

École polytechnique de Louvain

Influence of vibratory pile driving on offshore wind turbine mechanical behaviour

Author : Antoine Piersoel
Supervisor : Hadrien Rattez
Readers : **Mohamed Shahrour, Jean-François Vanden Berghe**
Academic year 2025–2026
Master [120] in Civil Engineering

Abstract

Master thesis :

Influence of vibratory pile driving on offshore wind turbine mechanical behaviour

by Antoine Piersoel

As the offshore wind sector continues to expand, monopile foundations remain one of the most widely used support solutions for fixed-bottom wind turbines. Their design is strongly governed by soil–structure interaction, particularly under lateral loading generated by wind, waves and turbine operation. While impact driving is still the reference installation method for offshore monopiles, vibratory driving is increasingly considered as a promising alternative due to its potential advantages in terms of installation time, underwater noise reduction and operational flexibility.

However, the influence of vibratory driving on the mechanical state of the surrounding soil and on the subsequent lateral behaviour of monopile foundations remains less clearly established. Since pile installation can modify the density, stress state and contact conditions around the pile, the installation process should not be considered only as a construction stage, but also as a factor that may affect the later performance of the foundation.

This thesis investigates the influence of impact driving and vibratory driving on the post-installation lateral response of model monopiles in sand. The experimental work is carried out at reduced scale under controlled laboratory conditions. Particular attention is given to the preparation of the sand mass, using a pluviation procedure assessed through cone penetration tests in order to improve the repeatability of the initial soil state. The model piles are then installed using either impact or vibratory driving and subsequently subjected to monotonic lateral loading.

The study focuses on the comparison of the force–displacement response, initial stiffness, moment–rotation behaviour and pile kinematics after installation. In addition to comparing the two installation methods, the influence of key parameters such as embedment ratio, impact-driving configuration and vibratory-driving frequency is investigated. By combining installation observations with post-installation lateral loading tests, this work aims to provide experimental insight into how different driving procedures may influence the mechanical behaviour of monopile foundations in sand.

Acknowledgements

First of all, I would like to thank my supervisor, Prof. Hadrien Rattez, for his guidance, availability and advice throughout this thesis. His feedback helped me to structure the work, refine the experimental approach and interpret the results with the necessary scientific caution.

I would also like to express my sincere gratitude to the people who supported me during the experimental part of this work. In particular, I would like to thank Pauline André, Guillaume Flood-Page and Mohamed Shahrour for their help, their explanations and their valuable advice during the preparation of the tests, the use of the laboratory equipment and the interpretation of the experimental procedure. Their experience and availability were especially valuable in the context of an experimental thesis.

I am also grateful to all the technicians of the LEMSC laboratory for their help, their practical advice and their support during the experimental campaign. Their contribution was essential for the preparation of the setup, the adaptation of the equipment and the successful completion of the tests.

I would also like to thank the previous students whose work provided an important basis for this thesis. Their experimental developments and observations helped to guide the present work and to place it within the continuity of the research carried out at UCLouvain.

I would also like to acknowledge the use of artificial intelligence tools, in particular ChatGPT, during the preparation of this thesis. These tools were used as support for reformulating certain parts of the text, improving the clarity of the writing, generating non-data illustrative figures and identifying potentially relevant sources, which were subsequently checked. All scientific content, interpretations, calculations and final decisions remained under my responsibility and were carefully reviewed before being included in this work.

Finally, I would like to thank my family and friends for their support, patience and encouragement throughout this thesis and during my studies. Their presence and confidence were very important to me during the completion of this work.

Contents

List of Figures	vi
List of Tables	viii
Nomenclature	x
1 Global Environmental and Climate Context	1
1.1 Climate change, emissions and the role of the energy system	1
1.2 Expansion of renewable electricity	2
2 Offshore Electricity Production Systems	3
2.1 Overview of Offshore Electricity Generation Technologies	3
2.1.1 Why produce electricity offshore?	4
2.1.2 Additional challenges specific to offshore deployment	4
2.1.3 Main families of offshore electricity production devices	5
2.1.4 Technology maturity and current deployment	6
2.2 Wind-based offshore electricity generation devices	7
2.2.1 Operating principle	7
2.2.2 Main components of an offshore wind turbine	7
2.2.3 Key performance notions	8
2.3 Offshore support solutions for wind turbines	8
2.3.1 Why a support system is needed offshore	8
2.3.2 Two main categories: fixed-bottom vs floating systems	9
2.3.3 Fixed-bottom support technologies	9
2.3.4 Floating support technologies	10
2.3.5 Cross-cutting comparison and selection drivers	11
3 Monopile foundations for offshore wind turbines	11
3.1 General overview and reasons for widespread adoption	11
3.2 Structural role and load transfer mechanisms	12
3.3 Domain of application and current limitations	14
3.4 Installation methods for offshore monopiles	14
3.4.1 Impact driving	15
3.4.2 Vibratory driving	16
3.5 Physical mechanisms and governing parameters of monopile installation . .	17
3.5.1 Impact driving of monopiles	17
3.5.2 Vibratory driving of monopiles	21
4 Literature review and objectives of the present study	28
4.1 Synthesis of the literature	28
4.2 Objectives of the present study	30

5	Experimental methodology and test programme	32
5.1	Overview of the experimental programme	32
5.2	Scaling considerations for model tests	32
5.3	Sand sample preparation: principles and governing parameters	36
5.3.1	Importance of the initial sand state	36
5.3.2	Main parameters governing the prepared sand mass	37
5.4	CPT device used in the present work	38
5.5	Sand used in the present work	39
5.6	Preliminary pluviation campaign	40
5.6.1	Pluviation tests carried out in the present work	40
5.6.2	Large tank test	40
5.6.3	Small tank tests: common setup and procedure	44
5.7	Optimisation of the large-tank pluviation procedure	50
5.7.1	First modified test in the large tank: diffuser placed directly on the tank rim	51
5.7.2	Second modified test in the large tank: trimming the protruding inner fins	53
5.7.3	Third modified test in the large tank: interrupted pluviation with intermediate surface levelling	56
5.8	Validation of the final pluviation procedure	58
5.8.1	Retained pluviation procedure	59
5.8.2	Statistical comparison between the initial and final pluviation pro- cedures	59
5.9	Experimental setup for pile installation and lateral loading	63
5.9.1	General presentation of the setup	63
5.9.2	Model pile	65
5.9.3	Impact driving system	66
5.9.4	Vibratory driving system	67
5.9.5	Motor-driven lateral loading system	68
5.9.6	Load transfer device between the pile and the force sensor	71
5.9.7	Force sensor	72
5.9.8	Laser 2: KEYENCE laser displacement sensor	74
5.9.9	Laser 1: SICK OD2 laser displacement sensor	81
5.10	Experimental procedure and data processing	82
5.10.1	Test procedure	82
5.10.2	Processing of the lateral loading measurements	85
5.11	Test programme	86
6	Results	88
6.1	Installation response	90
6.1.1	Impact driving tests	90
6.1.2	Vibratory driving tests	91
6.2	Influence of the lateral loading rate	93
6.3	Lateral response of the piles	95
6.3.1	Influence of the embedment ratio	95
6.3.2	Influence of the impact-driving configuration	98

6.3.3	Influence of the vibratory-driving frequency	100
6.4	Comparison between impact-driven and vibratory-driven piles	102
6.4.1	Force–displacement response	102
6.4.2	Moment–rotation response	104
6.4.3	Evolution of the apparent rotation point	105
7	Discussion and interpretation of the results	107
7.1	Role of the embedment ratio and initial sand state	107
7.2	Effect of the impact-driving configuration	108
7.3	Effect of the vibratory-driving frequency	108
7.4	Comparison between impact-driven and vibratory-driven piles	109
7.5	Kinematic interpretation	109
7.6	Summary of the interpretation	110
8	Conclusion and perspectives	110
A	CPT profiles	118
B	Technical drawings of the pluviation system	120

List of Figures

1	Greenhouse gas emissions by sector (World). Data: Our World in Data, based on Climate Watch [1].	1
2	World electricity generation by source (2000–2024). Data: Our World in Data energy dataset [2]. Processed by the author.	2
3	Share of global electricity generated from wind and solar. Data: Our World in Data energy dataset [2]. Processed by the author.	3
4	Offshore electricity production technologies. Author-generated schematic with AI assistance.	5
5	Deployment scale of offshore electricity technologies. Offshore wind installed capacity from IEA [3]. IRENA global ocean energy totals [4]. Processed by the author.	6
6	Main components of a wind turbine. Taken from [5]	8
7	Fixed-bottom support technologies. Author-generated schematic with AI assistance.	9
8	Floating support technologies. Author-generated schematic with AI assistance.	10
9	Environmental actions and main load effects acting on a monopile-supported offshore wind turbine. Source: Shahrouh [6].	13
10	Example of offshore monopile installation by impact driving using a hydraulic hammer. Source: Offshore Wind Biz [7].	15
11	Example of offshore monopile installation by vibratory driving. Source: CAPE Holland [8].	16
12	Schematic representation of the two main phases of a hammer blow and associated simplified impact force–time history during monopile installation by impact driving, adapted from Chen et al. [9].	18
13	Example of measured change in relative soil density around a vibratory driven monopile, adapted from Remspecher et al. [10].	22
14	Schematic representation of the operating principle of vibratory pile driving and associated cyclic excitation, adapted from Massarsch et al. [11].	24
15	Conceptual illustration of pile–soil interaction during vibratory driving under different vibration conditions, adapted from Massarsch et al. [11].	27
16	Portable electronic cone penetrometer used in the present work.	39
17	General view of the large tank used for the pluviation test and of the existing pluviation system available in the laboratory.	40
18	Measured free-fall height in the large-tank pluviation setup.	41
19	Top view after pluviation.	42
20	Surface of the sand mass in the large tank immediately after pluviation.	42
21	Examples of height measurements in the large tank after pluviation and before levelling.	43
22	CPT results obtained at the nine test locations in the large tank.	44
23	Adjustment of the geometric configuration.	45
24	Pluviation in progress.	45
25	Top view after pluviation: location of the nine CPT test points.	46
26	Surface condition after the first pluviation test in the small tank.	47

27	Superposition of the CPT profiles for the nine test locations after the first pluviation test in the small tank.	48
28	Top view of the final sand surface after the second pluviation test in the small tank.	49
29	Superposition of the CPT profiles for the nine test locations after the second pluviation test in the small tank.	50
30	First modified configuration in the large tank.	51
31	Surface of the sand mass in the large tank immediately after pluviation. . .	52
32	Superposition of the CPT profiles for the nine test locations after the first modified test in the large tank, with the diffuser placed directly on the tank rim.	53
33	Schematic illustration of the geometric disturbance caused by the protruding inner fins of the large tank.	54
34	Surface of the sand mass in the large tank immediately after pluviation. . .	55
35	Superposition of the CPT profiles for the nine test locations after the second modified test in the large tank, following the trimming of the protruding inner fins.	56
36	Surface of the sand mass in the large tank immediately after pluviation. . .	57
37	Superposition of the CPT profiles for the nine test locations after the interrupted-pluviation test.	58
38	Superposition of the CPT profiles obtained after the initial pluviation test in the large tank.	60
39	Superposition of the CPT profiles obtained after the first pluviation test performed with the final large-tank configuration.	61
40	Superposition of the CPT profiles obtained after the second pluviation test performed with the final large-tank configuration.	61
41	Coefficient of variation of the CPT results as a function of depth for the initial pluviation test and the two pluviation tests performed with the final configuration.	62
42	Simplified schematic representation of the lateral loading setup and main geometrical parameters.	64
43	Photographs of the experimental setup used for lateral loading tests. . . .	65
44	Impact driving system used for the installation of the model pile.	66
45	Vibratory driving system used during pile installation: global arrangement with the cardboard guide and detailed view of the motor, eccentric masses and accelerometers.	68
46	Motor-driven system used to impose the lateral displacement during the loading tests. The Bosch Rexroth servo motor is coupled to a planetary gearbox and drives the winding drum connected to the Dyneema cable. . .	69
47	Load-transfer device and force sensor connection used during the lateral loading tests.	72
48	Raw voltage signal delivered by the laser sensor during the calibration test.	75
49	Amplitude spectrum of the raw laser voltage signal obtained by FFT. . . .	76
50	Comparison between the raw voltage signal and the signal after application of a 5 Hz low-pass filter	76
51	Calibration curve of the laser displacement sensor.	78

52	Raw distance signal measured by the laser sensor during the validation test.	79
53	Comparison between the raw distance signal and the signal after application of a 5 Hz low-pass filter during the validation test.	79
54	Processed filtered signal used for the validation analysis.	80
55	Comparison between the theoretical imposed distance and the measured distance obtained from the calibrated laser sensor during the validation test.	81
56	Schematic arrangement of the three pile locations in the large tank, inspired by Shahrou [6].	83
57	Tarpaulin used to materialise the three pile locations at the surface of the sand mass.	84
58	Cumulative nominal impact energy for the impact-driven piles installed at $L/D = 7$	91
59	Influence of the embedment ratio on vibratory driving duration.	92
60	Influence of the motor speed on vibratory driving duration.	92
61	Influence of the lateral loading rate on the force–displacement response. . .	93
62	Effect of the lateral loading rate on the force mobilised at $u_0/D = 0.10$. . .	94
63	Effect of the lateral loading rate on the initial stiffness K_{initial}	94
64	Influence of the embedment ratio on the force–displacement response of the impact-driven piles.	95
65	Influence of the embedment ratio on the force–displacement response of the vibratory-driven piles.	96
66	CPT profiles measured before the installation of the vibratory-driven piles $T7$, $T8$, and $T9$	97
67	Initial stiffness K_{initial} as a function of the embedment ratio for impact-driven and vibratory-driven piles.	98
68	Influence of the impact-driving configuration on the force–displacement response for piles installed at $L/D = 7$ with a similar nominal impact energy per blow.	99
69	Initial stiffness K_{initial} for the different impact-driving configurations at $L/D = 7$	99
70	Influence of the vibratory-driving frequency on the force–displacement response.	100
71	CPT profiles measured before the installation of the vibratory-driven piles.	101
72	Initial stiffness K_{initial} as a function of the vibratory-driving motor speed. .	102
73	Comparison between impact-driven and vibratory-driven piles at the same embedment ratio.	103
74	Comparison between impact-driven and vibratory-driven piles installed with $L/D = 7$	103
75	Moment–rotation response for impact-driven and vibratory-driven piles installed close to $L/D = 7$	105
76	Evolution of the normalised apparent rotation point depth d/L during lateral loading for piles installed with $L/D = 7$	106
77	Mean value of the normalised apparent rotation point depth d/L during loading.	107
78	CPT profiles before impact-driven pile installation.	118
79	CPT profiles before vibratory-driven pile installation.	119

List of Tables

1	Summary of the preliminary pluviation tests.	59
2	Mean coefficient of variation of the CPT results for the three pluviation tests over the first 42 cm of penetration.	62
3	Main characteristics of the model pile used in the experimental campaign. .	65
4	Main characteristics of the motor used for the lateral loading system. . . .	69
5	Comparison between the theoretical applied force and the measured response of the force sensor for the first calibration test.	73
6	Validation of the force sensor calibration after application of the offset correction.	74
7	Representative plateau values retained for the calibration of the laser sensor.	77
8	Global error indicators obtained during the validation of the laser calibration.	81
9	Experimental test matrix for pile installation and lateral loading.	88
10	Summary of the impact driving installation results.	90
11	Mean response indicators for impact-driven and vibratory-driven piles with $L/D = 7$	104

Nomenclature

Symbol	Description	Unit
D	Outer diameter of the model pile	mm, cm
L	Embedded length of the pile	cm
L/D	Embedment ratio	–
h	Height of lateral load application above the mudline	cm
h_1, h_2	Heights of the two laser displacement sensors above the mudline	cm
u_0	Lateral displacement at mudline level	mm, cm
u_1, u_2	Lateral displacements measured by Laser 1 and Laser 2	mm, cm
θ	Rotation of the pile during lateral loading	rad, °
d	Apparent depth of the rotation point below the mudline	cm
F	Lateral force applied to the pile	N
$F_{0.01D}$	Force mobilised at $u_0/D = 0.01$	N
$F_{0.02D}$	Force mobilised at $u_0/D = 0.02$	N
$F_{0.05D}$	Force mobilised at $u_0/D = 0.05$	N
$F_{0.10D}$	Force mobilised at $u_0/D = 0.10$	N
K_{initial}	Initial secant stiffness of the lateral force–displacement curve	N/cm
M	Moment applied to the pile during lateral loading	N m
q_c	Cone resistance measured during CPT tests	MPa
E	Nominal impact energy	J
m	Falling mass used during impact driving	kg
H_{drop}	Drop height used during impact driving	cm
N_{drops}	Number of hammer drops required during impact driving	–
M_e	Eccentric moment of the rotating masses	g mm
v	Motor speed during vibratory driving	krpm
f	Vibration frequency	Hz
ω	Angular frequency of the vibrator	rad/s
$\dot{u}_a$	Imposed actuator displacement rate	mm/s, mm/min
$\dot{u}_0$	Estimated mudline displacement rate	mm/min
U	Voltage signal delivered by a displacement sensor	V
ρ	Soil density	kg/m ³
σ_v	Vertical stress in the soil	kPa
N	Geometrical scale reduction factor	–
CV	Coefficient of variation	%

Abbreviations

Abbreviation	Meaning
CPT	Cone penetration test
DAQ	Data acquisition system
FFT	Fast Fourier Transform
HR	High-rate lateral loading test
LEMSC	Laboratory of Experimental Mechanics, Structures and Civil Engineering
OWT	Offshore wind turbine
PISA	Pile Soil Analysis framework
RMSE	Root mean square error
SRD	Soil resistance to driving

1 Global Environmental and Climate Context

The development of offshore wind energy is part of a broader transformation of the global energy system. This transformation is driven by the need to reduce greenhouse gas emissions, to decarbonise electricity production, and to supply a growing demand for low-carbon electricity. The objective of this chapter is therefore not to provide an exhaustive review of climate policy, but to briefly explain why renewable electricity, and more specifically offshore wind, has become strategically important.

1.1 Climate change, emissions and the role of the energy system

Climate change is one of the major systemic risks for societies and infrastructure systems in the 21st century. The IPCC Sixth Assessment Report concludes with very high confidence that human activities, primarily through greenhouse gas emissions, have unequivocally caused global warming. Over the 2011–2020 decade, global surface temperature was about 1.1°C higher than the 1850–1900 pre-industrial baseline [12].

A large share of global greenhouse gas emissions is directly related to the production and use of energy. Electricity and heat generation, transport and industrial activities remain major contributors to anthropogenic emissions, as shown in Fig. 1. Consequently, the decarbonisation of the energy system is a central component of climate mitigation strategies. These strategies generally rely on two complementary pathways: reducing fossil-fuel demand through efficiency and electrification, and increasing the share of low-carbon electricity generation.

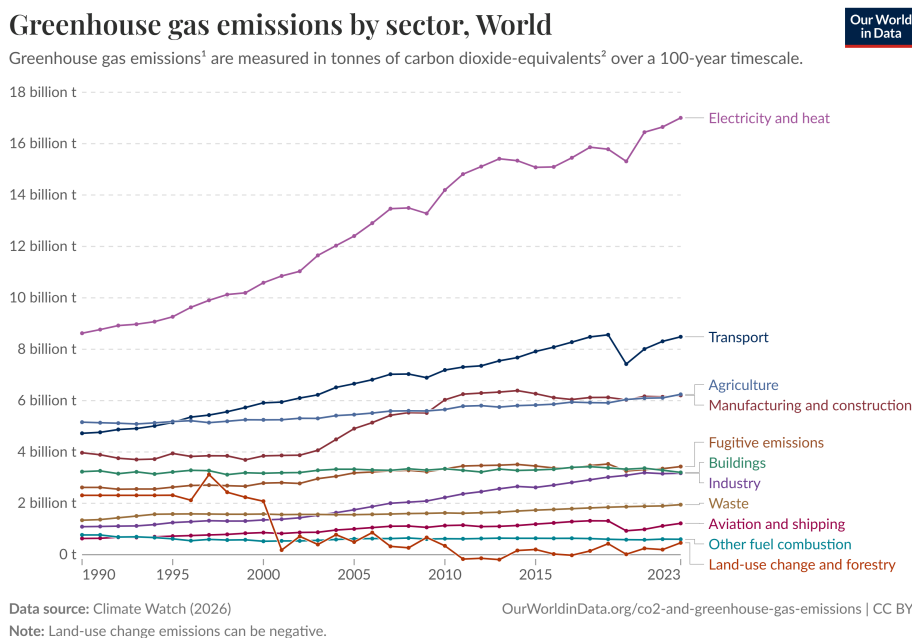


Figure 1: Greenhouse gas emissions by sector (World). Data: Our World in Data, based on Climate Watch [1].

This transition is also supported by policy commitments. At the international level, the Paris Agreement aims to hold the increase in global average temperature to well below

2°C above pre-industrial levels, while pursuing efforts to limit warming to 1.5°C [13]. In the European Union, the European Climate Law makes climate neutrality by 2050 legally binding and sets an intermediate target of at least 55% net greenhouse gas emissions reduction by 2030 compared to 1990 levels [14]. These targets create a strong incentive for the large-scale deployment of renewable electricity technologies.

1.2 Expansion of renewable electricity

The energy transition is accompanied by an increasing role of electricity in final energy demand. Transport, heating, industrial processes and emerging uses such as data centres and hydrogen production are expected to rely increasingly on electricity. As a result, decarbonising electricity generation becomes particularly important: electricity must not only replace part of fossil-fuel consumption in other sectors, but also remain reliable, affordable and sufficiently flexible.

Renewable electricity technologies have expanded rapidly in this context. Solar photovoltaics and wind energy have led recent deployment because of their scalability, technological maturity and decreasing costs. The International Energy Agency estimates that global renewable capacity additions reached around 700 GW in 2024, with solar PV representing the majority of new installations [15]. In parallel, IRENA reports that renewable power capacity additions have accelerated markedly in recent years [16].

The progressive evolution of the global electricity generation mix is illustrated in Fig. 2. Although fossil fuels still represent a substantial share of global electricity production, the contribution of wind and solar has increased strongly since the early 2000s. This confirms that renewable technologies are not only expanding in installed capacity, but are also contributing increasingly to actual electricity generation.

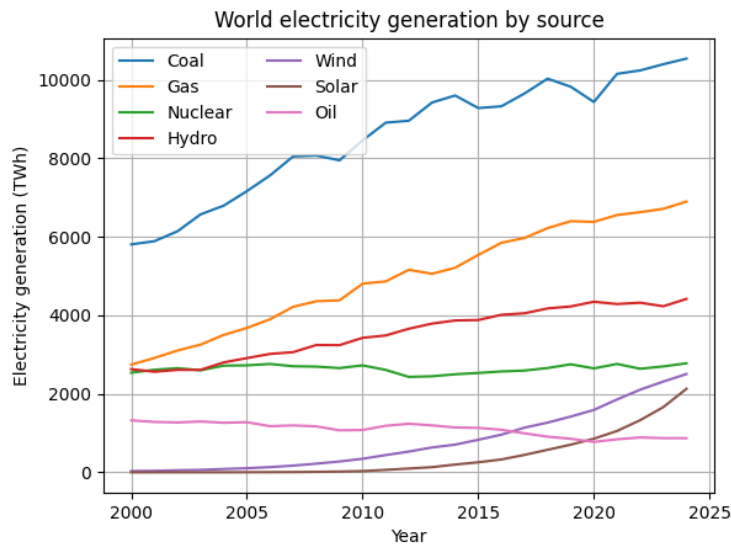


Figure 2: World electricity generation by source (2000–2024). Data: Our World in Data energy dataset [2]. Processed by the author.

A key enabling factor behind this growth is the significant decrease in renewable electricity costs over the past two decades, particularly for solar PV and wind power.

Improvements in manufacturing, turbine and panel efficiency, supply-chain optimisation and economies of scale have reduced the levelised cost of electricity, making solar PV and wind power competitive with, or cheaper than, new fossil-based generation in many regions [17]. This economic evolution is consistent with the increasing share of global electricity generated from wind and solar, shown in Fig. 3.

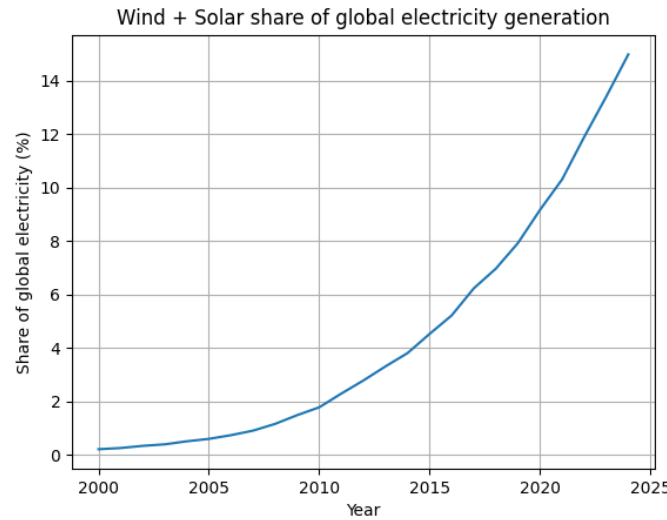


Figure 3: Share of global electricity generated from wind and solar. Data: Our World in Data energy dataset [2]. Processed by the author.

However, wind and solar are variable renewable energy sources. Their production depends on weather conditions and therefore requires power systems to become more flexible. This flexibility can be provided by storage, demand-side management, interconnections, dispatchable low-carbon generation and reinforced transmission infrastructure. These integration constraints do not reduce the strategic value of renewables, but they show that their large-scale deployment must be accompanied by suitable infrastructure and system planning.

2 Offshore Electricity Production Systems

2.1 Overview of Offshore Electricity Generation Technologies

Offshore environments provide access to vast renewable resources and large deployment areas, making them an attractive option to supply low-carbon electricity at scale. In addition to resource availability, offshore development can relieve land-use constraints and enable coordinated planning of generation assets and shared infrastructure (e.g., offshore grids, interconnectors, and energy hubs) [18].

This section introduces (i) the main advantages of offshore electricity production, (ii) the additional challenges associated with offshore deployment, (iii) the principal technology families used to generate electricity offshore, and (iv) a brief positioning of these technologies in terms of maturity and deployment. Although several offshore renewable technologies exist, they differ strongly in technological maturity, deployment scale, and

structural requirements. This is why the following sections progressively narrow the discussion toward offshore wind, which currently dominates offshore renewable deployment [3].

2.1.1 Why produce electricity offshore?

Several factors support offshore electricity production [18]:

- **High-quality resources.** Offshore wind speeds are generally higher and more regular than onshore due to reduced surface roughness and fewer obstacles. Offshore environments also offer access to wave energy and tidal resources that can be significant in certain regions [19].
- **Space and scalability.** Large marine areas enable utility-scale projects and clustering of multiple farms, potentially supporting long-term expansion strategies.
- **Reduced land-use constraints.** Offshore siting can mitigate some conflicts associated with land availability and competing onshore uses, while moving certain nuisances (e.g., visual footprint) away from densely populated areas.
- **Proximity to coastal demand and industrial hubs.** Many major demand centres and industrial zones are located in coastal regions, where offshore generation can integrate into regional energy strategies.
- **Infrastructure and multi-use opportunities.** Offshore development may leverage shared infrastructure (cables, substations, offshore hubs) and support emerging energy vectors such as offshore-produced hydrogen in some strategies.

2.1.2 Additional challenges specific to offshore deployment

Offshore deployment also introduces constraints that significantly shape technology choices and project implementation:

- **Installation logistics and weather windows.** Offshore construction requires specialised vessels, ports, lifting operations, and schedule planning constrained by sea state and wind conditions.
- **Harsh marine environment.** Corrosion, fatigue, biofouling, and extreme meteorological events increase design and maintenance requirements compared with onshore systems.
- **Support and seabed interaction.** Most offshore generators require robust support solutions (foundations or moorings) and must account for seabed conditions and processes such as scour.
- **Grid connection and reliability.** Offshore projects require export cables, offshore substations and protection systems; cable faults and offshore repairs can be costly and time-consuming [18].
- **Permitting and environmental constraints.** Marine spatial planning, environmental impact assessments, and coexistence with other sea users can strongly influence project timelines and layouts [18].

These challenges mean that offshore technologies cannot be compared solely on the basis of resource potential: support requirements, marine operations, and infrastructure complexity are also key differentiating factors.

2.1.3 Main families of offshore electricity production devices

Offshore electricity can be produced through several technology families, which exploit different physical resources [19]. While all contribute to the broader offshore renewable landscape, they differ markedly in maturity, scale, and engineering requirements. Some of these technologies are presented on Fig. 4.

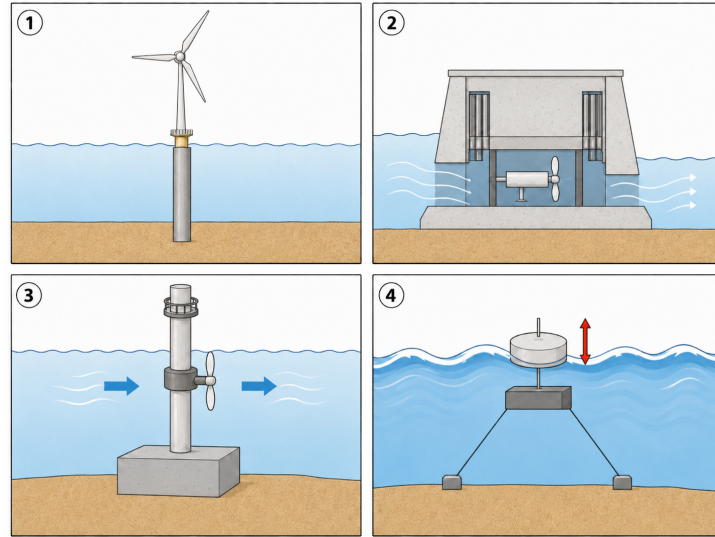


Figure 4: Offshore electricity production technologies. Author-generated schematic with AI assistance.

1 Offshore wind.

Wind turbines convert the kinetic energy of marine winds into electricity. Offshore wind farms are deployed either on *fixed-bottom* support structures (typically in shallow to intermediate waters) or on *floating* platforms (for deeper waters). Offshore wind is currently the most mature and widely deployed offshore renewable electricity technology [3].

2: Tidal range.

Tidal range schemes use the potential energy created by differences in water level between high and low tide. Typical concepts include barrages and lagoons, where turbines generate during filling and/or emptying cycles. These projects are highly site-specific and relatively rare.

3: Tidal stream and marine currents.

Tidal stream technologies extract kinetic energy from tidal currents, often using turbine concepts analogous to wind turbines but operating in denser seawater. Other concepts (ducted turbines, oscillating hydrofoils, kites) also exist. Deployment remains limited compared to offshore wind, with ongoing development and pilot-scale projects in several regions.

4: Wave energy converters.

Wave energy aims to harvest energy from surface waves using a wide diversity of device families (e.g., point absorbers, attenuators, oscillating water columns, overtopping de-

vices). Despite significant theoretical potential, wave energy remains largely pre-commercial due to survivability and cost challenges in harsh offshore conditions.

Other ocean energy pathways.

Additional concepts include ocean thermal energy conversion (OTEC) and salinity-gradient power, which remain less mature and have limited deployment to date [19].

Taken together, these technology families illustrate the diversity of offshore renewable options. However, their present industrial relevance is not equivalent, which motivates the following discussion on maturity and deployment.

2.1.4 Technology maturity and current deployment

A key distinction across offshore electricity technologies is their maturity and deployment scale. Offshore wind is commercially deployed at large scale, supported by an established industrial supply chain, standardised project development practices, and strong growth trajectories [3]. In contrast, most ocean energy technologies (wave, tidal stream, and other concepts) remain at earlier development stages, with smaller installed capacities and continuing innovation and demonstration efforts [19]. This contrast in deployment scale is synthesised in Fig. 5.

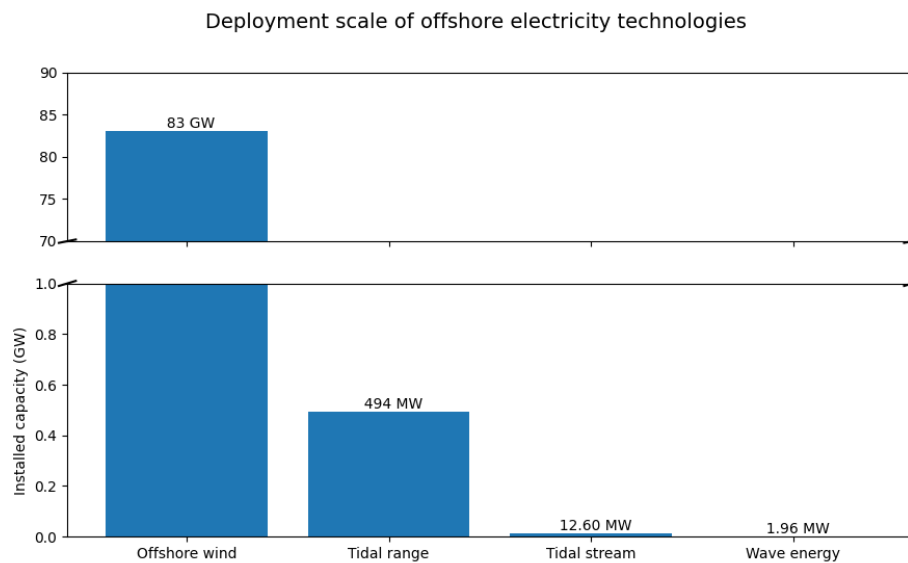


Figure 5: Deployment scale of offshore electricity technologies. Offshore wind installed capacity from IEA [3]. IRENA global ocean energy totals [4]. Processed by the author.

This difference in maturity is important for the remainder of this work: offshore wind is not only the leading offshore renewable technology, but also the one for which support structures, installation methods, and soil–structure interaction are most immediately relevant at industrial scale.

2.2 Wind-based offshore electricity generation devices

This section introduces the wind-turbine device level, from the operating principle and main components to basic performance indicators.

2.2.1 Operating principle

An offshore wind turbine converts the kinetic energy of the wind into electrical power through (i) aerodynamic lift generated by the rotor blades, (ii) mechanical power transmission through the drivetrain, and (iii) electrical conversion and grid-compliant delivery through power electronics and transformers. In modern turbines, active control systems continuously adjust the rotor speed, blade pitch, and nacelle yaw to optimise power capture, limit loads, and maintain safe operation across varying wind conditions [20].

2.2.2 Main components of an offshore wind turbine

As shown in Figure 6, an offshore wind turbine is composed of several main subsystems that act together from energy capture at the rotor to load transfer into the support structure:

- **Rotor assembly** (*rotor blades* and *hub*), which captures the kinetic energy of the wind and converts it into rotational motion.
- **Pitch mechanism**, located at the blade roots, which adjusts the blade angle to regulate aerodynamic performance, optimize power production, and reduce loads under varying wind conditions.
- **Nacelle**, which houses the main drivetrain and conversion equipment, including in particular the *gearbox*, the *generator*, and part of the *electrical and control system*.
- **Yaw mechanism**, installed between the nacelle and the tower, which rotates the nacelle so that the rotor remains aligned with the incoming wind direction.
- **Wind measurement and control devices**, such as the *anemometer* and *wind vane*, which provide the data required for turbine control and orientation.
- **Tower**, which supports the rotor–nacelle assembly and provides sufficient elevation to access stronger and more stable winds.
- **Electrical systems**, including internal *power cables* and associated control equipment, which transmit the generated electricity and support turbine operation and monitoring.
- **Foundation**, which transfers the loads from the turbine and support structure to the seabed; this aspect is addressed in more detail in the next section [21].

Figure 6 therefore provides a synthetic overview of the main components of a wind turbine and their respective roles in energy capture, control, power conversion, and structural support.

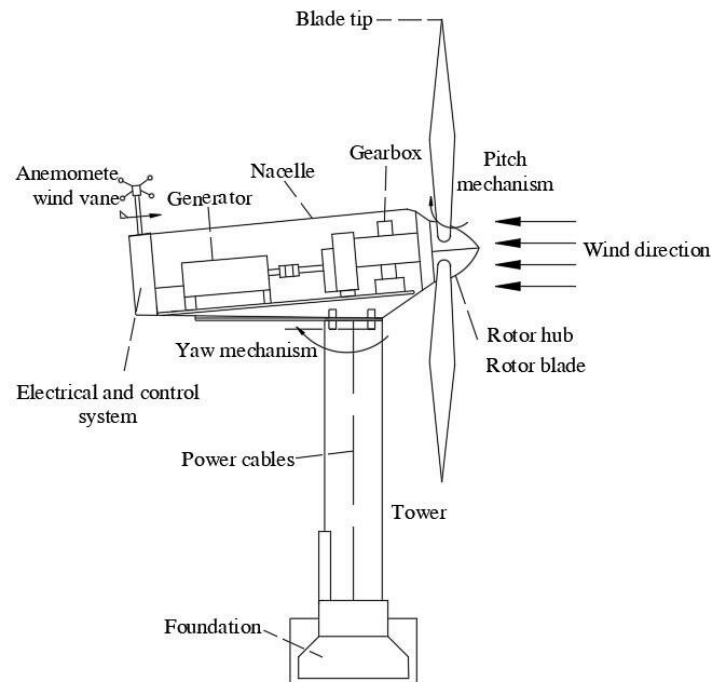


Figure 6: Main components of a wind turbine. Taken from [5]

2.2.3 Key performance notions

Two notions are frequently used to characterise the operational performance of wind farms: *availability*, describing the proportion of time the system is able to produce electricity when resource conditions allow, and *capacity factor*, describing the ratio between actual energy production and the theoretical maximum output at rated power over a given period. In offshore wind, these indicators are particularly meaningful because they reflect not only the quality of the wind resource, but also the impact of access limitations, maintenance logistics, and operational reliability on the effective electricity delivered to the grid [3].

2.3 Offshore support solutions for wind turbines

2.3.1 Why a support system is needed offshore

Offshore wind turbines must remain stable and operational under combined wind and metocean actions (waves and currents), while ensuring acceptable serviceability over a typical design life. The support system therefore has a dual role: (i) provide global stability and stiffness to limit motions and long-term deformations, and (ii) transfer loads safely to the seabed or to a station-keeping system. In addition, offshore conditions introduce specific constraints such as fatigue, corrosion, biofouling, restricted access for maintenance, and seabed processes such as scour, which all influence support design and lifecycle performance [22].

2.3.2 Two main categories: fixed-bottom vs floating systems

Offshore wind support solutions can be grouped into two main categories [22, 23]:

- **Fixed-bottom systems**, where the turbine is supported by a structure connected to the seabed through a foundation. These systems are typically used in shallow to intermediate water depths.
- **Floating systems**, where the turbine is supported by buoyancy and maintained in position through mooring lines connected to seabed anchors. These systems enable deployment in deeper waters where fixed-bottom solutions become less practical.

This distinction is primarily driven by water depth and site constraints, and it strongly affects installation strategy, operations and maintenance logistics, and overall project architecture [23].

2.3.3 Fixed-bottom support technologies

Several fixed-bottom concepts are used in offshore wind. Each solution reflects a trade-off between water depth, turbine size, soil conditions, fabrication and installation constraints, and cost [22].

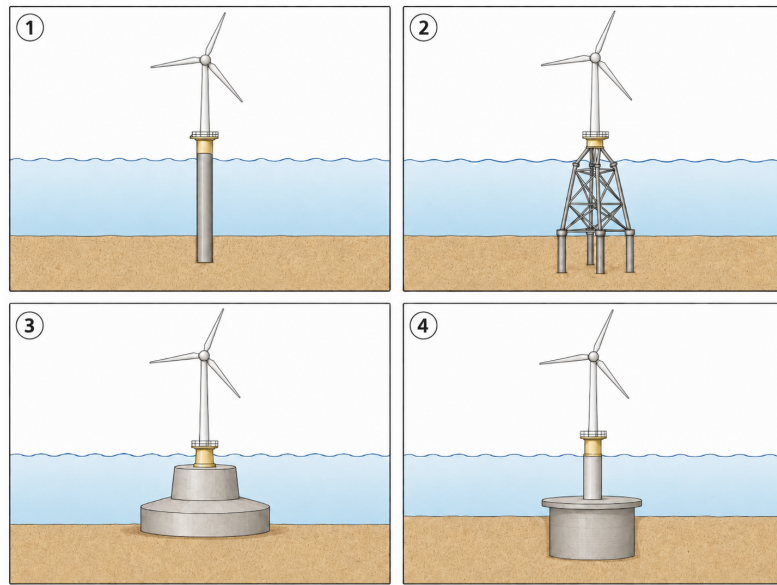


Figure 7: Fixed-bottom support technologies. Author-generated schematic with AI assistance.

1: Monopile.

A monopile consists of a single large-diameter steel tubular pile driven into the seabed, on top of which the tower and transition piece are installed. It has historically been the dominant fixed-bottom concept in many offshore wind farms due to its relative simplicity, industrial scalability, and compatibility with large-scale deployment. Because monopiles remain the reference solution in a large share of fixed-bottom offshore wind projects, their installation and soil–structure interaction are of primary interest for the present work.

2: Jacket.

