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Centrifuge modelling of an offshore monopile in saturated sand under storm loading: WESDOM test programme

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A series of centrifuge model tests of an offshore wind turbine (OWT) monopile foundation were conducted using the 500 gtonne capacity beam centrifuge of the ETH Zurich Geotechnical Centrifuge Centre. The paper establishes a meticulous experimental procedure for realistic simulation of OWT monopile foundations subjected to storm loading, revealing soil drainage effects on cyclic monopile response. The model steel monopile, with a ratio of embedded length over diameter, L/D = 3.75, was installed in fully saturated dense Ottawa F-65 sand of target relative density D_r = 75%. The key aspects of the centrifuge modelling campaign, including: (i) construction of the monopile model including internal strain gauges; (ii) air-pluviation of the sand through a custom-built curtain raining system; (iii) extensive instrumentation with pore pressure transducers and LVDTs; (v) sand saturation with 60 cp viscous fluid; and (vi) 1 *g* jacking of the monopile. After spinning up the model to 60 *g*, the monopile was subjected to lateral one-way sinusoidal and random storm loading using a specially designed hydraulic dynamic actuator that can apply load-controlled time histories in a frequency range of 4–100 Hz in model scale. Selected test results are presented, demonstrating that the experiments successfully captured the partial drainage effects on soil-monopile system response.

Keywords: centrifuge modelling/dynamics/geotechnical engineering/liquefaction/offshore engineering/piles and piling/pore pressures/sands/soil–structure interaction

1. Introduction

Offshore wind energy is experiencing significant growth globally, with offshore wind farms being continuously developed across Europe, Asia and America, including regions prone to earthquakes and cyclones. Currently, Europe is at the forefront of offshore wind power production, expected to produce at least 240 gigawatts (GW) of global offshore wind power capacity by 2050. Most European offshore wind farms are founded in dense North Sea sands, with monopiles accounting for about 80% of the installed offshore wind turbine (OWT) foundations. The monopile size (length and diameter) is typically controlled by either serviceability limit state (SLS) or ultimate limit state (ULS) cyclic loading. A normalized lateral displacement of y/D = 0.1 (where y is the displacement at the mudline and D is the pile diameter) is typically considered a failure criterion for OWT monopile design in practice, research (Byrne *et al*., 2015, 2020; Li *et al*., 2019; Liu and Kaynia, 2022) and code standards (such as DNV, 2019; DNV, 2021). This threshold is typically associated with the ULS, representing a geotechnical failure where the soil surrounding the monopile has suffered excessive degradation, leading to system instability. Moreover, while y/D = 0.1 is a ULS threshold, OWTs are generally more sensitive to SLS failure, which is characterized by excessive permanent tilt in the order of 0.5 degrees at the mudline. For monopiles with L/D = 3–5 that practically rotate rigidly with minimal bending, a lateral displacement of y/D = 0.1 can result in rotation in the order of 2 degrees, greater than the SLS criterion. This rotation range for such tall structures including second-order effects ($P{-}\Delta$) impact system performance. Thus, in the context of standard geotechnical push-over analyses, y/D = 0.1 is widely recognized as 'failure' state, while for realistic cyclic loading, permanent deformations, even less than y/D = 0.1 can lead to instability especially when combined with significant soil strength loss (i.e. due to excess pore pressure (EPP) build-up). Therefore, the level of accuracy in predicting both the system capacity at failure, as well as the deformation (tilt) at specific load levels (before failure) has a significant impact on the economic viability of offshore wind farms.

Real performance optimisation of OWT foundations in saturated sand stems from acquiring deep understanding of the complex soil–foundation–superstructure interaction mechanisms that govern the response of the system under various combinations of cyclic loading. It is theoretically well-recognized that the cyclic response of the soil–OWT foundation system is partially drained. Depending on the loading rate, amplitude and duration, the foundation dimensions, as well as the soil stratigraphy and permeability, the dissipation of pore water pressure may or may not compensate for the tendency for pore water pressure generation (e.g. de Groot *et al*., 2006; Seed and Rahman, 1978; Tasiopoulou *et al*., 2021). Thus, partially drained conditions cover a broad range of responses, from undrained response leading to significant loss of soil strength and stiffness or even local liquefaction, to fully drained response, where stiffening accompanied by densification is the dominant mechanism (Chaloulos *et al*., 2024; Tasiopoulou *et al*., 2021, 2022).

However, despite acknowledging the importance of drainage effects, the current state of design practice for storm loading is often unable to accurately capture such complicated partially drained response. Simplified multi-step monotonic analyses are typically employed, with degraded soil strength estimated using empirical laboratory test-dependent charts, combined with 'rain flow' counting algorithms. The adopted procedures have varying complexity regarding the way the empirical charts are developed, and/or the way the degraded soil parameters are assigned to the soil during the monotonic analyses, but they are all fundamentally based on the *a priori* binary assumption of either *fully drained* or *undrained* conditions, aiming for conservatism in design (e.g. Andersen, 2015; Peralta *et al*., 2017). Given the limitations of the models and the uncertainties arising from the drainage-conditions assumptions, it is difficult to determine whether the end products are truly *over-designed* or *under-designed* (Chaloulos *et al*., 2024). The fact that no failure case-histories have been recorded to-date may indicate that existing design methods are adequately 'safe'; nevertheless, the Offshore Wind industry is relatively young and future design optimisation is calling for better understanding of how far the design is from 'failure'.

Major roadblocks to assess the reliability of the current design processes and adopt a more elaborate design method are: (a) the lack of well-documented published case studies and/or experimental data for OWT–foundations cyclically loaded to failure, or at least to significant deformation under partial drainage conditions; and (b) the limitations in the capability of numerical tools and/or the availability of computing power to fully capture the key mechanisms of the complex partially drained soil–foundation–structure interaction. In this context, there is a strong scientific motivation to conduct experiments that realistically reproduce the key aspects of OWT foundations, soil saturation and the resultant stress field, as well as loading and drainage conditions. Such experiments may shed light on the failure mechanisms of soil–OWT foundation systems, and can ultimately be used as a basis for development, improvement and validation of more advanced and robust numerical models that can be adopted in OWT design.

