or damaging the motor.

Additionally, the embedment in saturated soil is slower compared to dry conditions, potentially influenced by several factors:

- **Confinement Effects:** In a small tank, soil confinement is higher, leading to stiffer behavior. This confinement can result in an increased soil resistance, especially in saturated conditions where water increases the confinement effect. Moreover, the reflection of stress waves from the tank walls can interfere with natural soil responses, unlike in larger setups where such reflections are minimal.
- **Pore Water Pressure:** Vibrations during driving generate excess pore water pressure, reducing effective stress and temporarily decreasing soil shear strength. However, this also increases resistance to penetration due to the interaction between water and soil grains, leading to slower embedment.
- **Damping Effect of Water:** Water within the soil absorbs vibration energy, reducing the energy available for displacing soil particles. In dry conditions, energy transfer is more efficient, resulting in faster embedment.
- **Soil Particle Rearrangement:** In dry conditions, soil particles can rearrange freely under vibrations. In contrast, water hinders this rearrangement in saturated conditions, slowing down the soil's response during driving.

5.2.2.1 Future Considerations

For future work, it's advisable to focus on tests involving saturated conditions. The small quantity of sand used allows for quick drying, enabling multiple tests to be conducted efficiently as the sand dries between trials.¹

¹This page was written with the help of ChatGpt

5.2.3 The embedment vs the frequency

The chart below analyzes the total pile embedment as a function of frequency over a 100-second duration, using 4 g.mm and 8 g.mm eccentric masses.

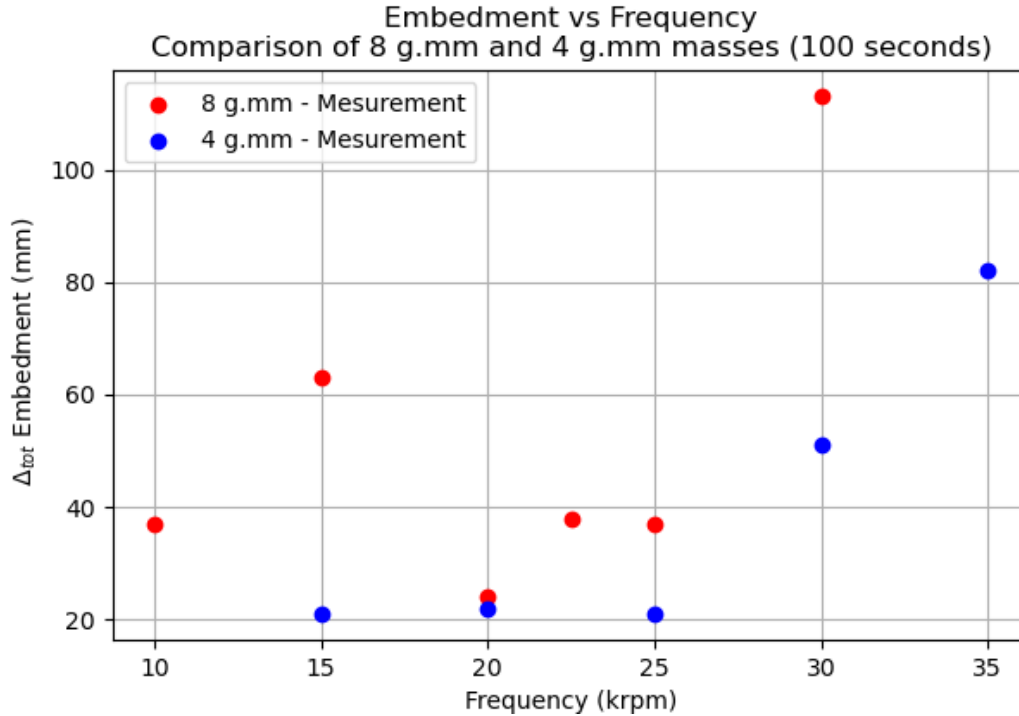


Figure 5.13: Embedment vs frequency during 100 seconds for the 4 g.mm and 8 g.mm eccentric masses

In these tests, the vertical vibratory force increases quadratically with the frequency. The increase of the final embedment with frequency above 20krpm thus appears logical.

However, a particularly interesting observation was made at 15 krpm, where the system exhibited noticeable vibrations, likely indicating resonance. This resonance may have caused a reduction in soil resistance, thereby facilitating greater embedment compared to other frequencies (as discussed with Luc Simonin [47]). The vibration might induces a temporary loss of grain-to-grain contact, reducing friction and therefore lateral and tip resistance, which in turn facilitates pile penetration.

It is important to note that this resonance frequency is specific to this lab setup and cannot be generalized. However, previous studies, such as those cited by (Rodger and Littlejohn 1980 [21]), have explored the concept of utilizing resonance to improve vibro-driving performance by adjusting the vibro-driver frequency to the pile's natural frequency or the pile-soil system frequency. For instance, Barkan 1959 attempted to enhance vibratory driver capacity by leveraging pile-soil resonance, leading to the development of machines designed on this principle.

In summary, these observations highlight the critical role of resonance in vibro-driving, emphasizing the need to consider these effects when optimizing installation performance in various soil conditions.

Chapter 6

Conclusion

As the European Union aims to transition to renewable energy sources, offshore wind energy plays a critical role in meeting its ambitious climate targets. With vast wind resources available offshore, the deployment of offshore wind turbines (OWTs) is key to increasing clean energy production. Among the various foundation options, monopiles have become the preferred solution due to their simplicity, cost-effectiveness, and scalability.

However, the installation of monopiles is a complex challenge that requires balancing efficiency with environmental impact. Traditionally, impact hammering has been the standard method for driving monopiles into the seabed. While reliable, this technique has several drawbacks, including significant underwater noise pollution that can harm marine life, extended installation times, and potential structural fatigue on the monopile.

In response to these challenges, vibro-driving has emerged as an alternative installation method. Vibro-driving offers potential advantages, such as faster installation, lower noise levels, and reduced mechanical stress on the pile. Despite these benefits, the precise behavior of vibro-driven monopiles remains less understood compared to conventional methods. Given the high stakes associated with offshore installations, it is essential to conduct controlled laboratory experiments to systematically compare these techniques and identify the conditions under which each method performs optimally.

6.1 Key Findings

This thesis has investigated the comparative effectiveness of impact hammering and vibro-driving techniques for the installation of offshore wind turbine monopiles. Through a series of controlled laboratory experiments, valuable insights were gained into the mechanical behavior of monopiles under varying conditions. The data collected were meticulously analyzed using Python tools specifically developed for this study, which enabled real-time visualization, filtering, and detailed interpretation of the results.

- **Design of a New Pluviation System:** A novel sand pluviation system was designed to achieve consistent sand deposition in the test tank, ensuring uniform density and reproducible test conditions. This system significantly improved the accuracy of the experimental results and allowed for more reliable comparisons

between different installation methods.