Jacket structures are lattice frameworks (derived from offshore oil and gas practice) typically connected to the seabed through multiple piles. Jackets can be efficient in intermediate depths and for larger turbines, but generally involve more complex fabrication and installation compared with single-pile concepts .

3: Gravity-based foundation (GBF).

Gravity-based foundations rely primarily on self-weight and seabed preparation to provide stability. They are typically installed on a prepared seabed (often with scour protection) and can be attractive where driving piles is constrained, but they require significant seabed preparation and heavy marine operations.

4: Suction bucket / suction caisson.

Suction caisson foundations (also called suction buckets) are installed by penetrating a large steel caisson into the seabed using under-pressure (suction). They can be used as a standalone foundation or in combination with jacket or multi-leg structures. Advantages often include potentially faster installation and reduced noise compared to impact pile driving, although their applicability depends on soil conditions and project context .

Other fixed-bottom concepts.

Other concepts such as tripods or tripiles have been used in some projects, but they are less common compared with monopiles and jackets in current large-scale deployment [22].

2.3.4 Floating support technologies

Floating offshore wind systems use buoyancy for vertical support and mooring systems (connected to seabed anchors) for station-keeping. Several platform families have emerged from offshore engineering practice, with ongoing innovation and demonstration projects [22, 23].

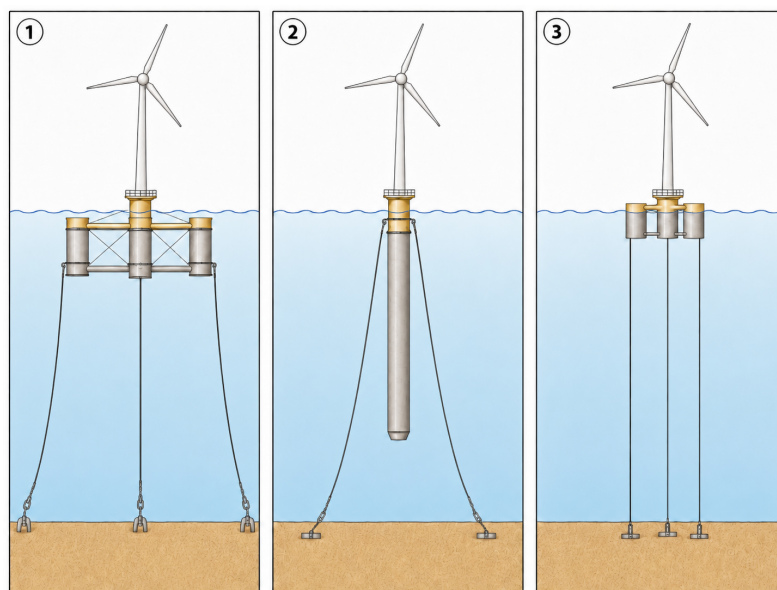


Figure 8: Floating support technologies. Author-generated schematic with AI assistance.

1: Semi-submersible.

Semi-submersibles rely on multiple columns and pontoons to provide stability with relatively low draft, enabling assembly in port and tow-out. Their stability characteristics and logistics make them one of the leading floating concepts in several demonstration and early commercial projects.

2: Spar.

Spar platforms use a long vertical cylindrical hull with deep draft to achieve stability. They can offer favourable motion characteristics, but may require deeper ports or offshore assembly constraints depending on the project context.

3: Tension-leg platform (TLP).

TLP concepts use buoyancy combined with taut mooring lines to limit vertical motions. They may offer reduced heave response, but require specific anchoring solutions and installation approaches.

2.3.5 Cross-cutting comparison and selection drivers

The selection of an offshore support solution is influenced by multiple interacting factors, including water depth, metocean conditions, seabed characteristics, turbine size, installation fleet availability, and operation and maintenance constraints. For fixed-bottom systems, soil stratigraphy and variability play a key role in determining feasible foundation concepts and installation strategies. For floating systems, platform motions, mooring design, and anchor solutions become central drivers [22, 23].

More broadly, support selection results from a compromise between structural performance, constructability, installation risk, and lifecycle cost. In practice, this means that no single concept is universally optimal: the most appropriate solution depends on the combination of site conditions and project objectives. Within this framework, fixed-bottom concepts—and especially monopiles—remain central for shallow to intermediate water depths, which explains their importance in the present study.

3 Monopile foundations for offshore wind turbines**3.1 General overview and reasons for widespread adoption**

Among fixed-bottom support concepts, the monopile has historically been the most widely used foundation solution for offshore wind turbines. As reviewed by Wu et al. [22], this widespread adoption is mainly explained by their structural simplicity, industrial maturity and compatibility with large-scale offshore deployment. A monopile consists of a single large-diameter steel tubular pile installed into the seabed, on top of which the transition piece and tower are connected. In offshore wind applications, monopiles differ significantly from conventional slender piles used in other marine structures, as their large diameter and lateral loading sensitivity make stiffness, rotation control, and soil–structure interaction central design considerations. This distinction is also emphasised by Arany et al. [24], who discuss the specific design challenges associated with offshore wind monopiles.

The widespread adoption of monopiles in offshore wind can be explained by several factors [22, 24]:

- **Conceptual simplicity.** The monopile relies on a relatively simple structural concept compared with multi-leg or gravity-based alternatives.
- **Industrial scalability.** Its tubular steel geometry is well suited to industrial fabrication, transport, and serial production.
- **Straightforward offshore installation chain.** Compared with more complex support systems, monopiles can be integrated into relatively streamlined marine installation procedures.
- **Suitability for many early offshore wind sites.** Monopiles proved particularly competitive for shallow to intermediate water depths and for sites with favourable seabed conditions.
- **Extensive experience feedback.** Their broad deployment has progressively generated design experience, installation know-how, and a stronger technical basis for subsequent projects.

This combination of relative simplicity, industrial maturity, and broad applicability explains why monopiles became the reference fixed-bottom solution during the early large-scale development of offshore wind. Their success has also encouraged continuous upscaling, with increasing pile diameters and wall thicknesses to accommodate larger turbines and more demanding sites. However, this evolution has progressively pushed monopiles toward new structural, geotechnical, and installation challenges, which are discussed in the following subsections.

3.2 Structural role and load transfer mechanisms

The monopile constitutes the main structural interface between the offshore wind turbine and the seabed. Its role is not limited to carrying the vertical weight of the structure, but also includes resisting horizontal loads and overturning moments generated by wind, waves, currents and turbine operation. The design of offshore wind turbine foundations therefore requires the combined consideration of structural, geotechnical, environmental and operational actions.

In real offshore conditions, these actions are not constant. They vary in time, amplitude, direction, frequency and eccentricity, leading to a cyclic and irregular loading history at the foundation level. As discussed by Richards [25], monopile foundations are subjected to complex cyclic lateral loading conditions, which strongly influence the long-term response of the soil–pile system. Despite this complexity, the monotonic lateral response remains an essential reference for understanding the initial stiffness, rotational behaviour and ultimate resistance of the foundation. For this reason, the present work focuses on controlled monotonic lateral loading tests performed after pile installation.

In offshore wind applications, the foundation is subjected to combined loading transmitted from the rotor–nacelle assembly, the tower, and the environmental actions acting along the exposed height of the structure. These actions generate a global response in which the monopile must ensure overall stability, limit deflections at seabed level, and maintain rotations within acceptable serviceability limits. This is particularly important for offshore wind turbines, for which excessive tilt or flexibility may affect not only structural safety, but also turbine performance and fatigue behaviour.

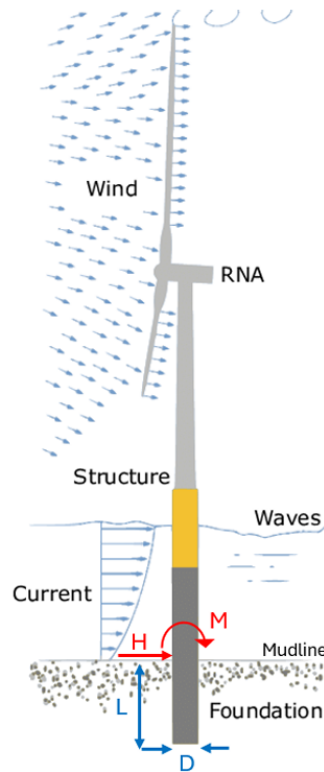


Figure 9: Environmental actions and main load effects acting on a monopile-supported offshore wind turbine. Source: Shahrour [6].

As shown in Figure 9, wind, waves and currents generate horizontal loading and overturning moment demands that are transferred to the foundation and resisted through soil–structure interaction.

The transfer of loads from the monopile to the surrounding ground is governed by soil–structure interaction. Unlike slender piles mainly designed for axial loading, offshore wind monopiles are large-diameter embedded members for which lateral soil reaction plays a dominant role. Under horizontal loading H and overturning moment M , the pile mobilises distributed soil resistance along its embedded length, while the coupled response of the steel tube and the surrounding soil controls the overall stiffness of the support system. The resulting behaviour depends on both structural parameters, such as pile diameter, wall thickness, and embedment length, and geotechnical parameters, such as soil stiffness, density, strength, and stratigraphy.

An important consequence of this behaviour is that monopile response cannot be described solely through the structural properties of the steel element. The foundation performance is strongly influenced by the evolution of soil reaction around the pile, especially under repeated environmental loading. The structural design of monopiles therefore requires an integrated approach in which the pile and the surrounding soil are considered as a coupled system. This central role of soil–structure interaction is one of the main reasons why installation effects and post-installation soil state are particularly relevant for offshore monopile foundations.

3.3 Domain of application and current limitations

Monopiles have historically been widely adopted in shallow to intermediate water depths and at sites with relatively favourable seabed conditions. As reviewed by Wu et al. [22], they progressively became the reference fixed-bottom foundation solution during the early large-scale development of offshore wind owing to their relative simplicity, industrial maturity, and broad applicability.

However, the continuous increase in turbine capacity, hub height, rotor diameter, and project water depth has progressively pushed monopile foundations toward more demanding conditions. Arany et al. [24] highlight that this upscaling requires larger diameters, greater wall thicknesses and longer embedment lengths in order to maintain sufficient stiffness and resistance. While this evolution has extended the practical range of monopile application, it has also highlighted important structural, geotechnical, and installation-related limitations.

From a geotechnical point of view, the performance of monopiles remains highly dependent on site conditions. Soil stiffness, density, layering, and strength strongly influence lateral response, rotation, and long-term serviceability behaviour. In addition, offshore monopiles are particularly sensitive to scour, cyclic loading effects, and uncertainties associated with the prediction of soil reaction for large-diameter piles. These aspects become increasingly critical as turbine size grows and design margins become more constrained.

From a structural and practical perspective, the upscaling of monopiles also raises challenges related to fabrication, transport, drivability, fatigue and installation tolerances. Moreover, several design approaches historically adapted from the offshore oil and gas sector are not always fully suited to the specific behaviour of large-diameter monopiles supporting wind turbines, especially under combined lateral loading, overturning moment and cyclic loading.

This limitation is particularly important for the prediction of the lateral soil reaction. Classical p - y approaches were initially developed for long and slender piles, whereas modern offshore monopiles often behave as short and relatively rigid foundations. The PISA framework proposed by Burd et al. [26] addresses this issue by introducing additional soil reaction components, including distributed moment reactions as well as base shear and base moment reactions. These components become increasingly relevant when the embedment ratio L/D decreases, which explains why L/D is a key parameter in the present experimental programme.

These limitations are also directly connected to the question of installation. As monopiles become larger and are deployed in increasingly demanding environments, installation procedures play a growing role not only in construction feasibility, but also in the resulting state of the soil around the pile and, consequently, in the later mechanical behaviour of the foundation system.

3.4 Installation methods for offshore monopiles

Monopile installation is a critical stage in offshore wind projects, as it directly governs construction feasibility, weather sensitivity, offshore logistics, and the initial state of the soil surrounding the foundation. In his technical review of offshore wind turbine installation, Jiang [27] shows that installation procedures are a central component of offshore wind project feasibility. In practice, monopile installation generally includes transporta-

tion to site, upending, positioning, and penetration into the seabed using a dedicated installation method. Although several techniques exist, impact driving has historically remained the dominant method in offshore wind, while vibratory driving has received increasing attention as a promising alternative [27, 28].

The choice of installation procedure is not only a construction issue. During penetration, the pile interacts strongly with the surrounding soil and may modify its stress state, density and local fabric. These installation-induced changes can subsequently affect the lateral response and serviceability behaviour of the monopile foundation. This link between installation process and post-installation behaviour was directly investigated by Gavin et al. [29], who studied the influence of installation method on the monotonic lateral loading response of monopiles in sand. For this reason, installation methods are of particular interest in the present work.

3.4.1 Impact driving

Impact driving is currently the most established installation method for offshore wind monopiles [27]. In this approach, a hydraulic hammer applies repeated impacts at the pile head, generating stress waves that propagate along the steel tube and progressively drive the pile into the seabed. The method is widely used because it is well proven in offshore practice and can provide a clear installation record during penetration.

However, impact driving is associated with several well-known drawbacks. It induces strong transient loading during installation and is also linked to high underwater noise emissions, which explains the growing interest in alternative techniques. From a geotechnical perspective, the numerical work of Chan et al. [28] also suggests that impact-driven installation may significantly modify the surrounding soil response and may have a marked influence on the later lateral behaviour of the monopile.



Figure 10: Example of offshore monopile installation by impact driving using a hydraulic hammer. Source: Offshore Wind Biz [7].

Figure 10 highlights the typical installation arrangement: the pile is held vertically while a large hydraulic hammer is positioned at the pile head to deliver repeated blows.

3.4.2 Vibratory driving

In vibratory driving, the pile is installed by applying vertical oscillations at the pile head rather than a succession of discrete hammer blows. As explained by Remspecher et al. [10] and Le and Rackwitz [30], this technique relies on cyclic excitation to reduce penetration resistance and promote progressive embedment. It is particularly attractive because it may reduce underwater noise and, in some soil conditions, facilitate penetration and shorten installation time compared with conventional impact driving [27]. In favourable cases, field and industry experience reported by Shahrour [6] indicates that vibratory installation may reduce installation time by a factor of about 3 to 4, while the clamped vibrator system may also facilitate handling, partial extraction or repositioning during installation.

At the same time, vibratory driving raises specific engineering questions. In their experimental investigation, Remspecher et al. [10] showed that vibratory installation can induce density changes in the direct vicinity of the pile, highlighting the importance of near-pile soil evolution during penetration. More recently, Le and Rackwitz [30] presented a case study on the prediction of monopile driveability under vibratory driving using wave equation analysis, illustrating the current effort to improve the prediction and control of vibratory installation in offshore practice. These aspects are directly relevant to the present research, since any installation-induced modification of the surrounding soil may influence the later mechanical behaviour of monopile-supported offshore wind turbines.



Figure 11: Example of offshore monopile installation by vibratory driving. Source: CAPE Holland [8].

Figure 11 shows the same general offshore configuration, but with the vibrator clamped at the pile head instead of a hammer. Visually, this underlines the main conceptual dif-

ference with impact driving: penetration is achieved through continuous cyclic excitation and static surcharge rather than through a succession of short impulsive blows.

3.5 Physical mechanisms and governing parameters of monopile installation

The installation techniques introduced above are examined here from a more physics-based perspective in order to clarify how the interaction between the installation system, the pile and the surrounding soil may influence the subsequent mechanical behaviour of the foundation. For impact driving and vibratory driving, the main mechanisms, governing quantities and relevant operating parameters are presented, with particular attention to the quantities likely to affect the post-installation soil state and the later lateral response of the monopile.

3.5.1 Impact driving of monopiles

General principle and hammer–pile–soil interaction.

Impact driving is the most established installation method for offshore wind monopiles and remains the reference solution in current offshore practice [27]. In this method, a hydraulic hammer repeatedly strikes the pile head, transmitting short-duration high-intensity impulses to the steel tube. During each blow, the ram is lifted to a given height and released; its kinetic energy is transferred to the pile head upon impact, generating a compressive stress wave that propagates along the pile and mobilises the resistance of the surrounding soil at the shaft and at the pile toe.

From a physical point of view, impact driving is therefore a highly dynamic and transient hammer–pile–soil interaction process. Compared with a purely static penetration process, it involves repeated loading and unloading cycles, progressive mobilisation of shaft and base resistance, redistribution of stresses around the pile, and dissipation of energy through soil deformation, interface friction and other mechanisms. As shown by Chen et al. [9], impact driving can be interpreted as an energy-dissipating process in which the system evolves from one equilibrium state to another after each blow. This interpretation is particularly relevant for monopiles, for which the driving process may require a large number of blows and significant cumulative energy.

Governing equations and key quantities.

The mechanics of a single hammer blow are summarised in Figure 12. The left-hand sketch distinguishes the main stages of the blow, from ram impact to stress-wave transmission into the pile, while the right-hand plot shows the corresponding idealised force–time history. In practice, the loading is strongly transient: a short-duration force pulse is generated at the pile head, reaches a peak value, and then decays rapidly as the wave propagates and energy is dissipated in the pile–soil system. This figure is therefore useful to interpret the simplified equations introduced below, which aim to relate the applied impact force, the maximum force level, and the hammer energy.

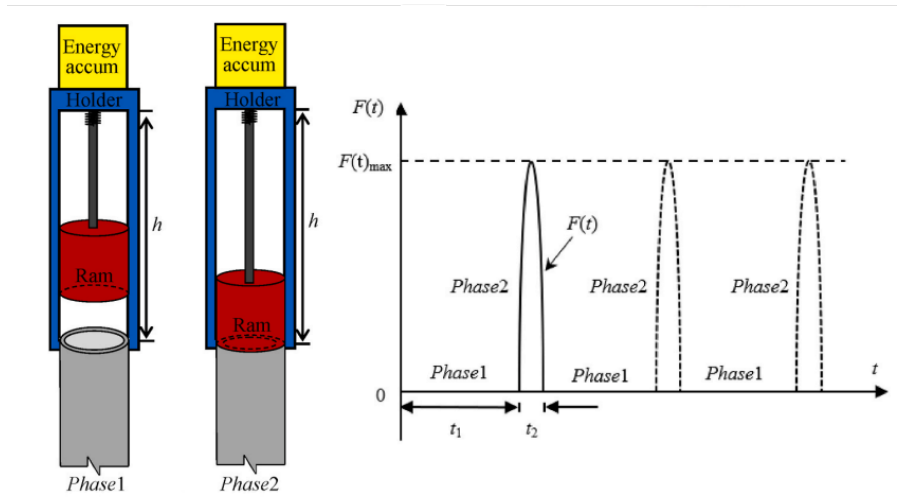


Figure 12: Schematic representation of the two main phases of a hammer blow and associated simplified impact force–time history during monopile installation by impact driving, adapted from Chen et al. [9].

Force transmitted at the pile head.

A first simplified description of impact loading consists in expressing the total load applied at the pile head as

$$F_c(t) = F(t) + Mg \quad (1)$$

with:

- $F_c(t)$: total axial load acting at the pile head [N];
- $F(t)$: time-dependent dynamic impact force [N];
- Mg : static contribution associated with the hammer deadweight [N].

This expression is useful because it separates the transient force generated by the hammer blow from the static vertical force due to the hammer system itself.

Following the simplified representation adopted by Chen et al. [9], the impact force during the short hammer–pile contact phase may be approximated by a sinusoidal law:

$$F(t) = f_{\max} \sin\left(\frac{\pi t}{t_2}\right) \quad (2)$$

where:

- $f_{\max}$: maximum instantaneous impact force [N];
- t_2 : impact duration [s].

Although simplified, this law is physically meaningful because it reproduces the short rise and decay of the contact force during a hammer blow and provides a direct bridge between the idealised force–time curve of Figure 12 and the analytical description of the loading pulse.

By combining impulse and free-fall considerations, Chen et al. [9] obtained the following expression for the peak impact force:

$$f_{\max} = \frac{\pi m \sqrt{2gh}}{2t_2} \quad (3)$$

with:

- m : ram mass [kg];
- g : gravitational acceleration [m/s^2];
- h : drop height [m];
- t_2 : impact duration [s].

This relation shows clearly that the peak force increases with ram mass and drop height, whereas it decreases when the impact duration becomes longer. It is therefore a convenient way to connect hammer settings to the force level effectively transmitted to the pile head.

Hammer energy.

At a more general level, the delivered hammer energy may be related to the ram mass and drop height through:

$$E_h \approx \eta_h mgh \quad (4)$$

where:

- E_h : hammer energy effectively delivered to the system [J];
- η_h : hammer efficiency factor [-];
- mgh : nominal gravitational potential energy of the ram before impact [J].

This equation is important because it distinguishes nominal hammer energy from the smaller energy actually transferred to the pile. In practice, part of the energy is dissipated in the hammer, anvil, cushions and surrounding medium, so drivability analyses must always account for this difference between rated and transferred energy. This distinction is consistent with the CPT-based drivability considerations discussed by Byrne et al. [31] for monopiles in North Sea sands.

Energy balance during penetration.

A useful contribution of Chen et al. [9] is the explicit formulation of the energy balance during a hammer blow:

$$E_{\text{Balance}} = E_I + E_F + E_V + E_K - E_W \quad (5)$$

where:

- E_I : internal energy of the system;
- E_F : energy dissipated by pile–soil contact forces;
- E_V : energy dissipated by viscous effects;
- E_K : kinetic energy of the pile–soil system;
- E_W : cumulative driving energy delivered by the hammer.

This formulation shows how the supplied energy is partitioned between deformation, dissipation and transient motion. For monopiles, only part of the driving energy contributes to immediate pile motion, while a major share is dissipated through soil deformation and interface mechanisms. Chen et al. [9] also showed that the relative contribution of kinetic energy becomes smaller as penetration proceeds and resistance increases with depth.

Penetration resistance and drivability.

The ability of a pile to advance under repeated hammer blows is commonly discussed in terms of drivability. In that context, the resistance opposing installation may be decomposed into shaft resistance and base resistance:

$$F_t = F_{\text{shaft}} + F_b \quad (6)$$

where:

- F_t : total penetration resistance [N];
- F_{shaft} : shaft resistance [N];
- F_b : base resistance [N].

This decomposition is fundamental because it makes explicit that penetration is governed by both side friction and toe resistance. In offshore drivability analyses, the same idea underlies the concept of Soil Resistance to Driving (SRD), often derived from CPT data while accounting for monopile-specific effects such as partial mobilisation of base resistance, friction fatigue and possible plugging. These aspects are discussed in detail by Byrne et al. [31] for monopile driveability in North Sea sands. For large-diameter open-ended piles, the balance between shaft and base resistance may differ markedly from that of smaller piles, which justifies dedicated monopile formulations.

Installation record variables.

Two practical quantities are particularly important during impact driving:

- the *hammer blow count*, i.e. the number of blows required to reach a given penetration depth;
- the *penetration depth per blow* (or set per blow), which quantifies the permanent advance produced by one impact.

These two variables are direct indicators of installation efficiency. Chen et al. [9] showed that, as penetration depth increases, hammer blow counts increase while penetration depth per blow decreases, reflecting the progressive growth of penetration resistance. This trend is fully consistent with the general understanding of monopile drivability in offshore practice.

Indicative offshore operating ranges.

As practical orders of magnitude, full-scale hydraulic hammers used for monopile installation may involve ram masses of about 100 to 320 t, blow frequencies of roughly 20 to 30 blows per minute, delivered energies of about 1000 to 6000 kJ per blow, and drop heights commonly in the range of 1 to 2 m. These values are only indicative and depend on pile size, soil profile and hammer type, but they illustrate the high energy levels involved in offshore impact driving.

Main parameters governing impact driving.

The main parameters governing impact driving can be grouped into hammer-related, pile-related and soil-related quantities. Hammer-related parameters include ram mass, drop height, impact duration, blow frequency and hammer efficiency. According to the simplified force and energy expressions above, increasing the ram mass or the drop height increases both the peak impact force and the energy delivered to the pile, while hammer efficiency controls the difference between rated and actually transferred energy [31, 9].

Pile-related parameters mainly include pile diameter, wall thickness, embedded length, mass and stiffness. Larger diameters generally increase the amount of soil displaced and the pile–soil interface area, leading to higher penetration resistance and more demanding installation conditions. For open-ended monopiles, plugging may also modify the effective base and shaft resistance mobilised during installation, although its importance remains case-dependent.

Finally, soil-related parameters such as relative density, layering, state of stress, saturation, interface properties and cone resistance q_c control the resistance developed during

penetration. Denser or stronger soils generally increase the required number of blows and reduce the penetration achieved per blow. Soil response may also evolve during installation because repeated impacts modify stress levels and interface conditions around the pile, which may later influence the lateral response compared with wished-in-place assumptions. This possible influence of impact-driven installation on the subsequent response is also highlighted by Chan et al. [28].

Relevance for the present research.

For the purposes of this thesis, impact driving provides a useful reference installation method because its governing mechanisms are comparatively well established and its installation record is relatively easy to interpret. The main variables of interest are the delivered hammer energy, the number of blows, the penetration depth per blow, and the corresponding evolution of penetration resistance with depth. Beyond construction feasibility, these quantities are relevant because they are linked to the amount of energy dissipated in the soil and therefore to the degree of installation-induced modification of the near-pile ground.

In this perspective, impact driving should not only be viewed as a means of reaching the target penetration depth. It should also be regarded as a dynamic soil–structure interaction process capable of altering the mechanical state of the surrounding soil mass. This makes it particularly relevant when comparing different installation methods and when investigating the potential influence of installation on the post-installation lateral response of monopiles.

3.5.2 Vibratory driving of monopiles

General principle.

Vibratory driving is an installation technique in which the pile is driven into the ground by the application of continuous vertical oscillations at the pile head rather than by a succession of discrete hammer blows. In practice, the vibrator is clamped to the pile and generates a harmonic vertical force through the rotation of eccentric masses. Under the combined action of this dynamic excitation and the static load applied by the pile–vibrator system, the penetration resistance of the surrounding soil may be reduced sufficiently to allow progressive embedment of the monopile. This vibrator–pile–soil interaction is described in detail by Bienen et al. [32] in the context of vibro-driven installation in sand.

For offshore monopiles, vibratory driving has attracted increasing interest because it may reduce underwater noise emissions and, in favourable soil conditions, shorten installation time compared with conventional impact driving [10, 30]. However, the physical mechanisms governing penetration remain more difficult to interpret than in impact driving, since the installation process depends on the coupled response of the vibrator, the pile, the surrounding soil, and the applied crane load. As a result, vibratory installation must be understood as a dynamic soil–structure interaction problem in which the driving outcome is controlled not only by the force generated by the vibrator, but also by the way this force is transmitted through the pile and resisted by the soil.

Physical description of the vibrator–pile–soil interaction.

The vertical oscillatory force applied during vibratory driving is produced by counter-rotating eccentric masses within the vibrator. Their rotation generates a sinusoidal ver-

tical excitation while the horizontal force components cancel each other out, as described by Bienen et al. [32]. This cyclic loading induces repeated acceleration and deceleration of the pile, which affects the stress state, contact conditions and soil fabric at the pile–soil interface.

In granular soils, vibratory driving may temporarily reduce penetration resistance through repeated changes in inter-particle contact conditions, stress redistribution around the shaft and toe, and rearrangement of the soil fabric in the direct vicinity of the pile. This mechanism should therefore not be interpreted only as a means of facilitating penetration: it can also modify the local state of the surrounding soil. This point was experimentally investigated by Remspecher et al. [10], who showed that vibratory driving can induce measurable density changes around the pile, with local densification and local loosening depending on the position relative to the shaft and along the embedded depth.

This observation is directly relevant to the present work. The lateral response of a monopile is governed mainly by the near-pile soil mobilised during lateral loading. Consequently, if vibratory driving modifies the density, contact conditions or stress state in this region, it may also modify the force–displacement response measured after installation. This is one of the reasons why the experimental programme compares vibratory-driven and impact-driven piles using the same lateral loading procedure.

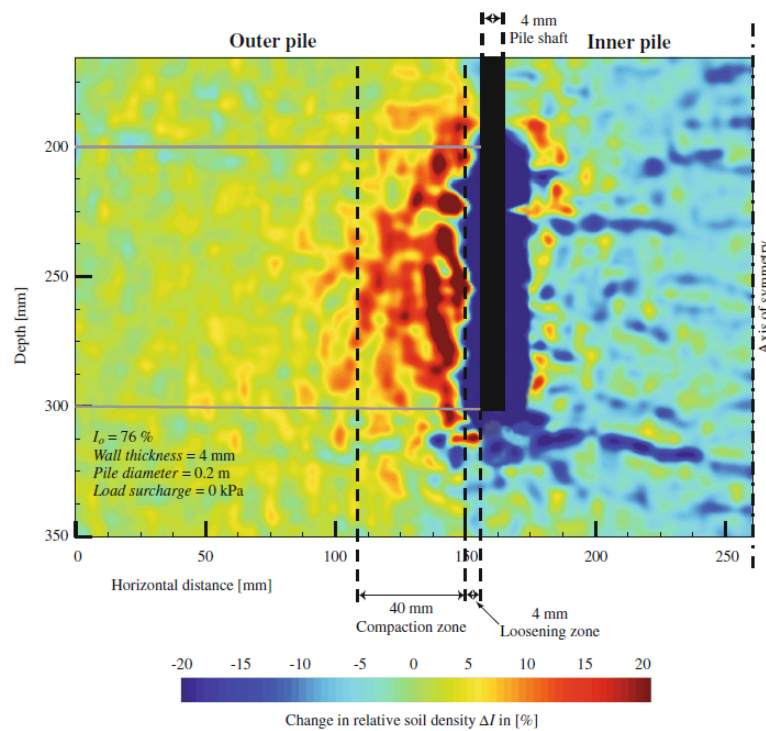


Figure 13: Example of measured change in relative soil density around a vibratory driven monopile, adapted from Remspecher et al. [10].

Figure 13 illustrates that the installation effect is spatially dependent. The soil state after vibratory driving cannot therefore be represented by a single global density value, which reinforces the need to consider installation effects when interpreting the lateral loading tests.

Additional insight into the local soil mechanisms is provided by the model tests of Vogelsang et al. [33] on soil deformation around pressed-in and vibrated pile tips. They reported that the soil below the pile tip is first pushed downward and then progressively displaced sideways, while a slight uplift of grains may occur after the tip has passed. In addition, both cyclic and vibro-driven penetration showed a tendency of the soil adjacent to the shaft to move towards the pile after passage of the tip. This behaviour was interpreted as an indication of stress relaxation and friction fatigue. In physical terms, this suggests that the radial stresses close to the pile shaft may relax during cyclic or vibratory penetration, which can reduce shaft friction and modify the final stress state around the installed pile.

A further difficulty is that the outcome of vibratory driving is not governed by a unique parameter. The penetration process depends on the interplay between dynamic force amplitude, vibration frequency, dynamic mass, static surcharge, crane tension, pile geometry and soil conditions [32]. In other words, two tests carried out with similar frequencies may still lead to different installation responses if the static force, hook load or pile mass differ. This complexity explains why the driveability of monopiles under vibratory driving is still an active research topic.

Figure 14 helps visualise this mechanism. The left-hand side shows the vibrator clamped to the pile head and imposing repeated vertical oscillations to the pile–soil system. The upper-right plot highlights the approximately harmonic time history of the dynamic force, while the lower-right plot recalls that the excitation is dominated by a controlled vibration frequency. Read together, these three views explain why vibratory driving is analysed through quantities such as dynamic force amplitude, angular frequency and displacement amplitude rather than through blow-by-blow impact variables.

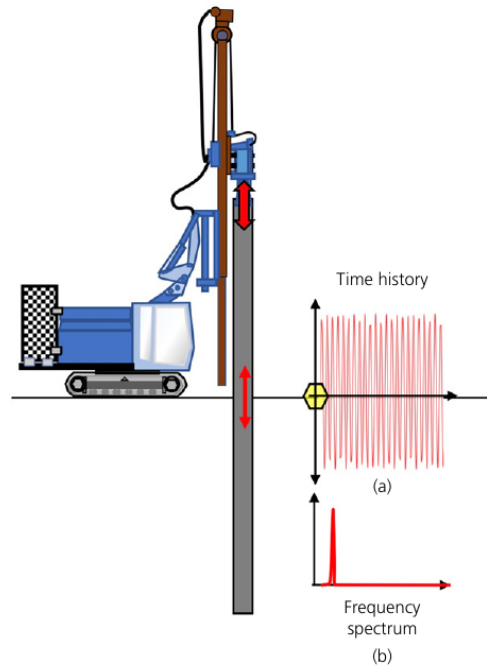


Figure 14: Schematic representation of the operating principle of vibratory pile driving and associated cyclic excitation, adapted from Massarsch et al. [11].

Governing equations and key quantities.

Vertical dynamic force generated by the vibrator.

A simplified expression for the harmonic vertical force generated by the vibrator is:

$$F_v(t) = M_e \omega^2 \sin(\omega t) = F_e \sin(\omega t) \quad (7)$$

where:

- $F_v(t)$: instantaneous vertical dynamic force applied by the vibrator [N];
- M_e : eccentric moment of the counter-rotating masses [kg m];
- ω : angular frequency of the vibrator [rad/s];
- $F_e = M_e \omega^2$: maximum centrifugal force generated by the vibrator [N].

This equation describes the sinusoidal vertical excitation produced by the rotating eccentric masses. It is central to vibratory driving because it links the vibrator characteristics directly to the oscillatory force transmitted to the pile [32].

The angular frequency is related to the vibration frequency f by:

$$\omega = 2\pi f \quad (8)$$

so that frequency has a particularly strong influence on the force level applied to the pile. Since the force amplitude is proportional to ω^2 , even a moderate increase in frequency may significantly increase the vibratory excitation.

However, the use of $F_{\text{dyn,max}}$ as a comparison parameter does not imply that the vibratory installation process is fully governed by this quantity alone. As shown by the experimental observations of Remspecher et al. [10], the final state of the soil around the

pile depends on the coupled response between the imposed vibration, the pile motion and the surrounding granular medium.

Total load applied during vibratory driving.

In an equipment-oriented notation commonly used to describe vibratory hammers, the net stationary load applied to the pile may be written as:

$$F_s = (M_{eb} + M_{ib} + M_{clamp})g - T \quad (9)$$

where:

- F_s : net stationary load on the pile [N];
- M_{eb} : mass of the exciter block [kg];
- M_{ib} : mass of the isolator block [kg];
- M_{clamp} : mass of the clamping device [kg];
- T : suspension force applied by the crane [N].

This expression is useful because it makes explicit how the weight of the vibrator assembly and the crane support combine to produce the net downward stationary load on the pile. It is fully consistent with the more general form commonly used in the recent literature:

$$F_0 = (m_{dyn} + m_{sup})g - F_{hook} \quad (10)$$

where m_{dyn} is the dynamically excited mass, m_{sup} the mass of the suppressor housing or upper part of the system, and F_{hook} the crane hook load. The important point is that crane control modifies the net static contribution without changing the vibration frequency itself.

Free-hanging amplitude.

In practical vibrator specifications, the free-hanging double amplitude is often written as:

$$S_{sp} = 2S_0 = \frac{2M_e}{M_{eb} + M_{clamp}} \quad (11)$$

where:

- S_{sp} : specified free-hanging double amplitude [m or mm];
- S_0 : corresponding single amplitude [m or mm];
- $M_{eb} + M_{clamp}$: effective vibrating mass of the free-hanging system [kg].

This form is useful because manufacturer data are frequently reported in terms of double amplitude rather than single amplitude. It conveys the same physical idea as the simpler expression for the single free-hanging amplitude:

$$s_0 = \frac{M_e}{m_{dyn}} \quad (12)$$

where m_{dyn} is the dynamically excited mass of the vibrator. Both expressions show that, for a given eccentric moment, the displacement amplitude decreases when the vibrating mass increases. During actual installation, the pile and clamping system become part of the vibrating assembly, and the effective amplitude is therefore usually smaller than the free-hanging value.

Driveability and wave-equation-based interpretation.

Although vibratory driving differs fundamentally from impact driving, recent work has shown that one-dimensional wave-equation-based approaches can still be used to analyse monopile driveability under vibratory loading. For example, Le and Rackwitz [30] showed that GRLWEAP-based simulations using CPT-derived site information can reproduce the measured penetration behaviour of a real offshore monopile with reasonable agreement. This confirms that vibratory penetration can still be interpreted through the balance between applied loading and soil resistance, provided that both dynamic quantities, such as frequency and eccentric moment, and installation-control quantities, such as hook load and operational mode, are considered.

Indicative offshore operating ranges.

As practical orders of magnitude, full-scale vibratory hammers for large pile installation generally operate at frequencies of several tens of hertz, with representative offshore values around 20 to 30 Hz. Large offshore vibrators may generate centrifugal forces of several MN, while free-hanging displacement amplitudes are typically of the order of a few millimetres to a few centimetres. These values remain indicative because the actual motion transmitted to the pile depends on pile mass, clamp conditions, crane control, soil resistance and vibration mode.

Main parameters governing vibratory driving.**Vibrator-related parameters.**

The first family of governing parameters is directly related to the vibrator. The most important ones are the vibration frequency, angular frequency, eccentric moment, dynamic mass and displacement amplitude. Among them, frequency plays a central role because the dynamic force amplitude is proportional to ω^2 . Increasing frequency therefore generally increases the force transmitted to the pile. Likewise, increasing the eccentric moment leads to a higher dynamic force amplitude. As an order of magnitude, Shahrou [6] reports that a representative operating frequency for offshore vibratory installation is often around 23 Hz, although the actual working range depends on the equipment, the pile properties and the desired soil response.

However, these effects cannot be interpreted independently of the system mass. A larger dynamically excited mass reduces the free-hanging amplitude and may alter the actual vibratory response during driving. In practice, the pile itself forms part of the vibrating system, so the response evolves as installation progresses.

The influence of vibration conditions on the pile–soil interaction can also be interpreted in terms of resonance-related mechanisms, as illustrated in Figure 15. This figure is particularly useful because it does more than show a pile vibrating in the ground: it suggests that different vibration conditions may mobilise different soil-response patterns around the shaft and toe. In particular, it highlights the possible development of distinct zones around the pile, including regions associated with strong cyclic shearing, arching effects, compaction and resonance-related response. The figure therefore supports the idea that frequency and amplitude do not simply change the force level applied to the pile; they may also change the way stresses are redistributed and how the surrounding granular skeleton evolves during penetration.

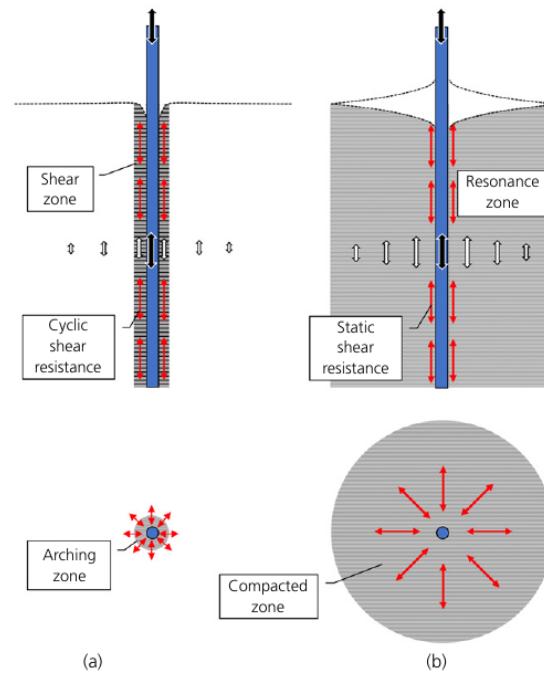


Figure 15: Conceptual illustration of pile–soil interaction during vibratory driving under different vibration conditions, adapted from Massarsch et al. [11].

Installation-control parameters.

A second family of parameters is related to the way vibratory driving is controlled in practice. These include the hook load, the operational mode (free-hanging or crane-controlled), the penetration speed, and the duration of vibration. Among them, hook load is especially important because it directly changes the net static force applied to the pile without changing the dynamic excitation.

The distinction between free-hanging and crane-controlled driving is also particularly relevant. In free-hanging conditions, the weight of the pile–vibrator system is fully transferred to the soil, whereas under crane-controlled conditions part of this weight is supported by the crane. Experimental work in dense sand showed that this distinction may significantly influence the subsequent lateral behaviour of the pile. In the tests reported by Peccin da Silva et al. [34], crane-controlled vibrated piles exhibited an initial lateral stiffness similar, although slightly lower, to that of impact-driven piles, whereas free-hanging vibrated piles showed a markedly softer monotonic response.

Pile- and soil-related parameters.

As for impact driving, pile geometry and soil conditions remain essential. The pile diameter, embedment length, mass and stiffness influence the response of the vibrating system, while soil density, saturation state, layering and stress level control the resistance developed against penetration. Vibratory driving is generally considered more favourable in granular and relatively loose to medium dense soils than in stiff cohesive soils, where the reduction in penetration resistance is less pronounced [30].

Experimental observations also indicate that soil saturation and the surrounding bound-

ary conditions may affect the installation response. More fundamentally, the near-pile soil behaviour under vibratory penetration differs from that of monotonic penetration because cyclic loading may induce stress relaxation effects and a specific evolution of contact conditions along the shaft.

Specific limitations, engineering challenges and relevance for the present work.