Over the past two decades, various experiments on monopiles in sand have been conducted, either at large-scale (field tests) or at model-scale (calibration chamber, centrifuge). While a variety of factors have been studied experimentally (especially in model scale), including monopile diameter, embedment depth, soil properties and cyclic loading characteristics, most of them were conducted under dry or fully drained conditions (as summarized in Klinkvort *et al*., 2012; Choo *et al*., 2014; Choo & Kim, 2016; Baek *et al*., 2017). Only a few tests were performed in saturated sand, but without focusing on drainage effects and without recording EPPs (Choo *et al*., 2014; Klinkvort & Hededal, 2014; Zhu *et al*., 2016).

Very recently, the attention of academia and industry has been shifted to capturing the more critical and fundamental system response due to partial drainage by ensuring soil saturation and EPP measurements. For example, the PISA project (McAdam *et al*., 2020) included field tests of monopiles embedded predominantly above the water table, subjected to stress-controlled monotonic and cyclic loading. After its completion, it was followed by the PICASO project, which had a similar concept, but the monopiles were embedded in submerged sand, and the pore pressures were monitored (Byrne *et al*., 2025); the results are yet to be published. All field tests conducted so far were carried out on small diameter piles of up to 2 m diameter in the PISA project and 2.5 m in the PICASO project, requiring the development of custom scaling relationships.

Centrifuge model testing has been the choice for many researchers in the past decades, as it is considered more suitable for the simulation of cyclic response of large diameter piles in saturated sand under a realistic stress field. Zhu *et al*. (2021) conducted centrifuge experiments investigating the response of monopiles under cyclic loading, tracking the generation of excess pore water pressures. However, the tests were slow-cyclic with a maximum prototype loading frequency of 0.05 Hz, being applicable primarily for wave loading. Takahashi *et al*. (2022) performed centrifuge model tests on monopiles in saturated sand, targeting a higher prototype frequency range (0.2–0.4 Hz). As indicated by the high EPP recorded in the vicinity of the pile, the response was clearly partially drained; however, the applied cyclic loading was *displacement-controlled*, deviating from the real nature of OWT cyclic loading, which is *force-controlled*. Moreover, the monopiles were wished-in-place, which may affect the results due to the formation of a looser soil zone around the pile possibly due to shadowing effects during pluviation around the pile, as highlighted by the validation numerical analyses performed by Chaloulos *et al*. (2024) and Sakellariadis *et al*. (2026).

The MIDAS centrifuge testing programme was recently concluded, with only selected test data published to-date (Pisanò *et al*., 2025; Wang *et al*., 2025). In these test series, wished-in-place monopiles with various diameters and aspect ratios founded in dry and saturated (with both water and viscous fluid) sand, were subjected to packages of sinusoidal loading applied in both a displacement- and load-controlled manner. In particular, the limited published experimental data obtained from the tests using viscous fluid – intended to investigate the effect of partial drainage – indicate that the applied load packages were displacement-controlled and restricted to both low number of loading cycles and narrow period range; not representative of OWT loading conditions encountered in the field.

To fill the gaps of the existing body of experimental work, a novel centrifuge testing programme entitled 'WESDOM' was designed and conducted in the Spring of 2024 at the ETH Zurich Geotechnical Centrifuge Centre (GCC), as part of the Horizon 2020 GEOLAB project. WESDOM simulated a relatively large-diameter OWT monopile embedded in dense sand saturated with viscous fluid, allowing proper scaling of permeability, which is essential for the simulation of partial drainage. The monopile was subjected to a range of sinusoidal and irregular cyclic loads, covering a broad frequency range, in a manner that is representative of the nature of the environmental (storm, typhoon, ice) loads affecting OWT foundations. The applied loading sequences consist of load-controlled packages with large number of cycles (of the order of 1000) and continuously increasing intensity, covering a broad range of loading amplitudes from SLS to ULS conditions. This scheme facilitated the development of significant deformations, investigating system response even close to 'failure' states.

This paper presents the design of the experiments and the developed physical modelling procedure, addressing the needs and challenges for successful and realistic simulation of OWT-monopile system response under storm loading and *partially drained* conditions. Besides the needs of the WESDOM experiments, the established experimental procedure opened up the road for more experiments in the same context, such as the ones of the 'DROMOS' programme (Fleminger *et al*., 2025), which were also part of the GEOLAB project and were conducted right after the WESDOM tests. Overall, the WESDOM centrifuge model tests reveal critical aspects of complex deformation mechanisms of OWT monopile foundations subjected to *partially drained* cyclic loading, while providing a high-quality, well-documented database for validation of state-of-the-art numerical tools that can eventually be utilized towards ULS and SLS design optimisation. While the principal focus of the paper is to describe the experimental procedure, selective results are presented to demonstrate that the system response accounting for partial drainage effects have been successfully captured.

2. Experimental setup

The WESDOM campaign was conducted at the ETHZ GCC, using the 500 gtonne capacity and 8.25 m dia. beam centrifuge (Figure 1), which can carry a payload of up to 5 tonnes at a centrifugal acceleration of 100 *g* (or equivalently 2 tonnes at 250 *g*). Thanks to its 500 gtonne capacity physical models of large dimensions can be tested, allowing for reduction of scale effects and facilitating the use of denser instrumentation. The models were prepared in cylindrical steel strong boxes, having an internal height $h = 0.75$ m and diameter $d = 0.75$ m, allowing for testing of large physical models (in this case an 80 mm dia. monopile) with minimal boundary effects. The boxes are accompanied by custom-made airtight lids, which were used for the saturation process.

An automated custom-built, curtain-type sand raining system was used for sand pluviation allowing for controllable and repeatable soil properties. The system is used to prepare sand models of controllable relative density D_r, with high repeatability. This is achieved by dry pluviation of the sand with controllable mass flow rate and drop height. The flow rate is adjustable by controlling the velocity and the aperture of the sand hopper. Equipped with servo-electric motors, the system allows fully automated control of lateral and vertical motion, ensuring constant velocity and drop height. To minimise the time required for model preparation, the system is designed to facilitate completion of the biggest possible model with a single fill-up of the 1.8 m wide hopper, which can carry up to 2 tonnes of sand. Six independent hopper openings offer the possibility to adjust the raining width, depending on the dimensions of the soil container. The system has a maximum operational capacity of 0.5 m^3/h, and can produce specimens with D_r ranging from 30% to 100%.