- **Development of a P-y Curve Analysis Tool:** A Python-based tool was developed for generating p-y curves, a critical component in offshore geotechnical engineering. This tool was tested using data from previous impact hammering studies, demonstrating its utility in predicting lateral soil resistance and pile deflection. The tool's adaptability for use with vibro-driving data offers a robust platform for future research.
- **Adaptation and Development of the Vibro-driver Setup:** The vibro-driver apparatus was adapted and optimized for use in laboratory conditions. This included modifications to the setup to accommodate different L/D ratio, eccentric masses, and vibration frequencies. The customized setup provided a controlled environment to study the effects of vibro-driving on monopile embedment and stability.
- **Effect of Frequency on Embedment:** The experiments revealed that monopile embedment increases rapidly within the first few seconds of vibro-driving before stabilizing. Higher frequencies generally resulted in greater embedment, although the rate of increase diminished beyond a certain threshold, indicating an optimal frequency range for efficient pile installation.
- **Impact of Saturation:** Tests conducted in saturated soil conditions showed slower embedment rates compared to dry conditions. This phenomenon is likely due to increased pore water pressure, higher soil viscosity, and the damping effect of water, which collectively reduce the efficiency of the vibro-driving process. The boundary conditions of the small tank may have amplified these effects, introducing additional complexities such as soil confinement and wave reflections.
- **Resonance Effects:** A resonance phenomenon was observed around 15,000 rpm, resulting in visible system vibrations and enhanced embedment efficiency. This aligns with theoretical studies on pile-soil resonance and suggests that leveraging resonance effects could optimize vibro-driving for specific applications.

6.2 Future Perspectives

Building on the insights gained from this study, several areas for future research and technical enhancements have been identified. First, further saturation tests are critical to fully understand the behavior of vibro-driven piles in realistic offshore conditions. This includes examining a broader range of vibration frequencies and eccentric masses, as well as improving the vibro-driver system by incorporating a more powerful motor. Addressing the current torque limitations and preventing overheating will enhance the reliability and effectiveness of the tests, leading to more robust conclusions about the vibro-driving method.

In terms of experimental setup, improvements to the sand pluviation system should be prioritized. Implementing multiple mesh diffusers with varying hole sizes would allow for better control over sand density, enabling a more precise replication of different soil conditions. Such enhancements would significantly improve the reproducibility and accuracy of the experimental results.

Additionally, the current setup can be optimized by introducing a hinged mechanism for the actuator system. This modification would allow for the easy replacement of piles without requiring the rotation of the tank, which can disturb the sand and affect the test results. This adjustment would streamline the experimental process while maintaining the integrity of the sand bed.

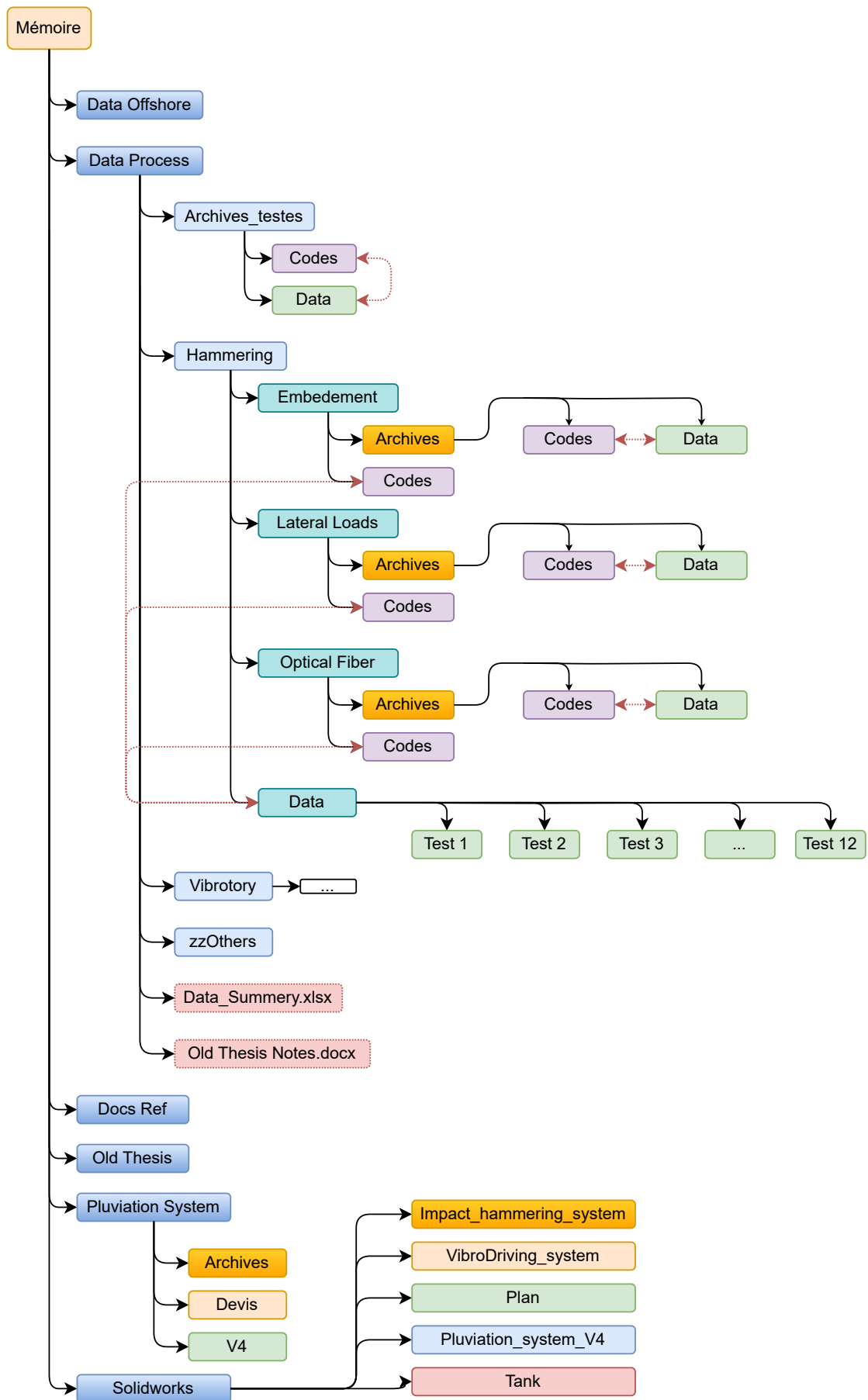
Finally, exploring the application of lateral loading on vibro-driven piles is a crucial next step. Coupling these tests with the p-y curve analysis tool developed in this study would provide deeper insights into the lateral behavior and overall performance of monopiles under various installation techniques. Such research would contribute to refining design methodologies and advancing the practical application of vibro-driving in offshore engineering.

6.3 Conclusion

In conclusion, this research has contributed to the field of offshore monopile installation by demonstrating the practical applicability of vibro-driving under controlled laboratory conditions. The development of specialized tools, proposed system modifications, and comprehensive data analysis have laid the groundwork for future investigations. The findings of this thesis hold promise for optimizing vibro-driving techniques, thereby supporting the sustainable and efficient development of offshore wind energy projects.

6.4 Folder tree

The following page presents the folder structure used throughout this project. This hierarchical organization serves as a clear roadmap for accessing various project components, including datasets, Python scripts, results, and documentation. Maintaining a well-organized file structure is essential for reproducibility, efficient navigation, and collaboration, ensuring that each step of the research process can be easily traced and understood.



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Appendix A

Container Sheet Calculation

A.1 Stress and Pressure Calculations in the Container

To design the container, a 2 mm thick steel plate with a width of 125 cm was used. This thickness was selected based on the availability of materials and the capability of laboratory equipment. The chosen thickness is sufficient to support the load of wet sand.