Despite its advantages, vibratory driving raises specific engineering challenges. Bearing capacity is not directly inferred from installation records in the same way as for impact driving, which complicates field control and performance assessment. Installation efficiency may also be highly sensitive to hook load and operational choices near refusal, while cyclic loading may simultaneously affect contact forces, stress redistribution, pore pressure generation in saturated soils, and the dynamic response of the vibrating system.

For this reason, vibratory driving is promising but still less standardised than impact driving for offshore monopiles. In the present work, the vibration frequency is the main controllable parameter of the vibratory driving system, while the eccentric moment is imposed by the geometry of the device. The corresponding theoretical dynamic force is therefore used only as an indicative reference parameter, not as a directly measured installation force. The resulting penetration behaviour and post-installation lateral response are then used to assess how vibratory driving modifies the pile–soil system.

4 Literature review and objectives of the present study

4.1 Synthesis of the literature

The literature reviewed in the previous chapters shows that the mechanical behaviour of offshore monopiles is governed by the coupled response of the pile and the surrounding soil. Unlike conventional slender piles, offshore wind monopiles are large-diameter foundations mainly subjected to horizontal loads and overturning moments. Their response is therefore strongly influenced by the mobilisation of lateral soil resistance, pile rotation and the evolution of the stress state around the embedded part of the pile. This specific behaviour of offshore monopiles is discussed in particular by Wu et al. [22] and Arany et al. [24], who emphasise the role of pile geometry, embedment length, stiffness and soil–structure interaction in monopile design.

Classical design approaches based on $p - y$ curves have long been used to represent the response of laterally loaded piles. However, several authors have shown that these methods were originally developed for long and slender piles and are not fully adapted to the behaviour of modern large-diameter monopiles. Large offshore monopiles may behave as relatively short and rigid foundations, for which global rotation, base reactions and distributed moment reactions become important. This limitation motivated the development of more advanced design frameworks, such as the PISA approach proposed by Burd et al. [26]. This framework introduces additional soil reaction components and provides a more complete description of monopile response in sand. These developments also highlight the importance of the embedment ratio L/D , since the relative embedded length controls the balance between shaft resistance, base reaction and global pile rotation.

Although real offshore monopiles are subjected to complex cyclic and irregular loading conditions, monotonic lateral loading remains an essential reference for understanding

their mechanical response. Richards [25] discusses the importance of cyclic lateral loading for offshore wind monopiles, but monotonic tests remain useful for characterising the initial stiffness, lateral resistance, rotation pattern and global kinematics of the pile–soil system. In this context, the force–displacement response at mudline level, the moment–rotation response and the position of the apparent rotation point provide useful indicators for comparing different pile configurations and different installation histories.

A central conclusion of the literature is that the installation method should not be considered only as a construction operation. During penetration, the pile interacts strongly with the surrounding soil and may modify its density, stress state, fabric and interface conditions. These installation-induced modifications may subsequently affect the lateral response of the pile. This point was directly addressed by Gavin et al. [29], who studied the influence of installation method on the monotonic lateral loading response of monopiles in sand. Their work supports the idea that the post-installation behaviour of a monopile cannot be interpreted independently from the process by which it was installed.

Impact driving is the most established installation method for offshore monopiles and therefore provides an important reference case. Chen et al. [9] analysed impact driving as an energy-dissipating hammer–pile–soil interaction process, showing how the delivered energy is partitioned into pile motion, soil deformation and dissipation mechanisms. Byrne et al. [31] focused on CPT-based driveability assessment for monopiles in North Sea sands, which is particularly relevant because it links penetration resistance to soil characterisation data. More recent numerical work by Chan et al. [28] also indicates that impact-driven installation can significantly modify the surrounding soil response and may influence the subsequent lateral behaviour of the monopile. Taken together, these studies show that impact driving can be interpreted through installation indicators such as impact energy, number of blows, penetration per blow and cumulative driving energy, but also that these indicators may be linked to changes in the near-pile soil state.

Vibratory driving has received increasing attention as an alternative installation method, but its influence on post-installation monopile behaviour remains less clearly established. Massarsch et al. [11] provide a detailed state-of-practice review of vibratory pile driving and identify the main parameters governing the process, including frequency, eccentric moment, dynamic force, displacement amplitude, static surcharge and crane control. Remspecher et al. [10] experimentally showed that vibratory installation can induce measurable changes in relative density around a monopile, with zones of densification and loosening depending on the position relative to the pile. Vogelsang et al. [33] also investigated soil deformation around pressed-in and vibrated pile tips, highlighting local soil displacement, stress redistribution and rearrangement of the granular material during penetration. These studies are particularly relevant because they show that vibratory driving does not only affect the ease of penetration, but may also modify the soil that is later mobilised during lateral loading.

Recent studies have also focused on the prediction and modelling of vibratory installation. Le and Rackwitz [30] investigated the prediction of monopile driveability under vibratory driving using wave-equation analysis, while Bienen et al. [32] studied vibro-driven pile installation in sand using numerical modelling. These works illustrate the current effort to better predict the penetration behaviour of vibratory-driven monopiles. However, they also show that the response depends on the coupled behaviour of the vibrator, the pile and the surrounding soil, and that the post-installation consequences of

vibratory driving remain more difficult to assess than the installation process itself.

Among the available experimental studies, the work of Peccin da Silva et al. [34] is particularly relevant to the present thesis. Their laboratory study directly investigated the effect of installation parameters on the lateral behaviour of monopiles in sand. Their results suggest that the installation configuration, including the way the vibrator is operated, may significantly affect the stiffness and resistance measured during subsequent lateral loading. This confirms that the installation method and its operating parameters must be considered when interpreting the mechanical response of monopile foundations.

The present work also builds on a sequence of experimental investigations carried out at UCLouvain. Hamoir and Hoet [35] initiated the experimental study of the influence of pile installation on the lateral behaviour of model piles in sand. Their work included the development of an initial laboratory setup, soil characterisation and preliminary lateral loading tests after impact driving. Deckers and Defer [36] then developed a more complete reduced-scale apparatus and investigated the influence of impact-driving parameters on the monotonic lateral response of offshore monopiles. Their results showed that the embedment ratio L/D has a strong influence on lateral resistance, while the influence of ram mass alone appeared more limited. More recently, Shahrou [6] extended the research direction towards vibratory driving by developing and testing a model-scale vibro-driving system.

Despite these contributions, a direct experimental comparison between impact-driven and vibratory-driven model monopiles under a common laboratory protocol remains limited. In particular, the previous UCLouvain studies progressively developed the experimental basis for impact driving and vibratory driving, but did not provide a systematic comparison of the two installation methods using the same sand preparation procedure, pile geometry and monotonic lateral loading configuration. This gap motivates the present study, which focuses on the relative influence of installation method and installation parameters on the post-installation lateral response of model monopiles in sand.

Overall, the literature indicates that monopile behaviour cannot be described solely from pile geometry or initial sand density. The installation procedure may significantly modify the near-pile soil state and therefore influence the subsequent lateral response. Impact driving is relatively well documented and provides a useful reference installation method, whereas vibratory driving remains less fully understood, especially regarding its consequences for post-installation lateral stiffness, resistance and pile kinematics. This observation provides the basis for the objectives of the present experimental study.

4.2 Objectives of the present study

Based on the literature synthesis, the present study focuses on the influence of pile installation on the subsequent lateral response of model monopiles in sand. The general motivation is that the installation procedure may modify the near-pile soil state and therefore affect the mechanical behaviour measured during lateral loading. This issue is particularly relevant when comparing impact driving and vibratory driving, since the two methods mobilise the soil through different physical mechanisms.

Impact driving is used as the reference installation method because it is widely used for offshore monopiles and has already been investigated in previous experimental work. Vibratory driving is considered as an alternative method because it may offer practical

and environmental advantages, but its influence on the post-installation lateral response remains less clearly established. The objective is therefore not only to compare the two techniques in terms of penetration behaviour, but also to evaluate whether they lead to different lateral stiffness, resistance and pile kinematics after installation.

The experimental programme is structured around five main objectives.

The first objective is to compare the post-installation lateral response of impact-driven and vibratory-driven piles under the same monotonic lateral loading procedure. The comparison is mainly based on the force–displacement response at mudline level, with particular attention to characteristic force levels, initial stiffness and resistance mobilised up to a reference displacement. This allows the influence of the installation method to be assessed while keeping the pile geometry, sand preparation procedure and loading configuration as consistent as possible.

The second objective is to study the influence of the embedment ratio L/D . This parameter is central for offshore monopiles because it controls the relative embedded length of the pile and therefore influences the balance between lateral soil resistance, pile rotation and base effects. In the present work, several embedment ratios are tested for both installation methods in order to determine whether the effect of L/D follows similar trends for impact-driven and vibratory-driven piles.

The third objective is to investigate the influence of the impact-driving configuration. For this purpose, several combinations of falling mass and drop height are considered while keeping the nominal impact energy approximately constant. This makes it possible to assess whether the post-installation response depends only on the nominal impact energy, or whether different impact configurations can produce different installation histories and different lateral responses.

The fourth objective is to evaluate the influence of the vibratory-driving frequency. In the vibratory test series, the eccentric moment is kept constant, while the motor speed is varied. Since the dynamic force generated by the vibrator depends strongly on the excitation frequency, this parameter is expected to influence both the penetration process and the soil state around the pile. The corresponding lateral loading tests are therefore used to determine whether changes in vibratory frequency lead to measurable differences in the post-installation response.

The fifth objective is to provide a kinematic interpretation of the pile response during lateral loading. In addition to the force–displacement curves, the two laser displacement measurements are used to estimate the mudline displacement, the pile rotation and the apparent depth of the rotation point. These quantities make it possible to go beyond a purely global comparison of resistance and stiffness, and to analyse how the pile moves within the soil during loading. This kinematic interpretation is particularly useful for assessing whether the pile response remains consistent with a mainly rigid-body rotation mechanism.

Overall, the present study aims to contribute to the experimental understanding of installation effects on monopile behaviour by comparing impact-driven and vibratory-driven model piles under controlled laboratory conditions. The results are intended to identify relative trends at model scale rather than to provide direct full-scale design values. They are therefore interpreted as comparative evidence of how installation method and installation parameters may influence the subsequent lateral response of monopile foundations in sand.

5 Experimental methodology and test programme

5.1 Overview of the experimental programme

The experimental programme was designed to provide a controlled comparison of the post-installation lateral response of model monopiles in sand. Rather than introducing new theoretical aspects, this section presents the general experimental logic followed in the laboratory: sand preparation, pile installation, lateral loading and data processing.

The campaign was organised around two installation methods, namely impact driving and vibratory driving. For each method, the pile was first installed to a target embedment ratio in a prepared sand mass. A monotonic lateral loading test was then performed using the same loading configuration for all tests. This sequential procedure makes it possible to compare the lateral response of the piles after installation while keeping the pile geometry, sand preparation procedure and loading arrangement as consistent as possible.

A key requirement of the programme was the reproducibility of the sand mass. Before the pile installation tests, the sand pluviation procedure was therefore developed and assessed using visual observations and CPT measurements. This preliminary step was necessary to ensure that differences observed during the lateral loading tests could be interpreted mainly in relation to the installation procedure and its parameters, rather than to uncontrolled variations in the initial sand state.

Once the sand preparation procedure was validated, the installation tests were carried out. The impact-driving series was used to study the influence of embedment ratio and impact configuration, while the vibratory-driving series was used to investigate the influence of embedment ratio and vibration frequency. The penetration behaviour during installation was recorded qualitatively and quantitatively when relevant, as it provides useful information for interpreting the subsequent lateral response.

Finally, the installed piles were subjected to monotonic lateral loading. The response was characterised not only through global force–displacement curves, but also through kinematic quantities derived from the displacement measurements. In particular, the mudline displacement, pile rotation and apparent depth of the rotation point were used to interpret how the pile moved within the soil during loading. The overall experimental programme therefore provides both a mechanical and a kinematic basis for comparing the influence of impact and vibratory installation on model monopiles.

5.2 Scaling considerations for model tests

The experimental campaign presented in this work is carried out at model scale under normal gravity conditions. The model pile used in the tests has a diameter of $D_m = 6$ cm, whereas full-scale offshore monopiles typically have diameters of several metres. The tests must therefore be interpreted within the framework of reduced-scale physical modelling. The aim is not to reproduce a specific offshore monopile at full scale in a strict one-to-one manner, but rather to reproduce the main mechanisms of interest under controlled laboratory conditions.

The scaling approach adopted in the present work is directly inspired by the previous experimental investigation conducted at UCLouvain by Deckers and Defer [36]. Their work is particularly relevant because it addressed the same general problem of reduced-

scale laboratory testing on offshore monopiles and included a detailed discussion of the corresponding scaling laws. In their study, model piles with diameters of 6 cm and 11 cm were used to represent prototype monopiles with diameters of 6 m and 11 m, respectively. This corresponds to a geometrical scale reduction factor equal to $N = 100$. The same order of magnitude is adopted in the present work, so that the 6 cm-diameter model pile can be interpreted as a reduced-scale representation of a 6 m-diameter offshore monopile.

The length scale factor is written as

$$n_L = \frac{L_m}{L_p} = \frac{1}{N},$$

where:

- n_L is the scale factor for lengths;
- L_m is a characteristic length in the model;
- L_p is the corresponding characteristic length in the prototype;
- N is the geometrical scale reduction factor;
- the subscripts m and p refer to the model and the prototype, respectively.

With $N = 100$, the diameter of the prototype corresponding to the model pile is

$$D_p = ND_m = 100 \times 0.06 = 6 \text{ m},$$

where D_m is the model pile diameter and D_p is the corresponding prototype pile diameter. This value is representative of conventional offshore monopile diameters. It should however be noted that modern XXL monopiles may reach larger diameters. The present model should therefore be understood as representative of the lower range of full-scale offshore monopiles rather than of the largest current monopile designs. This scaling interpretation is mainly used for the pile diameter and for the dimensionless ratios governing the tests, such as L/D and h/D , rather than as a strict scaling of every geometrical detail of the experimental setup.

From the length scale factor, the scale factors for areas and volumes follow directly:

$$n_A = \frac{A_m}{A_p} = \frac{1}{N^2}, \quad n_V = \frac{V_m}{V_p} = \frac{1}{N^3},$$

where:

- n_A is the scale factor for areas;
- A_m and A_p are corresponding areas in the model and prototype;
- n_V is the scale factor for volumes;
- V_m and V_p are corresponding volumes in the model and prototype.

These relations follow directly from geometrical similarity, since areas and volumes scale with the square and the cube of the length scale, respectively. If the density of the sand and the density of the pile material are kept unchanged between model and prototype, the mass scale factor also follows the volume scale factor:

$$n_m = \frac{m_m}{m_p} = \frac{1}{N^3},$$

where:

- n_m is the scale factor for mass;
- m_m is a mass in the model;

- m_p is the corresponding mass in the prototype.

However, achieving complete similitude in geotechnical model tests performed under normal gravity is not possible in a strict sense. As discussed by Deckers and Defer [36], physical modelling should ideally satisfy geometrical, kinematic and dynamic similarity. Geometrical similarity concerns the relative dimensions of the model and prototype. Kinematic similarity concerns the similarity of displacement and deformation patterns. Dynamic similarity concerns the similarity of the force ratios governing the mechanical response. In practice, these conditions cannot generally be satisfied simultaneously in a $1g$ laboratory test. Therefore, the experimental design must prioritise the similitude conditions that are most relevant for the objective of the study.

This limitation is particularly important for the present work because the experimental programme involves dynamic installation procedures, namely impact driving and vibratory driving. In a strict similitude framework, reproducing the full-scale installation process would require a consistent scaling of time, velocity, acceleration, force transmission, energy dissipation and soil response during penetration. Such a complete dynamic similitude is not achieved in the present $1g$ model tests.

For impact driving, the imposed quantity is the nominal impact energy,

$$E = mgH_{\text{drop}},$$

where m is the falling mass and H_{drop} is the drop height. This energy is used as a practical control parameter for comparing different impact configurations. However, the energy actually transmitted to the pile, the contact duration, the peak impact force and the detailed stress-wave propagation in the pile–soil system are not scaled in a strict prototype-to-model sense. The impact driving tests should therefore be interpreted as controlled model-scale installation procedures rather than as exact dynamic replicas of offshore hammer driving.

A similar limitation applies to vibratory driving. In the present experimental programme, the vibration frequency is the main controllable parameter of the vibro-driving system, while the eccentric moment is imposed by the eccentric mass installed in the vibro-driver. These quantities define the imposed excitation at model scale, but the coupled response of the vibrator, the pile and the surrounding sand is not a direct scaled reproduction of a full-scale offshore vibratory installation. In particular, the actual pile motion, the force transmitted to the soil and the evolution of penetration resistance depend on the interaction between the model vibrator, the pile geometry and the prepared sand mass.

Consequently, the installation procedures used in this thesis are not intended to satisfy complete dynamic similitude with offshore pile driving. Their purpose is instead to generate different, controlled model-scale installation conditions that may produce different initial states of the soil around the pile. The subsequent lateral loading tests can then be used to compare how these installation-induced states influence the measured pile response. The interpretation of the results must therefore remain comparative and model-scale, rather than directly predictive of full-scale offshore installation behaviour.

A major limitation of $1g$ reduced-scale geotechnical modelling is related to the stress level in the soil. In a dry sand deposit, the vertical stress at a depth z may be written as

$$\sigma_v = \rho g z,$$

where:

- σ_v is the vertical stress in the soil;
- ρ is the soil density;
- g is the gravitational acceleration;
- z is the depth below the sand surface.

In the present model tests, the soil density ρ is not scaled and the gravitational acceleration g is the same as in the prototype, because the experiments are performed under normal gravity. Since the characteristic depth z scales as a length, the scale factor for the soil stress becomes

$$n_{\sigma_s} = n_{\rho} n_g n_L = 1 \cdot 1 \cdot \frac{1}{N} = \frac{1}{N},$$

where:

- n_{σ_s} is the scale factor for stresses in the soil;
- n_{ρ} is the scale factor for density;
- n_g is the scale factor for gravitational acceleration;
- n_L is the scale factor for lengths.

This means that, at corresponding depths, the stress level in the model is approximately N times smaller than in the prototype. This point is important because the mechanical behaviour of sand is stress-dependent. Soil stiffness, dilatancy and strength mobilisation may therefore differ between model and prototype. As a consequence, the laboratory tests cannot be used as a direct quantitative reproduction of the full-scale offshore response without accounting for scale effects.

The use of a 1g model is nevertheless appropriate for the objective of this thesis, because the work focuses mainly on comparative trends. The same pile diameter, sand preparation procedure, installation framework and lateral loading configuration are used throughout the campaign. This makes it possible to compare different installation procedures and test conditions while limiting the number of uncontrolled variables. In other words, the experimental programme is designed to identify relative changes in pile response rather than to provide direct prototype-scale design values.

In this context, dimensionless ratios are especially important. The embedment ratio L/D is used to describe the relative penetration depth of the pile, where L is the embedded length of the pile and D is the pile diameter. The loading height may also be expressed through h/D , where h is the height at which the lateral load is applied above the mudline. These ratios are more meaningful than the absolute values of L , D and h alone, because they describe the relative geometry of the soil–pile system independently of the chosen model scale.

The interpretation of the results must therefore remain consistent with the limitations of the model scale. The measured force–displacement responses should not be directly transposed to a full-scale offshore monopile without additional scaling assumptions. Instead, the results are interpreted as model-scale evidence of how the installation procedure affects the surrounding sand and the subsequent lateral response of the pile. This approach is consistent with the continuity between the present thesis and the previous experimental work carried out at UCLouvain, while extending the comparison between impact and vibratory driving.

5.3 Sand sample preparation: principles and governing parameters

5.3.1 Importance of the initial sand state

General importance of the initial state.

Before studying the installation and post-installation behaviour of model monopiles, particular attention must be paid to the initial state of the sand mass. Indeed, the response of the pile does not only depend on the installation method itself, but also on the condition of the surrounding soil prior to penetration.

In geotechnical model tests, the initial state of the soil is mainly characterised by:

- the density distribution within the sample,
- the stress state inside the soil mass.

As pointed out by Vogelsang et al., this initial state must be known as accurately as possible, especially when the results are intended to be interpreted quantitatively or compared with numerical simulations [37].

Need for reproducibility in the present work.

In the present work, this aspect is particularly important because the objective is not only to install piles in sand, but also to compare the influence of different installation procedures on the later mechanical response of the monopile. If the initial state of the sand is not sufficiently controlled, it becomes difficult to distinguish:

- the effect of the installation method,
- from the effect of variations in the prepared soil sample itself.

For this reason, preparing a sand mass that is as homogeneous and reproducible as possible is a necessary preliminary step before carrying out pile installation tests.

Influence of confinement and container effects.

Another important point highlighted in the literature is that the initial state of sand in laboratory containers cannot simply be assumed to correspond to ideal free-field conditions. In confined devices, the filling process resembles that of a silo, which may lead to a stress distribution different from that of an infinite soil mass [37]. In particular, wall friction may induce so-called *silo effects*, leading to an under-linear increase of vertical stress with depth.

As a consequence:

- two samples may have a similar average density,
- while still exhibiting different internal stress states,
- depending on the geometry of the container and the deposition conditions.

Interest of pluviation for dry sand preparation.

The density distribution itself must also be controlled. For dry cohesionless soils, pluviation is commonly used to prepare samples that are as uniform and repeatable as possible [37]. Nevertheless, even with a well-controlled raining process, a perfectly uniform density cannot be guaranteed. Some fluctuation of void ratio remains unavoidable.

According to Vogelsang et al., these fluctuations can remain limited when the preparation method is carefully executed [37]. This confirms that sand preparation should be

considered as an integral part of the experimental methodology rather than as a secondary practical issue.

5.3.2 Main parameters governing the prepared sand mass

Several parameters govern the quality of the prepared sand mass and must therefore be considered before starting pile installation tests.

Density state.

The first key parameter is the *density state* of the soil. In practice, it may be described through:

- dry density,
- void ratio,
- relative density.

This parameter is essential because it directly affects the mechanical behaviour of sand. Vogelsang et al. also note that the void ratio influences the stress state through the unit weight of the material [37]. In the context of monopile tests, controlling density is therefore necessary to ensure repeatability and meaningful comparison between experiments.

Spatial homogeneity.

A second important aspect is the *spatial homogeneity* of the sample. The purpose of pluviation is precisely to obtain a density distribution that is as uniform as possible throughout the whole soil mass [37].

However, local variations may still appear because of:

- irregularities in the raining process,
- interruptions during filling,
- uneven distribution of the material.

As a result, obtaining a correct average density is not sufficient on its own. The sample must also remain reasonably uniform:

- over depth,
- and across the horizontal extent of the container.

Deposition conditions.

The deposition conditions are also major governing parameters. For dry cohesionless soils, the density achieved by pluviation depends not only on the free-fall height and the intensity of deposition, but also on the way the sand flow is distributed before reaching the sample surface. In air pluviation systems, the characteristics of the perforated plates, the opening size, the spacing between openings and the resulting flow rate may significantly influence the relative density and spatial uniformity of the prepared sand mass [38]. These parameters must therefore be controlled as carefully as possible if several pile installation tests are to be compared under similar initial sand conditions.

This point is particularly relevant in the present work because the objective is not only to fill the tank, but to prepare a sand mass that is sufficiently repeatable and homogeneous for subsequent pile installation and lateral loading tests. Local disturbances in the falling sand flow, changes in the diffuser position or obstacles close to the tank wall may all affect the final density distribution. The pluviation campaign is therefore considered as a necessary validation step before analysing the mechanical response of the installed piles.

Geometry of the container.

The *geometry of the test container* must also be taken into account. As discussed by Vogelsang et al., filling a confined sample may generate silo effects, and these effects become increasingly significant when the ratio between depth and container width increases [37].

This means that the container dimensions influence not only the practical organisation of the tests, but also the internal stress distribution within the sand mass. This point is especially relevant in the present work, since pluviation tests were carried out in:

- a large tank,
- and a smaller tank.

This makes it possible to evaluate whether the size of the container influences the quality and homogeneity of the prepared sample.

Quality control of the prepared mass.

Finally, the *quality control method* used to assess the prepared sample is itself an important part of the procedure. Vogelsang et al. indicate that CPT can be used to investigate density distribution and sample homogeneity, although the results must be interpreted carefully because of possible boundary effects in model tests [37].

Although Digital Image Correlation (DIC) can also be used to evaluate settlements and soil deformation during sample preparation, the present work relies on post-preparation observations and CPT measurements to compare the quality of the pluviation procedures.

5.4 CPT device used in the present work

The penetration tests presented in this work were carried out using a portable electronic cone penetrometer, namely an Eijkelkamp Penetrologger. Devices of this type are commonly used to assess soil penetration resistance and to provide an indication of the mechanical state and spatial variability of the tested material.

In the present work, the device was used with a cone of 1.0 cm^2 base area and a 60° apex angle. The penetration speed was set to 2.0 cm/s , which corresponds to the recommended operating conditions for this type of equipment [39]. These parameters were kept identical throughout the test campaign in order to ensure a consistent comparison between the different pluviation configurations.

The device consists of a handheld measuring unit connected to a slender penetration rod ending with a cone tip. The penetration resistance is recorded as a function of depth and is expressed here through the cone resistance q_c . By comparing the $q_c(z)$ profiles obtained at different locations in the same container, it becomes possible to evaluate, from a qualitative point of view, the apparent homogeneity of the prepared sand mass.

It should however be noted that the device is manually operated. As a result, maintaining a perfectly constant penetration rate throughout the whole test is not always easy in practice, especially when the resistance of the sand increases with depth. For this reason, some profiles do not reach exactly the same maximum penetration depth, and a few curves may also present slight discontinuities or irregularities. These features are not necessarily related to the soil state itself, but may partly reflect the practical difficulty of keeping the penetration perfectly smooth and continuous under manual loading.

These practical limitations must therefore be kept in mind when interpreting the results. In addition, the interpretation of CPT measurements performed at shallow depth

in sand is not straightforward. At low stress levels, the measured cone resistance may be influenced by scale effects, boundary conditions, local density variations and the limited vertical confinement of the sand. Krogh et al. showed that the relationship between shallow CPT resistance and relative density must therefore be interpreted with caution [40].

For this reason, the CPT results obtained in the present work are not used as a direct and absolute measurement of the relative density of the sand mass. Instead, they are mainly used as a comparative tool to assess the repeatability and spatial homogeneity of the pluviation procedure. The interpretation therefore focuses on the shape of the $q_c(z)$ profiles, their dispersion, their repeatability between measurement locations and the coefficient of variation over a selected depth range.



Figure 16: Portable electronic cone penetrometer used in the present work.

5.5 Sand used in the present work

The sand used in the present experimental campaign was the North Sea sand already available in the laboratory and previously used by Hamoir and Hoet [35]. In their work, this material was characterised as a sandy soil, with most of the grains between approximately 0.25 and 4 mm. Their experimental procedure also included sieving the sand with a 2.36 mm sieve in order to avoid clogging the openings of the sand raining system.

No additional grain-size distribution test was performed as part of the present study. The sand should therefore not be considered as freshly characterised material, but rather as the same laboratory sand stock used in the previous experimental work.

A few months before the present tests, the sand was filtered with a coarse mesh in order to remove most of the shell fragments that could block the pluviation system.

It should also be noted that, because the sand has been reused through a large number of pluviation campaigns over several years, part of the finest fraction may have been progressively lost during handling, sieving and cleaning operations. For this reason, the available soil characterisation is used only as a general description of the material, while the present work focuses mainly on the repeatability and homogeneity of the prepared sand mass assessed through CPT measurements.

5.6 Preliminary pluviation campaign

5.6.1 Pluviation tests carried out in the present work

In previous years, experimental work had already been carried out on the preparation of sand samples for model pile tests. As a result, an existing pluviation system was already available in the laboratory and had previously been used in a large tank. Rather than developing a completely new preparation method, the objective of the preliminary tests presented in the following subsections was therefore to assess this existing setup and to identify possible improvements in order to obtain a sand mass that is as homogeneous and reproducible as possible for the next stages of the experimental campaign.

5.6.2 Large tank test

Test setup.

The first pluviation test was carried out in the large tank available in the laboratory. The internal diameter of the tank was 110 cm, while its total height was 126 cm. This tank was equipped with an existing pluviation system already used in previous student works. The objective of the present preliminary tests was therefore not to develop a new preparation device, but rather to assess this existing configuration and identify possible improvements.

The general arrangement of the large tank and the associated pluviation setup is shown in Figure 17. In the test presented here, the distance between the lower end of the pluviation system and the top of the tank was 37 cm, as illustrated in Figure 18. This parameter is important because the free-fall height is known to influence the density of dry sand samples prepared by pluviation. A circular diffuser plate was positioned 16 centimetres above the container in order to distribute the sand flow more evenly and reduce direct jet effects .



Figure 17: General view of the large tank used for the pluviation test and of the existing pluviation system available in the laboratory.

The detailed drawings of the pluviation system are provided in Appendix B.



Figure 18: Measured free-fall height in the large-tank pluviation setup.

Test procedure.

The tank was first filled by pluviation using the existing setup described previously. Once the deposition was completed, the surface of the sample was observed. In order to document the initial state of the deposited sand mass, height measurements were first taken at several locations on the surface before any correction was made. These measurements were used to obtain a first indication of the surface irregularity produced by the pluviation process.

A careful manual levelling was then carried out in order to regularise the top surface before performing the penetration tests. This step was intended to obtain a more consistent reference surface and to limit the influence of local topographic irregularities on the subsequent CPT measurements.

After surface preparation, CPT tests were performed in nine locations distributed over the surface of the tank. The test layout is shown in Figure 19, where the red circles indicate the CPT locations.

The CPT positions were identified by means of two orthogonal strings materialising two perpendicular diameters of the tank. A total of nine penetration locations were defined:

- one central point,
- four points located at approximately $0.3R$,
- four points located at approximately $0.7R$.

This layout was chosen in order to compare the response in the central part of the sample with that measured closer to the container wall. CPT tests were then performed at the selected locations in order to obtain a first indication of the spatial variability of the prepared sand mass.

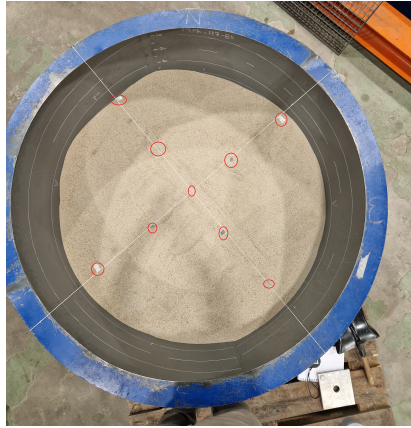


Figure 19: Top view after pluviation.

Results and first assessment.

Surface condition after pluviation. From a qualitative point of view, the final sand surface was clearly uneven. As shown in Figure 20, the deposited surface presented a marked relief, with important height variations across the tank.



Figure 20: Surface of the sand mass in the large tank immediately after pluviation.

Height measurements and filling level. These visual observations were confirmed by the first height measurements, which showed significant differences depending on the measurement location. This confirmed that the deposited surface could not directly provide a flat reference level for the following tests. Here, d denotes the vertical distance measured between the sand surface and the top edge of the tank; consequently, $d_{\max}$ corresponds to the location where the sand level was the lowest, while $d_{\min}$ corresponds to the location where the sand level was the highest.

- $d_{\max} \approx 57$ cm,
- $d_{\min} \approx 38.5$ cm,



Figure 21: Examples of height measurements in the large tank after pluviation and before levelling.

A manual levelling operation was therefore necessary before carrying out the CPT campaign.

CPT results. The CPT results obtained in the nine selected locations are presented in Figure 22. A visible scatter can be observed between the different curves, suggesting that the sample cannot be regarded as homogeneous.

It should be noted that not all CPT profiles reached exactly the same depth. Since the tests were performed manually, maintaining the required constant penetration rate was not always possible; when the rate became too irregular, the device could stop recording the measurement. In some cases, this occurred before the same depth as in the other tests was reached, for example due to a sudden change in penetration resistance or an unfavourable operator position. This remark applies to all CPT results presented in this document.

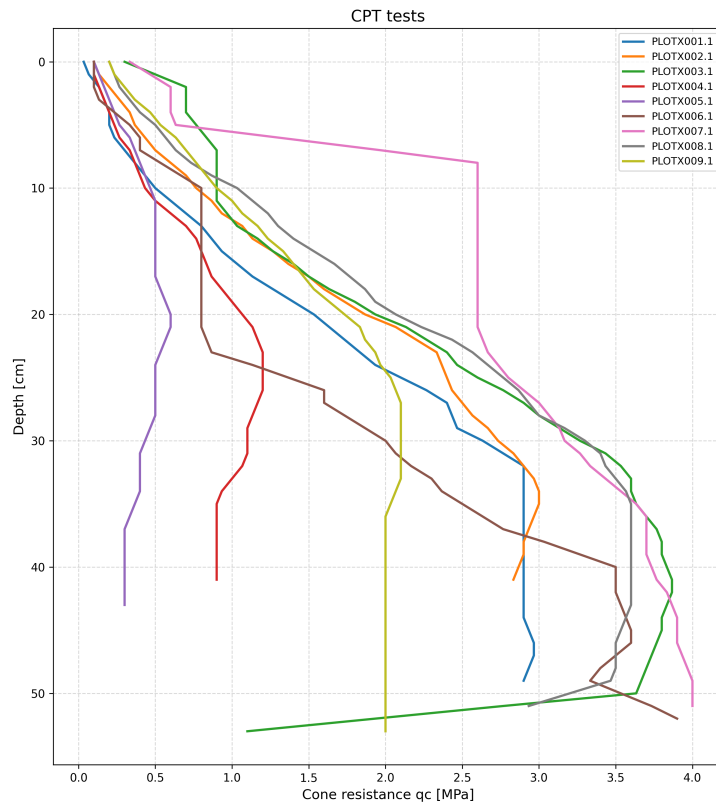


Figure 22: CPT results obtained at the nine test locations in the large tank.

General assessment. Overall, this first pluviation test showed that the existing large-tank setup was usable, but that it produced an irregular surface and noticeable spatial variability in the CPT response. The procedure therefore still had clear limitations in terms of surface regularity and apparent homogeneity.

5.6.3 Small tank tests: common setup and procedure

After the preliminary observations made in the large tank, additional pluviation tests were carried out in a smaller cylindrical container. The objective of these tests was to evaluate whether a reduced container diameter could improve the quality of the deposited sand mass, especially in terms of surface regularity and apparent homogeneity. Two pluviation tests were performed in the small tank. Since the same general setup and procedure were used in both cases, they are presented first, while the results and first assessment are reported separately for each test.

Test setup.

The pluviation tests were carried out in a cylindrical small tank with an internal diameter of 59 cm. The total internal height of the container was 87 cm. The deposition was performed with the same pluviation system but the circular diffuser plate was positioned directly above the container.

The vertical distance between the hopper outlet and the diffuser was set to 37 cm. A protective sheet was also placed on the floor in order to facilitate cleaning of the spilled sand.



Figure 23: Adjustment of the geometric configuration.



Figure 24: Pluviation in progress.

Test procedure.

For each test, the small tank was filled by pluviation. After deposition, the final surface condition was photographed and the geometry of the sample was documented through height measurements. As for the large tank, a first objective was to evaluate the regularity of the final surface produced directly by the pluviation process.

The CPT positions were identified by means of two orthogonal strings materialising two perpendicular diameters of the tank. As for the large tank, a total of nine penetration locations were defined:

- one central point,
- four points located at approximately $0.3R$,
- four points located at approximately $0.7R$.

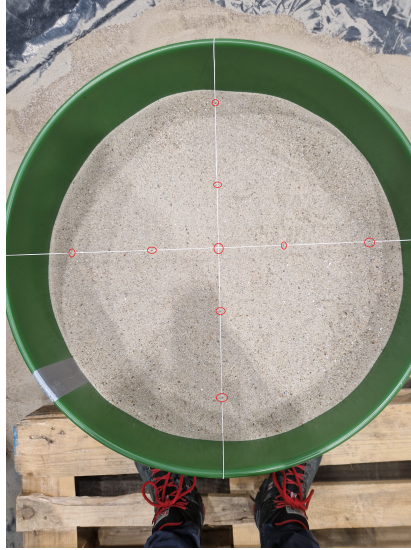


Figure 25: Top view after pluviation: location of the nine CPT test points.

First pluviation test in the small tank

Surface condition after pluviation. From a qualitative point of view, the first pluviation test in the small tank gave a clearly more satisfactory surface condition than the initial large-tank test. The final surface appeared globally plane, with no evident segregation zone or marked local defect.



(a) Side/general view of the final fill.



(b) Top view of the final surface.

Figure 26: Surface condition after the first pluviation test in the small tank.

Height measurements and filling level. This first visual impression was supported by the height measurements carried out at four locations. The measured distances between the upper rim of the barrel and the sand surface were approximately:

- $d_{\min} \approx 10.6$ cm,
- $d_{\max} \approx 13.5$ cm.

CPT results. The CPT results obtained at the nine selected locations also suggest a more coherent response than that observed in the large tank. Some scatter remains visible, especially at intermediate and larger depths, but the profiles follow a more consistent global trend.

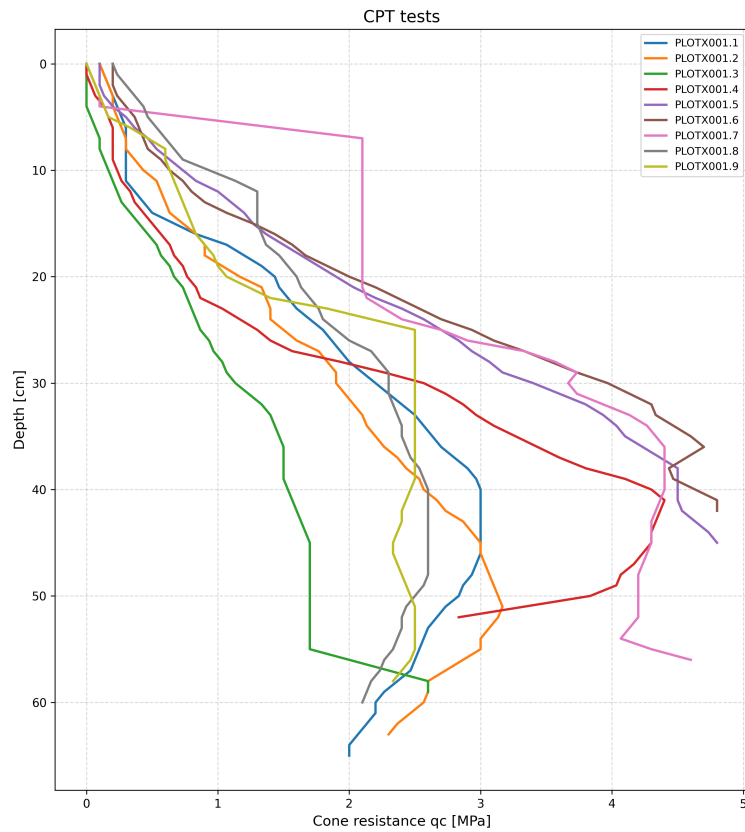


Figure 27: Superposition of the CPT profiles for the nine test locations after the first pluviation test in the small tank.

Second pluviation test in the small tank

Surface condition after pluviation. From a qualitative point of view, the second pluviation test in the small tank again led to a satisfactory final surface condition. As shown in Figure 28, the surface appeared globally plane, with no evident segregation zone or marked local defect. This second test also suggests good repeatability of the procedure in the smaller container.



Figure 28: Top view of the final sand surface after the second pluviation test in the small tank.

Height measurements and filling level. This visual impression was supported by the height measurements carried out at four locations. The measured distances between the upper rim of the barrel and the sand surface were approximately:

- $d_{\min} \approx 1.5$ cm,
- $d_{\max} \approx 3.8$ cm,

CPT results. The CPT results obtained at the nine selected locations are presented in Figure 29. The superposed $q_c(z)$ profiles show a globally coherent trend over a large part of the penetrated depth. A certain scatter remains visible, especially at intermediate and larger depths, but the general shape of the profiles is comparable from one CPT location to another.

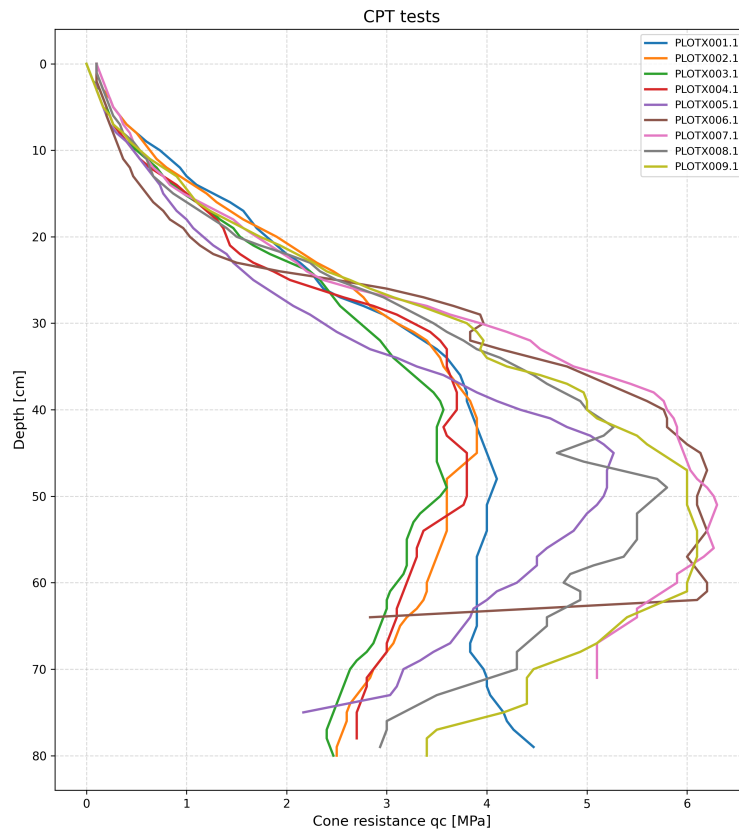


Figure 29: Superposition of the CPT profiles for the nine test locations after the second pluviation test in the small tank.