The GCC also offers a complete set of equipment for soil saturation with viscous fluid, including an assembly for deaired water, a heating apparatus, thermal cameras and thermometers to monitor temperature, as well as rotational viscometers to evaluate the achieved viscosity. After being prepared, the viscous fluid was used for soil saturation through a custom-built saturation system, which incorporates carbon dioxide (CO_2) flushing of the model and saturation with controlled flow rates under constant vacuum monitored via manometers (Kutter *et al*., 2020).

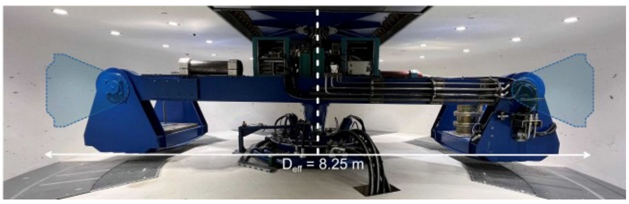

Figure 1. The 500 gtonne capacity ETHZ beam centrifuge

The data acquisition system of the beam centrifuge is equipped with 124 channels, allowing for extensive instrumentation. A wide range of newly acquired sensors, including laser sensors, pore pressure transducers (PPTs) and load cells were utilised for the tests. The ETHZ workshop built several customized parts, including support frames for the loading and the monitoring devices, as well as the model pile, and the specially designed sliding-hinge connection between the model structure and the actuator. The model pile was instrumented with pairs of strain gauges along the interior to measure bending moments. For each sensor, a calibration apparatus was made available and used before each test, to secure data quality. After building the model, the box was transported by a gantry crane and lowered onto the centrifuge swing ensuring minimal disturbance.

The novelty and key prerequisite of the WESDOM experimental campaign was testing the monopile under targeted irregular load time histories, representative of storm events containing a broad frequency band in a load-controlled mode. This was achieved by a custom-built hydraulic actuator, developed in collaboration with Hagenbuch Hydraulic Systems AG. The dynamic actuator can apply thousands of cycles of loading, used to model long-term wave and storm loading with frequencies of up to 150 Hz and loads up to 50 kN with a maximum stroke of 180 mm. The actuator can run in closed-loop force control with feedforward to produce smooth sinusoidal motions, or fully model-based open-loop to produce fast, arbitrary waveforms such as storm loading.

The WESDOM experimental series tested an OWT monopile foundation in dense saturated sand. To investigate partial drainage effects of the soil/foundation system under storm loading, it was critical that full saturation of the dense sand deposit was achieved, even while using a viscous fluid with a viscosity level equal to centrifugal acceleration value (in cp) to ensure appropriate scaling of dynamic and consolidation time. However, saturating fine sands with highly viscous fluids (e.g., more than 60 cp) can be challenging. Thus, rather than risking the achievement of fully saturated conditions, a 1:60 scale model was selected with an associated centrifugal acceleration of 60 *g* lower than the centrifuge capacity. Similarly, in order to minimise boundary effects from the 750 mm dia. centrifuge box, model pile of 80 mm dia. was selected allowing for a distance of approximately 4D between the monopile and the boundary of the container. In line with typical offshore practice an L/D ratio of ~3–4 was desirable for the model pile, and model pile embedment of 300 mm was selected. The prototype monopile thus had an embedded length, L, of 18 m, a diameter, D, of 4.8 m and a thickness of 15 cm, extending 24 m above seabed representing a substructure extending between the seabed and the transition piece. At the top of the monopile, where the storm loading was applied, a prototype mass of 216 tonnes, was attached simulating the mass of the tower and the rotor blades. These dimensions are representative of existing 2–2.3 MW OWTs in the North Sea.

The experimental models were built in 1:60 scale ensuring a centrifugal acceleration of 60 *g* at the reference level, corresponding to two-third of the embedded part of the monopile. The latter was selected to minimise the error in the achieved stress field associated with radial distortion (Madabushi, 2017). A hollow open-ended steel pile was used with a diameter of 80 and 2.5 mm thickness in model scale. The pile had a total length of 700 mm and consisted of two segments: a 300 mm segment, representing the foundation embedded in the soil; and a 400 mm segment extending above seabed. In the upper section of the monopile, an additional cylinder was inserted in the monopile weighting 1 kg in model scale. The conceptualization development of the experimental setup is illustrated in Figure 2. The dimensions of the model in prototype scale are shown in Table 1.

In addition to soil properties, partial drainage effects are highly dependent on loading characteristic such as amplitude, frequency content and duration (i.e. number of loading cycles). The applied loading sequences were designed to study the impact of all these parameters. A typical 10 min window of storm loading, used for estimating the design loads, consists of pulses of irregular loading amplitudes, random sequence with a broad frequency content. Thus, apart from storm loading sequences, sinusoidal loading was used to help isolate the impact of each of the above parameters. Sinusoidal packages of constant frequency and amplitude can investigate the impact of the number of loading cycles, while the loading amplitude effects can be explored by applying sinusoidal packages of a predefined number of cycles with continuously increasing loading amplitudes. In this line of thinking, the target loading conditions (in prototype scale) comprised:

- Slow monotonic lateral pushover to acquire the drained system capacity as a base reference, defined as the load level H_{ult}, corresponding to pile lateral displacement at the seabed y, equal to 10% of the monopile diameter D (i.e. at y/D = 0.1), as shown in Tasiopoulou *et al*. (2025). Limited unloading–reloading occurred during the pushover at three different load levels (TEST M2).
- Sequence of six 1-way sinusoidal packages (P1–P6) with 1 Hz frequency (1 s period) in prototype scale and a large number of loading cycles (320–1300). The loading amplitude of the packages continuously increased, as depicted in Figure 3 (a), as a percentage of the maximum load $H_{max/C1}$, applied at the last package (TEST C1).
- Sequence of one-way sinusoidal packages with varying frequencies from 0.33 to 1 Hz (1–3 s period) in prototype scale, containing a large number of load cycles (1000–3000). As shown in Figure 3(b), the sequence consisted of five packages (P1–P5) with 0.33 Hz frequency, two packages (P6–P7) with 0.5 Hz frequency and two more packages (P8–P9) with 1 Hz frequency. The loading levels of the sequence