A.1.1 Vertical Stress in the Soil

The vertical stress in the soil at a given depth z is calculated as:

$$\sigma_v = \gamma_{\text{sat}} \cdot z = \sigma'_v + u$$

where:

- γ_{sat} is the saturated unit weight of the soil.
- z is the depth below the surface.
- σ'_v is the effective vertical stress.
- u is the pore water pressure.

A.1.2 Horizontal Pressure in the Soil

The horizontal pressure exerted by the saturated soil on the container walls is:

$$p_{\text{sat, soil}} = \sigma_h = K_a \cdot \sigma'_v + u = K_a \cdot \sigma_v + (1 - K_a) \cdot u$$

where:

- K_a is the active lateral earth pressure coefficient, calculated using Rankine's theory:

$$K_a = \tan^2 \left(\frac{\pi}{4} - \frac{\phi}{2} \right)$$

- ϕ is the internal friction angle of the soil.

Which gives us a $p_{\text{sat, soil}} = 15.6 \text{ kPa}$

A.1.3 Circumferential Stress in the Container Wall

The circumferential stress (hoop stress) in the container wall can be determined by:

$$\sigma_{\theta} = \frac{p_{\text{sat, soil}} \cdot R}{t}$$

where:

- R is the radius of the container.
- t is the thickness of the container wall.

And so $\sigma_{\theta} = 4.3 \text{ Mpa}$ which is significantly bellow the yield strength of steel (235 MPa). Consequently, it can be inferred that a 2 mm thickness for the steel wall of the container is sufficient to withstand the lateral pressure exerted by the saturated sand as mentioned by (Cedric and Maxence) [37].

A.1.4 Application with North Sea Sand

Using the following properties of North Sea sand:

$$\gamma_{\text{sat}} = 18.9 \text{ kN/m}^3, \quad \phi = 34^{\circ}, \quad R = 550 \text{ mm}, \quad t = 2 \text{ mm}$$

the vertical stress, horizontal pressure, and circumferential stress on the container can be calculated.

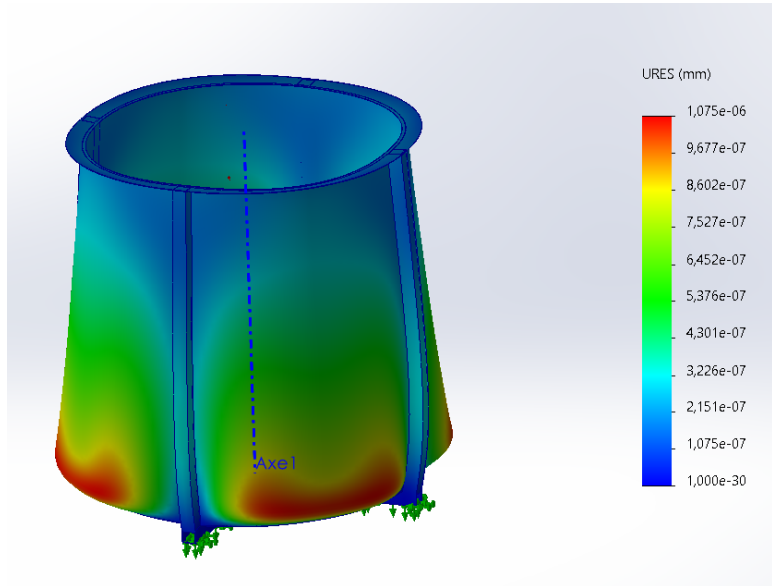


Figure A.1: Simulation of container sheet deformation (SolidWorks)

Appendix B

Impact hammering IQIP

B.1 EQ-Pilling

EQ-piling offers significant benefits for monopile installation projects. First of all, it significantly reduces noise by making bubbles that surround the installation zone. That means more efficient and less complex installations, lower costs, reduced energy consumption, and simplified project management.

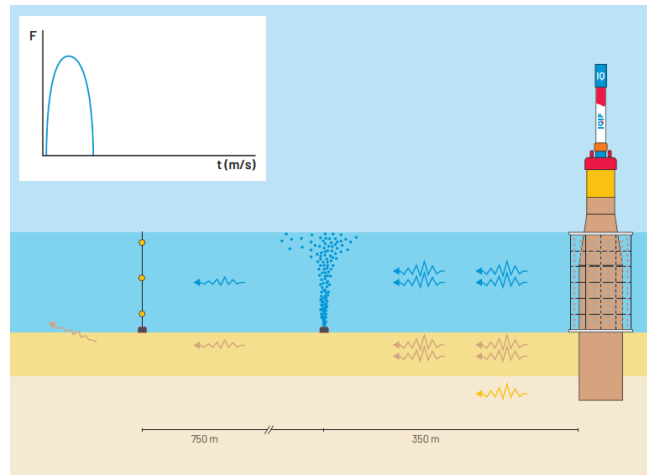


Figure B.1: EQ-Pilling system that generate bubbles around the monopile

B.2 PULSE

PULSE, a modular addition to the Hydrohammer, minimises noise during pile driving to deliver an environmentally friendly foundation installation. The elongated blow of PULSE is twice the duration of the blow of a conventional hammer, increasing piling efficiency while reducing pile fatigue and impact noise.

	PULSE RESULTS		PULSE EXPECTATIONS	
	S-90	IQ2	IQ4	IQ6
Weight PULSE	1 ton	125 ton	125 ton	125 ton
Height PULSE (add)	1 m	3.6 m	3.6 m	3.6 m
Noise reduction (SEL)*	6 - 10 dB	6 - 10 dB	6 - 10 dB	6 - 10 dB
Noise reduction (SPL)	10 - 12 dB	5 - 12 dB	5 - 12 dB	5 - 12 dB
Fatigue*	up to -60% improvement**	up to -60 % improvement**	up to -60 % improvement**	up to -60 % improvement**
Installation efficiency / blow	up to 10 % (depending on soil/pile)	up to 10% (depending on soil/pile)	up to 10% (depending on soil/pile)	up to 10% (depending on soil/pile)

Figure B.2: Reducing environmental impact with the modular PULSE

Appendix C

Motor specifications

Here are the data provided by Maxon for the used motor, they are necessary for parameterise the ESCON Studio software and here is a link for the software download and the parametrization :