5.7 Optimisation of the large-tank pluviation procedure

After the encouraging results obtained in the small tank, additional investigations were carried out in the large tank in order to better understand the origin of the poorer behaviour previously observed in this configuration. Since the large tank remains the preferred container for the subsequent pile installation tests because of its dimensions, the objective was to assess whether simple geometric modifications of the setup could improve the quality and apparent homogeneity of the prepared sand mass.

Three complementary tests were therefore performed in the large tank. The first one consisted in lowering the diffuser so that it rested directly on the tank rim, in order to reproduce more closely the configuration used in the small tank. The second one focused on the geometry of the tank opening itself, by trimming the inner steel fins protruding above the upper edge of the container. A third test was then carried out in this modified configuration by interrupting the pluviation at regular time intervals in order to manually flatten the sand surface during filling.

5.7.1 First modified test in the large tank: diffuser placed directly on the tank rim

Test setup.

A first difference identified between the large-tank and small-tank configurations concerned the position of the diffuser relative to the container opening. In the small tank, the diffuser was placed directly on the barrel rim, whereas in the initial large-tank configuration the diffuser remained above the tank opening. A first working hypothesis was therefore that this geometric difference could influence the way the sand entered the container and, consequently, the quality of the final deposit.

To test this hypothesis, the diffuser plate was lowered until it came directly into contact with the upper rim of the large tank, so as to reproduce more closely the configuration previously used in the small tank. A general view of this modified setup is shown in Figure 30. The same general CPT layout as in the previous large-tank test was then used in order to evaluate the spatial variability of the prepared sand mass.



Figure 30: First modified configuration in the large tank.

Surface condition after pluviation.

From a qualitative point of view, the final sand surface remained clearly uneven. As shown in Figure 31, the deposited surface still presented a marked relief, with important height variations across the tank.



Figure 31: Surface of the sand mass in the large tank immediately after pluviation.

Height measurements and filling level.

These visual observations were confirmed by the first height measurements, which showed significant differences depending on the measurement location. This confirmed that the deposited surface could not directly provide a flat reference level for the following tests.

- $d_{\max} \approx 57$ cm,
- $d_{\min} \approx 40$ cm,

CPT results.

The CPT results obtained after this first modified large-tank test are presented in Figure 32. Overall, the scatter remains important, with no clear improvement compared with the initial large-tank configuration.

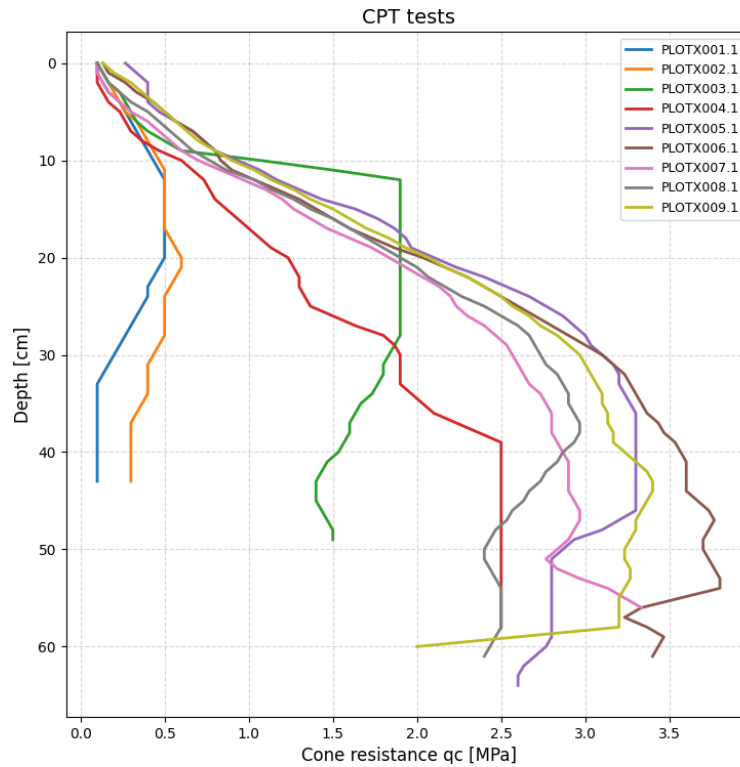


Figure 32: Superposition of the CPT profiles for the nine test locations after the first modified test in the large tank, with the diffuser placed directly on the tank rim.

General assessment.

This first modification isolated the role of the diffuser position, but the CPT results do not show a clear improvement. The prepared sand mass still exhibited marked spatial variability, indicating that the poorer behaviour initially observed in the large tank cannot be explained by the diffuser position alone.

5.7.2 Second modified test in the large tank: trimming the protruding inner fins

Problem identification.

Since the first modified configuration did not lead to a clear improvement, attention was then turned to the geometry of the large tank itself. More specifically, the upper part of the tank included inner steel fins protruding above the cylindrical opening. These elements locally altered the geometry of the opening and were therefore considered as a possible source of disturbance during pluviation.

The geometric origin and the possible consequences of this problem are illustrated in Figure 33. In top view, the protruding steel elements locally reduced the free circular opening of the tank. As a result, the sand curtain could not be distributed over the whole tank section in a fully regular way. Part of the peripheral area near the wall was effectively screened by these elements, so that the effective deposition area was smaller than the full cylindrical section of the tank.

The same geometry may also have affected the air evacuation during filling. As the sand fell into the tank, the air initially present inside the container had to escape upward

through an opening whose geometry was locally disturbed by the protruding fins. This upward air flow may therefore have interacted with the falling sand curtain and further affected the regularity of the pluviation. At this stage, this mechanism should be regarded as a plausible interpretation rather than as a directly measured phenomenon.

Based on these observations, a second modified test was carried out after trimming the inner fins protruding above the upper edge of the tank. The objective of this intervention was to recover a more regular cylindrical opening, so that the sand could be deposited over the whole tank section and the displaced air could escape in a less disturbed manner during filling.

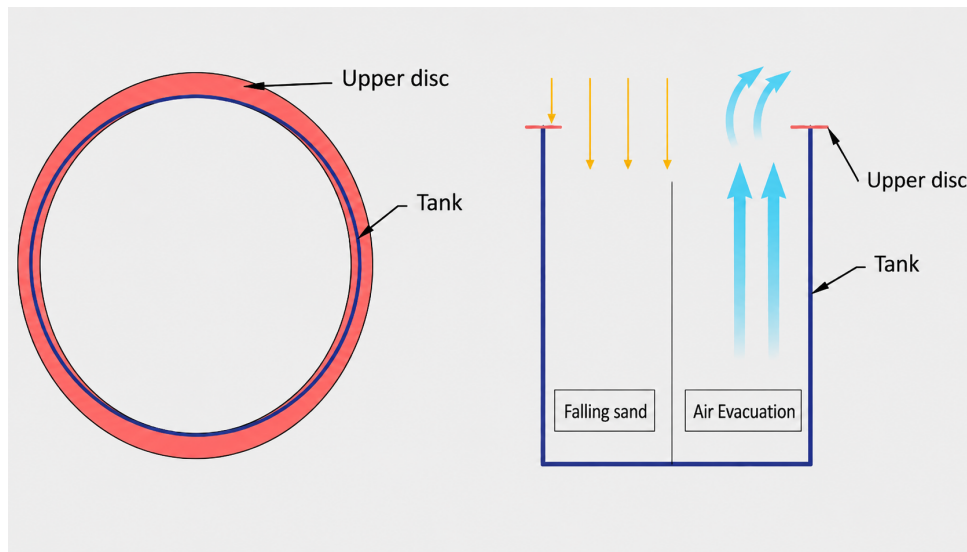


Figure 33: Schematic illustration of the geometric disturbance caused by the protruding inner fins of the large tank.

Surface condition after pluviation.

From a qualitative point of view, this pluviation test in the large tank gave a more satisfactory surface condition than the previous large-tank tests. The final surface appeared globally plane at the scale of the large tank, with no evident segregation zone or marked local defect.



Figure 34: Surface of the sand mass in the large tank immediately after pluviation.

Height measurements and filling level.

These visual observations were confirmed by the first height measurements, which showed no significant differences depending on the measurement location.

- $d_{\max} \approx 54$ cm,
- $d_{\min} \approx 50$ cm,

CPT results.

The CPT results obtained after trimming the protruding inner fins are presented in Figure 35. The profiles show an acceptable grouping in the near-surface zone and several curves follow a broadly comparable increasing trend at larger depths. A significant scatter nevertheless remains, with some low-response profiles still present. Even so, this second modified configuration appears more encouraging than the previous one.

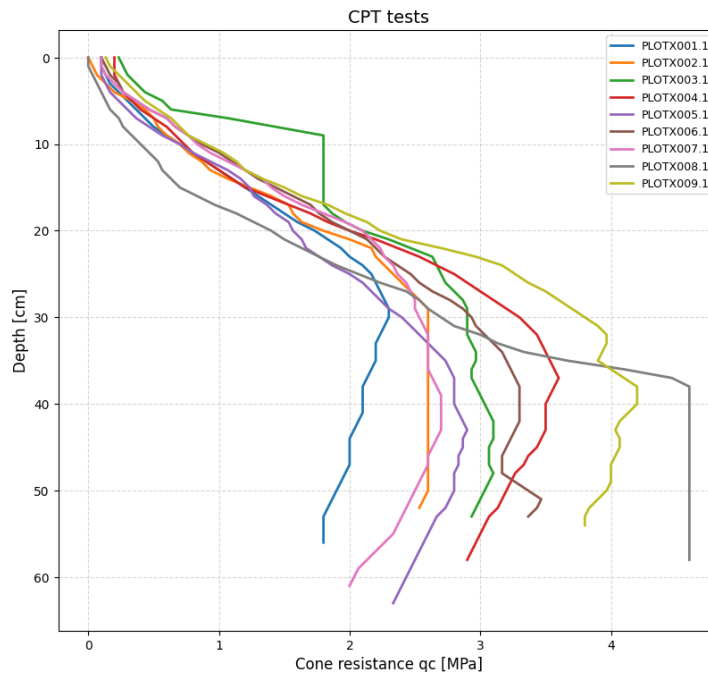


Figure 35: Superposition of the CPT profiles for the nine test locations after the second modified test in the large tank, following the trimming of the protruding inner fins.

General assessment.

This second modification appears more relevant than simply lowering the diffuser onto the tank rim. Trimming the protruding inner fins did not fully eliminate the variability observed in the large tank, but the CPT profiles became more coherent than in the initial configuration. This supports the idea that the geometry of the tank opening played an important role in the poorer behaviour initially observed in the large tank.

5.7.3 Third modified test in the large tank: interrupted pluviation with intermediate surface levelling

Test procedure.

After the test carried out in the large tank with the protruding inner fins trimmed, an additional pluviation test was performed in the same modified geometry. In this third test, the filling was not carried out continuously. Instead, the sand fall was interrupted approximately every 1 min 30 s, and the surface of the sand mass was manually flattened before resuming the pluviation.

The objective of this procedure was to assess whether repeated intermediate surface corrections during filling could improve the final surface condition and, possibly, lead to a more homogeneous deposited sand mass. The underlying idea was that, if the local relief created during pluviation was corrected progressively rather than only at the end of the filling process, the final state of the sample might become more regular.

However, this modified procedure also differs significantly from the logic of a continuous pluviation process. Interrupting the sand fall and manually reworking the surface during filling may create additional disturbances in the deposited material and modify the natural evolution of the sample as it is being formed.

Surface condition after pluviation.

From a qualitative point of view, the final sand surface was again clearly uneven. As shown in Figure 36, the deposited surface presented a marked relief, with important height variations across the tank.



Figure 36: Surface of the sand mass in the large tank immediately after pluviation.

Height measurements and filling level.

These visual observations were confirmed by the first height measurements, which showed significant differences depending on the measurement location. This confirmed that the deposited surface could not directly provide a flat reference level for the following tests.

- $d_{\max} \approx 56$ cm,
- $d_{\min} \approx 41$ cm,

CPT results.

The CPT results obtained after this interrupted-pluviation test are presented in Figure 37. From a qualitative point of view, the profiles appear less coherent than those obtained during the continuous-pluviation test performed in the same modified large tank geometry. A marked scatter remains visible, with some curves staying at relatively low cone resistance levels while others increase more significantly with depth. Several local drops in cone resistance can also be observed during penetration; these losses of resistance are likely related to disturbances introduced during the interruptions of the pluviation process, when the sand surface was manually levelled.

As in the previous tests, the maximum penetration depth was not identical for all CPT soundings, which should be kept in mind when comparing the deepest parts of the curves. Nevertheless, the overall dispersion of the profiles suggests that the interrupted pluviation procedure did not improve the homogeneity of the prepared sand mass.

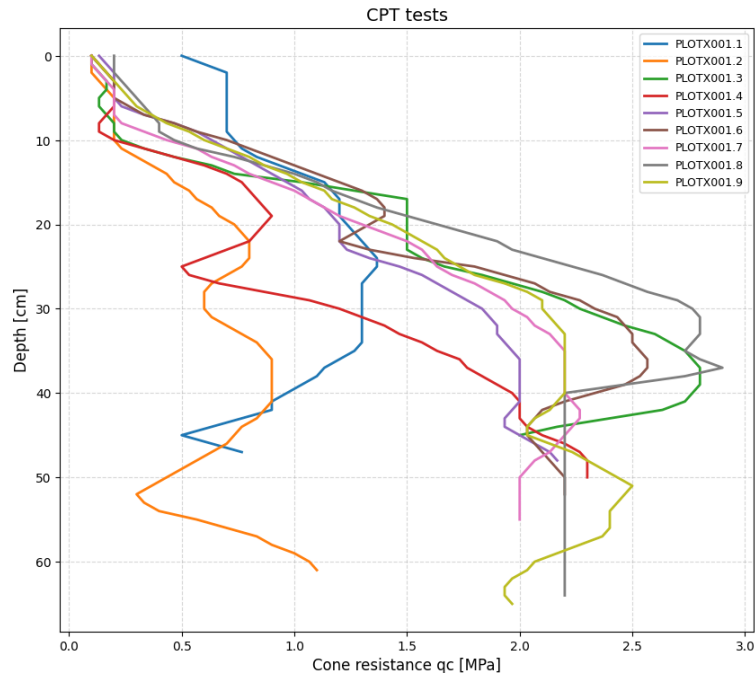


Figure 37: Superposition of the CPT profiles for the nine test locations after the interrupted-pluviation test.

General assessment.

This third test does not suggest any improvement associated with repeated manual flattening during filling. On the contrary, the CPT results appear less satisfactory than those obtained in the continuous-pluviation test carried out in the same modified geometry. These observations indicate that, once the tank geometry has been improved, the pluviation process should remain as continuous as possible.

5.8 Validation of the final pluviation procedure

The main observations from the preliminary and optimisation tests are summarised in Table 1, in order to highlight the role of each configuration in the selection of the final pluviation procedure.

Table 1: Summary of the preliminary pluviation tests.

Test configuration	Surface condition	CPT results
Initial large-tank setup	Irregular surface	Strong scatter; poor profile consistency
Small-tank tests	More regular	More coherent profiles
Diffuser placed on the large-tank rim	Irregular surface	No clear improvement in homogeneity
Large tank with trimmed inner fins	More regular surface	Better grouping of the CPT profiles
Interrupted pluviation with manual levelling every 1 min 30 s	Irregular surface	Increased scatter; less coherent profiles

5.8.1 Retained pluviation procedure

Based on the previous observations, a final pluviation procedure was retained for the preparation of the sand mass used in the pile installation and lateral loading tests. This procedure was selected in order to improve the repeatability of the initial sand state while limiting unnecessary disturbances during filling.

The retained configuration uses the modified large tank, after trimming the protruding inner fins at the top of the tank. The sand raining system is positioned 37 cm above the upper rim of the tank, while the diffuser mesh is placed 16 cm above the tank opening. This configuration is intended to promote a more regular distribution of the sand flow over the tank section.

The pluviation is then carried out continuously, without intermediate interruption or manual levelling during filling. This choice follows from the previous optimization tests, which showed that interrupting the deposition process did not improve the homogeneity of the prepared sand mass. After pluviation, if necessary, a careful final levelling is performed before the CPT measurements and pile installation tests.

The same procedure is repeated for each sand preparation in order to ensure comparable initial conditions between the different tests.

5.8.2 Statistical comparison between the initial and final pluviation procedures

After defining the retained pluviation procedure, its effectiveness must be verified quantitatively. The objective of this subsection is therefore to assess whether the final configuration actually improves the homogeneity and repeatability of the prepared sand mass compared with the initial large-tank configuration.

For this purpose, the CPT results obtained after the initial pluviation test are compared with those obtained after two independent pluviation tests performed with the final procedure. These two final tests were carried out using the modified large tank, after trimming the protruding inner fins, and with a continuous pluviation procedure. The comparison is based first on the visual grouping of the CPT profiles and then on a statistical analysis of their dispersion.

Figures 38, 39 and 40 show the superposed CPT profiles obtained for the three pluviation tests. A first visual comparison already indicates that the initial pluviation led to a much more scattered response between the nine measurement locations. In the initial configuration, some CPT curves remain at very low resistance over a large part of the explored depth range, whereas others increase much more rapidly, which reflects an important spatial variability in the prepared sample.

By contrast, the two pluviation tests performed with the final configuration show a noticeably more consistent response. Although some dispersion remains, especially near the surface and at a few isolated depths, the overall grouping of the CPT curves is clearly improved. The curves obtained after the first final pluviation are more coherent than those of the initial test, while the second final pluviation shows the most regular and best-grouped set of CPT profiles among the three tests.

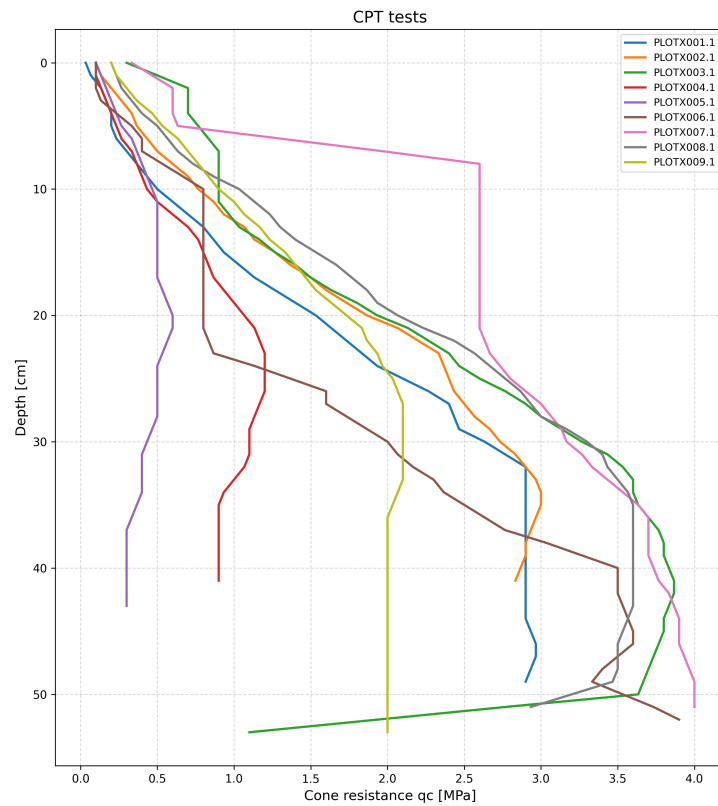


Figure 38: Superposition of the CPT profiles obtained after the initial pluviation test in the large tank.

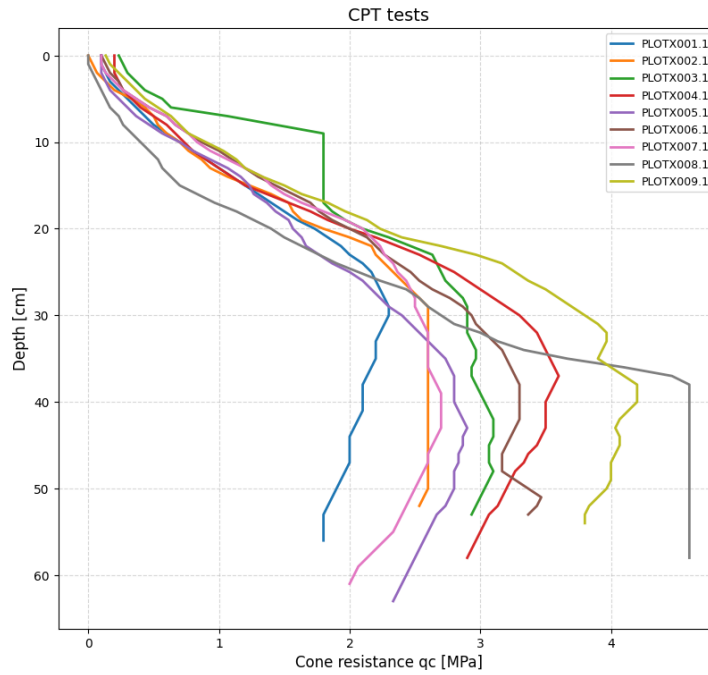


Figure 39: Superposition of the CPT profiles obtained after the first pluviation test performed with the final large-tank configuration.

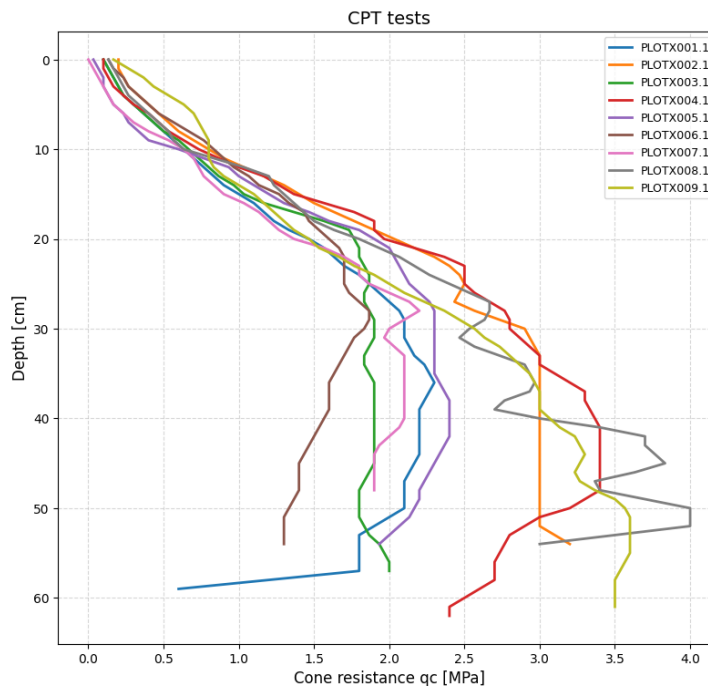


Figure 40: Superposition of the CPT profiles obtained after the second pluviation test performed with the final large-tank configuration.

To quantify these observations, the coefficient of variation of the CPT resistance was computed as a function of depth for the three pluviation tests over the common depth range from 0 to 42 cm. The corresponding results are shown in Figure 41. Since the

coefficient of variation expresses the dispersion of the measurements relative to their mean value, a lower value indicates a more homogeneous response between the different CPT locations.

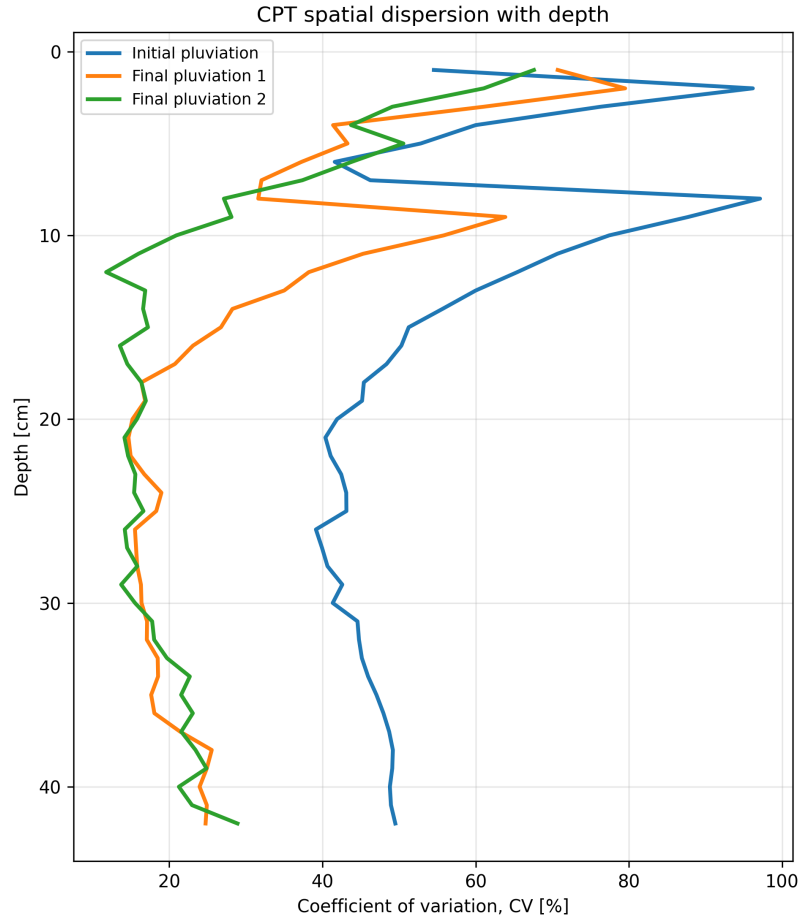


Figure 41: Coefficient of variation of the CPT results as a function of depth for the initial pluviation test and the two pluviation tests performed with the final configuration.

Table 2: Mean coefficient of variation of the CPT results for the three pluviation tests over the first 42 cm of penetration.

Indicator	Initial pluviation	Final pluviation 1	Final pluviation 2
Mean CV over 0–42 cm [%]	52.6	28.5	23.8
CV reduction relative to initial [%]	–	45.7	54.7

The coefficient-of-variation profiles confirm the qualitative interpretation drawn from the raw CPT curves. The initial pluviation test systematically exhibits the largest dispersion. Over most of the investigated depth range, its coefficient of variation remains clearly higher than that of both final pluviation tests, and it reaches very high values

at shallow depth. This behaviour reflects the strong heterogeneity already visible in the initial CPT profiles.

The two final pluviation tests both show a marked reduction in spatial dispersion compared with the initial test. After approximately the first 10 to 12 cm, their coefficients of variation are generally much lower than those of the initial pluviation and remain mostly in a significantly narrower range. This indicates that the modified large-tank configuration produces a sand mass that is more uniform across the different measurement locations.

Overall, both the qualitative comparison of the CPT profiles and the statistical analysis based on the coefficient of variation confirm the improvement brought by the final pluviation configuration. The reduction of the mean CV from 52.6% in the initial configuration to 28.5% and 23.8% in the two final tests indicates a substantially lower spatial variability of the prepared sand mass. The final procedure can therefore be considered sufficiently homogeneous and repeatable for the comparative purpose of the subsequent pile installation and lateral loading tests.

5.9 Experimental setup for pile installation and lateral loading

5.9.1 General presentation of the setup

A simplified schematic representation of the lateral loading configuration is shown in Figure 42. The horizontal load H is applied at a height h above the mudline, corresponding to the level of the actuator and the force sensor. The lateral displacement of the pile is monitored by two laser sensors located at heights h_1 and h_2 . The embedded pile length is denoted by L , while d represents the depth of the rotation point below the mudline. This idealised representation is used to interpret the pile kinematics during loading and to estimate the displacement at the mudline from the measured quantities.

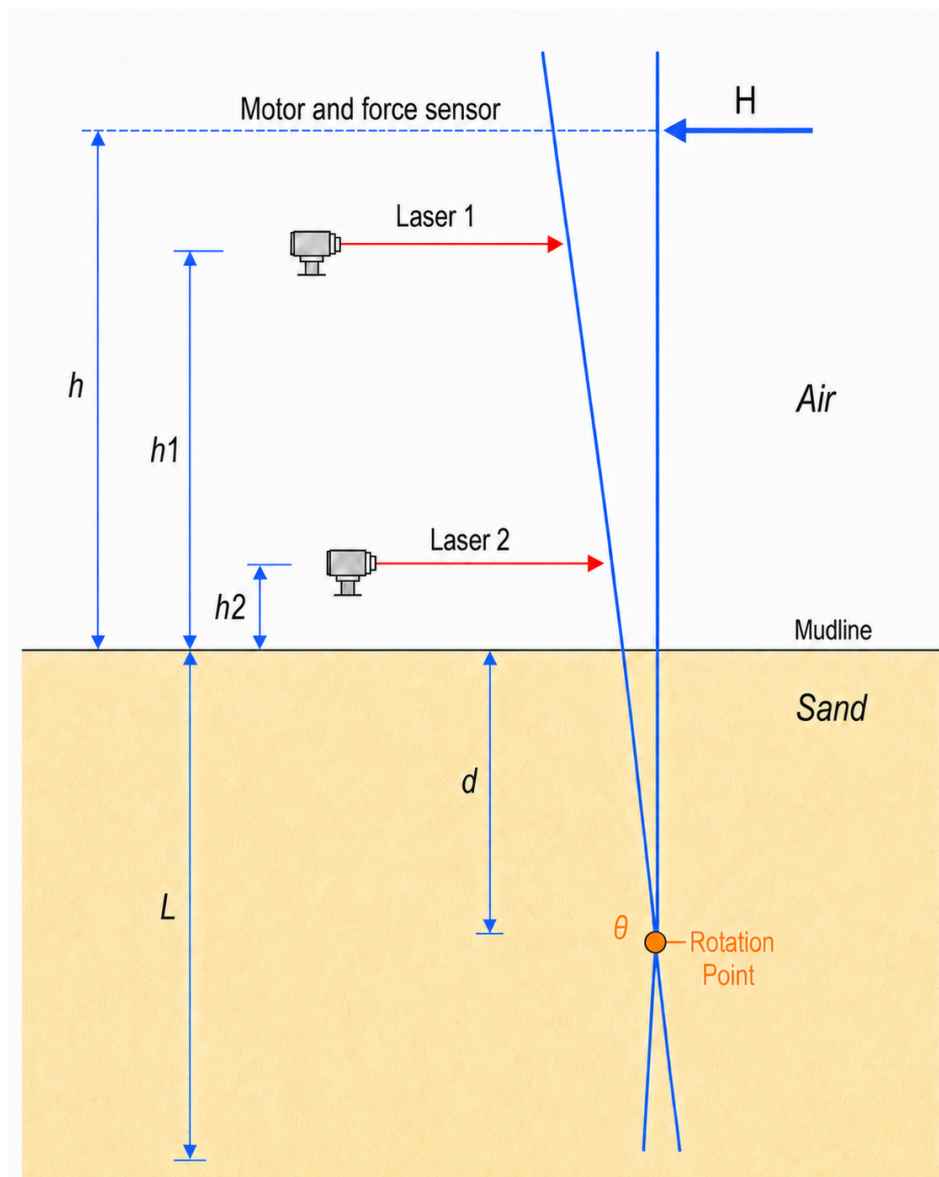


Figure 42: Simplified schematic representation of the lateral loading setup and main geometrical parameters.

Photographs of the experimental setup are presented in Figure 43. The model pile is installed in the sand deposit contained in the tank. The lateral loading system is connected to the pile at the actuator level, while the displacement sensors are positioned at different elevations in order to monitor the pile movement during loading. The force sensor is placed in line with the loading cable, allowing the applied horizontal force to be measured during the test.

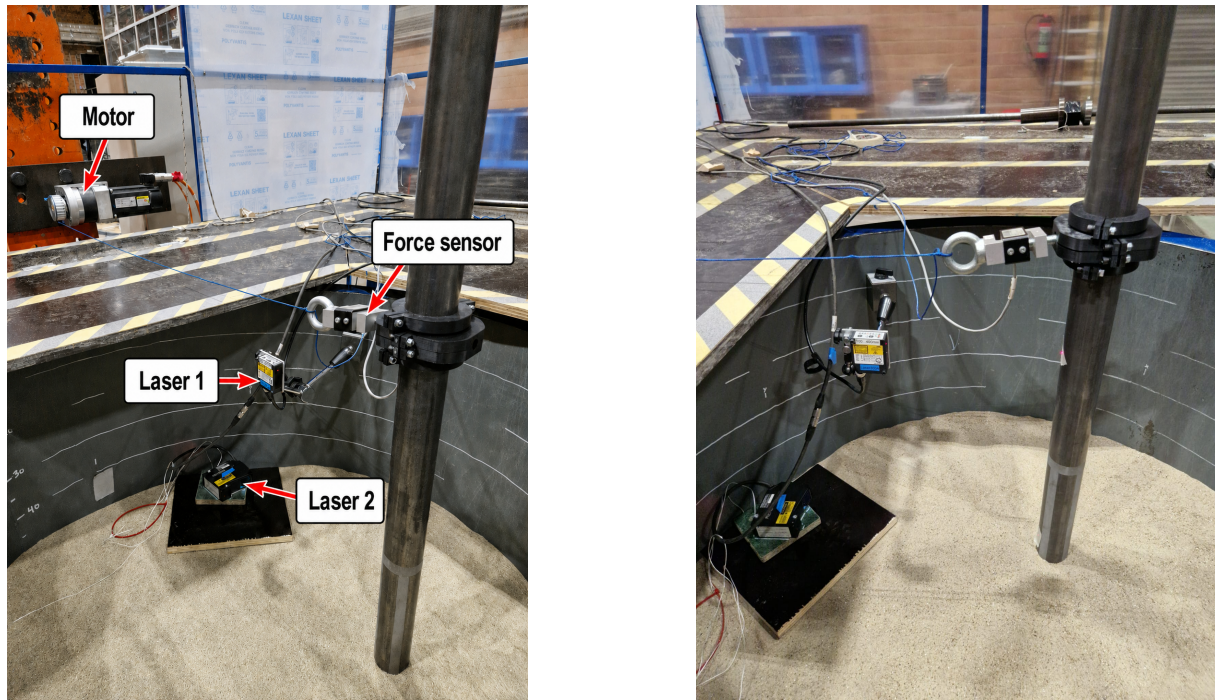


Figure 43: Photographs of the experimental setup used for lateral loading tests.

The following subsections describe in more detail the different components of the setup, including the impact driving system, the vibratory driving system and the lateral loading system. The force sensor and the laser displacement sensors are also presented separately in dedicated subsections.

5.9.2 Model pile

The model pile used in the present experimental campaign was a hollow steel tube. Its outer diameter was $D = 60$ mm, its wall thickness was $t = 1$ mm, and its total length was 1.45 m. The corresponding inner diameter was therefore approximately 58 mm. This geometry was kept constant for all the tests.

Table 3: Main characteristics of the model pile used in the experimental campaign.

Parameter	Value
Material	Steel
Outer diameter, D	60 mm
Wall thickness, t	1 mm
Inner diameter	58 mm
Total length	1.45 m

5.9.3 Impact driving system

The impact driving system was developed to install the model pile by applying repeated vertical impacts at the pile head. This installation method serves as a reference case for comparison with vibratory driving. The principle is based on a falling mass that is lifted manually and then released, so that its impact drives the pile progressively into the sand.

The system consists of a cylindrical steel mass sliding along a vertical guiding rod aligned with the pile axis. During installation, the mass is first raised to a prescribed height and then released, allowing it to fall freely onto the impact head connected to the pile. By repeating this operation, the pile is progressively driven into the sand until the desired embedment depth is reached.

A string fitted with a metallic washer is attached to the moving mass and serves as a visual marker for the drop height. This simple device makes it possible to lift the mass to the same elevation before each blow, thereby improving the repeatability of the installation procedure. As a result, the impact energy remains approximately constant from one blow to another.

Figure 44 illustrates the impact driving system in its two main configurations. In Figure 44a, the falling mass is shown in its raised position before release. In Figure 44b, the mass is shown in its lower position after the impact.

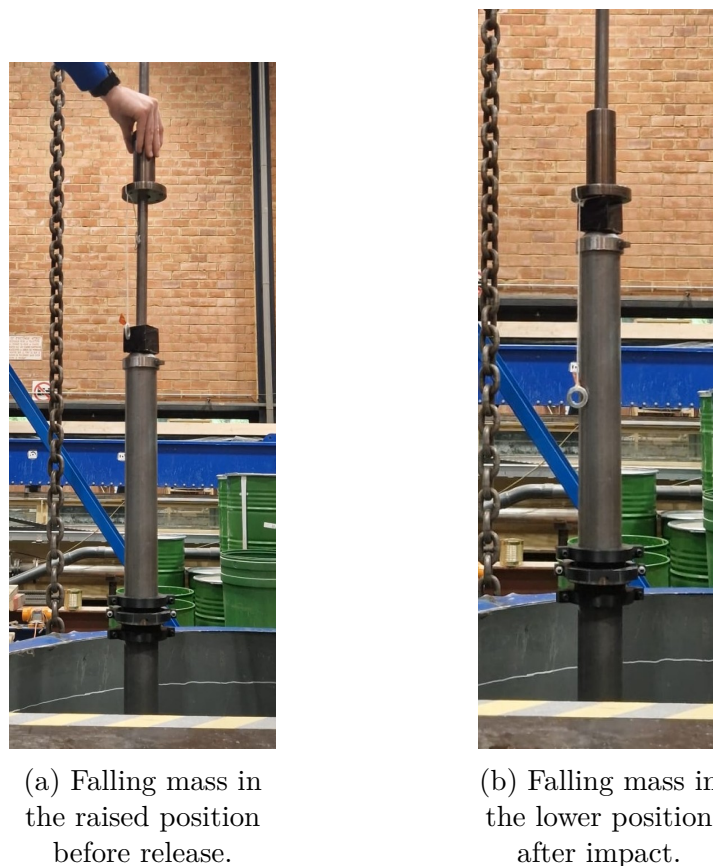


Figure 44: Impact driving system used for the installation of the model pile.

5.9.4 Vibratory driving system

The vibratory driving system was used to install the model pile by applying a vertical harmonic excitation at the pile head. In the present setup, the excitation was generated by a small brushless DC motor driving a pair of eccentric masses mounted on the vibratory head. The motor used for this purpose was a maxon *ECX SPEED 13 L*, with a diameter of 13 mm. This motor belongs to the ECX SPEED family of brushless DC motors, which are designed for high-speed applications. According to the manufacturer's documentation, the ECX SPEED 13 L is available in several configurations, including sensorless versions, and is suitable for operation at very high rotational speeds [41].

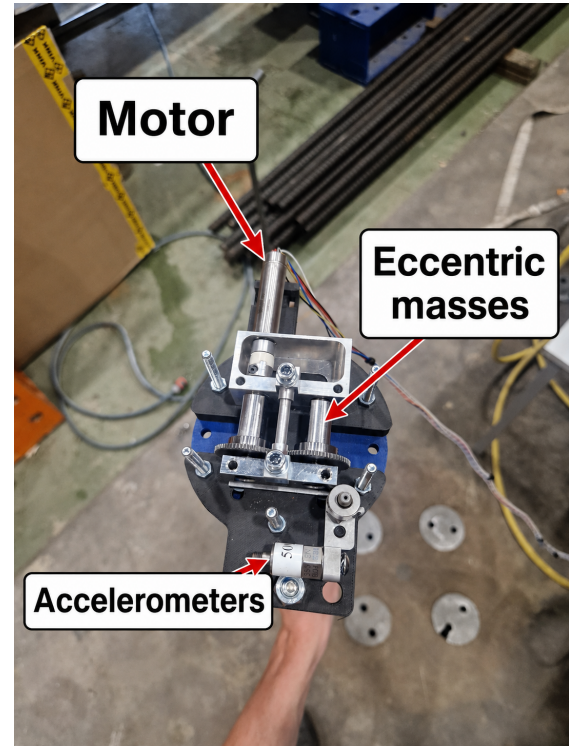
The motor was controlled using a maxon *ESCON 50/5* servo controller. This controller is intended for the control of permanent-magnet brushed DC motors and brushless EC motors up to approximately 250 W. Its nominal operating voltage range is 10–50 VDC, and its output current capability is 5 A in continuous operation and 15 A for short-duration operation [42]. In the present work, the controller was configured and operated using the *ESCON Studio* software installed on a separate computer.

Unlike the motor used for lateral loading, the vibratory driving motor was not commanded from the main data acquisition system. The vibration frequency was imposed through the ESCON controller using another computer, while the other measurements were recorded separately during the experimental campaign. The vibratory driving system therefore acted as an independent actuator, whose main controlled parameter was the motor rotational speed.