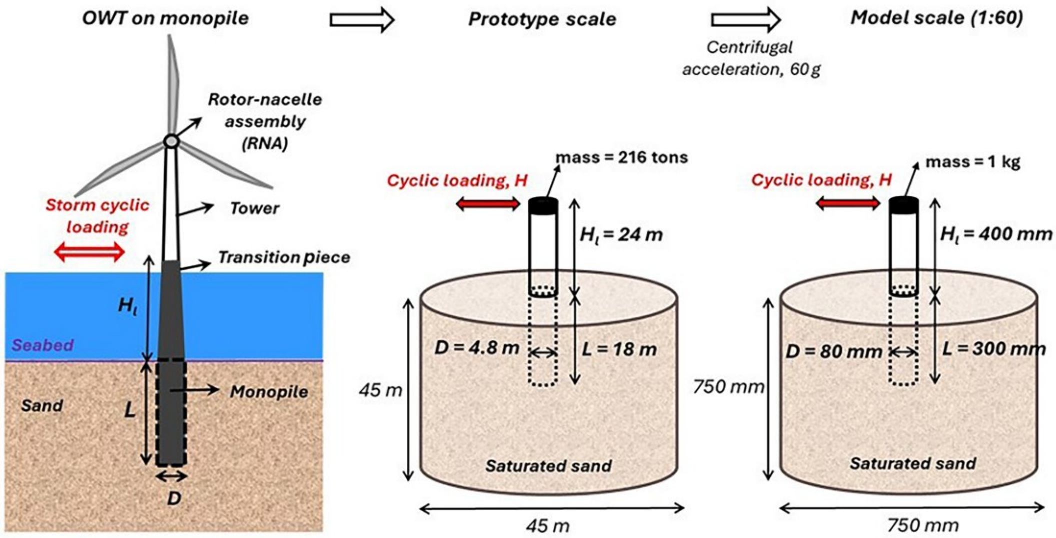

Figure 2. Conceptualization of the experimental setup

Table 1. Characteristics of the studied system at prototype and model scale

System characteristics	Prototype scale: m	Model scale: mm (1:60)
Monopile length	18	300
Superstructure length	24	400
Diameter	5	80
Thickness	0.15	2.5

systematically increased in time, and they are plotted as a percentage of the maximum load $H_{max/C2}$, applied in the last package (TEST C2).

- Sequence of packages representative of an irregular storm-load time-history with varying intensity, illustrated in Figure 3(c). All packages were based on a typical 10-min window of a storm event containing a broad range of frequencies between 0.67 and 1.67 Hz (0.6–15) s, plotted in Figure 4. The first six packages (P1–P6) of the applied loading sequence simulated the target 10-min window (Figure 4) at continuously increasing load levels up to an $H_{max/C3}$ value, while each of the following three packages (P7–P9) consisted of the target time-history with two times higher frequency content, repeated five times in each package. For the last three packages, load levels ranging from 60% to 130% of $H_{max/C3}$ were achieved (TEST C3).

Adequate waiting periods were maintained between load packages to ensure dissipation of excess pore water pressures developed during the previous load package, monitored through PPTs.

In summary, the experimental series tested a steel monopile founded in dense, saturated, Ottawa F-65 sand, under monotonic and cyclic loading at 60 *g* centrifugal acceleration. Four physical models were built in total, subjected to four different loading conditions – one monotonic and three cyclic, as summarized in Table 2.

Ottawa F-65 sand was selected due to the extensive documentation available in literature (e.g. El Ghoraiby *et al*., 2017; Morales, 2018; Vasko, 2015) based on laboratory testing in both element- and centrifuge experiments. During pluviation, the target relative density was set to D_r = 75%; the achieved values varied from 72% to 76%, as summarized in Table 2. The dense sand deposit of the cyclic tests was saturated using viscous fluid with approximately 60 centipoise (cp) target viscosity, allowing for proper scaling of permeability, which is a prerequisite for realistic modelling of excess pore water pressure generation and dissipation times. The achieved values of viscosity are also summarized in Table 2. For the monotonic test, the sand deposit was saturated with water, since the drained response under the correct stress level was targeted.

The instrumentation layout for each experimental setup is illustrated in Figure 5. The monopile is instrumented with 10 pairs of strain gauges (SG) along its embedded part to capture the bending moment distribution. The practically rigid body movement (rotation, lateral translation and settlement) of the superstructure is monitored by 4 displacement laser sensors (LS). Sensors LS1–LS3 were positioned in the plane of the actuator, as shown in Figure 5, while LS4 was installed out of plane, behind the pile at the illustration, aligned with the pile centre and located at a radial distance of 70 mm. In total, 26 PPTs were carefully installed at each test to record pore pressure build-up, particularly in the