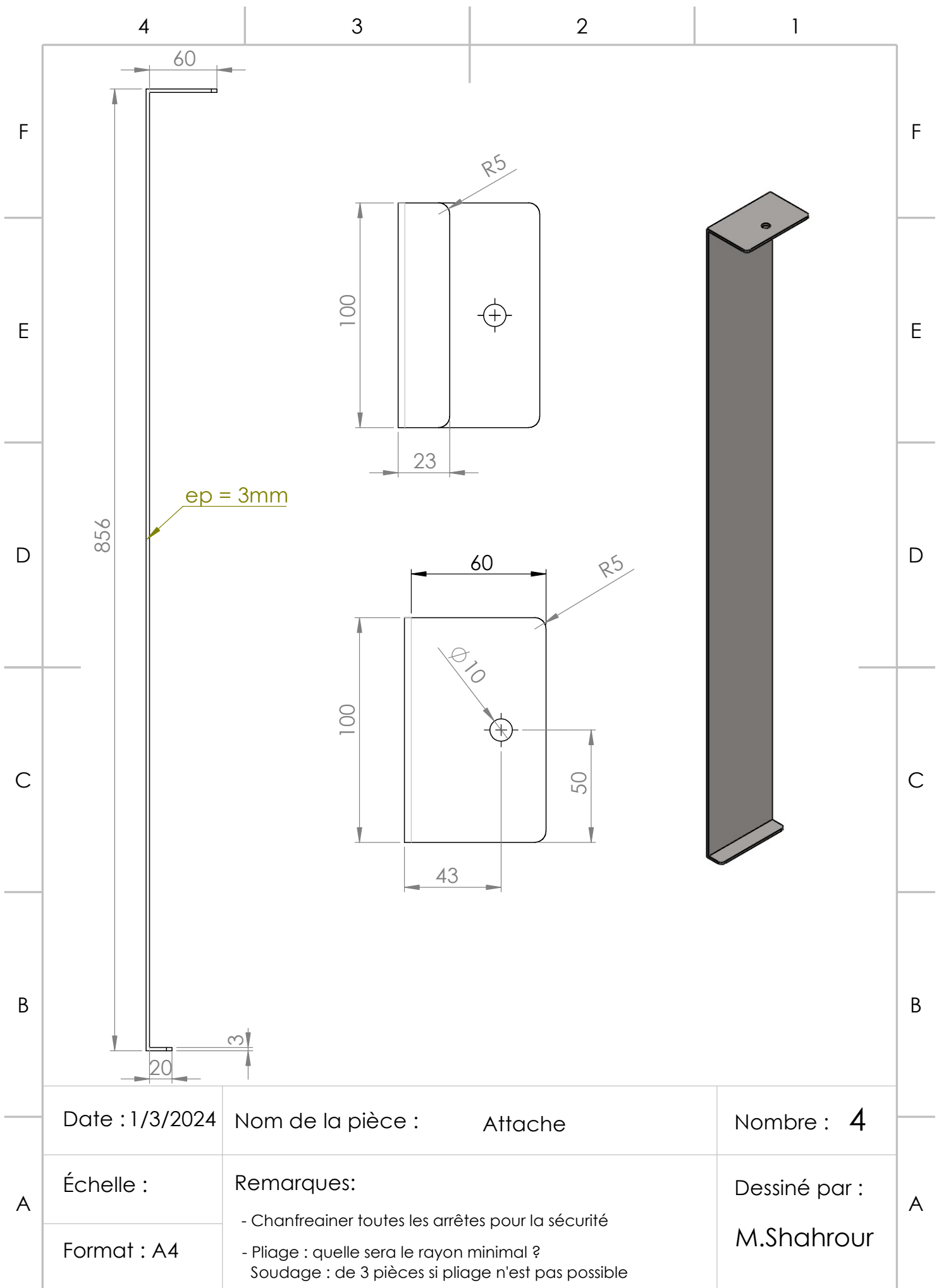
- Download link for the software : https://www.maxongroup.com/medias/sys_master/root/9122561261598/ESCON-Setup.zip
- parametrization link : <https://photos.app.goo.gl/ExxvwDkfu1n933ir7>

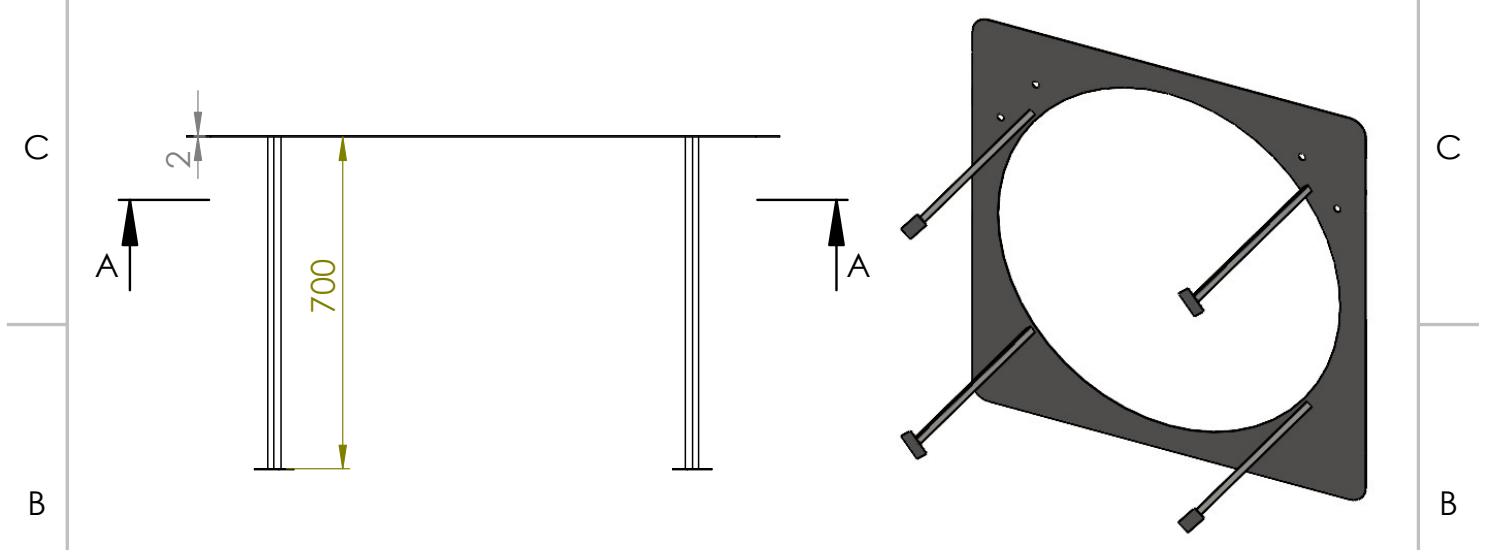
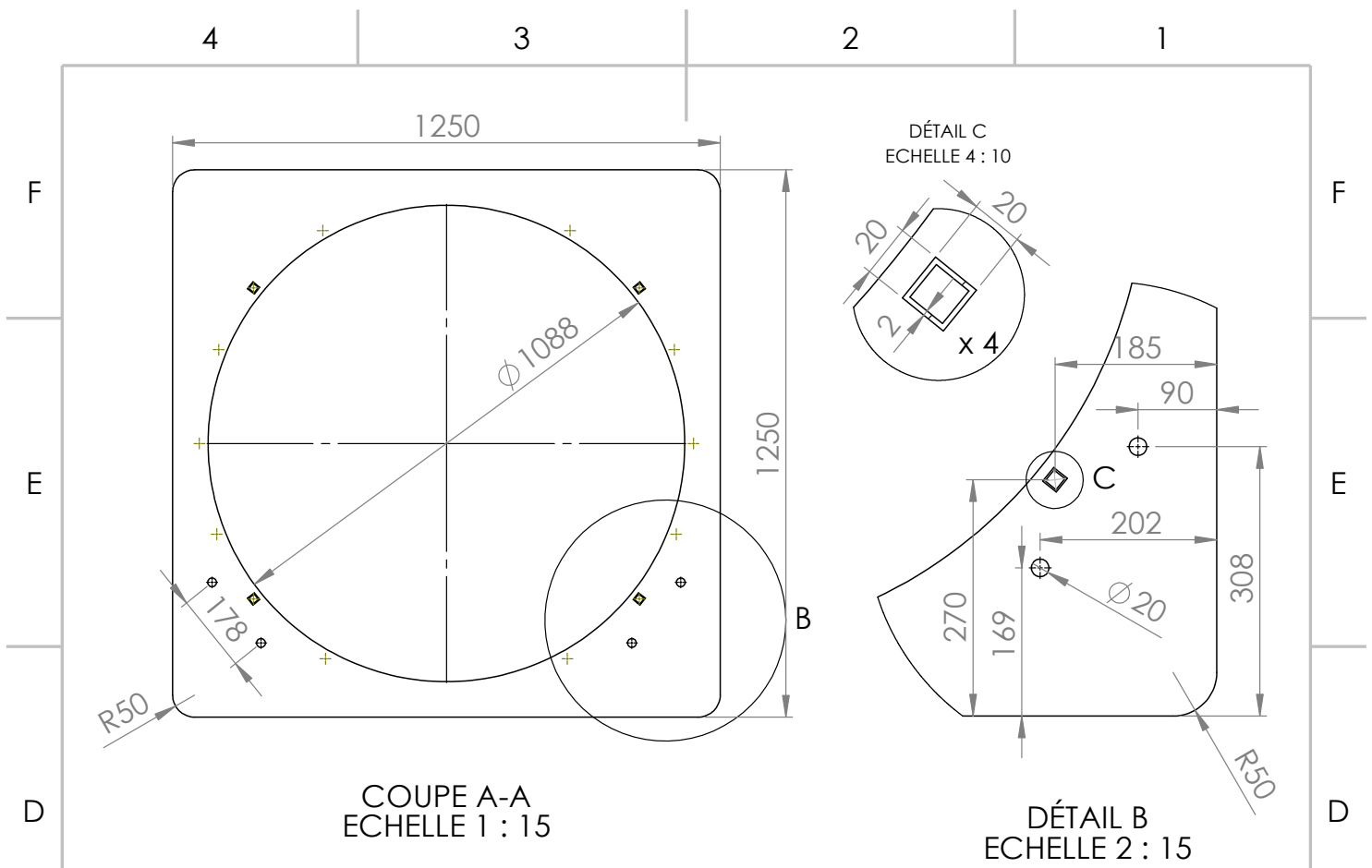
Paramètres du moteur					
1_ Tension nominale	V	18	24	36	48
2_ Vitesse à vide	tr/min	67900	66400	67900	62300
3_ Courant à vide	mA	223	161	112	72.1
4_ Vitesse nominale	tr/min	64400	63100	64600	59000
5_ Couple nominal (couple max. permanent)	mNm	6.89	7.15	6.88	7.12
6_ Courant nominal (courant charge perm. max.)	A	2.93	2.22	1.46	1.03
7_ Couple de démarrage	mNm	149	161	161	151
8_ Courant de démarrage	A	59.1	46.8	31.9	20.5
9_ Rendement max.	%	88.3	88.8	88.7	88.7
10_ Résistance aux bornes	Ω	0.304	0.512	1.13	2.34
11_ Inductance aux bornes	mH	0.012	0.0223	0.0479	0.101
12_ Constante de couple	mNm/A	2.52	3.44	5.04	7.33
13_ Constante de vitesse	tr/min/V	3790	2780	1890	1300
14_ Pente vitesse/couple	tr/min/mNm	457	414	424	415
15_ Constante de temps mécanique	ms	1.67	1.51	1.55	1.52
16_ Moment d'inertie du rotor	gcm ²	0.349	0.349	0.349	0.349