The complete vibratory driving arrangement and a detailed view of the vibratory head are shown in Figure 45. During installation, the vibratory head was fixed at the top of the pile. In order to keep the pile vertical during vibro-driving, a simple perforated cardboard guide was placed around the pile at the tank opening. This guide provided a practical alignment reference during installation, while allowing the pile to penetrate vertically through the central hole.



(a) General view of the vibratory driving arrangement.



(b) Detailed view of the vibratory head.

Figure 45: Vibratory driving system used during pile installation: global arrangement with the cardboard guide and detailed view of the motor, eccentric masses and accelerometers.

The eccentric mass configuration was kept unchanged throughout the vibratory test series. The corresponding eccentric moment was equal to 28 gmm. As a result, the eccentric moment was constant for all vibratory driving tests, and the motor speed, or equivalently the excitation frequency, was the main parameter varied in the present experimental programme.

The vibration frequency imposed to the pile is directly related to the motor rotational speed. If the motor speed is denoted by n , expressed in revolutions per minute, the corresponding excitation frequency is

$$f = \frac{n}{60},$$

where f is expressed in hertz.

This configuration made it possible to compare the influence of vibration frequency on pile installation and on the subsequent lateral response, while keeping the pile geometry and eccentric mass configuration unchanged.

5.9.5 Motor-driven lateral loading system

The lateral displacement during the monotonic loading tests was imposed by a motor-driven system. The motor used in the setup was a Bosch Rexroth IndraDyn S synchronous servo motor, type *MSK050C-0600-NN-S1-UG0-NNNN*. This motor is a three-

phase permanent-magnet servo motor designed for controlled motion applications. In the present setup, it was mounted horizontally on the rigid loading frame and coupled to a planetary gearbox driving the winding drum connected to the Dyneema cable.



Figure 46: Motor-driven system used to impose the lateral displacement during the loading tests. The Bosch Rexroth servo motor is coupled to a planetary gearbox and drives the winding drum connected to the Dyneema cable.

The main characteristics of the motor are summarised in Table 4. The motor has a maximum speed of 6000 min^{-1} , a standstill torque of 5.0 N m , a standstill current of 6.2 A , and a mass of approximately 5.4 kg . It is naturally cooled and has an IP65 protection class, which is suitable for laboratory use in the present experimental setup [43].

Table 4: Main characteristics of the motor used for the lateral loading system.

Parameter	Value
Manufacturer	Bosch Rexroth
Motor series	IndraDyn S MSK
Motor type	MSK050C-0600-NN-S1-UG0-NNNN
Motor technology	Three-phase permanent-magnet servo motor
Cooling mode	Natural convection
Maximum speed, $n_{\max}$	6000 min^{-1}
Standstill torque, M_0	5.0 N m
Standstill current, I_0	6.2 A
Torque constant, K_m	0.89 N m/A
Maximum voltage, $U_{\max}$	600 V AC
Protection class	IP65
Motor mass	5.4 kg

The motor was connected to the control and acquisition system used during the lateral loading tests. The imposed motion was therefore commanded from the same control interface used during the tests, which made it possible to impose a controlled and repeatable displacement rate. This arrangement ensured that the lateral loading procedure was displacement-controlled, while the force and displacement measurements were recorded during the same test sequence.

Choice of the imposed lateral loading rate

The lateral loading stage is intended to provide a monotonic reference response of the installed pile. In the present experimental programme, this response is mainly interpreted through the relationship between the horizontal force H and the lateral displacement at mudline level, denoted here as u_0 . The imposed displacement rate during this stage must therefore be defined carefully, because a change in loading rate may modify the apparent stiffness and resistance of the soil–pile system.

For the purpose of the present work, the most important requirement is to impose a controlled and repeatable displacement-controlled loading procedure. The same lateral loading rate must be used for all the reference tests so that the measured force–displacement curves can be compared consistently. This is particularly important because the objective of the experimental programme is not to study the lateral loading process itself, but to compare the post-installation response of piles installed under different conditions.

Previous experimental work at UCLouvain by Deckers and Defer [36] also adopted a displacement-controlled lateral loading procedure. In their study, the actuator displacement was imposed at a constant velocity of 0.4 mm/s, which allowed the results to be analysed through force–displacement curves. In the present work, a lower velocity is preferred in order to reduce possible rate effects while remaining compatible with the available motor and with the practical duration of the tests.

The selected value is also consistent with displacement rates used in several reduced-scale experimental studies on laterally loaded monopiles in sand. For example, Lee et al. [44] performed static lateral loading tests using a constant displacement rate of 0.1 mm/s. The MIDAS centrifuge testing programme on monopiles in sand also used a loading rate of 0.1 mm/s for the monotonic tests reported by Wang et al. [45]. Similarly, Blanc et al. [46] used a displacement-controlled actuator velocity of 0.1 mm/s in centrifuge tests on an instrumented monopile foundation in saturated sand.

Based on these considerations and on the limitations of the motor, the retained actuator velocity for the main lateral loading tests is

$$\dot{u}_a = 0.1 \text{ mm/s} = 6 \text{ mm/min.}$$

This velocity is imposed at the load application point, located at a height h above the mudline. Since the displacement of interest is the mudline displacement u_0 , the corresponding mudline velocity can be estimated by assuming that the pile behaves approximately as a rigid body rotating around a point located at a depth d below the mudline. Under this assumption, the displacement rates are related by

$$\dot{u}_0 \simeq \dot{u}_a \frac{d}{h + d},$$

where $\dot{u}_a$ is the actuator velocity and $\dot{u}_0$ is the corresponding mudline velocity.

In the present experimental programme, the pile diameter is kept equal to $D = 60$ mm and the lateral loading height is kept equal to $h/D = 10$, corresponding to

$$h = 10D = 600 \text{ mm.}$$

For the reference embedment ratio $L/D = 7$, the embedded length is

$$L = 7D = 420 \text{ mm.}$$

The depth of the rotation point is not strictly constant during lateral loading and is determined a posteriori from the displacement measurements. However, for the purpose of estimating the order of magnitude of the mudline velocity, an indicative value $d \simeq 0.7L$ is adopted, giving

$$d \simeq 0.7L = 294 \text{ mm.}$$

The corresponding mudline displacement rate is therefore estimated as

$$\dot{u}_0 \simeq 6 \frac{294}{600 + 294} = 1.97 \text{ mm/min.}$$

The first tests performed during the campaign were carried out with an imposed displacement rate of 2 mm/s at the load application point, corresponding to 120 mm/min. For the same reference geometry, this gives an estimated mudline displacement rate of approximately

$$\dot{u}_{0,\text{high}} \simeq 120 \frac{294}{600 + 294} = 39.5 \text{ mm/min.}$$

The retained velocity of 6 mm/min at the actuator level is therefore about twenty times lower than the initial high-rate tests. It is consequently adopted as the reference lateral loading rate for the main experimental programme. The initial tests performed at 2 mm/s are not used directly in the main comparison between installation methods and installation parameters; instead, they are retained as a complementary high-rate dataset to assess the possible influence of the imposed lateral loading rate on the measured force–displacement response.

5.9.6 Load transfer device between the pile and the force sensor

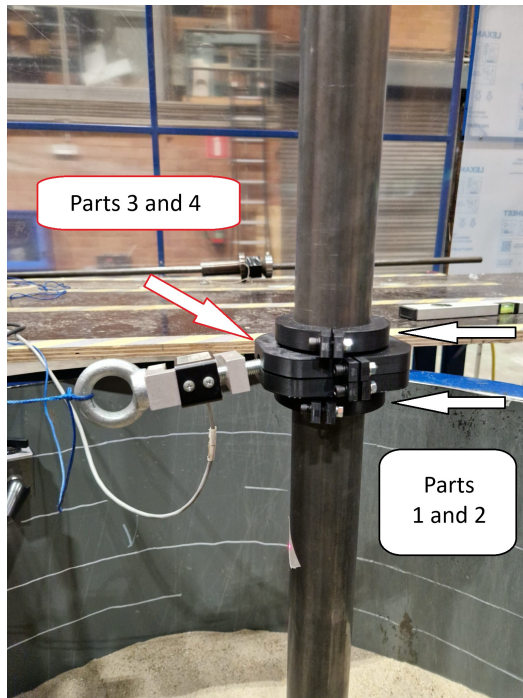
In order to apply the lateral load to the pile and to measure it correctly, a specific connection device was designed and mounted around the pile at the loading height. This device ensured the mechanical connection between the pile, the tensile force sensor and the Dyneema cable used to apply the lateral load.

The device is composed of two main assemblies. The first assembly, corresponding to parts 1 and 2, is clamped directly onto the pile. Its role is to prevent any vertical movement of the connection device during the lateral loading test. This clamping system therefore fixes the loading point at the prescribed height h above the mudline and ensures that the applied load is transmitted to the pile at a well-defined elevation.

The second assembly, corresponding to parts 3 and 4, is clamped around a ball bearing. This bearing allows the force sensor and the Dyneema cable to rotate freely around the

pile. As a result, the force sensor and the cable can naturally align with the direction of traction during the test. The use of the ball bearing therefore limits parasitic bending or torsional effects in the connection and helps ensure that the force sensor mainly works in axial tension.

A general view of the connection device is shown in Figures 47a and 47b. The parts 1 and 2 are fixed to the pile, while the parts 3 and 4 are connected to the rotating bearing. The tensile force sensor is then attached to the rotating part of the assembly and connected to the Dyneema cable.



(a) Connection device mounted around the pile at the lateral loading height.



(b) Detailed view of the force sensor connected to the load-transfer device.

Figure 47: Load-transfer device and force sensor connection used during the lateral loading tests.

5.9.7 Force sensor

The lateral force applied to the pile was measured using a tensile load cell mounted in line with the loading cable. The sensor used in the setup has a rated capacity of 3000 N. This capacity is significantly higher than the force levels expected during the lateral loading tests, ensuring that the sensor remains within its safe operating range throughout the experimental campaign.

The load cell was connected to the data acquisition system and recorded the tensile force transmitted through the cable. Since the lateral load is applied by pulling the pile, the sensor mainly operates in tension during the tests. In the subsequent analysis, the sign convention was therefore chosen so that tensile forces applied to the pile are considered positive.

Force sensor: first calibration test

The first calibration test of the force sensor was performed by applying a series of known dead weights in a stepwise manner. The sensor was loaded by suspending masses from it, starting from an initial mass of 445 g. Five additional masses of 4 kg were then added successively, so that the loading history consisted of six distinct force levels.

The theoretical applied loads were thus 0.445 kg, 4.445 kg, 8.445 kg, 12.445 kg, 16.445 kg and 20.445 kg, corresponding to approximately 4.37 N, 43.61 N, 82.85 N, 122.09 N, 161.33 N and 200.57 N. The sensor output was recorded continuously during the test.

The objective of this test was to evaluate the ability of the sensor to reproduce the increase in applied load in a consistent manner and to establish a calibration relationship between the measured signal and the actual force. Since the loading was applied in a series of plateaus, the test also provided suitable time intervals over which the sensor response could be averaged in order to obtain one representative measured value for each loading step. The mean values obtained for the six loading plateaus are reported in Table 5. In order to facilitate the comparison with the theoretical applied force, the sensor output is expressed below using the same sign convention as the applied load. The difference reported in the last column is defined as the theoretical applied force minus the measured sensor value.

Table 5: Comparison between the theoretical applied force and the measured response of the force sensor for the first calibration test.

Loading step	Applied force [N]	Measured force [N]	Difference [N]
0.445 kg	4.37	-13.01	17.38
4.445 kg	43.61	26.00	17.61
8.445 kg	82.85	64.92	17.93
12.445 kg	122.09	104.00	18.09
16.445 kg	161.33	143.00	18.33
20.445 kg	200.57	182.22	18.35

The results show that the variations in the applied load are very well captured by the sensor. Indeed, the increase in the measured response from one loading step to the next is highly consistent with the increase in the theoretical applied force. This indicates that the sensor reproduces the load increments in a reliable manner over the considered range.

The remaining discrepancy is mainly associated with an approximately constant vertical offset. The difference between the applied force and the measured force remains close to 18 N for all loading steps, with a mean value of about 17.95 N. This suggests that the calibration curve does not require a change of slope, but rather a vertical translation. In the present representation, the curve should therefore be shifted downward by 17.95 N.

Validation of the calibration

After applying the previously identified offset correction of 17.95 N, a final stepwise loading test was carried out in order to validate the calibration of the force sensor. The corrected signal was again recorded continuously under a series of increasing dead weights, and

the mean value of each retained loading plateau was compared with the corresponding theoretical applied force. In addition to the previous loading steps, a last plateau was obtained by adding an extra mass of 2 kg after the 24.445 kg loading level.

The results are reported in Table 6. For clarity, the sensor response is expressed below using the same sign convention as the applied load. The difference reported in the last column is defined as the theoretical applied force minus the measured force.

Table 6: Validation of the force sensor calibration after application of the offset correction.

Loading step	Applied force [N]	Measured force [N]	Difference [N]
0.445 kg	4.37	4.25	0.12
4.445 kg	43.61	43.31	0.30
8.445 kg	82.85	82.43	0.42
12.445 kg	122.09	121.52	0.57
16.445 kg	161.33	160.65	0.68
20.445 kg	200.57	199.66	0.91
24.445 kg	239.81	238.87	0.94
26.445 kg	259.43	258.32	1.11

The corrected results show that the force sensor reproduces the applied load in a highly consistent manner over the considered range. The evolution of the measured force follows the successive load increments very closely, and the difference between the theoretical and measured values remains close to 1 N or less over the whole loading range.

These results validate the offset correction applied after the first calibration test. They show that the force sensor is able to capture the load variations reliably and that the corrected signal can be considered sufficiently accurate for the subsequent lateral loading tests. The agreement remains very good even for the last loading step obtained after adding the extra 2 kg mass, which confirms the validity of the calibration over the full investigated loading range.

5.9.8 Laser 2: KEYENCE laser displacement sensor

One of the laser displacement sensors used during the experimental campaign was a KEYENCE LK-G502 sensor, belonging to the LK-G series. This sensor was used to measure the lateral displacement of the pile during the loading tests. The sensor emits a semiconductor laser beam with a wavelength of 650 nm and a maximum output power of 0.95 mW, as indicated on the sensor casing.

The KEYENCE laser was connected to the main data acquisition system and provided a continuous displacement-related signal during the tests. Since the measured output signal had to be converted into a physical displacement, a specific calibration procedure was carried out before the lateral loading tests. This calibration made it possible to establish the relationship between the laser output signal and the actual distance between the sensor and the target surface.

The calibration test was carried out in order to establish the relationship between the voltage signal delivered by the laser sensor to the acquisition system and the actual distance to the target. During this test, the target was initially positioned at a distance of 56.2 cm from the sensor. The distance was then progressively reduced by successive steps of 2 cm, leading to a total of 16 imposed positions. At each step, the voltage transmitted by the laser sensor to the acquisition system was recorded continuously. In parallel, the distance indicated directly by the laser sensor was also noted as an additional reference during the test.

The raw voltage signal obtained during the calibration test is shown in Figure 48. A clear stepwise evolution can already be identified, which reflects the successive imposed positions of the target. However, the signal also contains a significant amount of high-frequency noise, which makes the plateaus less easy to interpret directly. In addition, sharp peaks appear at the moments where the target position was changed. These peaks were intentionally created by briefly passing a hand in front of the laser in order to generate visible markers on the recorded signal and to facilitate the identification of the successive plateaus.

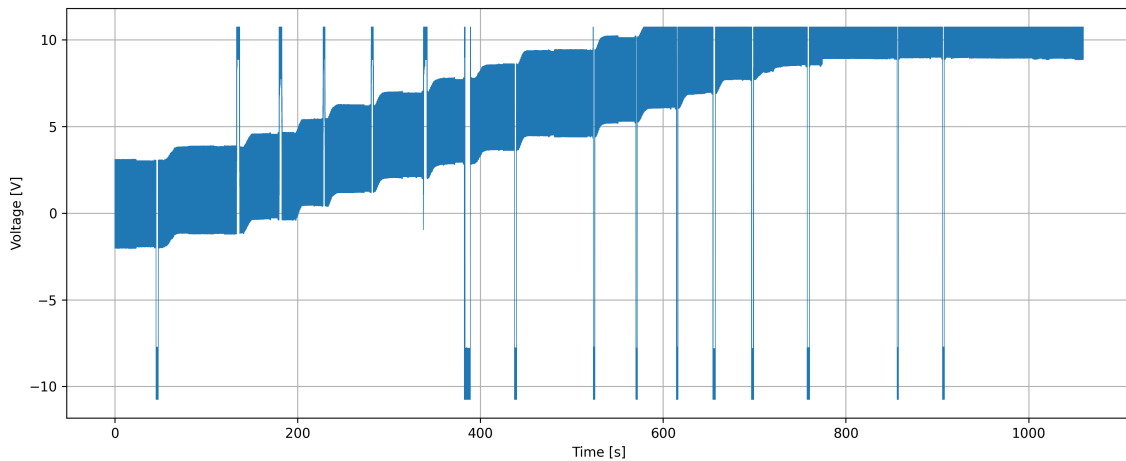


Figure 48: Raw voltage signal delivered by the laser sensor during the calibration test.

In order to better characterise the noisy content of the signal, a frequency-domain analysis was performed by computing its Fast Fourier Transform (FFT). The corresponding amplitude spectrum is presented in Figure 49. The spectrum shows that the useful information associated with the successive quasi-static plateaus is mainly concentrated at very low frequencies. By contrast, the higher-frequency content is predominantly associated with measurement noise. A distinct component can also be observed close to 50 Hz, which is consistent with electrical interference from the surrounding environment.

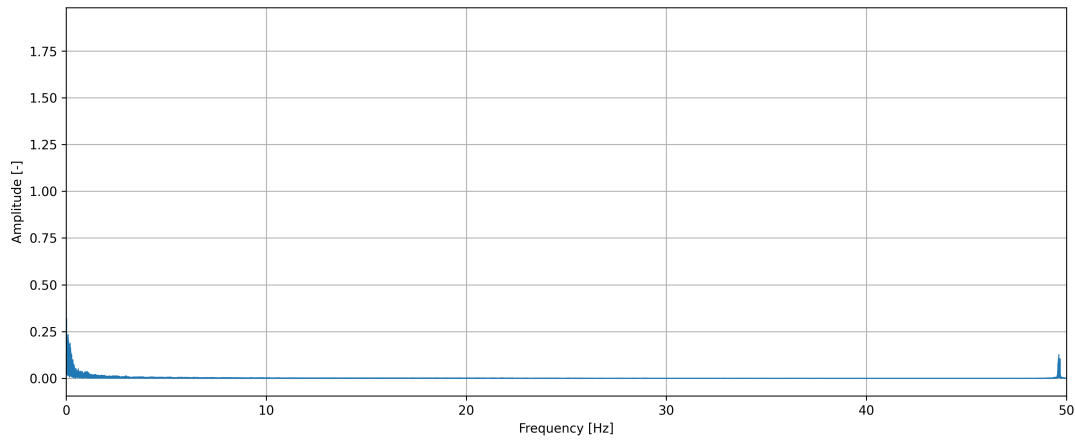


Figure 49: Amplitude spectrum of the raw laser voltage signal obtained by FFT.

On the basis of this frequency-domain analysis, a low-pass filter with a cut-off frequency of 5 Hz was applied to the voltage signal. The objective of this operation was to preserve the low-frequency evolution of the signal associated with the successive plateaus while attenuating the high-frequency fluctuations. The filtered signal is shown in Figure 50. Compared with the raw signal, the plateau levels become much more stable and easier to identify. The overall stepwise evolution of the calibration test is preserved, while the noise affecting the direct interpretation of the signal is significantly reduced. This filtered signal was therefore used in the next stage of the analysis to extract representative plateau values and to establish the calibration relationship between voltage and distance.

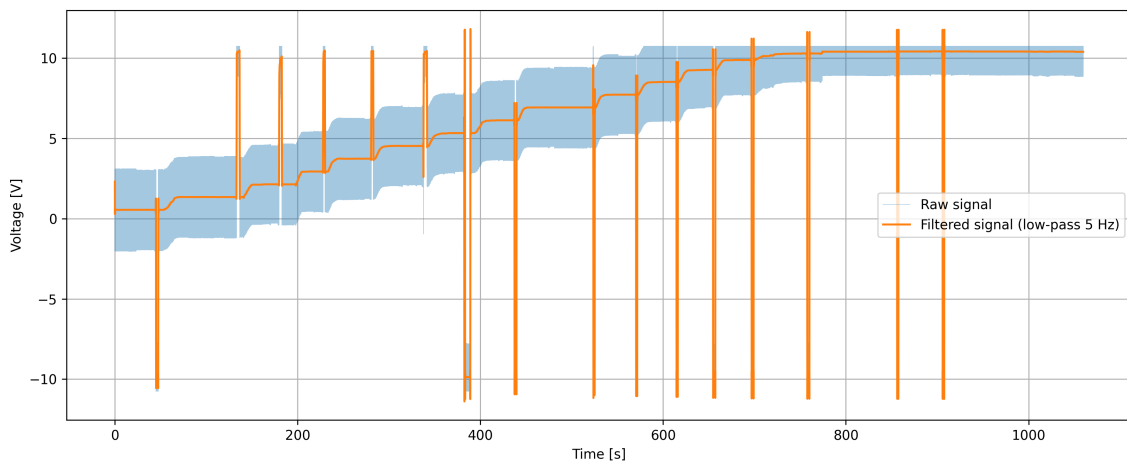


Figure 50: Comparison between the raw voltage signal and the signal after application of a 5 Hz low-pass filter

After filtering, the signal was processed in order to extract one representative voltage value for each imposed distance level. To make the plateau detection more robust, the filtered signal was first reduced by grouping the data into time blocks of 0.5 s and replacing the signal within each block by its median value. This reduced signal made the stepwise evolution easier to identify and limited the influence of the residual local fluctuations that remained after low-pass filtering.

The successive plateaus were then identified automatically from the positive jumps of the reduced signal. For each remaining stabilised interval, both the mean and the median voltage were computed. Since a few plateaus still exhibited some residual perturbations, the median voltage was retained as the representative value for the calibration, because it is more robust than the mean with respect to local fluctuations.

A total of 14 valid plateaus were retained within the effective measuring range of the sensor. The corresponding theoretical distances and representative voltages are reported in Table 7. The calibration curve obtained from these 14 points is shown in Figure 51. The relation between the imposed distance and the measured voltage is clearly linear over the retained range.

A linear regression performed on these 14 plateaus led to the following calibration law:

$$d = -2.573901 U + 57.797542 \quad (13)$$

where d is the distance from the laser sensor to the target in centimetres and U is the voltage delivered by the acquisition system in volts. The coefficient of determination associated with this regression is $R^2 = 0.998844$, which indicates an excellent agreement between the experimental points and the fitted linear law.

These results confirm that, once the high-frequency noise is attenuated and the plateau values are extracted over stabilised time intervals, the laser sensor provides a highly consistent and nearly linear response over its effective operating range. The calibration law given by Equation 13 was therefore retained for the conversion of the voltage signal into distance in the subsequent lateral loading tests.

Table 7: Representative plateau values retained for the calibration of the laser sensor.

Plateau	Theoretical distance [cm]	Median voltage [V]
1	56.2	0.5515
2	54.2	1.3476
3	52.2	2.1393
4	50.2	2.9385
5	48.2	3.7347
6	46.2	4.5318
7	44.2	5.3110
8	42.2	6.1201
9	40.2	6.9309
10	38.2	7.7278
11	36.2	8.5111
12	34.2	9.2593
13	32.2	9.8846
14	30.2	10.4111

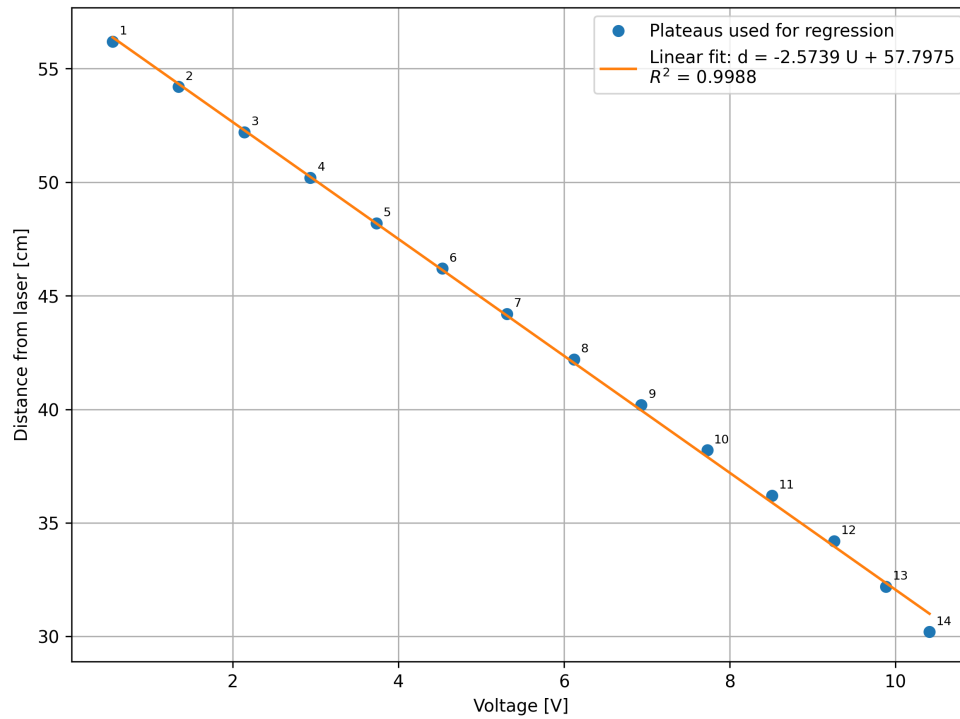


Figure 51: Calibration curve of the laser displacement sensor.

The very high value of R^2 shows that the retained calibration law provides an excellent representation of the experimental data over the considered range.

Validation of the calibration

After establishing the calibration law of the laser displacement sensor, a validation test was carried out in order to assess its accuracy over the working range considered for the subsequent lateral loading tests. For this purpose, the target was initially positioned at a distance of 34 cm from the sensor. The distance was then increased step by step by successive increments of 1 cm, leading to a total of 35 imposed positions, from 34 cm to 68 cm.

The raw signal recorded during this validation test is shown in Figure 52. As in the calibration phase, the stepwise evolution associated with the imposed distances can already be identified, but the signal remains strongly affected by high-frequency fluctuations.

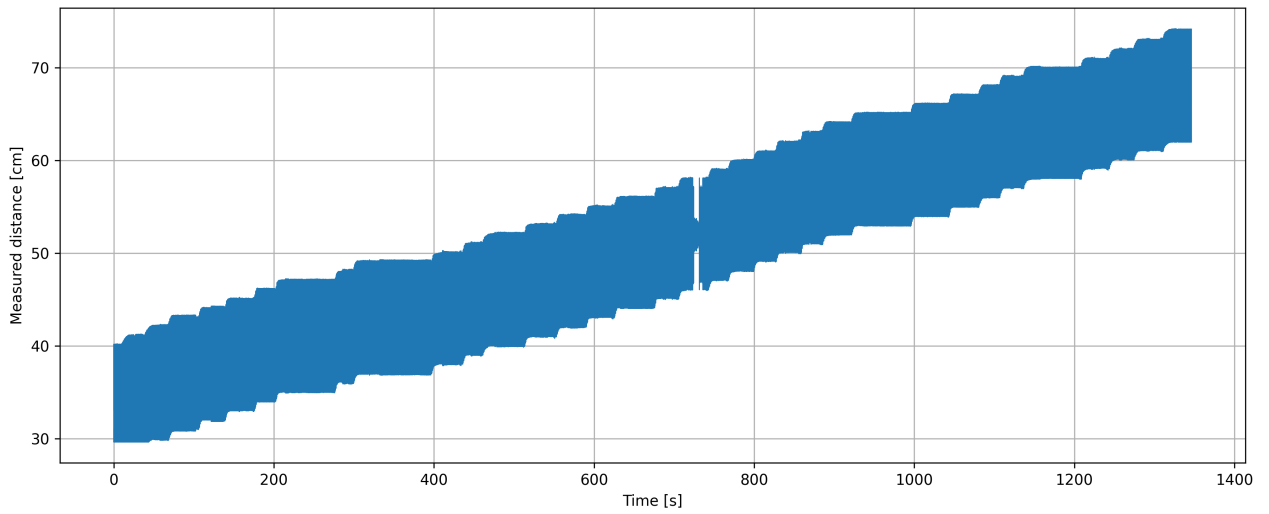


Figure 52: Raw distance signal measured by the laser sensor during the validation test.

In order to obtain a more stable representation of the distance evolution, the same low-pass filtering procedure as for the calibration stage was applied, with a cut-off frequency of 5 Hz. The comparison between the raw and filtered signals is presented in Figure 53. The filtered signal exhibits a much clearer stepwise structure, while preserving the successive plateaus corresponding to the imposed positions.

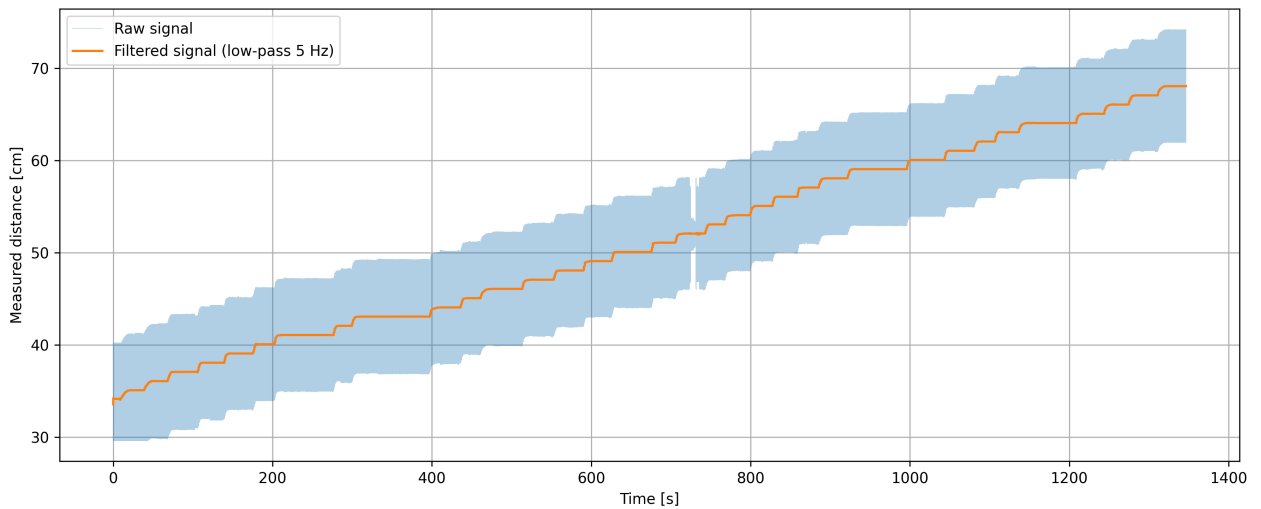


Figure 53: Comparison between the raw distance signal and the signal after application of a 5 Hz low-pass filter during the validation test.

In a second step, the filtered signal was processed in order to extract one representative value for each imposed distance level. To make plateau detection more robust, the filtered signal was first reduced by grouping the measurements into time blocks of 0.5 s and by replacing the signal within each block by its median value. The successive jumps between plateaus were then detected automatically from this reduced signal. In order to avoid contaminating the representative values with transient effects, the first and last 2 s of

each detected plateau were excluded from the calculation. For each stabilised interval, both the mean and the median distance were computed. The resulting processed signal, together with the 35 detected plateaus, is shown in Figure 54.

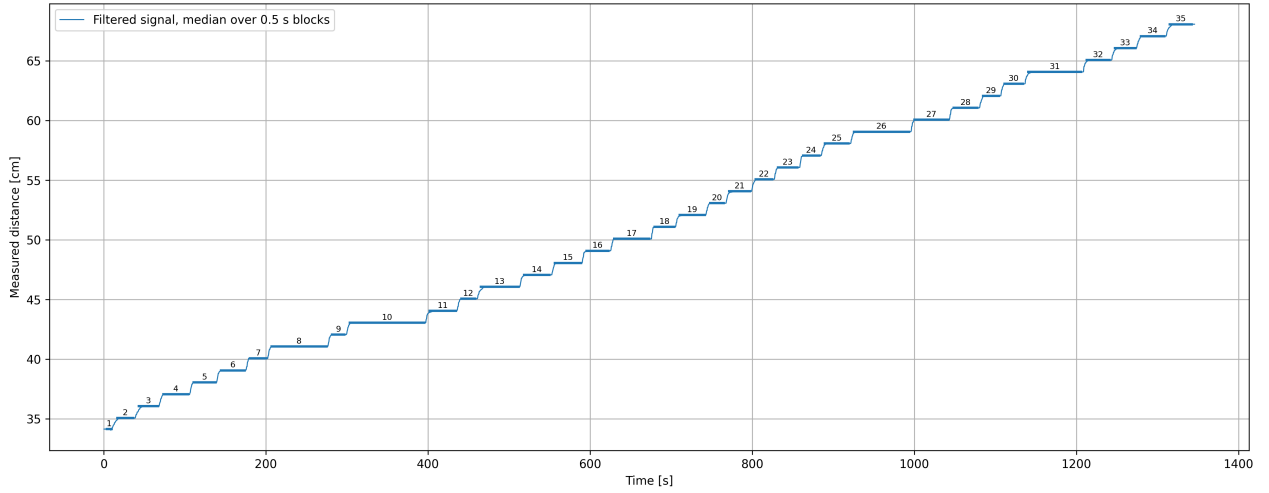


Figure 54: Processed filtered signal used for the validation analysis.

The measured plateau values were then compared with the corresponding theoretical imposed distances. Figure 55 presents this comparison using the plateau median values. The experimental points lie almost perfectly on the line of ideal agreement $y = x$, which indicates that the calibrated laser sensor is able to reproduce the imposed distance variations with very high accuracy over the whole range from 34 cm to 68 cm.

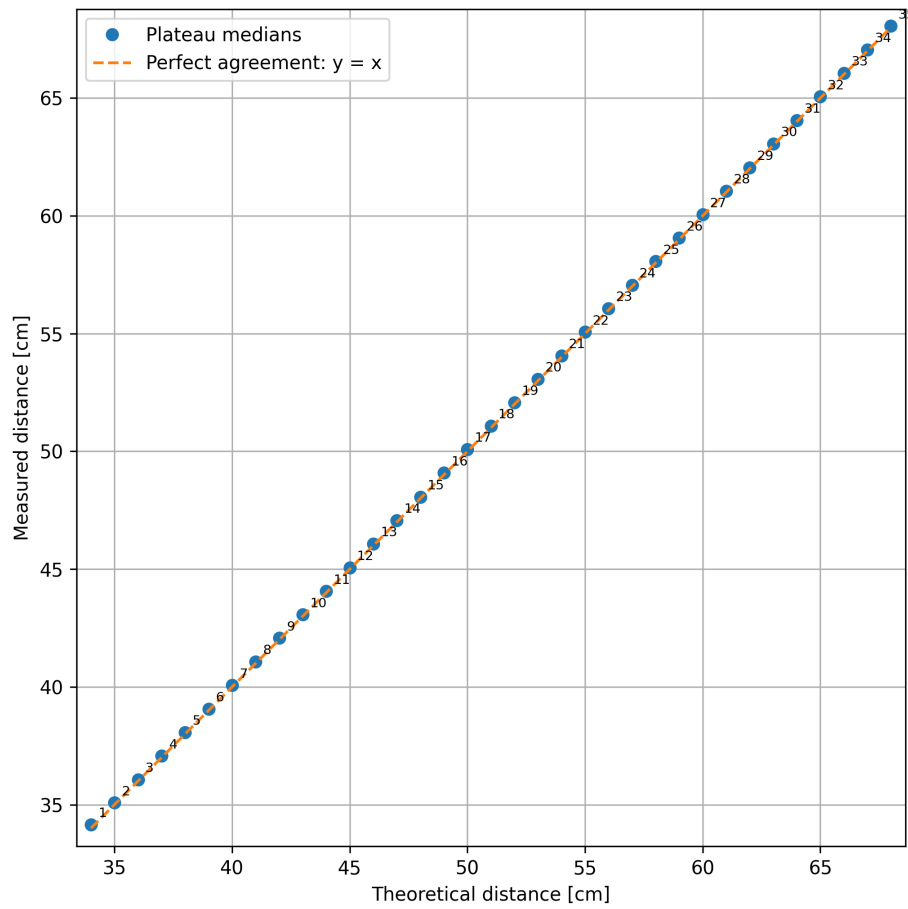


Figure 55: Comparison between the theoretical imposed distance and the measured distance obtained from the calibrated laser sensor during the validation test.

The corresponding global error indicators are summarised in Table 8. Very small deviations were obtained in all cases. When the plateau mean is used as representative value, the mean absolute error is 0.0605 cm and the root mean square error is 0.0646 cm. When the plateau median is used, the mean absolute error is 0.0694 cm and the root mean square error is 0.0714 cm. These values correspond to errors of the order of a few tenths of a millimetre, which confirms the very good quality of the adopted calibration law.

Table 8: Global error indicators obtained during the validation of the laser calibration.

Representative plateau value	MAE [cm]	RMSE [cm]
Mean distance	0.0605	0.0646
Median distance	0.0694	0.0714

5.9.9 Laser 1: SICK OD2 laser displacement sensor

An additional laser displacement sensor was used to measure the lateral displacement of the pile during the lateral loading tests. The sensor used for this purpose was a SICK

OD2 laser displacement sensor, identified in the laboratory as *Laser300A*. Its measuring range is 100–400 mm, and the sensor provides an analogue output signal between 0 and 10 V.

This second laser was used as a complementary displacement measurement point along the pile. Together with the first laser sensor, it made it possible to follow the horizontal movement of the pile at two different elevations during lateral loading. These two measurements were then used to estimate the pile rotation, the mudline displacement and the apparent depth of the rotation point, as described in Section 5.10.2.

No specific calibration test was performed for this laser sensor as part of the present experimental campaign. The sensor is regularly checked and calibrated within the laboratory, and was therefore considered reliable for the displacement measurements required in this work. Before testing, a functional check was carried out to verify that the sensor response was stable and that the measured distance was correct.

5.10 Experimental procedure and data processing

5.10.1 Test procedure

Each test campaign started with the preparation of the sand mass using the pluviation procedure retained in the previous chapter. As discussed in Section 5.8.1, the final preparation method was selected in order to obtain the most homogeneous and repeatable sand deposit possible within the large tank. Once the tank was filled, three pile tests were carried out in the same prepared sand mass.

The three pile locations were predefined in the tank and kept identical for all pluviations. Their arrangement was chosen so as to maximise the distance between the tests while maintaining a sufficient distance from the tank wall. This layout, inspired by the arrangement proposed by Shahrour [6], made it possible to perform three tests in one tank while limiting both boundary effects and possible interactions between neighbouring test zones.

The selected spacing is also consistent with experimental observations on laterally loaded pile groups in sand, for which pile-to-pile interaction effects become limited when the spacing is of the order of six pile diameters or larger [47]. In the present layout, the minimum spacing between two pile locations is 43 cm, corresponding to approximately $7.2D$ for a pile diameter of $D = 6$ cm. The distance to the tank wall was also kept as large as possible within the available geometry. Although some boundary influence cannot be completely excluded in a finite laboratory container, the adopted layout was considered acceptable for a comparative experimental programme. Similar model tests on laterally loaded stiff piles in sand have also emphasised the need to keep a sufficient pile-to-wall distance in order to reduce boundary effects [48]. A schematic view of the selected pile arrangement is shown in Figure 56.

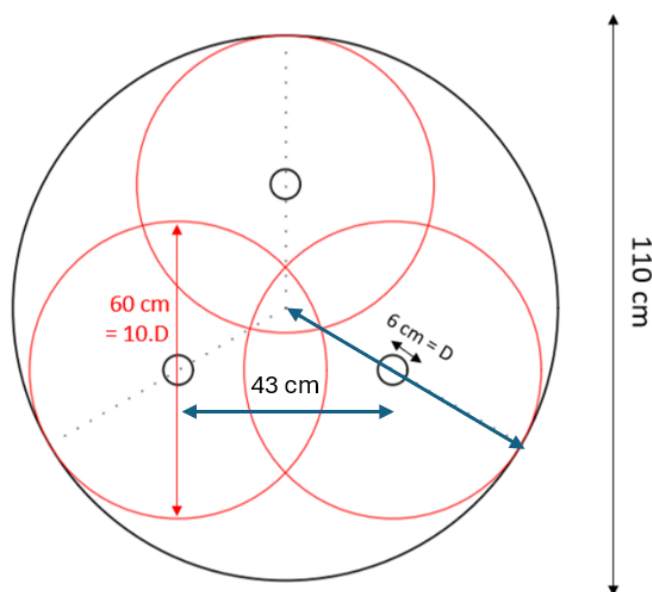


Figure 56: Schematic arrangement of the three pile locations in the large tank, inspired by Shahrour [6].

In practice, the position of the three pile locations was materialised by means of a tarpaulin placed on the sand surface. Openings were made in the tarpaulin at the pile locations so that the pile could be installed at the intended position. A reference mark was placed both on the tarpaulin and on the tank, which ensured that the tarpaulin was always positioned in the same orientation from one pluviation to another. This provided a simple and repeatable way of locating the pile positions before each test series. A photograph of the tarpaulin used for this purpose is given in Figure 57.