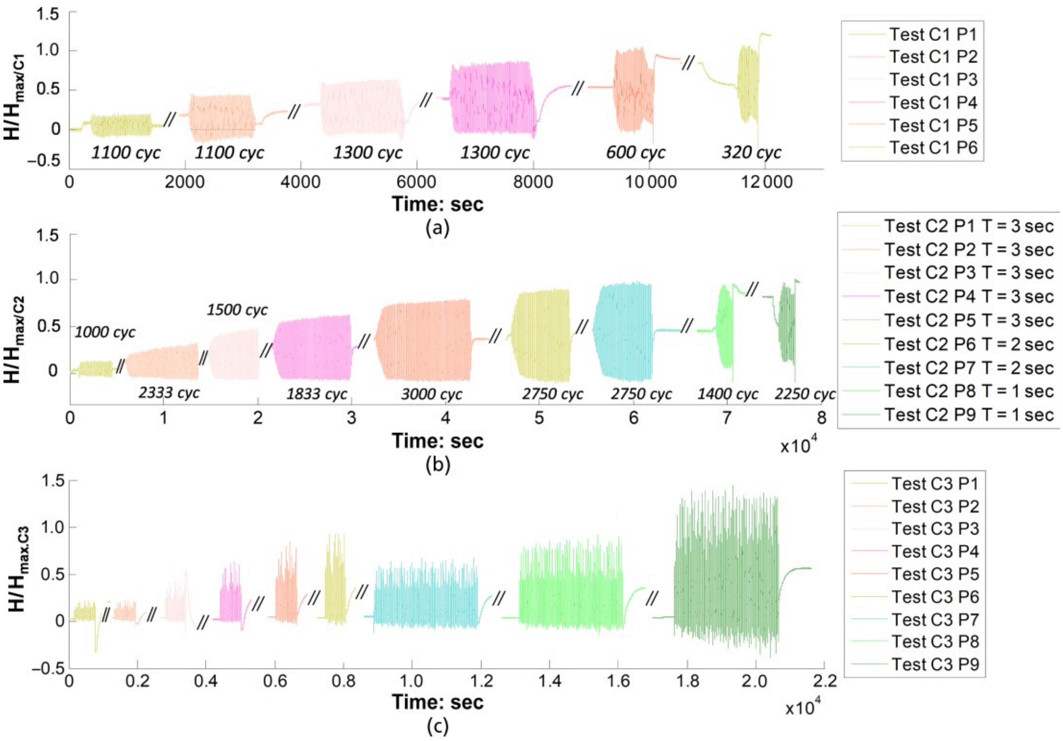

Figure 3. Applied loading sequences plotted in terms of lateral load levels against time (s) in prototype scale for: (a) Test C1; (b) Test C2; and (c) Test C3

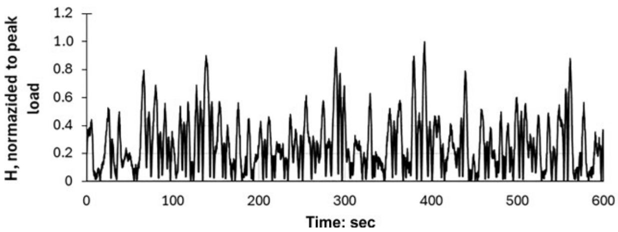

Figure 4. Target load history of a 10-min window of a storm event plotted in terms of lateral force normalized to the peak value and time (s) in prototype scale

vicinity of the monopile. For storm loading, where the load is transferred to the soil through the monopile, pore pressure development generally occurs in a certain soil zone in the vicinity of the monopile. This zone decreases with increasing depth. Thus, PPTs were arranged to cover: (i) the full embedded length of the monopile, as well as the soil area close to the tip within one D distance from the monopile; and (ii) a wider soil zone around the monopile close to the surface. A load cell (LC) was mounted at the connection of the substructure and the hydraulic actuator. At this connection, a sliding hinge was incorporated, combining a roller bearing with a linear slider. The latter not only allows the model to settle or uplift freely during lateral loading, but also ensures zero bending moment and shear loading of the load cell, preventing errors and/or damage.

3. Model preparation

This section outlines the preparation and assembly of the centrifuge model prior to testing. Specifically, it covers the process of sand air-pluviation, in parallel with the installation of PPTs, the preparation of the viscous fluid and the saturation of the model, as well as the monopile installation through 1 g jacking, its

Table 2. Characteristics of the centrifuge tests

Test	Type of lateral loading	Prototype frequency: Hz	Model frequency: Hz	D_r: %	Viscosity: cP
M2	Monotonic	—	—	72	Water
C1	One-way sinusoidal	1	60	73	64
C2	One-way sinusoidal	0.33–1	20–60	76	65
C3	Random loading – severe storm event	0.067–1.67	4–100	73	63

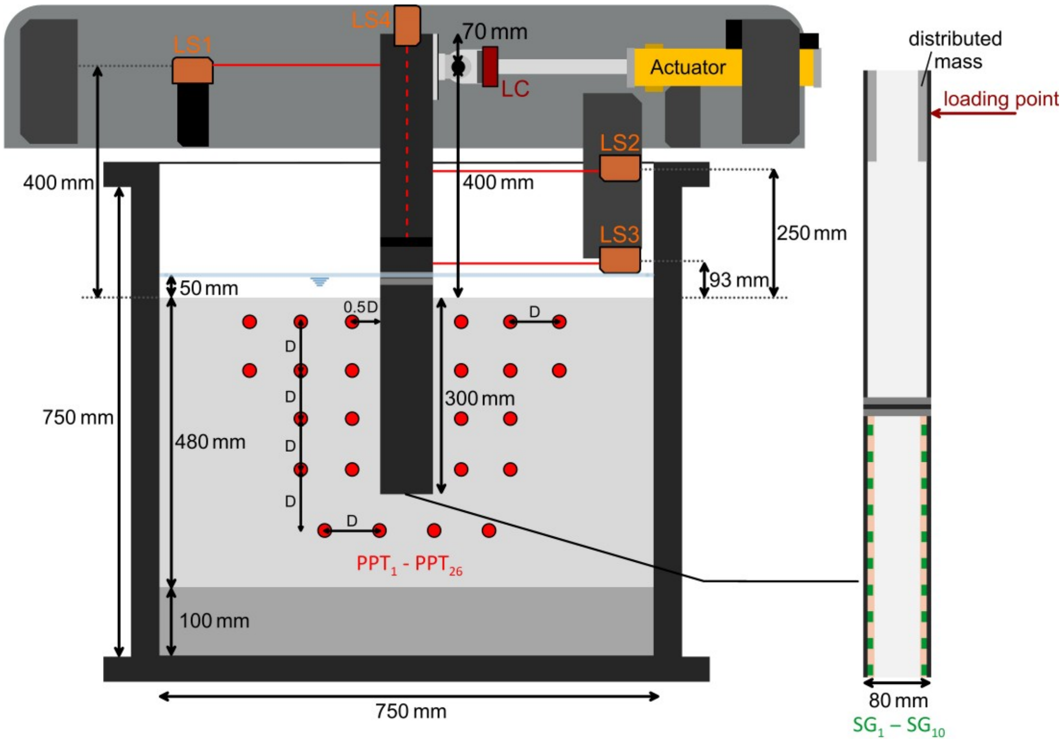

Figure 5. Overview of the experimental setup

connection with the actuator and the installation of the displacement sensors, once the model was placed on the centrifuge swing.

3.1 Sand pluviation

A 100-mm-thick gravel filter layer was first placed at the bottom of the strong box (Figure 6(a)). This layer was introduced to prevent clogging of the tank inlet valve, through which model saturation is carried out, by inhibiting the migration of sand particles towards it. A stainless-steel sieve was then placed above the gravel layer to prevent intermixing of the gravel and sand at their interface.