Caractéristiques thermiques		
17_ Résistance therm. boîtier/air ambiant	K/W	24.4
18_ Résistance therm. bobinage/boîtier	K/W	1.6
19_ Constante therm. temps de bobinage	s	1.68
20_ Constante therm. de temps moteur	s	411
21_ Température ambiante	°C	-20...+100
22_ Température max. du bobinage	°C	155
Caractéristiques mécaniques roulement à billes		
23_ Vitesse max. admise	tr/min	70 000
24_ Jeu axial	mm	0...0.28
Précontrainte	N	1.5
Sens de la force		traction
25_ Jeu radial		précontraint
26_ Charge axiale max. (dynamique)	N	1.5
27_ Force de chassage axiale max. (statique) (arbre soutenu)	N	50 1500
28_ Charge radiale max. [mm du flasque]	N	6 [5]
Autres spécifications		
29_ Nombre de paires de pôles		1
30_ Nombre de phases		3
31_ Poids du moteur	g	33.7
32_ Niveau sonore typique [tr/min]	dBA	47 [50 000]

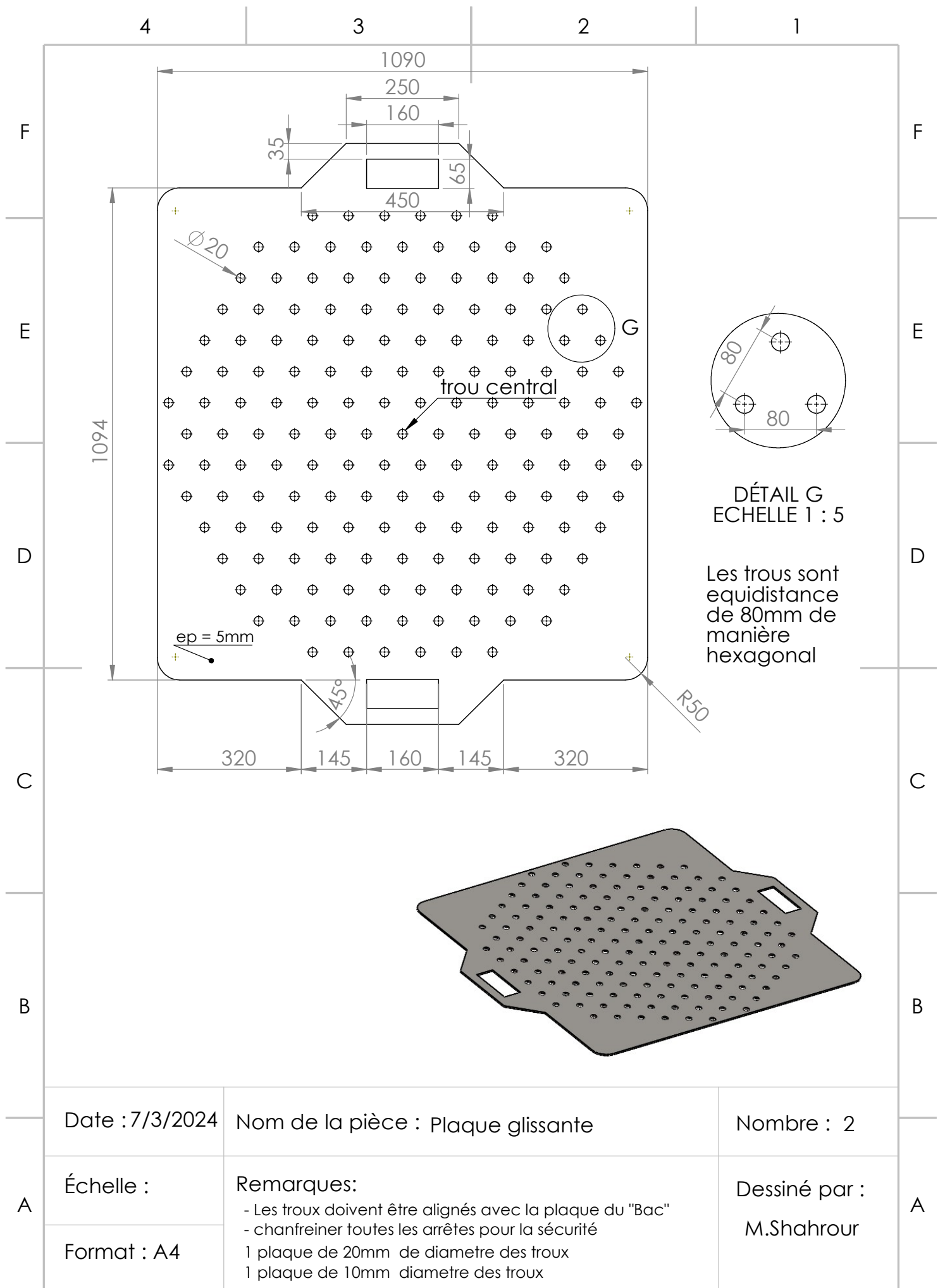
C.1 Controller specification

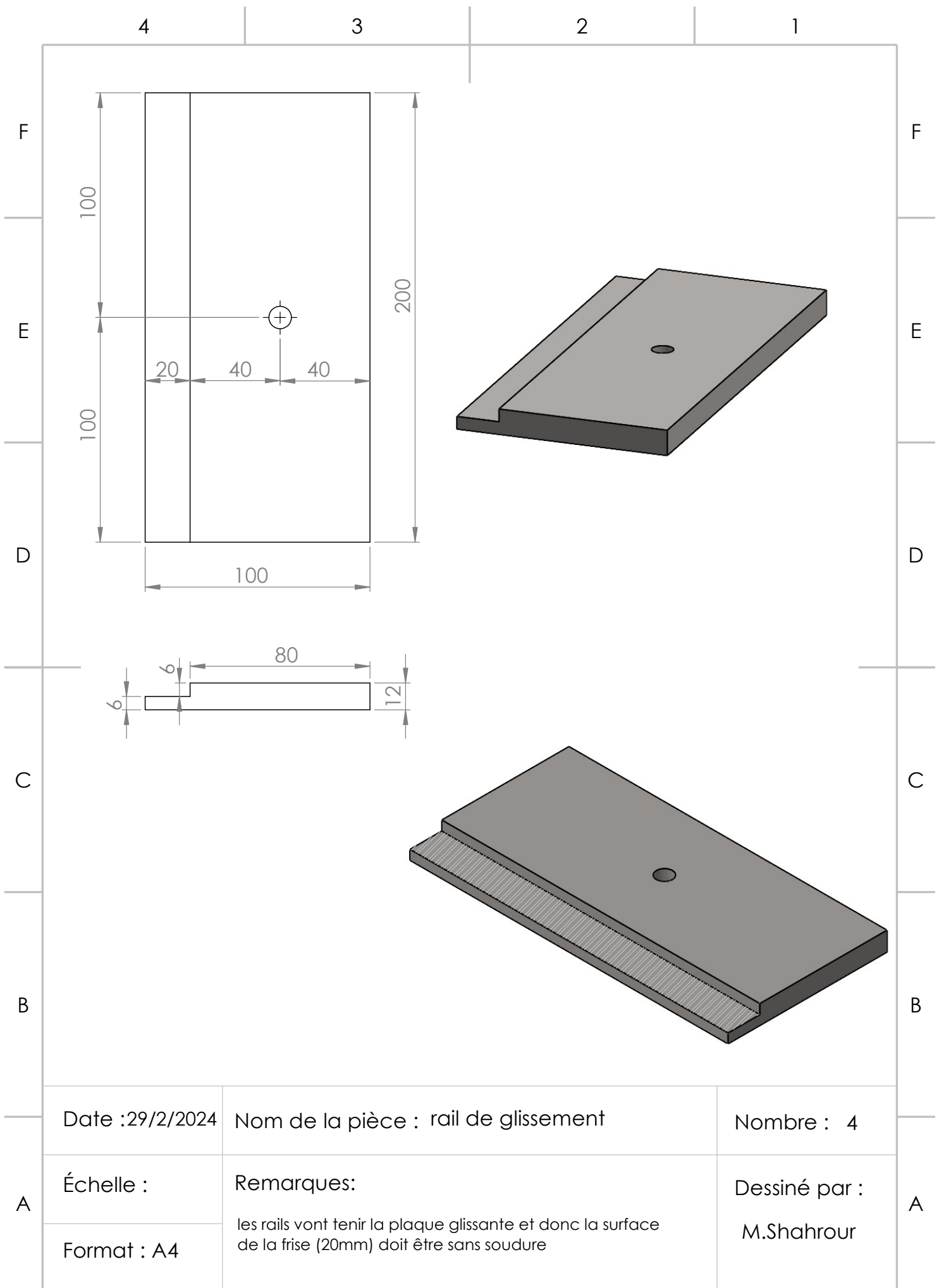
ESCON 50/5 (409510)		
Electrical Rating	Nominal operating voltage $+V_{CC}$	10...50 VDC
	Absolute operating voltage $+V_{CC\ min} / +V_{CC\ max}$	8 VDC / 56 VDC
	Output voltage (max.)	$0.98 \times +V_{CC}$
	Output current I_{cont} / I_{max} (<20 s)	5 A / 15 A
	Pulse Width Modulation frequency	53.6 kHz
	Sampling rate PI current controller	53.6 kHz
	Sampling rate PI speed controller	5.36 kHz
	Max. efficiency	95%
	Max. speed DC motor	limited by max. permissible speed (motor) and max. output voltage (controller)
	Max. speed EC motor	150'000 rpm (1 pole pair)
	Built-in motor choke	3 x 30 μ H; 5 A
Inputs & Outputs	Analog Input 1 Analog Input 2	resolution 12-bit; -10...+10 V; differential
	Analog Output 1 Analog Output 2	resolution 12-bit; -4...+4 V; referenced to GND
	Digital Input 1 Digital Input 2	+2.4...+36 VDC ($R_i = 38.5\ k\Omega$)
	Digital Input/Output 3 Digital Input/Output 4	+2.4...+36 VDC ($R_i = 38.5\ k\Omega$) / max. 36 VDC ($I_L < 500\ mA$)
	Hall sensor signals	H1, H2, H3
	Encoder signals	A, A $\bar$, B, B $\bar$, (max. 1 MHz)
Voltage Outputs	Auxiliary output voltage	+5 VDC ($I_L \leq 10\ mA$)
	Hall sensor supply voltage	+5 VDC ($I_L \leq 30\ mA$)
	Encoder supply voltage	+5 VDC ($I_L \leq 70\ mA$)
Potentiometers	Potentiometer P1 (on board) Potentiometer P2 (on board)	240°; linear
Motor Connections	DC motor	+ Motor, - Motor
	EC motor	Motor winding 1, Motor winding 2, Motor winding 3
Interface	USB 2.0 / USB 3.0	full speed
Status Indicators	Operation	green LED
	Error	red LED