Figure 57: Tarpaulin used to materialise the three pile locations at the surface of the sand mass.

Before the installation of each pile, a CPT was performed at the corresponding pile location. The objective of these CPTs was not to redefine the global pluviation procedure, which had already been validated beforehand, but rather to document the local state of the sand at each test position. This information was kept as a reference in case an unexpected pile response was observed during installation or lateral loading.

After the CPT, the pile was installed at the considered location using the selected installation procedure. Depending on the test, the pile was driven either by impact driving or by vibratory driving until the target embedment was reached. The target depth was defined through the prescribed embedment ratio L/D , where L is the embedded length of the pile and D is the pile diameter.

Once the installation stage was completed, the lateral loading system was connected to the pile head and the monotonic lateral loading test was carried out using the motor-driven loading system.

The same general procedure was therefore followed for each pile test:

1. The sand mass was prepared using the retained pluviation procedure, as defined in the previous section.
2. The tarpaulin was positioned on the sand surface using the reference marks placed on both the tarpaulin and the tank, in order to ensure the same orientation and pile locations for each pluviation.

3. A CPT was performed at the considered pile location in order to document the local state of the sand before installation. This information was kept as a reference in case an unexpected pile response was observed during installation or lateral loading. All CPT profiles measured before pile installation are provided in Appendix A.
4. The pile was installed at the selected location using the prescribed installation method, either impact driving or vibratory driving, until the target embedment depth was reached.
5. The lateral loading system was connected to the pile. The Dyneema cable was attached to the loading point.
6. A monotonic lateral loading test was performed using the motor-driven system, while the force and displacement signals were recorded by the sensors.

This systematic procedure was adopted in order to ensure the best possible consistency between tests and to make the subsequent comparisons between installation methods and installation parameters as reliable as possible.

5.10.2 Processing of the lateral loading measurements

After each pile installation, the pile was subjected to a monotonic lateral loading test. The objective of this stage was to obtain the global lateral response of the installed pile and to express it in terms of the applied horizontal force H , the mudline displacement u_0 , the pile rotation θ , and the depth of the rotation point d .

The lateral load was transmitted to the pile through a Dyneema cable connected to the loading system. During the test, the tensile force in the cable was measured by the force sensor and was interpreted as the horizontal force applied to the pile. The sign convention was chosen so that a positive force corresponds to the direction of pulling imposed by the cable.

The pile displacement was obtained from the two laser displacement sensors. The laser signals were transformed into relative horizontal displacements with respect to the initial position of the pile after installation. The sign of the displacement was chosen consistently with the force convention, so that a positive displacement corresponds to the direction of the applied lateral load.

Let $u_1(t)$ and $u_2(t)$ be the horizontal displacements measured by Laser 1 and Laser 2, respectively, at heights h_1 and h_2 above the mudline. The vertical coordinate z is taken positive upwards from the mudline. Assuming that the pile behaves as a rigid body during lateral loading, the horizontal displacement profile can be written as

$$u(z, t) = u_0(t) + \theta(t)z,$$

where $u_0(t)$ is the displacement at mudline level and $\theta(t)$ is the rotation of the pile. Under the small-rotation assumption, the rotation can be estimated from the two laser measurements as

$$\theta(t) = \frac{u_2(t) - u_1(t)}{h_2 - h_1}.$$

The mudline displacement is then obtained by extrapolating the measured displacement profile to $z = 0$:

$$u_0(t) = u_1(t) - \theta(t)h_1.$$

Equivalently, this can be written directly from the two laser measurements as

$$u_0(t) = \frac{h_2u_1(t) - h_1u_2(t)}{h_2 - h_1}.$$

The depth of the rotation point below the mudline is obtained by finding the depth at which the extrapolated displacement becomes zero. Since the rotation point is located at $z = -d$, one has

$$0 = u_0(t) - \theta(t)d(t),$$

and therefore

$$d(t) = \frac{u_0(t)}{\theta(t)}.$$

All sensors used during the lateral loading tests were connected to the same data acquisition system. This made it possible to record the force and displacement signals synchronously during each test. For each experiment, the acquisition system generated an Excel file containing the complete time history of the measured quantities. These files were then processed using Python scripts developed for this work.

The main curve extracted from each lateral loading test is the relationship between the horizontal force H and the mudline displacement u_0 . This representation is used as the reference response for comparing the different installation methods and installation parameters. Additional quantities, such as the pile rotation θ and the estimated depth of the rotation point d , are used to further interpret the kinematics of the pile during loading.

5.11 Test programme

The experimental campaign is designed to investigate the influence of the pile installation method on the subsequent lateral response of the pile.

A total of twelve reference tests were carried out. The tests are divided into four groups of three tests. Each group corresponds to one sand pluviation, meaning that three pile installations are performed in the tank after each preparation of the sand mass. This approach limits the number of complete sand preparations while still allowing the influence of selected installation parameters to be investigated.

The pile diameter is kept constant and equal to $D = 6$ cm for all tests. The lateral loading height is also kept constant, with $h/D = 10$, corresponding to a load application height of $h = 60$ cm above the mudline. After each installation, the pile is laterally loaded under the same loading configuration in order to compare the lateral response of piles installed under different conditions while limiting the number of parameters that vary between tests.

The experimental programme is organised in two main parts. The first part concerns impact-driven piles and investigates the influence of the embedment ratio and of the impact driving configuration. The second part concerns vibratory-driven piles and investigates the influence of the embedment ratio and of the vibration frequency.

For impact driving tests, the impact energy is defined as

$$E = mgH_{\text{drop}},$$

where m is the falling mass, g is the gravitational acceleration and H_{drop} is the drop height.

The first group of impact driving tests is used to study the influence of the embedment ratio L/D . In this group, the impact conditions are kept constant, with the same falling mass and the same drop height, while the target embedment ratio is varied. Three values of L/D are considered: 5.0, 7.0 and 8.5.

The second group of impact driving tests focuses on the influence of the impact driving configuration. In this group, the target embedment ratio is kept constant and equal to $L/D = 7.0$. Different combinations of falling mass and drop height are selected in order to obtain approximately comparable impact energies. This makes it possible to evaluate whether different impact configurations, leading to a similar nominal impact energy, produce a comparable lateral response after installation.

For vibratory driving tests, the eccentric moment M_e is kept constant throughout the vibratory test series. Consequently, the vibration frequency f is the only vibratory driving parameter varied in the present programme.

The first group of vibratory driving tests is used to study the influence of the embedment ratio L/D . In this group, the vibratory driving conditions are kept constant. A reference motor speed of 15 krpm, corresponding to a vibration frequency of approximately 250 Hz, is retained. As for the impact-driven piles, three values of L/D are considered: 5.0, 7.0 and 8.5.

The second group of vibratory driving tests focuses on the influence of the vibration frequency. In this group, the target embedment ratio is kept constant and equal to $L/D = 7.0$, and the eccentric moment is kept constant. Three motor speeds are selected: 10 krpm, 13.5 krpm and 17 krpm, corresponding respectively to approximately 166.7 Hz, 225 Hz and 283.3 Hz.

The complete reference test matrix is presented in Table 9.

The notation HR refers to the tests performed with the initial high-rate lateral loading procedure; these tests are included for comparison but are not used as the reference loading condition in the subsequent analysis.

Table 9: Experimental test matrix for pile installation and lateral loading.

Test	Pluviation	Installation	D [cm]	L/D [-]	H_{drop} [cm]	m [kg]	M_e [g mm]	v [krpm]	f [Hz]
1 _{HR}	1	Impact driving	6	7.0	25.0	1.6	–	–	–
2 _{HR}	1	Impact driving	6	5.0	25.0	1.6	–	–	–
3 _{HR}	1	Impact driving	6	8.5	25.0	1.6	–	–	–
1	1	Impact driving	6	7.0	25.0	1.6	–	–	–
2	1	Impact driving	6	5.0	25.0	1.6	–	–	–
3	1	Impact driving	6	8.5	25.0	1.6	–	–	–
4	2	Impact driving	6	7.0	25.0	1.6	–	–	–
5	2	Impact driving	6	7.0	5.4	7.4	–	–	–
6	2	Impact driving	6	7.0	33.3	1.2	–	–	–
7	3	Vibratory driving	6	7.0	–	–	28	15.0	250.0
8	3	Vibratory driving	6	5.0	–	–	28	15.0	250.0
9	3	Vibratory driving	6	8.5	–	–	28	15.0	250.0
10	4	Vibratory driving	6	7.0	–	–	28	13.5	225.0
11	4	Vibratory driving	6	7.0	–	–	28	17.0	283.3
12	4	Vibratory driving	6	7.0	–	–	28	10.0	166.7

6 Results

This section presents the experimental results obtained during the pile installation tests and the subsequent monotonic lateral loading tests. The objective is first to report the measured installation response, and then to compare the post-installation lateral behaviour of the piles as a function of the installation method and installation parameters. The results are therefore organised progressively, starting from the installation records and then focusing on the lateral force–displacement response, the influence of the embedment ratio, the influence of the impact- and vibratory-driving parameters, and finally the kinematic response of the pile during lateral loading.

For each lateral loading test, the main response curve is expressed as the positive lateral force F as a function of the normalised mudline displacement u_0/D . The mudline displacement u_0 is obtained from the displacement measurements described in Section 5.10.2.

The force–displacement curve $F(u_0/D)$ is the primary result used in the following sections. It provides a direct visual comparison of the post-installation lateral response of the pile–soil system. It allows differences in initial stiffness, progressive mobilisation of soil resistance and resistance level at larger displacement to be identified. In the figures, only the monotonic loading branch is retained.

In this work, the force–displacement curves are generally presented up to $u_0/D = 0.10$. This value is retained as a common reference displacement because all tests reached this displacement level and because a horizontal displacement equal to $0.1D$ is commonly used in the literature as a conventional ultimate limit state criterion for laterally loaded piles or monopiles. Frick and Achmus [49] summarise several failure criteria adopted in previous monopile studies, including criteria based on a pile displacement of $0.1D$ and on limiting pile rotations. In the present study, this criterion is not interpreted as a true physical failure of the model pile. It is used as a consistent comparison limit between tests. Since

the pile diameter is $D = 60$ mm, this reference displacement corresponds to

$$u_0 = 0.10D = 6 \text{ mm}. \quad (14)$$

In addition to the complete curves, several scalar indicators are extracted in order to compare the tests quantitatively. The first one is the lateral resistance at the reference displacement, denoted $F_{0.10D}$, defined as

$$F_{0.10D} = F\left(\frac{u_0}{D} = 0.10\right). \quad (15)$$

This value represents the force mobilised when the mudline displacement reaches 10% of the pile diameter. It is not interpreted as an ultimate resistance in a strict sense, but as a reproducible reference value allowing the different tests to be compared at the same deformation level.

The initial stiffness of the response is quantified through an initial secant stiffness, denoted K_{initial} . In this work, K_{initial} is not defined as a true tangent stiffness at the origin, since the very beginning of the force–displacement curve may be sensitive to measurement noise, cable tensioning and small initial adjustments of the loading system. Instead, it is defined at a small reference displacement corresponding to $u_0/D = 0.01$.

The force mobilised at this displacement is denoted $F_{0.01D}$ and is obtained by interpolation of the processed force–displacement curve:

$$F_{0.01D} = F\left(\frac{u_0}{D} = 0.01\right). \quad (16)$$

The initial stiffness is then calculated as the secant stiffness between the origin and this reference point:

$$K_{\text{initial}} = \frac{F_{0.01D}}{0.01D}. \quad (17)$$

For the model pile diameter used in this study, $D = 60$ mm, this reference displacement corresponds to

$$0.01D = 0.6 \text{ mm} = 0.06 \text{ cm}. \quad (18)$$

Since u_0 is expressed in centimetres in the data processing, K_{initial} is reported in N/cm. This indicator characterises the small-displacement response of the pile–soil system while avoiding the most noise-sensitive part of the signal at the very beginning of loading.

Additional force indicators, such as $F_{0.02D}$ and $F_{0.05D}$, are also reported in some tables. They correspond to the force mobilised at $u_0/D = 0.02$ and $u_0/D = 0.05$, respectively:

$$F_{0.02D} = F\left(\frac{u_0}{D} = 0.02\right), \quad F_{0.05D} = F\left(\frac{u_0}{D} = 0.05\right). \quad (19)$$

These intermediate values make it possible to compare the evolution of the response at different displacement levels and to distinguish between changes in initial stiffness and changes in resistance at larger displacement.

The kinematic interpretation of the lateral loading tests is based on the estimated pile rotation and on the apparent depth of the rotation point. These quantities are used in the last part of the results to complement the force–displacement analysis. They make it possible to assess whether differences in lateral resistance are associated with different

pile kinematics, for example through a deeper or shallower apparent rotation point during loading.

For the moment–rotation curves, the response is generally presented up to $\theta = 1.5^\circ$. This value is also consistent with conventional rotation-based failure criteria reported in the literature for monopile foundations. As for the displacement criterion, it is used here as a common comparison threshold rather than as evidence of a sudden physical failure of the model pile. The combined use of $u_0/D = 0.10$ and $\theta = 1.5^\circ$ therefore provides two complementary and reproducible reference levels for comparing the different tests.

6.1 Installation response

6.1.1 Impact driving tests

The impact driving tests were first analysed through the number of blows required to reach the target embedment depth. The complete installation results for the impact-driven piles are summarised in Table 10. For the reference impact series, the impact configuration was kept constant, with a falling mass of 1.6 kg, a drop height of 25 cm, and a nominal impact energy of approximately 3.92 J/drop. The corresponding installation response is therefore mainly assessed through the number of blows required to reach the target embedment ratio.

Table 10: Summary of the impact driving installation results.

Test	L/D [-]	m [kg]	H_{drop} [cm]	E [J/drop]	N_{drops} [-]
$T1_{\text{HR}}$	7.0	1.6	25.0	3.92	194
$T1$	7.0	1.6	25.0	3.92	181
$T2_{\text{HR}}$	5.0	1.6	25.0	3.92	112
$T2$	5.0	1.6	25.0	3.92	103
$T3_{\text{HR}}$	8.5	1.6	25.0	3.92	290
$T3$	8.5	1.6	25.0	3.92	223
T4	7.0	1.6	25.0	3.92	135
T5	7.0	7.4	5.4	3.92	105
T6	7.0	1.2	33.3	3.92	184

For the tests performed with the reference impact configuration, the number of blows generally increased with the target embedment ratio. The piles installed to $L/D = 5$ required 103 to 112 blows, whereas the piles installed to $L/D = 8.5$ required 223 to 290 blows. For $L/D = 7$, some scatter is observed, with values between 135 and 194 blows for the tests performed with the same nominal impact configuration. This confirms that the installation effort increases markedly with embedment depth when the nominal impact energy per blow is kept constant.

A second comparison was carried out for tests T4, T5 and T6, which were all performed at $L/D = 7$ but with different combinations of falling mass and drop height. Although the nominal impact energy per blow was the same for the three tests, the installation records were different. Test T5, performed with the largest falling mass and the smallest

drop height, required the lowest number of blows. In contrast, T6, performed with the smallest falling mass and the largest drop height, required the largest number of blows.

This difference is also visible in Figure 58, which compares the cumulative nominal impact energy required to reach the target embedment depth. The results show that the nominal impact energy per blow alone does not fully describe the installation process. The number of blows and the cumulative nominal impact energy are therefore retained as complementary indicators of the impact driving installation response.

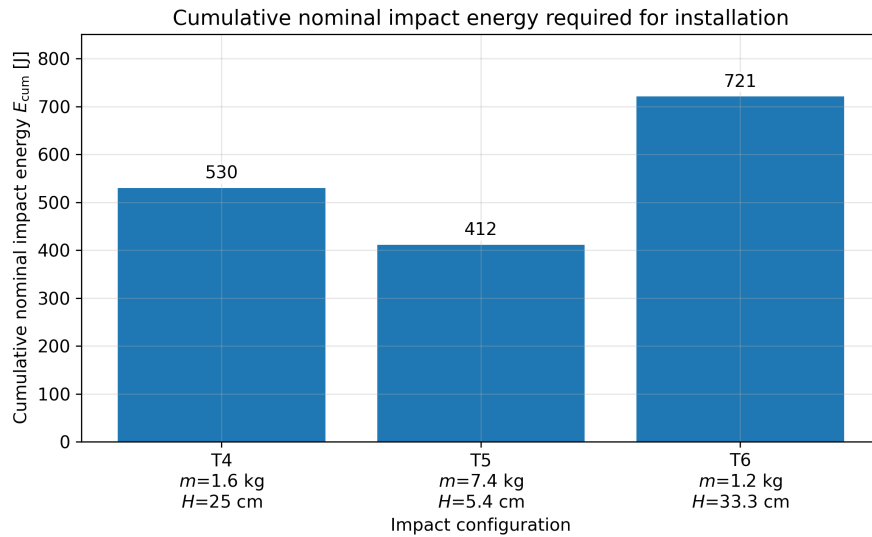


Figure 58: Cumulative nominal impact energy for the impact-driven piles installed at $L/D = 7$.

6.1.2 Vibratory driving tests

In contrast with the impact driving tests, for which the installation record is expressed through the number of blows, the direct installation indicator retained for vibratory driving is the duration during which the vibrator remained active.

Figure 59 shows the influence of the embedment ratio for the tests performed at a constant motor speed of $v = 15$ krpm. The active duration increases strongly with the target embedment ratio. This shows that the installation effort increases markedly with embedment depth.

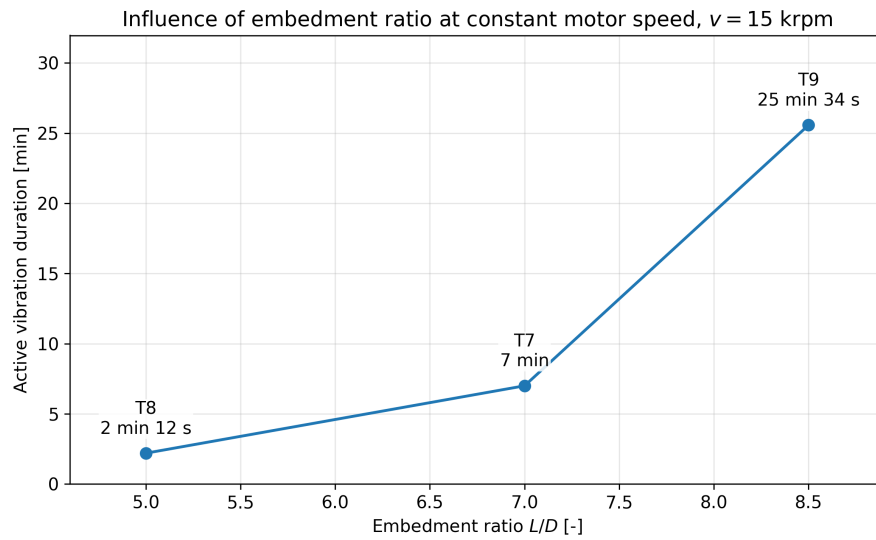


Figure 59: Influence of the embedment ratio on vibratory driving duration.

The influence of the motor speed is shown in Figure 60 for the tests targeting $L/D = 7$. Test T12, the lowest motor speed, $v = 10$ krpm, corresponding to an excitation frequency of approximately 167 Hz, led to refusal before the target embedment depth was reached. It stopped at an embedment ratio of $L/D = 6.17$ after 34 min of active vibration. At higher motor speeds, the target embedment ratio was reached.

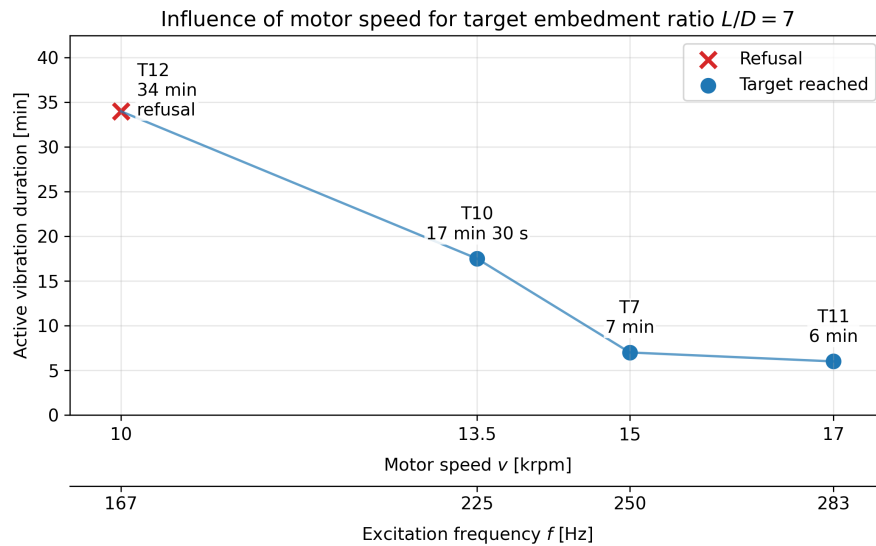


Figure 60: Influence of the motor speed on vibratory driving duration.

These results indicate that both the embedment ratio and the motor speed have a clear influence on the vibratory installation response. Increasing the embedment ratio leads to longer driving durations, while increasing the motor speed generally reduces the duration required to reach the target depth. Test T12 is kept as a specific case, since the target embedment ratio was not reached and refusal occurred before $L/D = 7$.

6.2 Influence of the lateral loading rate

Before analysing the influence of the installation method and installation parameters on the post-installation lateral response, the effect of the imposed lateral loading rate was first examined. This comparison was introduced because the first tests of the campaign were initially performed with a relatively high actuator velocity, whereas the subsequent tests were carried out using the lower reference loading rate defined in the methodology.

The comparison is based on the three impact-driven configurations used to study the influence of the embedment ratio. The tests denoted $T1$, $T2$ and $T3$ correspond to the reference low-rate lateral loading tests, while $T1_{HR}$, $T2_{HR}$ and $T3_{HR}$ correspond to the associated high-rate tests. The subscript HR is used to indicate the high-rate loading condition.

Figure 61 compares the force–displacement curves obtained for the two loading rates. For each embedment ratio, the same colour is used for the low-rate and high-rate tests, while the line style distinguishes the loading rate. Overall, the curves obtained at the two loading rates remain relatively close for a given embedment ratio, especially when compared with the differences observed between the three values of L/D . This suggests that the embedment ratio has a much stronger influence on the lateral response than the change in loading rate considered here.

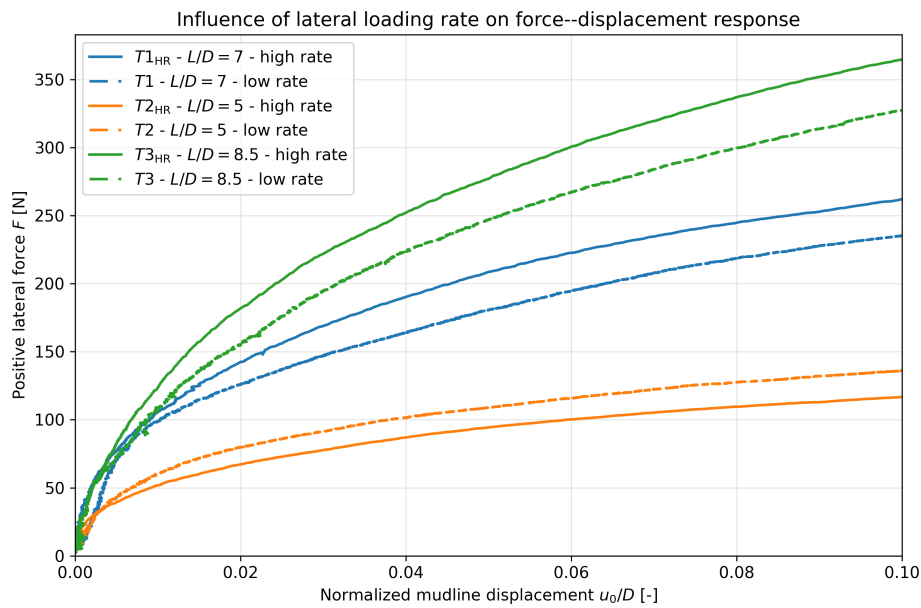


Figure 61: Influence of the lateral loading rate on the force–displacement response.

The comparison based on $F_{0.10D}$ is presented in Figure 62. The results do not show a systematic increase or decrease of the force with the loading rate. For $T1$ and $T3$, the high-rate tests lead to higher values of $F_{0.10D}$, whereas for $T2$, the low-rate test gives a higher value. Therefore, no clear or general trend can be established from these three pairs of tests regarding the influence of the imposed lateral loading rate on the force mobilised at $u_0/D = 0.10$.

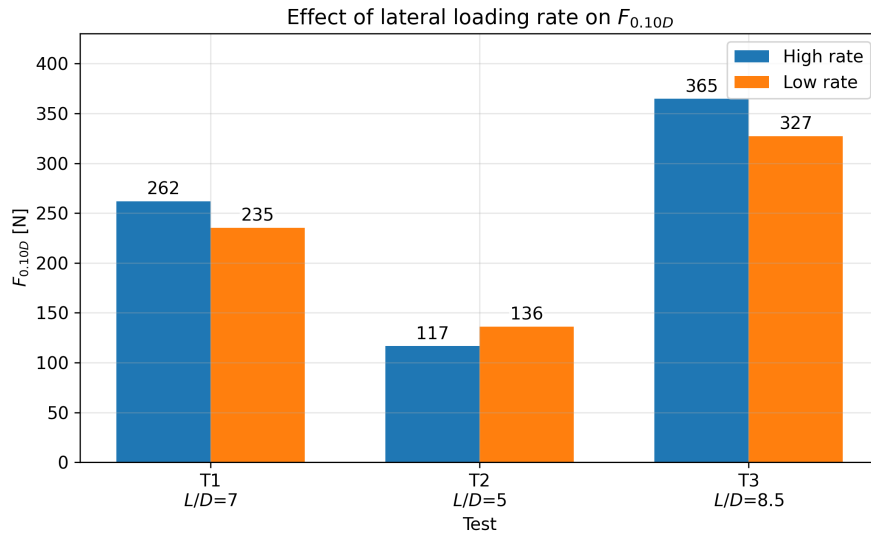


Figure 62: Effect of the lateral loading rate on the force mobilised at $u_0/D = 0.10$.

A similar observation can be made for the initial stiffness K_{initial} , shown in Figure 63. The high-rate tests give larger values of K_{initial} for $T1$ and $T3$, but a lower value for $T2$. As for $F_{0.10D}$, the results are therefore not sufficient to identify a consistent loading-rate effect over the investigated range.

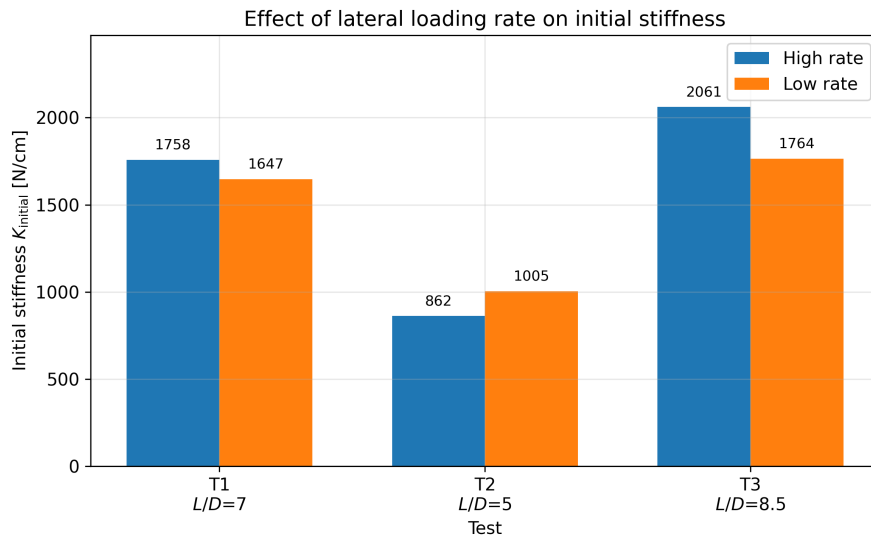


Figure 63: Effect of the lateral loading rate on the initial stiffness K_{initial} .

Based on these observations, the influence of the loading rate cannot be isolated in a definitive way from the present dataset. The two loading rates lead to broadly comparable force–displacement responses, while the embedment ratio remains the dominant parameter affecting the lateral resistance and stiffness. For the rest of the analysis, the low-rate tests $T1$, $T2$ and $T3$ are retained as the reference results, since they correspond to the loading rate selected in the methodology. The high-rate tests are kept as complementary results and are not used for the main comparison between installation conditions.

6.3 Lateral response of the piles

6.3.1 Influence of the embedment ratio

The influence of the embedment ratio was analysed separately for impact-driven and vibratory-driven piles. For each installation method, three embedment ratios were considered: $L/D = 5$, $L/D = 7$, and $L/D = 8.5$. The objective of this subsection is to compare the lateral response obtained after installation and to assess whether increasing the embedded length leads to a higher lateral resistance and initial stiffness.

Figure 64 presents the force–displacement response of the impact-driven piles. For this installation method, the effect of the embedment ratio is clear. The pile installed to $L/D = 5$, denoted $T2$, gives the lowest response. The resistance then increases for $T1$, installed to $L/D = 7$, and the highest force level is obtained for $T3$, installed to $L/D = 8.5$. This trend is visible over the whole displacement range and is consistent with the expected influence of the embedded length on the lateral response of the pile–soil system.

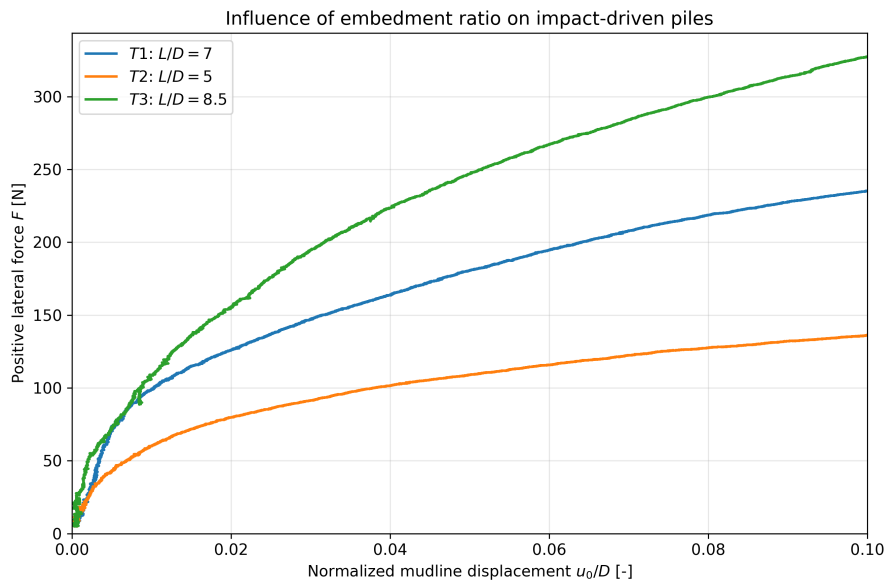


Figure 64: Influence of the embedment ratio on the force–displacement response of the impact-driven piles.

The response of the vibratory-driven piles is shown in Figure 65. In this case, the trend is less regular. The pile installed to $L/D = 5$, denoted $T8$, gives the lowest response, as expected. However, the pile installed to the largest embedment ratio, $T9$ with $L/D = 8.5$, does not mobilise a higher lateral resistance than $T7$, installed to $L/D = 7$. On the contrary, over a significant part of the displacement range, the response of $T9$ remains slightly lower than that of $T7$. This indicates that, for the vibratory-driven series, the increase in embedment ratio from $L/D = 7$ to $L/D = 8.5$ does not translate into an increase in measured lateral resistance.

It should also be noted that some local downward force drops remain visible on the vibratory curves, even after application of the 5 Hz low-pass filter. These drops were kept in the plotted results because they are still present after filtering and therefore cannot be

treated simply as high-frequency measurement noise. Their origin cannot be identified with certainty from the available measurements. Possible explanations include a small adjustment or tightening of a knot in the Dyneema cable, a transient variation in the motor-driven pulling system, local stick-slip in the loading system, or a sudden rearrangement of the soil-pile system during loading. These events are therefore mentioned as experimental features of the measured response.

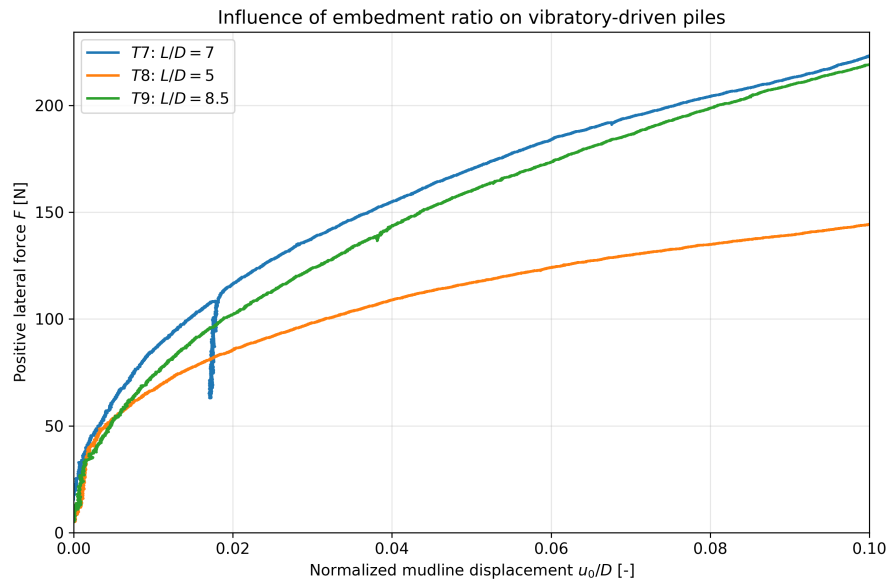


Figure 65: Influence of the embedment ratio on the force-displacement response of the vibratory-driven piles.

In order to better understand the relatively low resistance of $T9$, the CPT profiles measured before the installation of the vibratory-driven piles are compared in Figure 66. These CPTs were performed at the corresponding pile locations and provide an indication of the local state of the sand before installation. The CPT profile associated with $T9$ shows a lower cone resistance than $T7$ over a large part of the investigated depth range. This suggests that the sand at the $T9$ location was locally less resistant before pile installation. This difference in initial sand condition provides a plausible explanation for why $T9$, despite being installed deeper, did not develop a higher lateral resistance than $T7$.

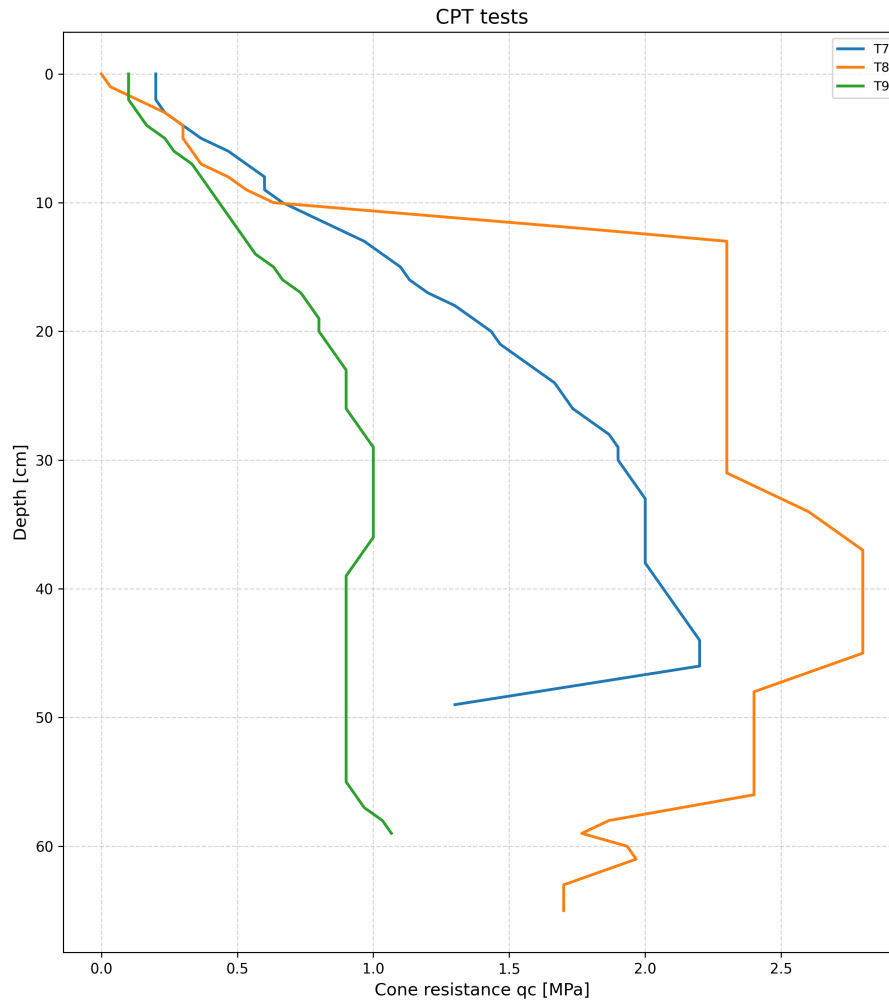


Figure 66: CPT profiles measured before the installation of the vibratory-driven piles $T7$, $T8$, and $T9$.

The same observation is reflected in the initial stiffness values shown in Figure 67. For the impact-driven piles, K_{initial} increases with the embedment ratio, consistently with the force-displacement response and with the values of $F_{0.10D}$. For the vibratory-driven piles, the stiffness increases from $T8$ to $T7$, but $T9$ does not show a higher stiffness than $T7$. This confirms that the expected beneficial effect of the larger embedment ratio is not clearly observed for $T9$.

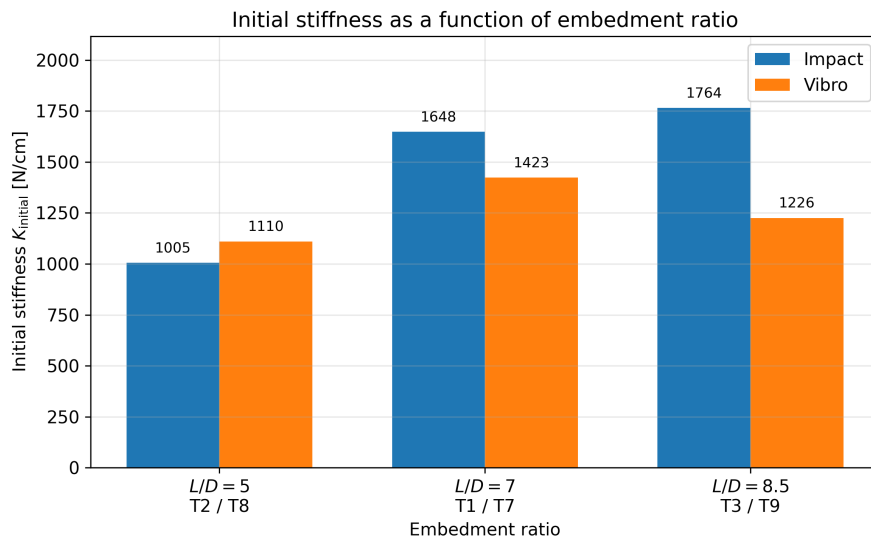


Figure 67: Initial stiffness K_{initial} as a function of the embedment ratio for impact-driven and vibratory-driven piles.

Overall, the embedment ratio clearly controls the lateral response of the impact-driven piles. For the vibratory-driven piles, the increase from $L/D = 5$ to $L/D = 7$ is visible, but the pile installed to $L/D = 8.5$ exhibits a slightly lower resistance than the pile installed to $L/D = 7$. The CPT measurements suggest that this result is likely influenced by the lower local cone resistance measured at the $T9$ location before installation. Consequently, the vibratory-driven series shows that the effect of L/D must be interpreted together with the local initial sand state.

6.3.2 Influence of the impact-driving configuration

The influence of the impact-driving configuration was analysed through tests $T4$, $T5$, and $T6$. These three piles were all installed to the same embedment ratio, $L/D = 7$, and with a nominal impact energy per blow of approximately 3.92 J/drop. The objective of this comparison is therefore to assess whether different combinations of falling mass and drop height, despite leading to the same nominal impact energy, produce different post-installation lateral responses.

The complete force–displacement curves are compared in Figure 68. Although the three tests were installed under the same nominal impact energy per blow, noticeable differences appear in the lateral response. Test $T6$, corresponding to the lowest mass and the largest drop height, mobilises the highest lateral resistance over most of the displacement range. Test $T5$, performed with the largest falling mass and the smallest drop height, shows an intermediate response. Test $T4$, corresponding to the reference impact configuration, gives the lowest force level of the three tests over a large part of the loading range.