The sand deposit was subsequently prepared in six layers using the curtain-type air-pluviation system (Figure 6(a)), with layer interfaces aligned with the target depths of the PPT sensors. The system was calibrated specifically for Ottawa sand to achieve a relative density of 75% by adjusting the hopper slot opening to 1.8 mm to control the sand flow, maintaining a passage velocity of the curtain equal to 40 mm/s, and keeping the drop height constant at 800 mm. All models were prepared in a controlled and repeatable manner using this calibrated air-pluviation process. To verify the pluviation system, a total of five pots were carefully placed across the plan view of the strong box to provide a representative assessment of the achieved relative density within the area of interest (Figure 6(b)). Measurements were conducted at three elevations, at the bottom, middle and top of the sand deposit (Figure 6(a)). Overall, the adopted preparation method provided consistent and uniform results in terms of the achieved relative density D_r (Figure 6(c) and Table 2).

At the end of the pluviation of each layer, the strong box was rotated around its vertical axis to change the direction of pluviation in the horizontal plane (Figure 7). This procedure was adopted to minimise surface unevenness arising from the interaction between the cylindrical geometry of the box, the curtain-type raining system and the associated aerodynamic effects. The effectiveness of this methodology was evaluated by surveying the sand surface at 37 points, distributed across the entire box area (Figure 8(a)) after the pluviation of each layer.

The depths measured at points located at the same radial distance from the centre were averaged, and the total soil volume was computed by integrating the volumes of the corresponding concentric rings. Subsequently, the strong box was lifted carefully with the use of a gantry crane, and the mass of the deposited sand was measured. Using these measurements, the relative density was determined macroscopically after the pluviation of each layer, providing an intermediate check to confirm that the target density was achieved. Figure 8(b) summarizes the achieved values of D_r at each stage of pluviation, for all three experiments. After the

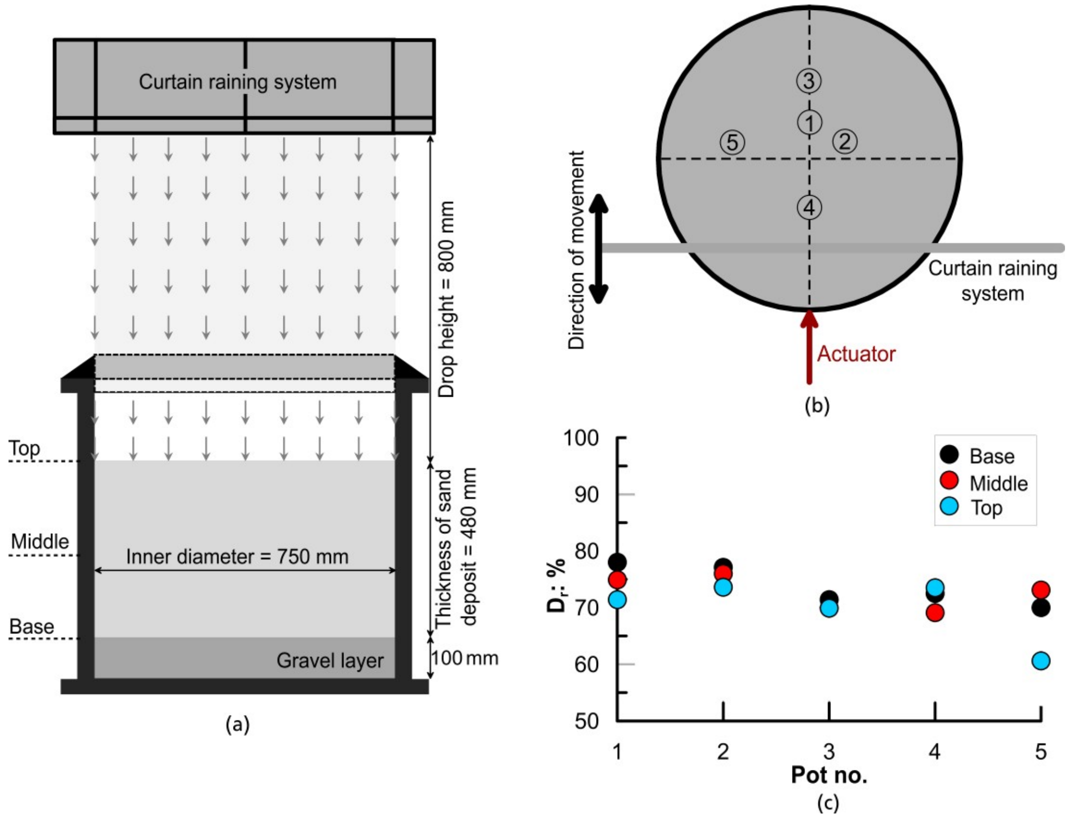

Figure 6. (a) Cross-section of the curtain-type air-pluviation system and the strong box; (b) plan view of the air-pluviation system and arrangement of pots used for verification; and (c) verification of the system and achieved relative densities

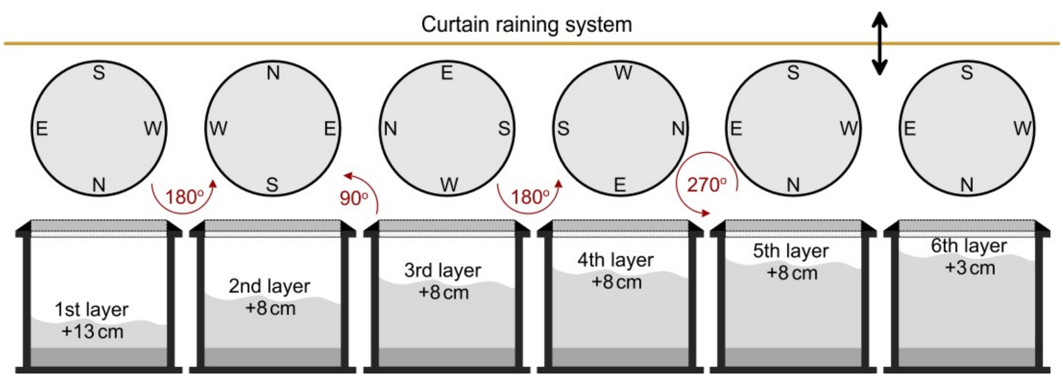

Figure 7. Illustration of successive pluviation stages and box rotation for deposition of sand layers

pluviation of the final sand layer, the surface of the model was levelled using a vacuum device equipped with a controlled-guidance system. The vacuum pipe was clamped to a frame allowing only for horizontal movement while keeping the vertical distance between the pipe and the model surface fixed. The vertical position of the vacuum nozzle was set large enough to minimise disturbance while also achieving a level surface at the target elevation.