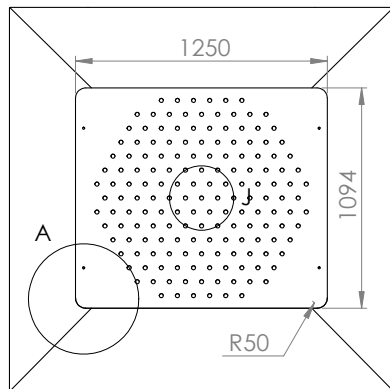




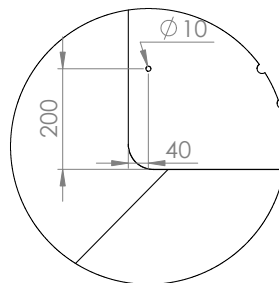
Date : 29/2/2024	Nom de la pièce : Plaque de positionnement	Nombre : 1
Échelle :	Remarques:	Dessiné par :
Format : A4	- pas besoin de faire les tubes uniquement les trous suffisent - Chanfreiner toutes les arrêtes pour la sécurité	M.Shahrour



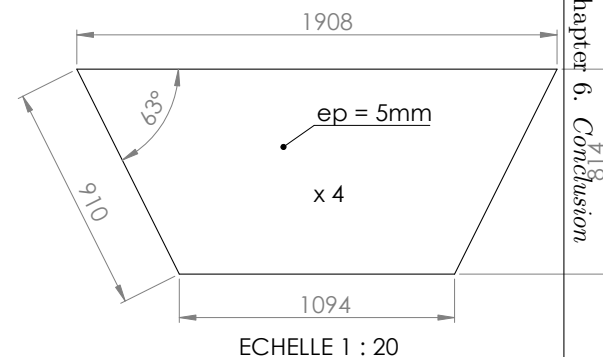
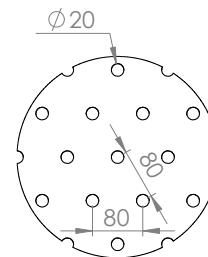




DÉTAIL A
ECHELLE 1 : 10

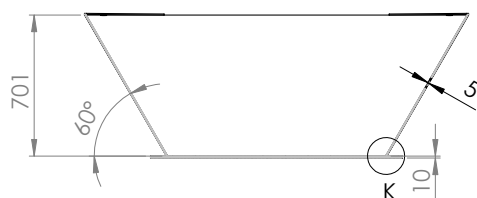


DÉTAIL J
ECHELLE 1 : 8

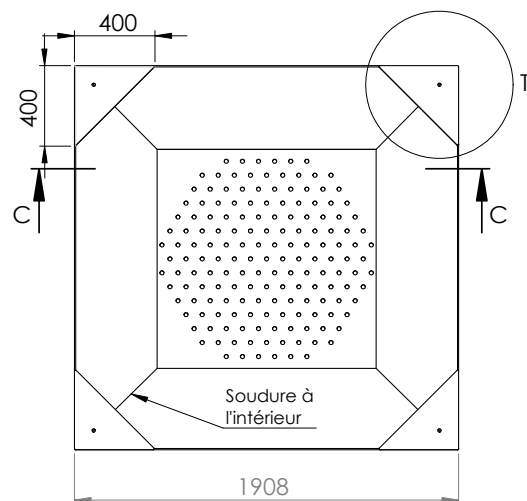
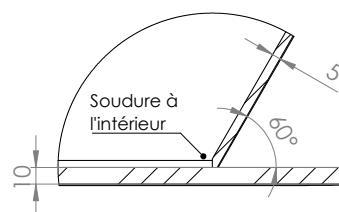


Chapter 6. Conclusion

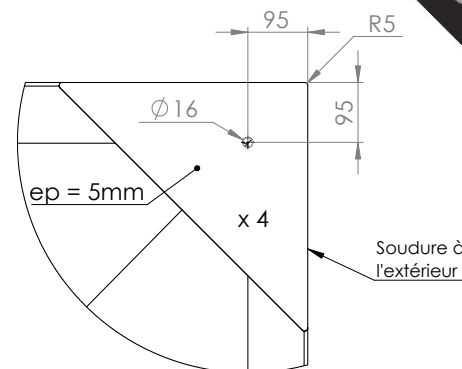
COUPE C-C
ECHELLE 1 : 25



DÉTAIL K
ECHELLE 1 : 3



DÉTAIL T
ECHELLE 1 : 8



Au bilan

4 trapèzes : 5mm
4 triangles : 5mm
une plaque : 10mm
avec trous 20mm tous les 80mm

Date : 7/3/2024

Nom de la pièce : Bac

Nombre : 8

Échelle :

Remarques : deux prix avec et sans les soudures
- nécessiter d'aligner les trous avec les trous du plaque glissante

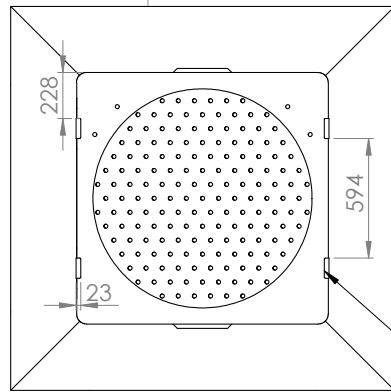
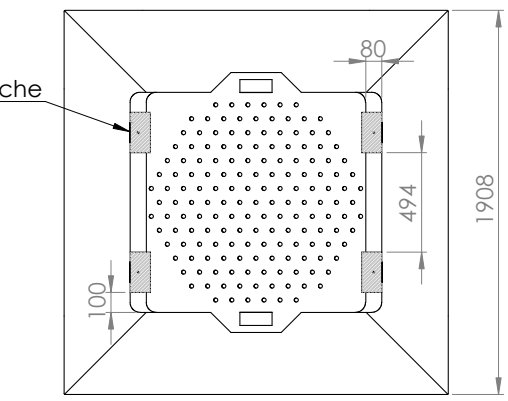
Dessiné par :
M.Shahrour