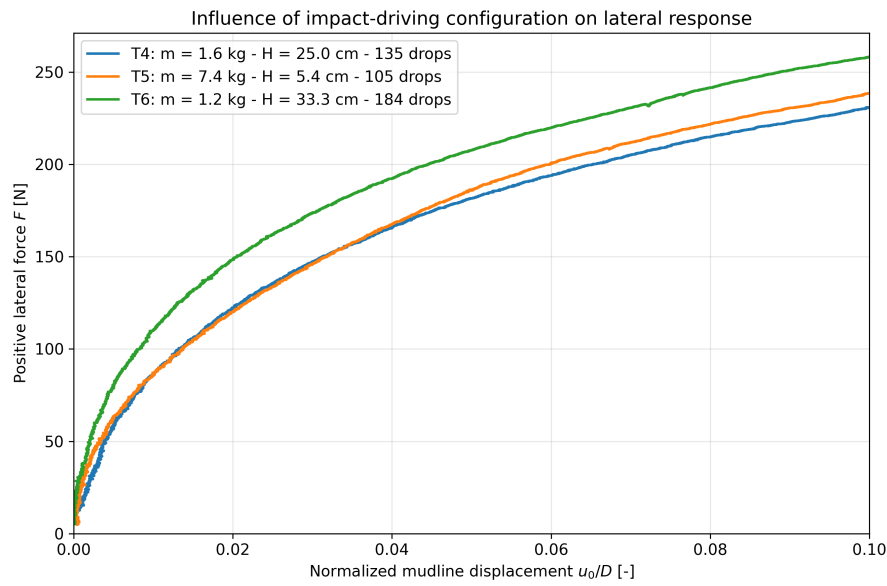


Figure 68: Influence of the impact-driving configuration on the force–displacement response for piles installed at $L/D = 7$ with a similar nominal impact energy per blow.

The same comparison is expressed in terms of initial stiffness in Figure 69. The values of K_{initial} remain of the same order of magnitude for the three tests, but some differences can still be observed. Test $T6$ gives the highest initial stiffness, while $T5$ gives the lowest. Test $T4$ lies between these two cases.

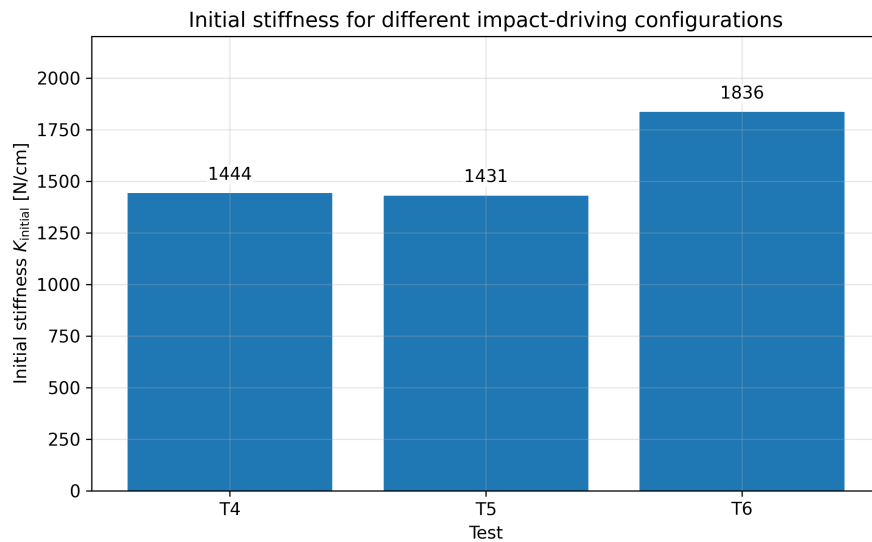


Figure 69: Initial stiffness K_{initial} for the different impact-driving configurations at $L/D = 7$.

Overall, the results show that the post-installation lateral response is not identical for the three impact-driving configurations, even though the nominal impact energy per blow was kept constant. In the present tests, the configuration associated with the lowest falling mass and the largest drop height led to the highest measured lateral resistance and

initial stiffness, whereas the largest mass and smallest drop height did not produce the highest response. These observations suggest that, at model scale, the nominal impact energy alone is not sufficient to fully characterise the effect of the installation conditions on the subsequent lateral behaviour.

6.3.3 Influence of the vibratory-driving frequency

The influence of the vibratory-driving frequency was analysed using the tests performed with different motor speeds. The comparison mainly focuses on the piles targeting $L/D = 7$, namely $T10$, $T7$, and $T11$, which were installed at motor speeds of 13.5, 15, and 17 krpm, respectively. Test $T12$, performed at the lowest motor speed, is also shown for completeness, but it must be interpreted separately since the target embedment ratio was not reached and refusal occurred at $L/D = 6.17$. This point is important because the previous section showed that the embedment ratio has a strong influence on the lateral response of the piles.

Figure 70 compares the force–displacement curves obtained for the different vibratory-driving frequencies. The three tests for which the target embedment ratio was reached, $T10$, $T7$, and $T11$, show responses of the same order of magnitude over the investigated displacement range. Among these tests, $T7$ tends to mobilise a slightly higher force level, while $T10$ and $T11$ remain relatively close. Test $T12$, corresponding to the lowest motor speed, gives a lower lateral resistance. However, this lower response cannot be attributed to the motor speed alone, since the pile did not reach the target embedment ratio and was therefore less deeply embedded than the other piles.

As for the previous vibratory-driven tests, local downward force drops remain visible on the curves. These features may correspond to transient physical events during loading and are therefore retained in the plotted response.

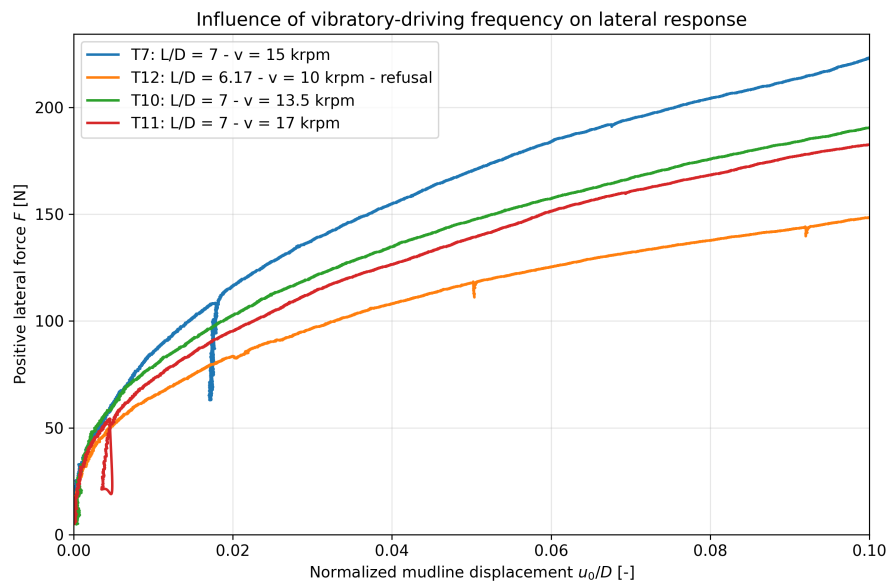


Figure 70: Influence of the vibratory-driving frequency on the force–displacement response.

In order to verify whether the slightly higher response of $T7$ could be related to a

more resistant initial sand state, the CPT profiles measured before the installation of $T10$, $T7$, $T11$, and $T12$ are compared in Figure 71. The profiles do not show a clear and systematic higher cone resistance for $T7$ over the investigated depth range. Therefore, the CPT measurements do not provide a direct explanation for the higher lateral resistance observed for $T7$. This difference may be related to local variability not fully captured by the CPT profiles, to installation-induced changes in the soil state, or to experimental effects during the lateral loading test. Based on the available measurements, the reason for the slightly higher response of $T7$ remains uncertain.

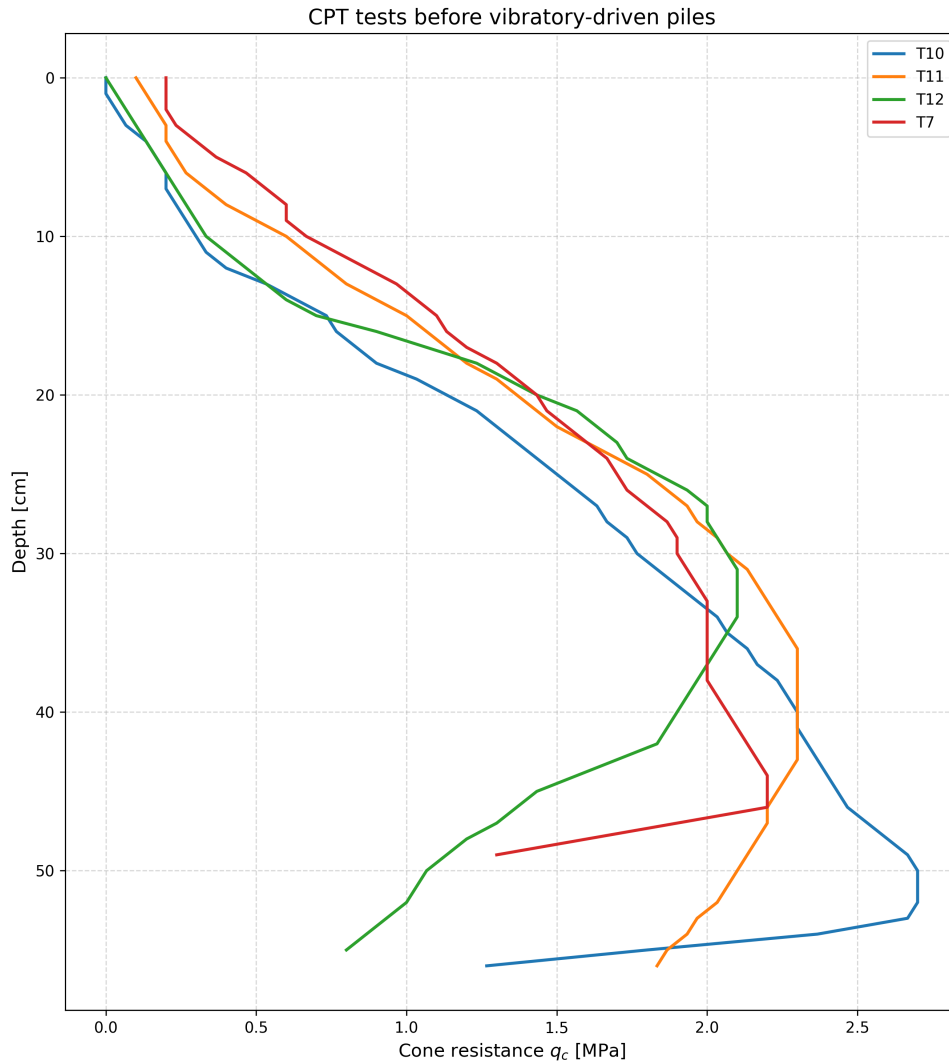


Figure 71: CPT profiles measured before the installation of the vibratory-driven piles.

The corresponding initial stiffness values are presented in Figure 72. The lowest value of K_{initial} is obtained for $T12$, which is consistent with its lower achieved embedment ratio and lower force–displacement response. For the piles that reached $L/D = 7$, the values obtained for $T10$, $T7$, and $T11$ remain relatively close, with $T7$ giving the highest value among the three. However, the evolution is not monotonic with motor speed, since the stiffness does not continue to increase from 15 to 17 krpm. Based on these results, no clear monotonic influence of the vibrodriver’s motor speed on the initial stiffness can be

established from the available tests.

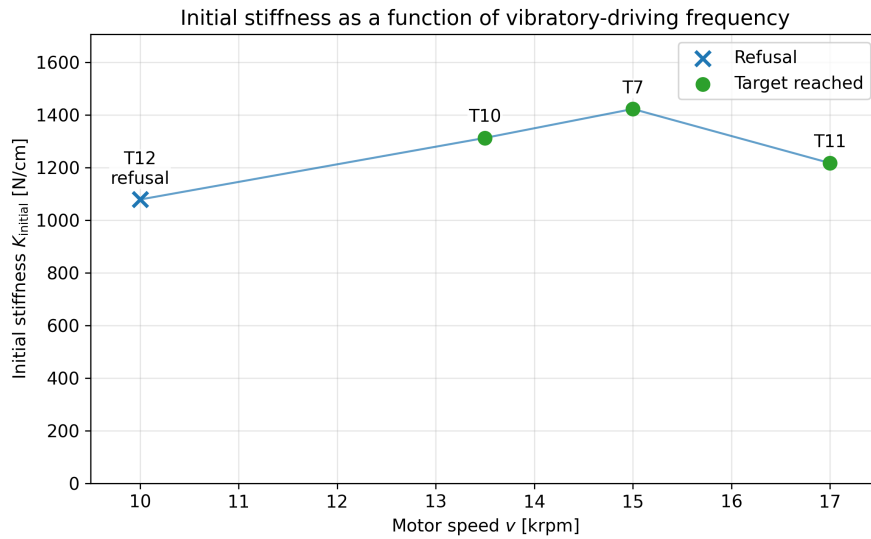


Figure 72: Initial stiffness K_{initial} as a function of the vibratory-driving motor speed.

Overall, the results do not allow a direct and isolated conclusion on the effect of the vibratory-driving frequency alone. The three piles that reached $L/D = 7$ show comparable lateral responses, without a clear monotonic trend with motor speed. Although $T7$ shows a slightly higher response, the CPT profiles do not indicate a clearly more resistant initial sand state at this location. The lowest-frequency test, $T12$, reached refusal before the target depth and consequently cannot be used as a direct comparison at constant embedment ratio.

6.4 Comparison between impact-driven and vibratory-driven piles

6.4.1 Force–displacement response

The force–displacement response of the impact-driven and vibratory-driven piles is first compared at similar embedment ratios. Figure 73 presents the curves obtained for $L/D = 5$, $L/D = 7$, and $L/D = 8.5$. For each embedment ratio, the impact-driven and vibratory-driven piles are plotted with the same colour, while the line style distinguishes the installation method.

As discussed previously, the lower response of $T9$ must be interpreted together with the lower local cone resistance measured before installation.

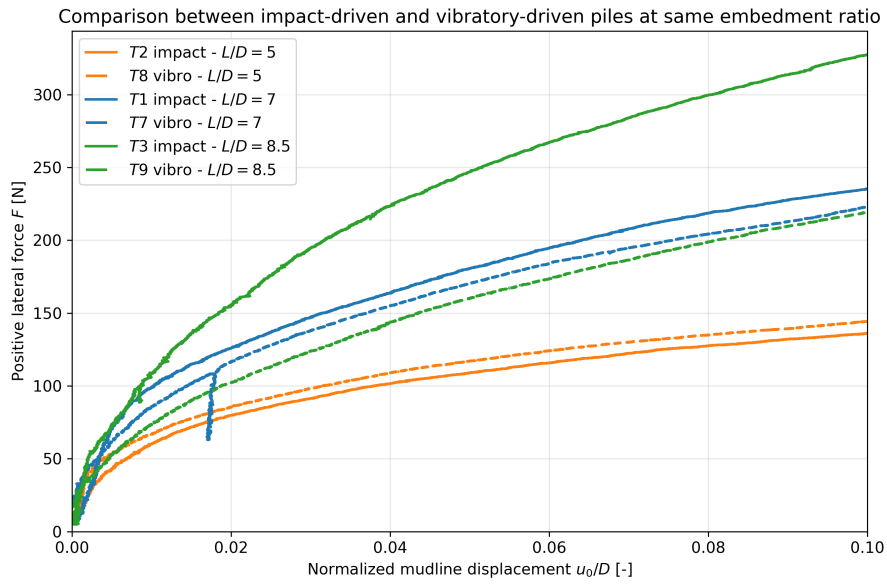


Figure 73: Comparison between impact-driven and vibratory-driven piles at the same embedment ratio.

A second comparison is made in Figure 74, focusing on the piles installed close to $L/D = 7$. This comparison includes the impact-driven piles $T1$, $T4$, $T5$, and $T6$, and the vibratory-driven piles $T10$, $T7$, and $T11$. Overall, the impact-driven piles tend to mobilise higher forces than the vibratory-driven piles over most of the displacement range. However, the difference is not extremely large, and the curves remain within the same order of magnitude. This indicates that the installation method affects the post-installation lateral response, but that the scatter between tests and the exact installation conditions also remain important.

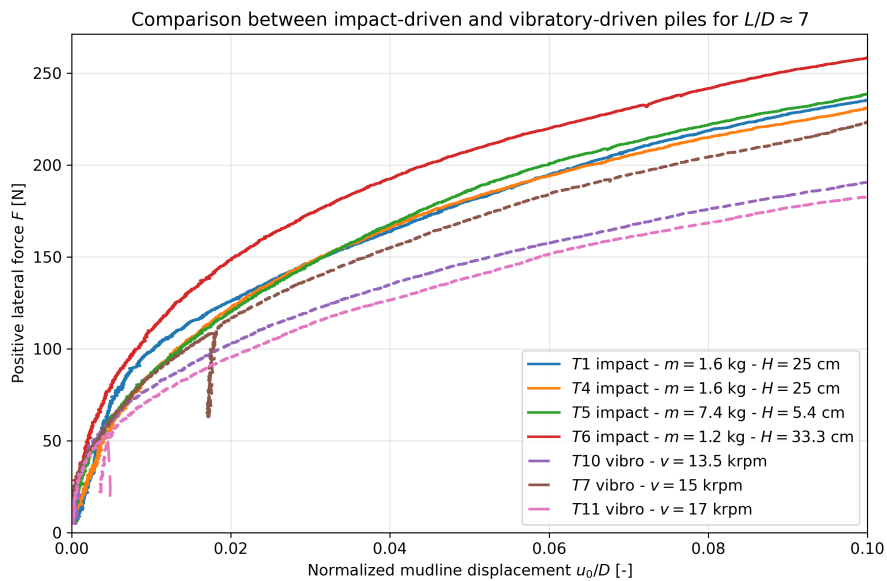


Figure 74: Comparison between impact-driven and vibratory-driven piles installed with $L/D = 7$.

The average response indicators for the piles installed close to $L/D = 7$ are summarised in Table 11. The mean values provide a compact summary of the tendency observed on the force–displacement curves. The impact-driven piles show higher average values of $F_{0.05D}$, $F_{0.10D}$, and K_{initial} than the vibratory-driven piles. This comparison is used as a global indicator of the difference between the two installation methods, while the detailed interpretation of the possible mechanisms is left for the discussion section.

Table 11: Mean response indicators for impact-driven and vibratory-driven piles with $L/D = 7$.

Method	Tests considered	Mean $F_{0.05D}$ [N]	Mean $F_{0.10D}$ [N]	Mean K_{initial} [N/cm]
Impact driving	T1, T4, T5, T6	189.09	240.80	1589.40
Vibratory driving	T10, T7, T11	152.18	198.83	1317.16

Given the limited number of tests, these average values should be interpreted as comparative indicators rather than as statistically representative mean responses.

6.4.2 Moment–rotation response

The comparison between impact-driven and vibratory-driven piles was also performed in terms of moment–rotation response. The moment at the mudline was computed from the measured lateral force as $M = Fh$, where h is the height of load application above the mudline. Figure 75 presents the moment–rotation curves for the piles installed close to $L/D = 7$.

The curves show that the impact-driven piles generally mobilise higher moments than the vibratory-driven piles for a given rotation. Among the impact-driven piles, $T6$ reaches the highest moment over most of the rotation range, while $T1$, $T4$, and $T5$ remain relatively close. The vibratory-driven piles $T10$, $T7$, and $T11$ show lower moment levels, although their curves follow a similar overall shape. This indicates that the difference between the two installation methods observed in the force–displacement response is also visible when the response is expressed in terms of rotation.

The comparison is shown up to $\theta = 1.5^\circ$, which was retained as a common rotation-based reference level. The objective here is not to identify a sudden failure point, but to compare the mobilised moment at a consistent rotation level for the different tests.

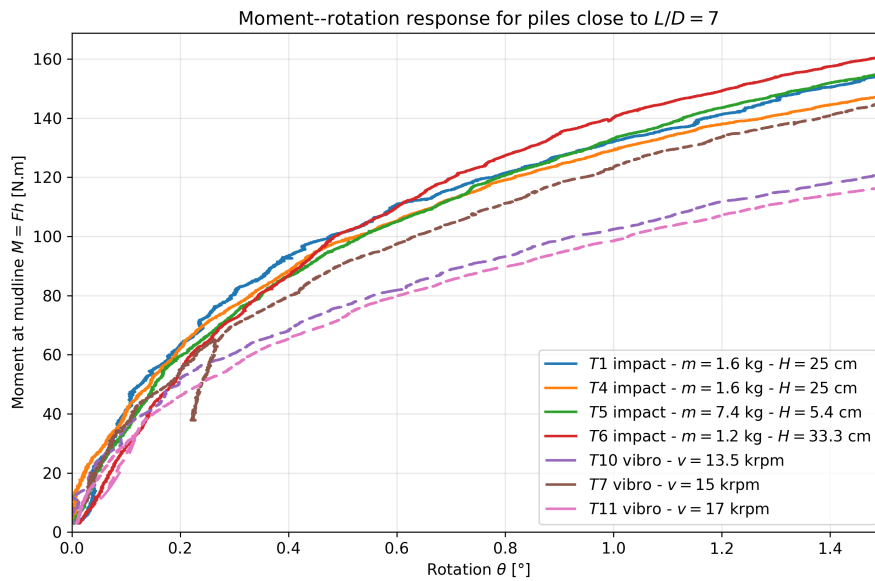


Figure 75: Moment–rotation response for impact-driven and vibratory-driven piles installed close to $L/D = 7$.

6.4.3 Evolution of the apparent rotation point

The apparent depth of the rotation point was used to complement the force–displacement and moment–rotation comparisons. The apparent rotation point depth d was normalised by the embedded length L , so that the results are expressed as d/L . This indicator provides information on the kinematic response of the pile during lateral loading. It should be recalled that d is an apparent kinematic quantity derived from two displacement measurements and from the rigid-body assumption. It should therefore not be interpreted as a directly observed physical hinge point in the soil.

Figure 76 shows the evolution of d/L as a function of the pile rotation for the tests installed close to $L/D = 7$. Most curves remain within a relatively narrow range, typically between approximately $d/L = 0.55$ and $d/L = 0.70$ after the initial part of loading. Test $T6$ is distinct at the beginning of the loading, with a shallower apparent rotation point, but its value progressively increases with rotation and approaches the range observed for the other tests. This indicates that the apparent rotation point is not perfectly constant during loading, especially at small rotations.

No clear separation between impact-driven and vibratory-driven piles can be identified from this graph alone. The curves tend to evolve towards comparable values of d/L , suggesting that the global rotation mechanism remains broadly similar for the different installation methods, even though the mobilised force and moment levels differ.

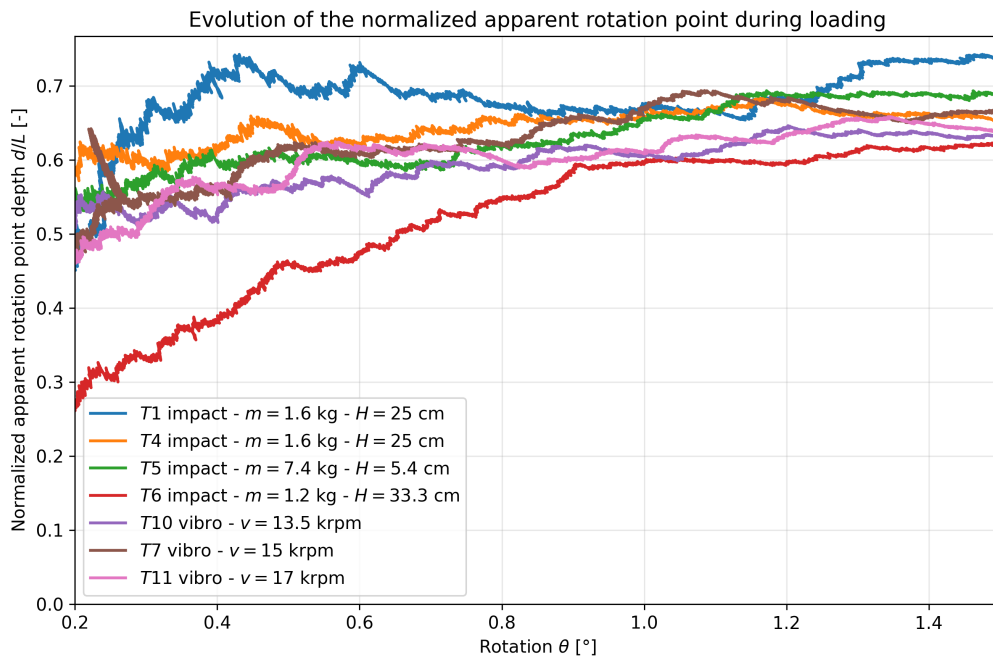


Figure 76: Evolution of the normalised apparent rotation point depth d/L during lateral loading for piles installed with $L/D = 7$.

The mean value of d/L during loading is compared for all selected tests in Figure 77. The vertical error bars correspond to one standard deviation of d/L over the analysed rotation interval and therefore represent the variability of the apparent rotation point during loading, rather than a direct measurement uncertainty.

The mean values remain mostly between approximately 0.5 and 0.8. The impact-driven and vibratory-driven tests do not form two clearly separated groups. Some individual tests show higher or lower values, but the overall range remains comparable for both installation methods. This suggests that the differences observed in lateral resistance are not associated with a completely different global rotation mechanism. Instead, the apparent rotation depth remains of the same order of magnitude for both installation methods.

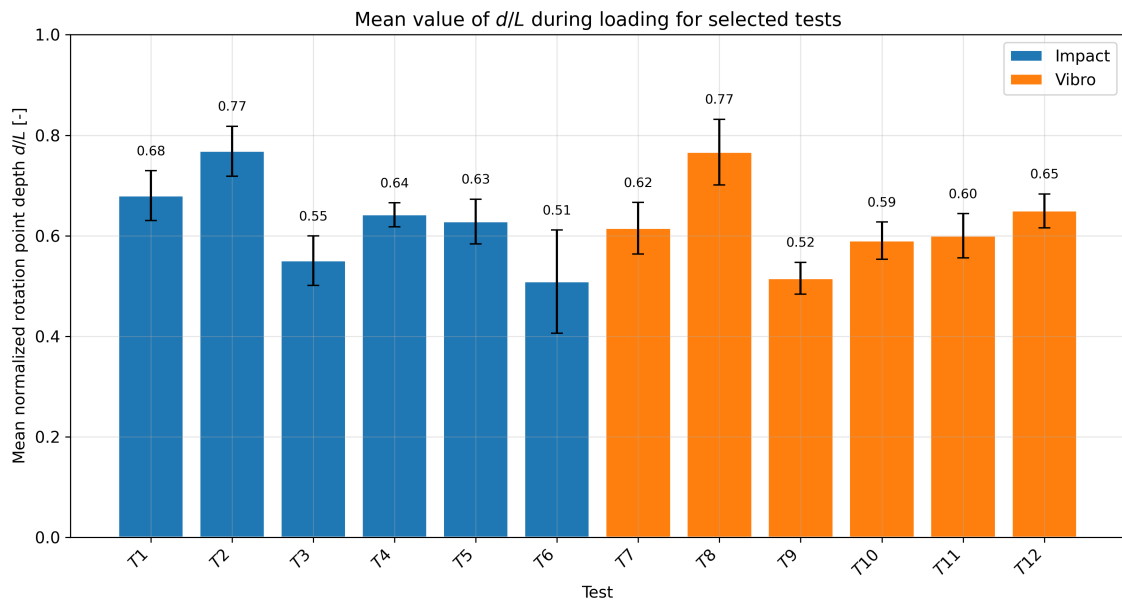


Figure 77: Mean value of the normalised apparent rotation point depth d/L during loading.

7 Discussion and interpretation of the results

The objective of this section is to interpret the main trends observed in the experimental results and to compare them with the behaviour that could be expected from soil–pile interaction mechanisms and from previous experimental studies. The discussion is intentionally focused on the results that are the most informative for the present work.

The present study is part of a progressive experimental research line developed at UCLouvain. Hamoir and Hoet first initiated the laboratory investigation of pile installation effects on the lateral behaviour of model piles in sand. Deckers and Defer then focused more specifically on impact-driven piles and on the influence of impact-driving parameters. Shahrour later extended the experimental approach towards vibratory driving by developing and testing a model-scale vibro-driving system [35, 36, 6]. The present work builds on these previous contributions by combining both installation routes in a single experimental programme and by comparing their post-installation lateral response.

7.1 Role of the embedment ratio and initial sand state

The influence of the embedment ratio is clearly observed for the impact-driven piles. Increasing L/D leads to a higher lateral resistance and a higher initial stiffness. This trend is consistent with the expected behaviour of laterally loaded piles in sand, since a larger embedded length allows a larger volume of soil to be mobilised during lateral loading.

This observation is also consistent with the previous work of Deckers and Defer, who showed that the embedment ratio had a strong influence on the lateral response of impact-driven model piles in sand [36]. In the present tests, this expected effect is clearly recovered for the impact-driven series.

For the vibratory-driven piles, the trend is less regular. The increase from $L/D = 5$ to $L/D = 7$ leads to a higher response, but the pile installed to $L/D = 8.5$, $T9$, does not mobilise a higher resistance than $T7$. The CPT profiles measured before installation indicate that the sand at the $T9$ location was less resistant than at the $T7$ location. This suggests that the effect of embedment ratio can be partly masked by local variations in the initial sand state. Therefore, L/D remains a major parameter, but its influence cannot be interpreted independently from the local soil conditions in the present experimental campaign.

7.2 Effect of the impact-driving configuration

The comparison between $T4$, $T5$, and $T6$ shows that different impact-driving configurations can lead to different post-installation lateral responses, even when the nominal impact energy per blow is kept approximately constant. This indicates that the nominal energy $E = mgH$ alone is not sufficient to fully describe the installation process.

This result is consistent with the conclusions of Deckers and Defer, who showed that the lateral response of impact-driven model piles can be affected by the installation parameters, although the embedment ratio remained one of the dominant factors [36]. The present tests extend this observation by showing that, even for a similar nominal energy per blow, different combinations of falling mass and drop height may lead to different post-installation responses.

This result is physically reasonable because two impacts with the same nominal energy may differ in terms of peak force, duration of impact, impulse, stress-wave transmission, and number of blows required to reach the target embedment depth. The soil around the pile is therefore not subjected to the same installation history. In the present tests, $T6$, corresponding to the smallest falling mass and largest drop height, gives the highest lateral resistance and initial stiffness. However, since only one test was performed for each impact configuration, this trend should be interpreted cautiously.

7.3 Effect of the vibratory-driving frequency

The vibratory-driving tests build directly on the previous work of Shahrour, who developed and tested a model-scale vibro-driving system at UCLouvain [6]. The present study extends this work by using vibratory installation as part of a broader comparison with impact-driven piles and by analysing the subsequent lateral response.

The vibratory-driving frequency clearly affects the installation phase. The lowest motor speed, used for $T12$, led to refusal before the target embedment ratio was reached, whereas the higher motor speeds allowed the piles to reach $L/D = 7$. This shows that the motor speed influences driveability.

However, the post-installation lateral response does not show a clear monotonic trend with motor speed. The piles $T10$, $T7$, and $T11$, all installed to $L/D = 7$, show responses of the same order of magnitude. Although $T7$ gives the highest lateral resistance and initial stiffness, the CPT profiles do not indicate a clearly more resistant initial sand state at this location. The reason for the slightly higher response of $T7$ therefore remains uncertain.

This suggests that the frequency alone is not sufficient to describe the effect of vibratory installation on the subsequent lateral response. Other factors, such as vibration duration, actual pile motion, soil rearrangement, contact conditions along the shaft, and local experimental variability, may also influence the final pile–soil state.

7.4 Comparison between impact-driven and vibratory-driven piles

For piles installed close to $L/D = 7$, the impact-driven piles generally mobilise higher lateral forces, higher moments, and higher mean response indicators than the vibratory-driven piles. In the present experimental setup, impact driving therefore appears to lead to a stiffer and stronger post-installation lateral response than vibratory driving.

This observation is consistent with previous studies showing that the installation method can influence the subsequent lateral response of monopiles in sand. Gavin et al. investigated the influence of installation method on the monotonic loading response of monopiles and showed that the post-installation behaviour cannot be interpreted independently from the installation process [29]. Peccin da Silva et al. also compared different installation conditions and showed that vibratory installation can lead to lateral responses that are comparable to, but in some cases softer than, those obtained after impact driving, depending on the installation parameters [34].

Compared with the previous UCLouvain studies, this direct comparison between impact-driven and vibratory-driven piles represents an additional contribution of the present work. Hamoir and Hoet, Deckers and Defer, and Shahrouh each contributed to the development of the experimental approach, but they did not provide a direct impact-versus-vibro comparison under the same lateral loading framework [35, 36, 6]. The present results therefore add new experimental information by comparing both installation methods within a common testing procedure.

The difference observed between impact-driven and vibratory-driven piles is mechanically plausible because the two installation methods impose different loading histories on the pile–soil system. Impact driving applies repeated transient blows, whereas vibratory driving imposes a continuous cyclic excitation. These different installation histories may modify the density, stress state and contact conditions around the pile, which are then mobilised during lateral loading.

However, the comparison should remain cautious. The literature does not show that impact driving always leads to a higher lateral resistance, and the response after installation depends on the soil state, the installation parameters and the achieved embedment ratio. The main conclusion is therefore limited to the present experimental conditions: for piles installed close to $L/D = 7$, the impact-driven piles tended to mobilise a higher lateral resistance and initial stiffness than the vibratory-driven piles.

7.5 Kinematic interpretation

The moment–rotation curves confirm the trend observed in the force–displacement response: for a given rotation, the impact-driven piles generally mobilise higher moments than the vibratory-driven piles. This supports the conclusion that the installation method affects the level of resistance mobilised during lateral loading.

In contrast, the apparent rotation point does not clearly separate the two installation

methods. The values of d/L remain of the same order of magnitude for both impact-driven and vibratory-driven piles. This suggests that the installation method mainly affects the stiffness and resistance level, rather than producing a completely different global rotation mechanism. It should nevertheless be recalled that d is an apparent kinematic quantity derived from two displacement measurements and from a rigid-body assumption; it should not be interpreted as a directly observed physical hinge point in the soil.

7.6 Summary of the interpretation

Overall, the results show that the embedment ratio is the dominant parameter controlling the lateral response of the model piles, but that local variations in the initial sand state can significantly affect the measured response. The impact-driving configuration also influences the post-installation behaviour, even when the nominal impact energy per blow is kept constant. For vibratory driving, the motor speed clearly affects installation success and duration, but no clear monotonic influence on the post-installation lateral response can be isolated.

The main comparison between installation methods indicates that, in the present tests, impact-driven piles installed close to $L/D = 7$ tend to mobilise higher lateral resistance and stiffness than vibratory-driven piles. This tendency is consistent with previous studies showing that installation method can affect post-installation lateral behaviour, although the effect of vibratory driving may vary depending on installation parameters and soil conditions.

8 Conclusion and perspectives

This thesis investigated the influence of pile installation method on the post-installation lateral behaviour of model monopiles in sand. The work focused on the comparison between impact-driven and vibratory-driven piles, with particular attention to the force–displacement response, initial stiffness, moment–rotation behaviour and pile kinematics during monotonic lateral loading.

A first important part of the work concerned the preparation of the sand mass. The preliminary pluviation tests showed that the initial configuration of the large tank led to significant spatial variability. The optimisation of the procedure, in particular the trimming of the protruding inner fins, improved the repeatability and apparent homogeneity of the prepared sand mass. This step was essential in order to make the subsequent comparison between pile installation methods more meaningful.

The experimental results confirmed that the embedment ratio L/D is a major parameter controlling the lateral response of the piles. For both installation methods, increasing the embedded length generally led to higher lateral resistance and higher initial stiffness. This confirms the importance of the relative embedded length in the behaviour of laterally loaded monopiles.

The impact-driving tests also showed that the nominal impact energy per blow is not sufficient on its own to describe the installation process. Different combinations of falling mass and drop height, even when associated with a similar nominal energy, produced different installation histories and different post-installation lateral responses.

This suggests that the detailed impact-driving configuration may influence the soil state around the pile and, consequently, the subsequent lateral behaviour.

For vibratory driving, the motor speed clearly influenced the installation phase. The lowest tested speed led to refusal before the target embedment was reached, while higher speeds allowed the pile to reach the prescribed depth. However, the post-installation lateral response did not show a clear monotonic trend with vibration frequency. The tests performed at similar embedment ratios led to responses of the same order of magnitude, indicating that frequency alone cannot fully explain the final behaviour of the pile–soil system.

The comparison between impact-driven and vibratory-driven piles showed that, for piles installed close to $L/D = 7$, the impact-driven piles generally mobilised higher forces and higher moments than the vibratory-driven piles. In the present experimental conditions, impact driving therefore appeared to lead to a stiffer and stronger post-installation response. Nevertheless, the differences remained within the same order of magnitude and must be interpreted with caution because of the limited number of tests and the unavoidable variability associated with reduced-scale geotechnical experiments.

Overall, this work confirms that the installation method should not be considered only as a construction stage. The way in which the pile is installed can modify the surrounding soil state and influence the subsequent lateral response of the foundation. The results obtained in this thesis provide experimental evidence of this effect at model scale and contribute to the ongoing comparison between impact driving and vibratory driving for offshore monopile foundations.

However, the conclusions drawn from this campaign must remain cautious. Each configuration was tested only a limited number of times, and some trends could not be isolated with sufficient statistical confidence. In order to continue this research, it would therefore be particularly valuable to carry out a larger and more systematic experimental campaign. Repeating each test configuration several times would make it possible to quantify the experimental scatter, distinguish true physical trends from local variability, and draw more robust conclusions regarding the influence of installation method, impact-driving configuration and vibratory-driving frequency.

Future work could also include additional measurements during installation, such as the actual pile acceleration, pile displacement amplitude, vibration transmission and possible changes in the near-pile soil state. Such measurements would help to better link the installation process itself with the lateral response measured afterwards. Extending the test matrix to more embedment ratios, vibration frequencies and impact configurations would also provide a stronger basis for interpreting the mechanisms governing the post-installation behaviour of monopiles in sand.

The present study should therefore be seen as a first comparative experimental step. It highlights relevant trends and methodological challenges, but also shows the need for a larger database of controlled tests before drawing definitive conclusions on the relative effects of impact and vibratory driving on the mechanical behaviour of offshore monopile foundations.

References

- [1] Our World in Data. Greenhouse Gas Emissions by Sector (World) – chart and data. Our World in Data, 2025. URL <https://ourworldindata.org/co2-and-greenhouse-gas-emissions>.
- [2] Our World in Data. OWID Energy Dataset (Codebook and Files). Our World in Data, 2025. URL <https://github.com/owid/energy-data>.
- [3] International Energy Agency. Wind. IEA Energy System, 2025. URL <https://www.iea.org/energy-system/renewables/wind>.
- [4] International Renewable Energy Agency. Ocean Energy. IRENA Technology overview, 2025. URL <https://www.irena.org/Energy-Transition/Technology/Ocean-energy>.
- [5] Adrian Nicolas Pozo Ayala, Eduardo Gabriel Muñoz Palomeque, and Edy Leonardo Ayala Cruz. Direct Speed Control of a 9 MW DFIG Wind Turbine. *Revista Técnica Energía*, 18(1):1–8, 2021. doi: 10.37116/revistaenergia.v18.n1.2021.435. URL <https://revistaenergia.cenace.gob.ec/index.php/cenace/article/download/435/526>.
- [6] Mohamed Shahrour. Influence of Vibratory Pile Driving on Offshore Wind Turbine Mechanical Behaviour. Master thesis, Ecole polytechnique de Louvain, Université catholique de Louvain, 2024. URL <https://thesis.dial.uclouvain.be/entities/masterthesis/ff2e838a-e94d-43a7-b490-df2a611ef5e6>.
- [7] Offshore Wind Biz. IHC Hammers Its Way in China. Offshore Wind Biz, 2014. URL <https://www.offshorewind.biz/2014/11/26/ihc-hammers-its-way-in-china/>.
- [8] CAPE Holland. Vibro driving: the next chapter in offshore wind, June 2020. URL <https://capeholland.com/news/vibro-driving-the-next-chapter-in-offshore-wind/>.
- [9] Fuquan Chen, Liyang Liu, Fengwen Lai, Kenneth Gavin, Kevin N. Flynn, and Yida Li. Numerical analyses of energy balance and installation mechanisms of large-diameter tapered monopiles by impact driving. *Ocean Engineering*, 266:113017, 2022. doi: 10.1016/j.oceaneng.2022.113017. URL <https://doi.org/10.1016/j.oceaneng.2022.113017>.
- [10] Fabian Remspecher, Viet Hung Le, Frank Rackwitz, Volker Herwig, and Benjamin Matlock. Vibratory Driven Installation of Monopiles – An Experimental Investigation of the Soil-Pile Interaction. In *Proceedings of the 1st Vietnam Symposium on Advances in Offshore Engineering*, volume 18 of *Lecture Notes in Civil Engineering*, pages 171–176. Springer, Singapore, 2019. doi: 10.1007/978-981-13-2306-5_22. URL https://doi.org/10.1007/978-981-13-2306-5_22.