A key objective of the WESDOM experimental campaign was to investigate the evolution of EPPs around the monopile under cyclic loading. To this end, a dense grid of 26 PPTs (P1–P26) was

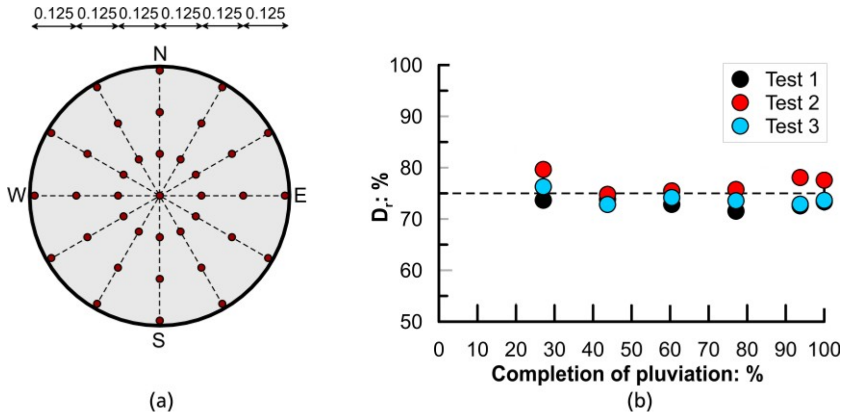

Figure 8. (a) Plan view of the points used for the surface survey; and (b) achieved relative density measured macroscopically

installed, each equipped with a specialised filter suitable for measuring pore pressures in granular materials (Figure 9). All PPTs were manufactured by Keller (type PAA-2Mi-140) covering a pressure range of 3–20 bar, and were calibrated before and after each test using an external manometer. At each target depth, the sensors were positioned at prescribed radial distances from the centre of the box – where the monopile would later be installed – primarily along the loading direction (24 out of 26), with two additional sensors placed in the out-of-plane direction (Figures 9(d) and 9(e)). The sensor locations within the model coordinate system were measured during their installation for each test, and their comparison to the designed layout is shown in Figures 9(d) and 9(e). Minor deviations from the intended positions occurred due to unavoidable imperfections associated with the sand pluviation process; however, these deviations were small, indicating a consistent and repeatable installation. The sensor cables were routed along the lateral wall of the strong box, secured with tape, and guided upwards to exit at the surface (Figure 9(b)). After completion of the pluviation process, the cables were arranged on a board mounted a few centimetres above the sand surface to ensure that they did not protrude from the box, allowing it to be properly sealed for saturation (Figure 9(c)).

3.2 Model saturation

To properly scale permeability, a viscous pore fluid was employed. A hydroxypropyl methylcellulose F50 solution in water was used as the pore fluid in the experiments, with a viscosity approximately 60 times that of water. The viscous fluid was prepared in a large (200 L) industrial vacuum mixer following the guidelines of Adamidis and Madabhushi (2015) and Stewart *et al*. (1998), targeting a hydroxypropyl methylcellulose concentration of 2.2%. Fine adjustments were made to account for the room temperature variations during preparation, which ranged between 22 and 24°C. The hydroxypropyl methylcellulose powder was gradually added to a continuously stirred batch of hot water to form a uniform suspension. The mixture was then slowly cooled and diluted with the required amount of water to achieve the target viscosity, while mixing under vacuum (0.2 bar absolute). As the suspension cooled, the methylcellulose hydrated and the fluid progressively thickened. The viscosity of the prepared fluid was checked by sampling and measuring at room temperature prior to model saturation using a benchtop rotational viscometer (Lamy Rheology B-ONE PLUS), suitable for the targeted range of high viscosities. The measured values ranged between 63 and 65 cP (see Table 2). Viscosity measurements were repeated after saturation, with practically negligible changes observed.

The saturation process was carried out under vacuum at 1 *g* (Figure 10). The fluid was introduced to the model through the gravel layer at the bottom of the box, saturating from bottom to the top. Achieving a high degree of saturation (ideally >99%) was critical, as partial saturation could significantly influence pore-pressure generation during cyclic loading. Before saturation, the strong box with the model was sealed with an airtight lid, and its integrity was verified to ensure complete isolation from external air. The fluid tank, model container and all connections and pipes were de-aired following the recommendations of Kutter (2013), under a vacuum pressure of 10 kPa (absolute). In accordance with the guidelines of Kutter *et al*. (2020), the model was then flushed twice with carbon dioxide to push out any remaining air or moisture from the pores. Following carbon dioxide flushing, vacuum was re-established in the model container (20 kPa absolute) and the viscous fluid was introduced via the basal filter layer. The saturation rate was carefully controlled by maintaining pore pressures below the threshold for soil liquefaction at the inlet. The flow rate was continuously regulated by maintaining a low and adjustable differential head between the model and the tank, achieved either by manually raising the tank with a crane (Figure 10(b)) or by directly regulating the pressure difference through adjustment of the applied vacuum levels in each vessel. The achieved flow rate was determined by continuously logging the fluid level in the vacuum tank