Format : A3

- Arrondir toutes les coins et
chanfreiner toutes les arrêtes pour la sécurité

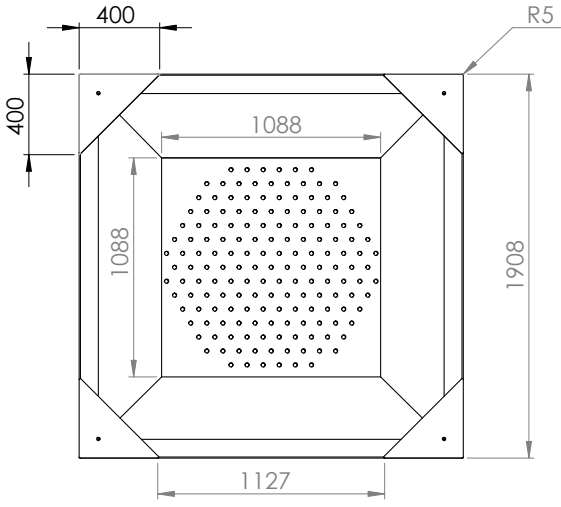
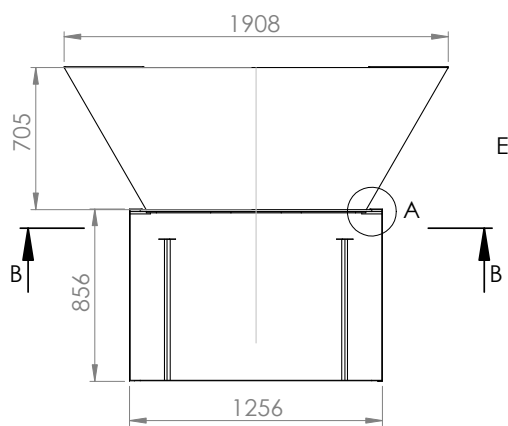
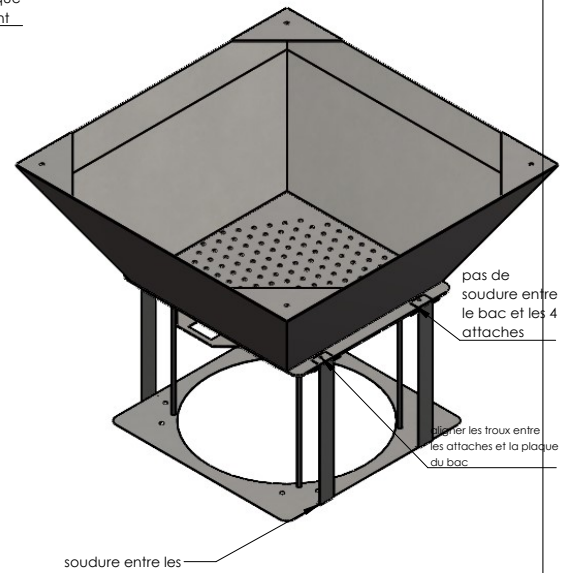
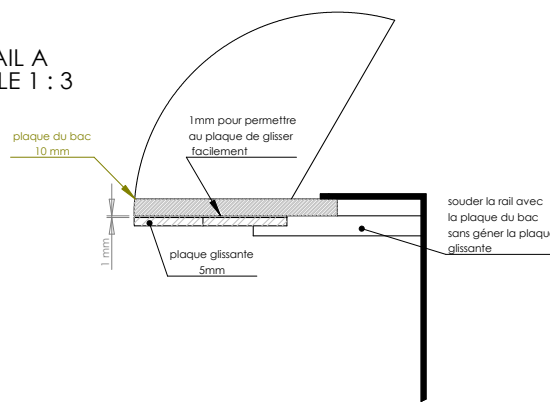
COUPE B-B
ECHELLE 1 : 25

Attache

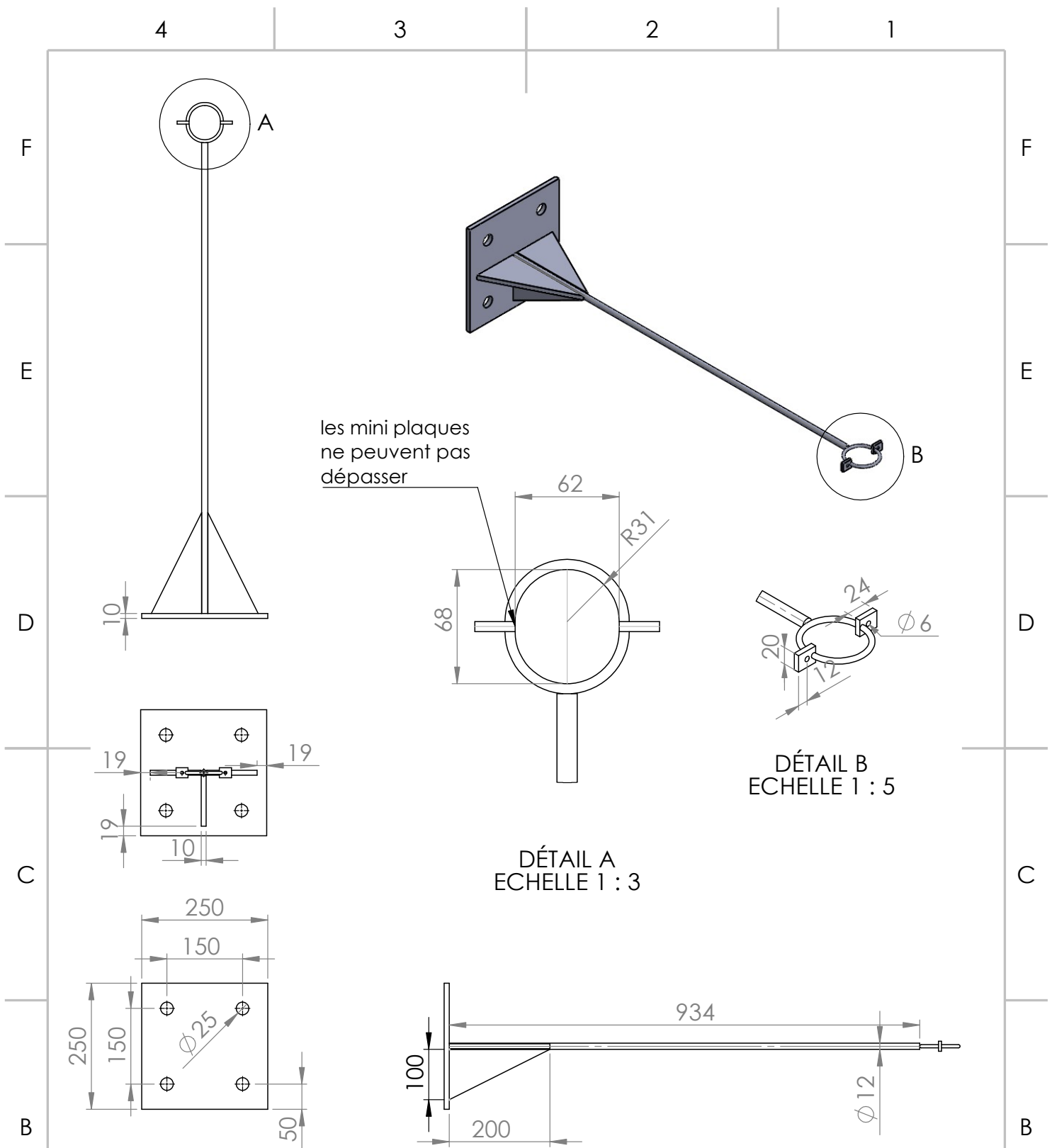


Attachs et la plaque
de positionnement

DÉTAIL A
ECHELLE 1 : 3



Date : 7/3/2024	Nom de la pièce : Le système de pluviation assemblage	Nombre : 1
Échelle :	Remarques : - si c'est possible d'avoir la durée et le prix avec et sans soudure de la fabrication - chanfreiner toutes les arrêtes pour la sécurité	Dessiné par : M.Shahrour
Format : A3		



Date : 17-6-2024	Nom de la pièce : Tige d'attache	Nombre : 2
Échelle : 1:10	Remarques:	Dessiné par :
Format : A4	- il faut que les triangles soient parfaitement angle droit - pour l'anneau il faut just assué une diamètre > 6cm - il ne faut pas souder le tige de Phi 12 (on le ferrera après avoir testé)	Shahrour Mohamed