- [11] K. Rainer Massarsch, Carl Wersäll, and Bengt H. Fellenius. Vibratory driving of piles and sheet piles – state of practice. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering*, 175(1):31–48, 2021. doi: 10.1680/jgeen.20.00127. URL <https://doi.org/10.1680/jgeen.20.00127>.
- [12] IPCC. Climate Change 2023: Synthesis Report. Contribution of Working Groups I, II and III to the Sixth Assessment Report of the Intergovernmental Panel on Climate Change. Technical report, Intergovernmental Panel on Climate Change, Geneva, Switzerland, 2023. URL <https://www.ipcc.ch/report/ar6/syr/>.
- [13] UNFCCC. The Paris Agreement. United Nations Framework Convention on Climate Change, 2015. URL <https://unfccc.int/process-and-meetings/the-paris-agreement/the-paris-agreement>.
- [14] European Union. Regulation (EU) 2021/1119 of the European Parliament and of the Council of 30 June 2021 Establishing the Framework for Achieving Climate Neutrality (European Climate Law). Official Journal of the European Union, 2021. URL <https://eur-lex.europa.eu/eli/reg/2021/1119/oj>.
- [15] International Energy Agency. Global Energy Review 2025. Technical report, International Energy Agency, 2025. URL <https://www.iea.org/reports/global-energy-review-2025>.
- [16] International Renewable Energy Agency. Renewable Capacity Statistics 2024. Technical report, IRENA, 2024. URL https://www.irena.org/-/media/Files/IRENA/Agency/Publication/2024/Mar/IRENA_RE_Capacity_Statistics_2024.pdf.
- [17] International Renewable Energy Agency. Renewable Power Generation Costs in 2023. Technical report, IRENA, 2024. URL https://www.irena.org/-/media/Files/IRENA/Agency/Publication/2024/Sep/IRENA_Renewable_power_generation_costs_in_2023.pdf.
- [18] European Commission. An EU Strategy to Harness the Potential of Offshore Renewable Energy for a Climate Neutral Future. Communication from the Commission, COM(2020) 741 final, 2020. URL <https://eur-lex.europa.eu/legal-content/EN/TXT/PDF/?uri=CELEX:52020DC0741>.
- [19] International Renewable Energy Agency. Innovation Outlook: Ocean Energy Technologies. Technical report, IRENA, 2020. URL https://www.irena.org/-/media/Files/IRENA/Agency/Publication/2020/Dec/IRENA_Innovation_Outlook_Ocean_Energy_2020.pdf.
- [20] International Renewable Energy Agency (IRENA) and ETSAP. Wind Power. Technical report, IRENA, 2016. URL https://www.irena.org/-/media/Files/IRENA/Agency/Publication/2016/IRENA-ETSAP_Tech_Brief_Wind_Power_E07.pdf.
- [21] Guide to an Offshore Wind Farm. Guide to a Fixed Offshore Wind Farm: Update 2025. Technical report, Crown Estate Scotland and partners, 2025. URL <https://guidetoanoffshorewindfarm.com/wp-content/uploads/2025/06/BVGA-16464-Fixed-Guide-rF.pdf>.

- [22] Xiaoni Wu, Yu Hu, Ye Li, Jian Yang, Lei Duan, Tongguang Wang, Thomas Adcock, Zhiyu Jiang, Zhen Gao, Zhiliang Lin, Alistair Borthwick, and Shijun Liao. Foundations of Offshore Wind Turbines: A Review. *Renewable and Sustainable Energy Reviews*, 104:379–393, 2019. doi: 10.1016/j.rser.2019.01.012. URL <https://doi.org/10.1016/j.rser.2019.01.012>.
- [23] International Renewable Energy Agency. Floating Offshore Wind Outlook. Technical report, IRENA, 2024. URL https://www.irena.org/-/media/Files/IRENA/Agency/Publication/2024/Jul/IRENA_G7_Floating_offshore_wind_outlook_2024.pdf.
- [24] Laszlo Arany, Subhamoy Bhattacharya, John Macdonald, and S. J. Hogan. Design of monopiles for offshore wind turbines in 10 steps. *Soil Dynamics and Earthquake Engineering*, 92:126–152, 2017. doi: 10.1016/j.soildyn.2016.09.024. URL <https://doi.org/10.1016/j.soildyn.2016.09.024>.
- [25] Iona A. Richards. *Monopile Foundations under Complex Cyclic Lateral Loading*. PhD thesis, University of Oxford, Oxford, United Kingdom, 2019. URL <https://ora.ox.ac.uk/objects/uuid:bc3fc931-1c15-4abf-a367-42b5b4971d2a>.
- [26] Harvey J. Burd, David M. G. Taborda, Lidija Zdravković, Christelle N. Abadie, Byron W. Byrne, Guy T. Houlsby, Kenneth G. Gavin, David J. P. Igoe, Richard J. Jardine, Christopher M. Martin, Ross A. McAdam, Antonio M. G. Pedro, and David M. Potts. PISA Design Model for Monopiles for Offshore Wind Turbines: Application to a Marine Sand. *Géotechnique*, 70(11):1048–1066, 2020. doi: 10.1680/jgeot.18.P.277. URL <https://doi.org/10.1680/jgeot.18.P.277>.
- [27] Zhiyu Jiang. Installation of Offshore Wind Turbines: A Technical Review. *Renewable and Sustainable Energy Reviews*, 139:110576, 2021. doi: 10.1016/j.rser.2020.110576. URL <https://doi.org/10.1016/j.rser.2020.110576>.
- [28] Ka Lok Chan, Susana Lopez-Querol, and Pedro Martin-Moreta. Numerical Analysis of Jacked and Impact-Driven Pile Installation Procedures in Offshore Wind Turbine Foundations. *Geotechnics*, 5(1):11, 2025. doi: 10.3390/geotechnics5010011. URL <https://doi.org/10.3390/geotechnics5010011>.
- [29] Kenneth G. Gavin, S. D. Blok, and G. R. Eiksund. The Impact of Installation Method on Monotonic Loading of Monopiles in Sand. In Z. Westgate, editor, *Proceedings of the Fourth International Symposium on Frontiers in Offshore Geotechnics*, pages 849–857, 2020. URL https://pure.tudelft.nl/ws/portalfiles/portal/86976132/Blok_et_al._ISFOG_2020.pdf. Article 3494.
- [30] Viet Hung Le and Frank Rackwitz. Prediction of Monopile Driveability under Vibratory Driving Using Wave Equation Analysis. In *Proceedings of the 5th International Symposium on Frontiers in Offshore Geotechnics (ISFOG 2025)*, Nantes, France, 2025. International Society for Soil Mechanics and Geotechnical Engineering. ISBN 978-2-85782-758-0. doi: 10.53243/ISFOG2025-398. URL <https://doi.org/10.53243/ISFOG2025-398>.

- [31] T. Byrne, K. Gavin, L. J. Prendergast, P. Cachim, P. Doherty, and S. Chenicheri Pulukul. Performance of CPT-based methods to assess monopile driveability in North Sea sands. *Ocean Engineering*, 166:76–91, 2018. doi: 10.1016/j.oceaneng.2018.08.010. URL <https://doi.org/10.1016/j.oceaneng.2018.08.010>.
- [32] Britta Bienen, Puorya Kazemi Esfeh, M. Fraser Bransby, and Patrick Staubach. Numerical study of vibro-driven pile installation in sand. In C. N. Abadie, Z. Li, M. Blanc, and L. Thorel, editors, *Proceedings of the 5th International Symposium on Frontiers in Offshore Geotechnics (ISFOG 2025)*, pages 1041–1046, Nantes, France, 2025. International Society for Soil Mechanics and Geotechnical Engineering. ISBN 978-2-85782-758-0. doi: 10.53243/ISFOG2025-57. URL <https://doi.org/10.53243/ISFOG2025-57>.
- [33] Jakob Vogelsang, Gerhard Huber, and Theodoros Triantafyllidis. On Soil Deformation and Stress Redistribution Around Pressed-in and Vibrated Displacement Pile Tips. In Theodoros Triantafyllidis, editor, *Holistic Simulation of Geotechnical Installation Processes*, volume 77 of *Lecture Notes in Applied and Computational Mechanics*, pages 41–57. Springer, Cham, 2015. doi: 10.1007/978-3-319-18170-7_3. URL https://doi.org/10.1007/978-3-319-18170-7_3.
- [34] Anderson Peccin da Silva, Mark Post, Ahmed S. Elkadi, Elias Kementzetzidis, and Federico Pisanò. A laboratory study of the effect of installation parameters on the lateral behaviour of monopiles in sand. In *Symposium on Energy Geotechnics 2023*, pages 1–2, 2023. doi: 10.59490/seg.2023.611. URL <https://doi.org/10.59490/seg.2023.611>.
- [35] Garance Hamoir and Nathan Hoet. Experimental Investigation of the Impact of the Installation Method on the Lateral Behavior of Piles. Master thesis, Ecole polytechnique de Louvain, Université catholique de Louvain, 2022. URL <http://hdl.handle.net/2078.1/thesis:37956>.
- [36] Maxence Deckers and Cédric Defer. Laboratory Investigation of the Effect of Installation Parameters on the Lateral Response of Offshore Monopiles. Master’s thesis, École polytechnique de Louvain, Université catholique de Louvain, Louvain-la-Neuve, Belgium, 2023. URL <http://hdl.handle.net/2078.1/thesis:43341>. Supervisor: Hadrien Rattez.
- [37] J. Vogelsang, H. Zachert, G. Huber, and Th. Triantafyllidis. Effects of Soil Deposition on the Initial Stress State in Model Tests: Experimental Results and FE Simulation. In Th. Triantafyllidis, editor, *Holistic Simulation of Geotechnical Installation Processes: Numerical and Physical Modelling*, volume 77 of *Lecture Notes in Applied and Computational Mechanics*, pages 1–19. Springer, Cham, 2015. doi: 10.1007/978-3-319-18170-7_1. URL https://doi.org/10.1007/978-3-319-18170-7_1.
- [38] Pooya Dastpak, Saeed Abrishami, Masoud Rezazadeh Anbarani, and Amirreza Dastpak. Effect of Perforated Plates on the Relative Density of Uniformly Graded Reconstituted Sands Using Air Pluviation Method. *Transportation Infrastructure Geotechnology*, 8(4):569–589, 2021. doi: 10.1007/s40515-021-00150-1. URL <https://doi.org/10.1007/s40515-021-00150-1>.

- [39] Royal Eijkelkamp. *Penetrologger: Operating Instructions*. Royal Eijkelkamp, 2022. URL <https://www.royaleijkelkamp.com/media/nrwjyah3/m-0615sae-penetrologger.pdf>. Recommended settings.
- [40] L. Krogh, S. Quinteros, H. K. Engin, and T. Lunne. Revisiting Interpretation of Relative Density from Shallow Depth CPTs in Sand. *Canadian Geotechnical Journal*, 59(6):808–826, 2021. doi: 10.1139/cgj-2021-0200. URL <https://doi.org/10.1139/cgj-2021-0200>.
- [41] maxon. *ECX SPEED 13 L, Diameter 13 mm, Brushless BLDC Motor: Catalog Page*, 2025. URL https://www.maxongroup.com/medias/sys_master/root/9399221420062/Cataloge-Page-EN-206.pdf.
- [42] maxon motor. *ESCON 50/5 Hardware Reference*, 2018. URL https://www.maxongroup.com/medias/sys_master/root/8834332262430/409510-ESCON-50-5-Hardware-Reference-En.pdf.
- [43] Bosch Rexroth AG. *Rexroth IndraDyn S MSK Synchronous Motors: Project Planning Manual*. Bosch Rexroth AG, 2014. URL https://www.lsa-control.com/pub/media//attach/pdf/Motors/Servo_Motors/IndraDyn_S_MSK_Project_Planning.pdf.
- [44] Ju-Hyung Lee, Tae-Young Kwak, Youn-Ju Jeong, Joonsang Park, and Jae-Hyun Kim. A Study on the Lateral Load Capacity of a Novel Hybrid Monopile via a Centrifuge Model Test. *Energies*, 16(21):7234, 2023. doi: 10.3390/en16217234. URL <https://doi.org/10.3390/en16217234>.
- [45] Huan Wang, Rob Zwaan, Anderson Peccin da Silva, Amin Askarinejad, and Federico Pisanò. Physical Modelling of Cyclically Loaded Monopiles in Sand: The MIDAS Centrifuge Testing Programme. In *Proceedings of the 5th International Symposium on Frontiers in Offshore Geotechnics (ISFOG 2025)*, Nantes, France, 2025. International Society for Soil Mechanics and Geotechnical Engineering. doi: 10.53243/ISFOG2025-440. URL <https://doi.org/10.53243/ISFOG2025-440>.
- [46] M. Blanc, Z. S. Li, and L. Thorel. Centrifuge Test on Instrumented FBGs Monopile Foundation in Sand. In *Proceedings of the XVII European Conference on Soil Mechanics and Geotechnical Engineering*, 2019. doi: 10.32075/17ECSMGE-2019-0263. URL <https://doi.org/10.32075/17ECSMGE-2019-0263>.
- [47] Mahdy Khari, Khairul Anuar Kassim, and Azlan Adnan. An Experimental Study on Pile Spacing Effects under Lateral Loading in Sand. *The Scientific World Journal*, 2013:734292, 2013. doi: 10.1155/2013/734292. URL <https://doi.org/10.1155/2013/734292>.
- [48] Sachin Jindal, Ulvi Rahmanli, Muhammad Aleem, Liang Cui, Saleh Jalbi, and Subhamoy Bhattacharya. Identification of Soil Damping for Stiff Piles Based on Experiments: Methodology and Results. *Ocean Engineering*, 301:117124, 2024. doi: 10.1016/j.oceaneng.2024.117124. URL <https://doi.org/10.1016/j.oceaneng.2024.117124>.

-
- [49] Dennis Frick and Martin Achmus. An Experimental Study on the Parameters Affecting the Cyclic Lateral Response of Monopiles for Offshore Wind Turbines in Sand. *Soils and Foundations*, 60(6):1570–1587, 2020. doi: 10.1016/j.sandf.2020.10.004. URL <https://doi.org/10.1016/j.sandf.2020.10.004>.

A CPT profiles

This appendix presents the CPT profiles measured before the installation of the impact-driven and vibratory-driven piles. These profiles are used as complementary information to document the local sand state before pile installation.

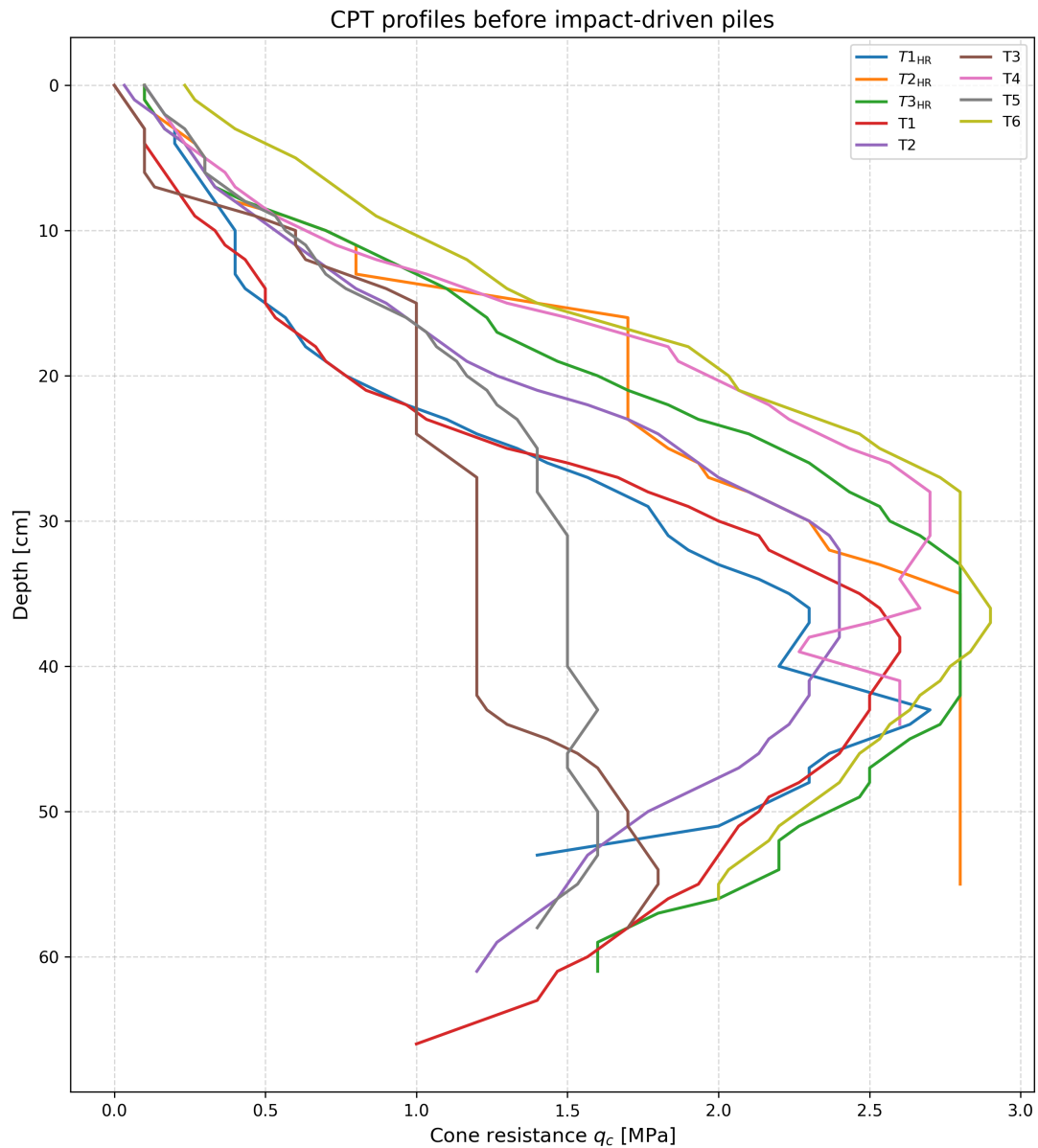


Figure 78: CPT profiles before impact-driven pile installation.

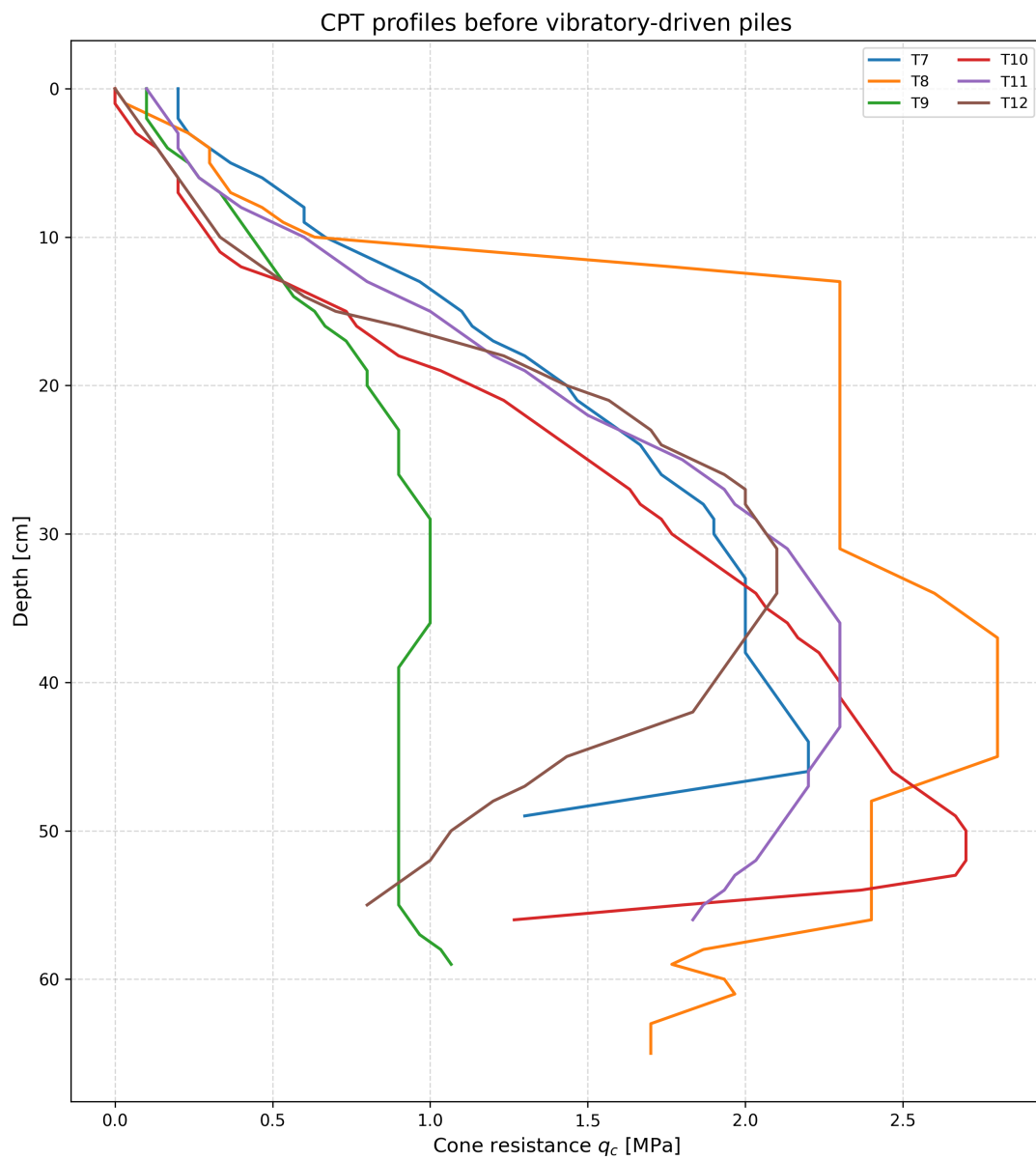
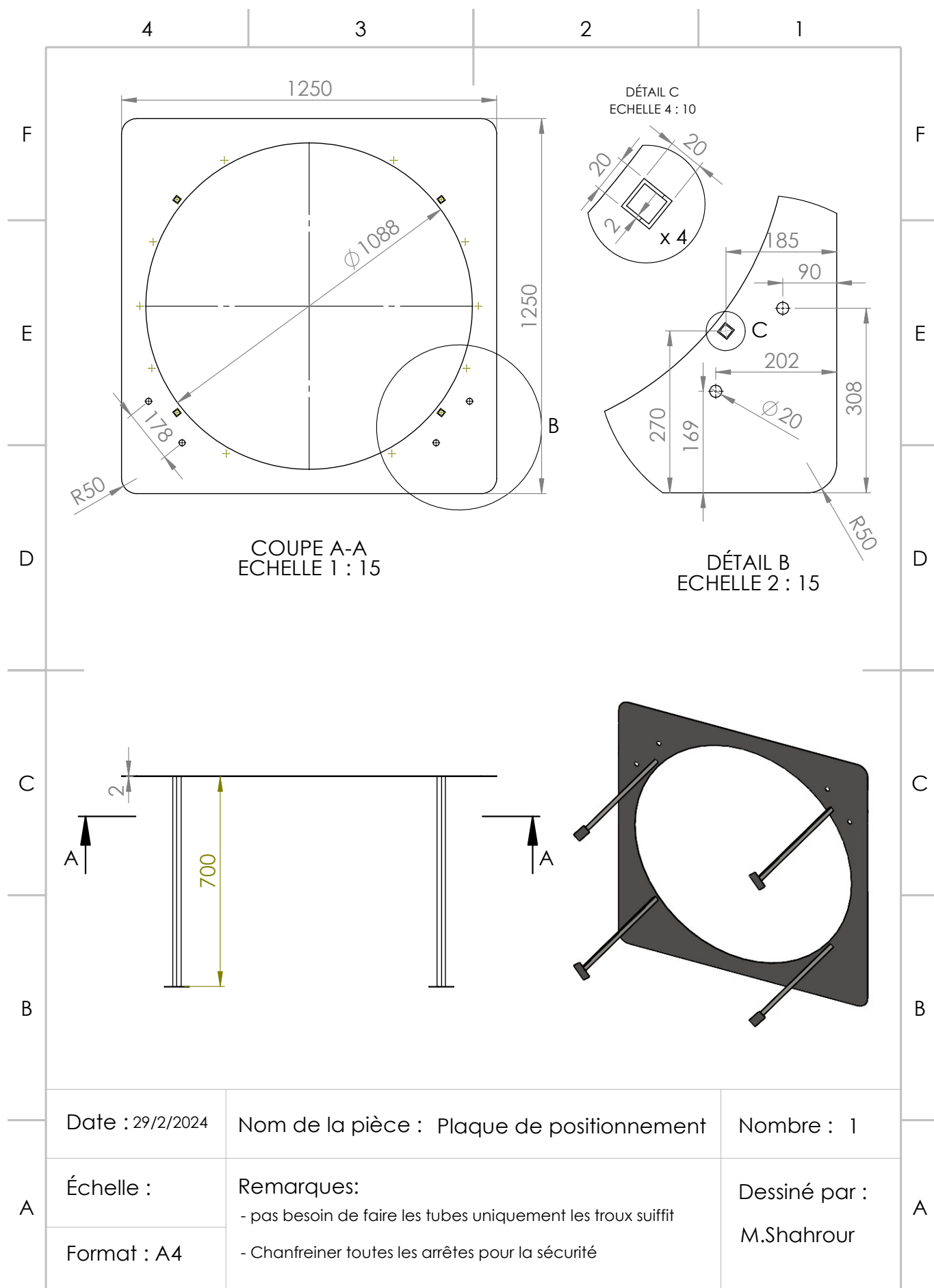
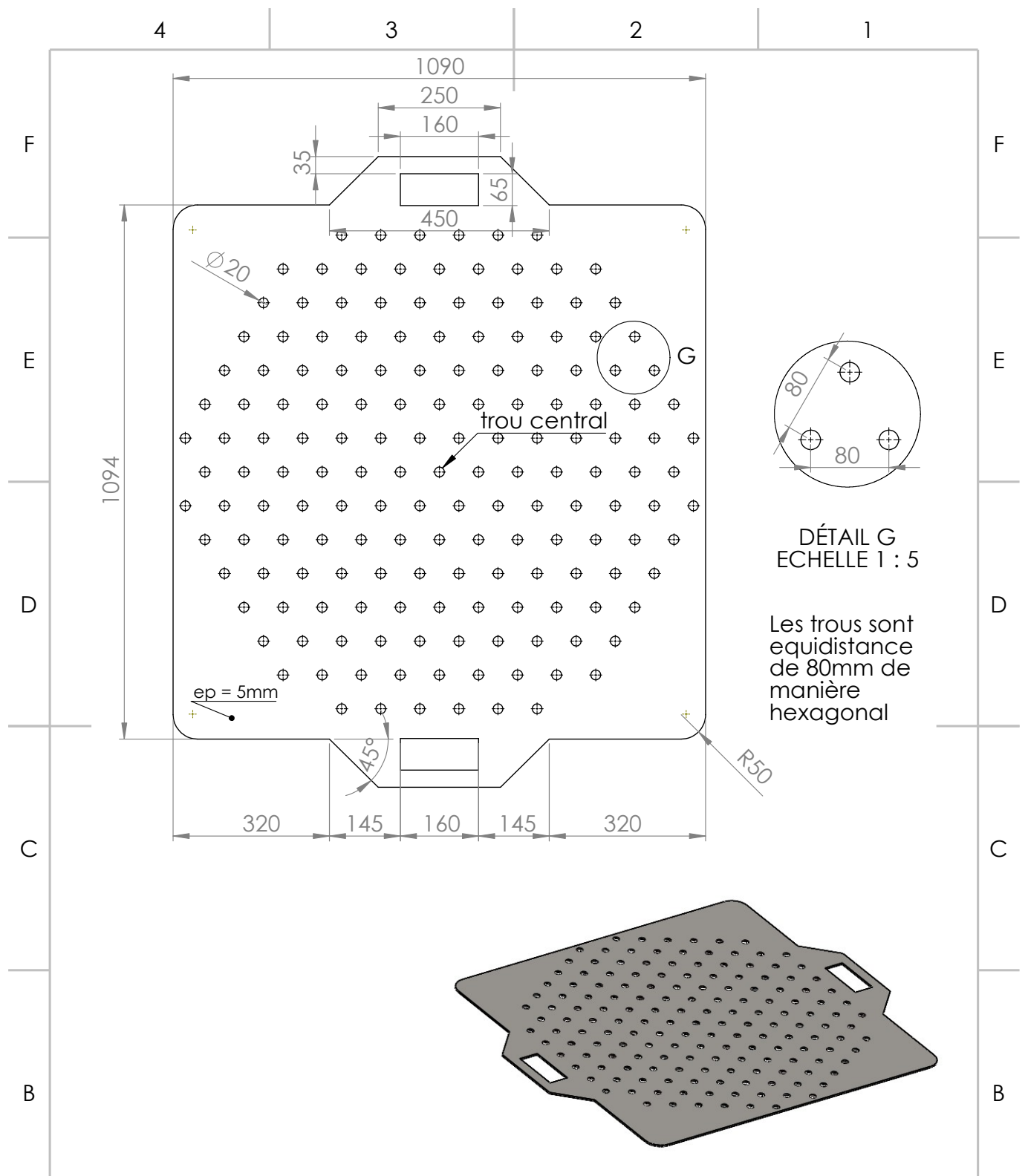


Figure 79: CPT profiles before vibratory-driven pile installation.

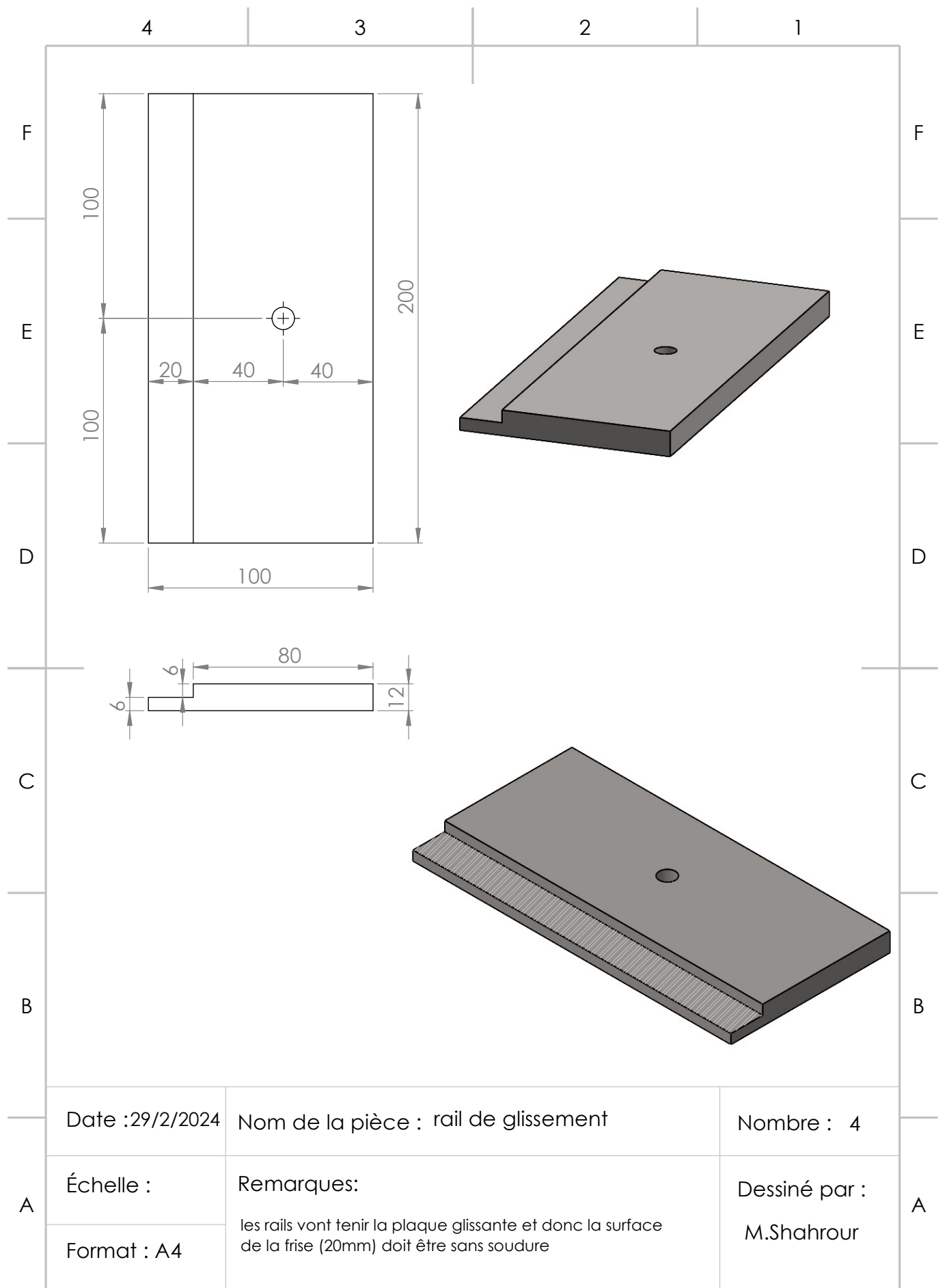
B Technical drawings of the pluviation system

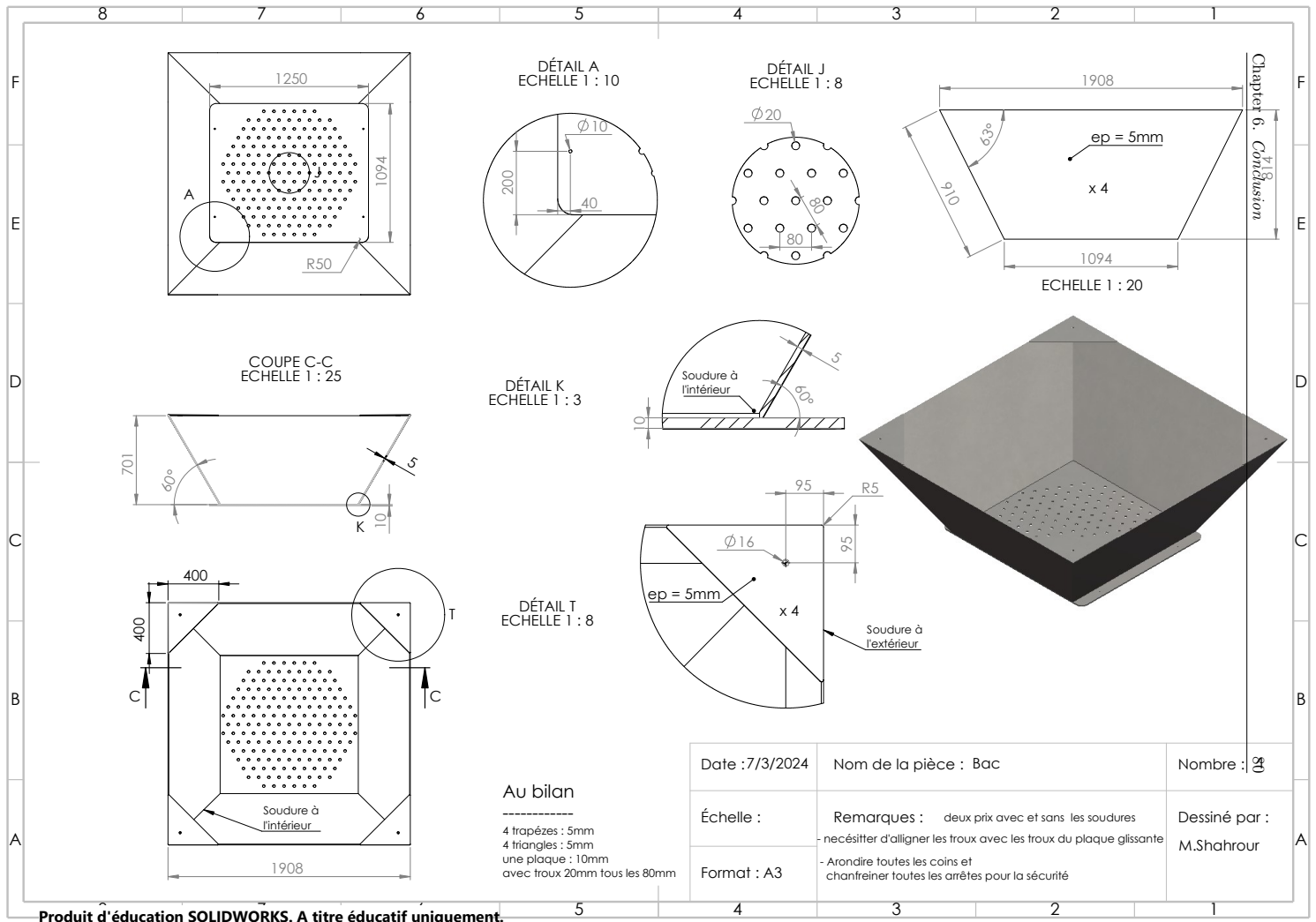
This appendix presents the technical drawings of the pluviation system used as a basis for the experimental setup. The drawings are taken from the previous work of Shahrour [6].

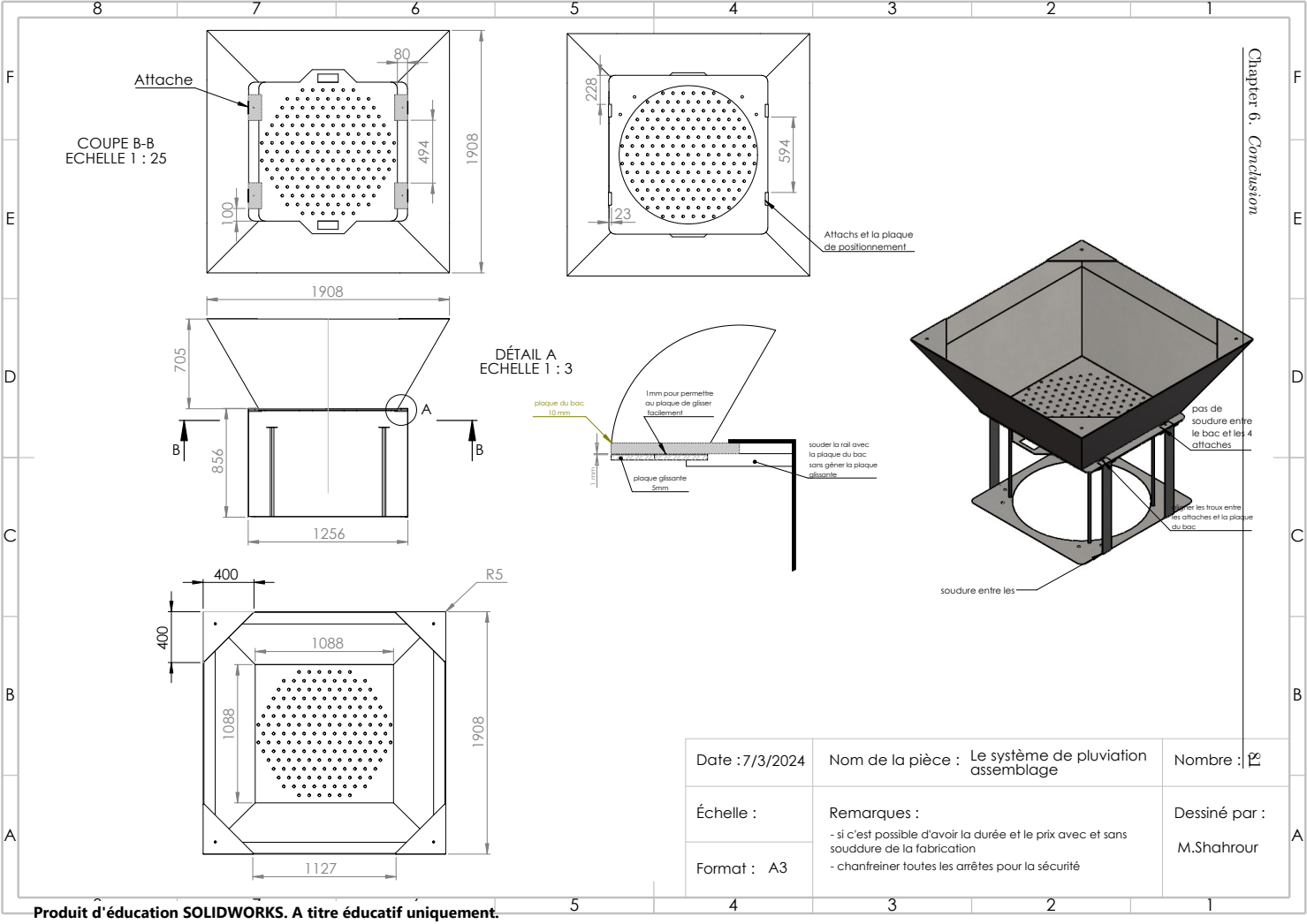




Date : 7/3/2024	Nom de la pièce : Plaque glissante	Nombre : 2
Échelle :	Remarques: - Les trous doivent être alignés avec la plaque du "Bac" - chanfreiner toutes les arrêtes pour la sécurité 1 plaque de 20mm de diamètre des trous 1 plaque de 10mm diamètre des trous	Dessiné par :
Format : A4		M.Shahrour







UNIVERSITÉ CATHOLIQUE DE LOUVAIN
École polytechnique de Louvain

Rue Archimède, 1 bte L6.11.01, 1348 Louvain-la-Neuve, Belgique | www.uclouvain.be/epl