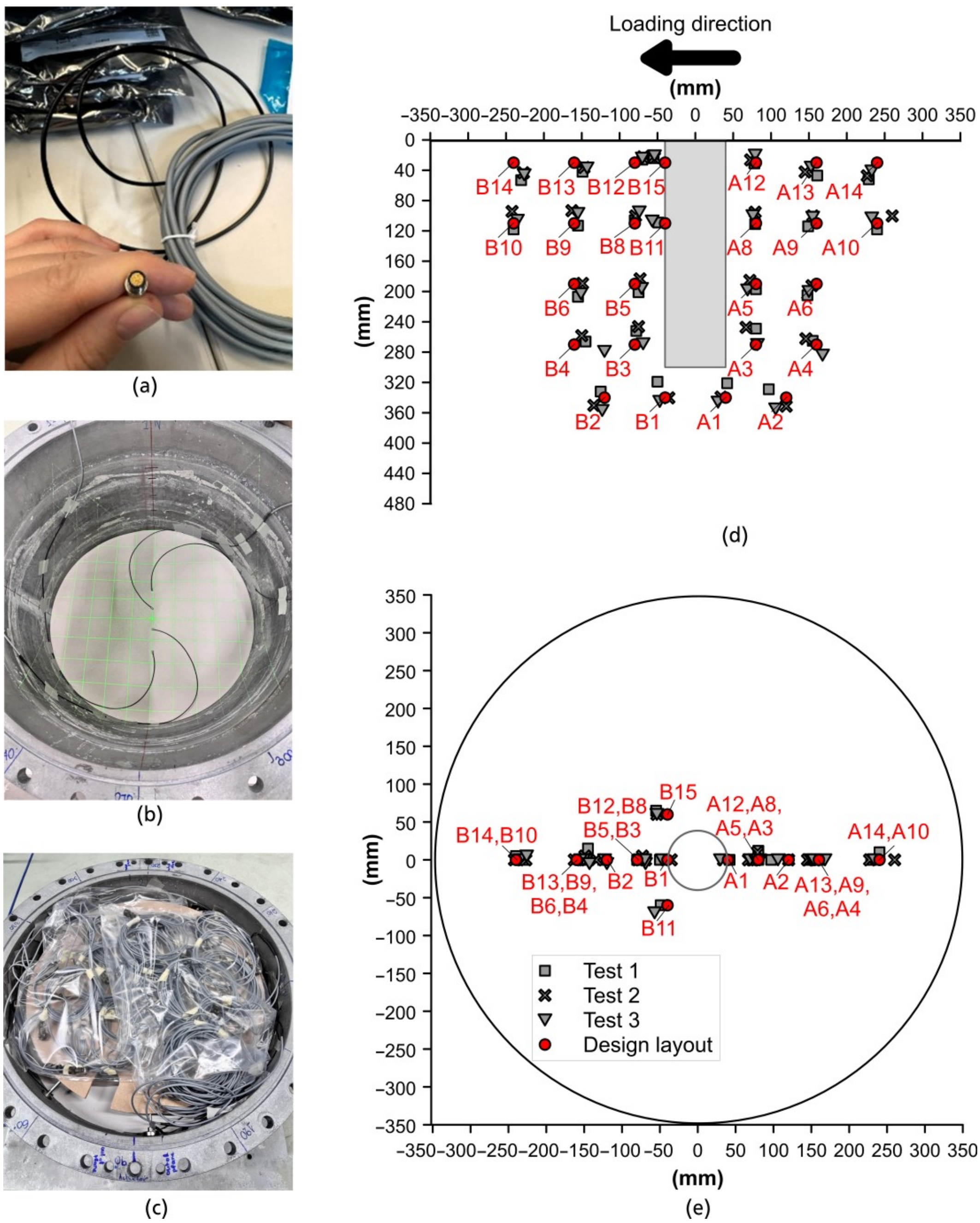

Figure 9. (a) Pore pressure transducer (PPT) with filter; (b) installation of PPTs during pluviation; (c) completed installation and securing of PPTs prior to saturation; (d) layout of PPTs in the model cross-section; and layout of PPTs in the model plan view

throughout the process. The measured flow rate was then compared against a conservative 'safe' volumetric flux, calculated using the Darcy's law and an estimate of the soil's permeability (Stringer and Madabhushi, 2009). Once the viscous fluid at the free surface of the model reached the desired level (~50 mm above 'seabed'), the flow was stopped.

Upon completion of the saturation process, the vacuum in both the fluid tank and the model was released, and the model was allowed to rest under atmospheric pressure for a few hours. The degree of saturation was then measured following the methodology of Okamura and Inoue (2012), to ensure that full saturation was achieved. Specifically, after the rigid box was released to

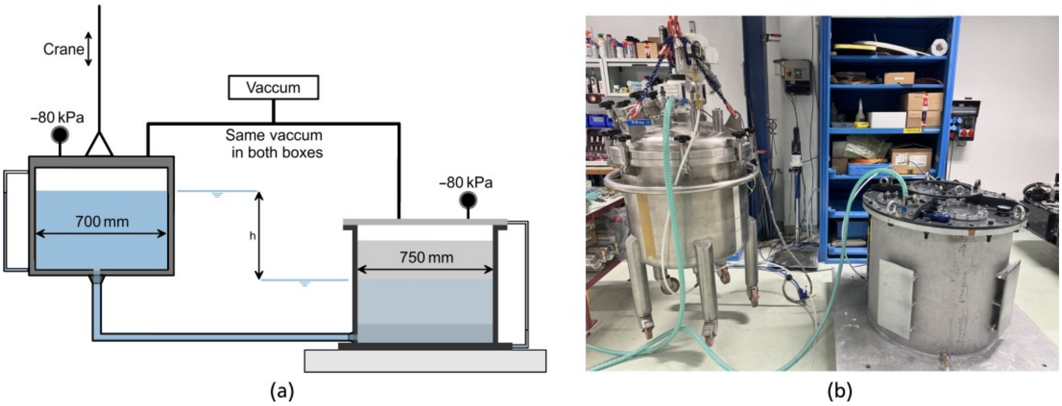

Figure 10. ETHZ saturation system: (a) schematic Illustration and (b) photo of the system

atmospheric pressure, a low vacuum (around 30 kPa absolute) was applied, and the resulting change in the level of viscous fluid was recorded. At this stage, the volume change of the viscous fluid corresponded to the volume of the remaining air trapped in the soil model, thus allowing the degree of saturation to be calculated. Negligible volume changes were observed, indicative of very high degrees of saturation (>99.5%). It is worth noting, that during centrifuge spinning the hydrostatic pressure within the model increases, and the remaining air trapped in the soil pores is dissolved (Kutter, 2013), thereby further improving the degree of saturation.

3.3 Pile installation and connection to the actuator

Following saturation, the strong box was transported using a gantry crane and carefully lowered onto the centrifuge swing (Figure 11(a)). The monopile installation was then carried out at 1 *g* by means of slow, monotonic jacking using a manual hydraulic jack. Temporary alignment guides were employed throughout the installation to ensure accurate positioning of the model pile with respect to the intended vertical and horizontal axes (Figures 11(b) and 11(c)). The entire slow hand-operated jacking process was continuously monitored using the laser displacement sensors and the embedded PPTs to track pore-pressure response of the soil to minimise the disturbance. Low jacking speed was achieved by ensuring minimal EPP build-up and allowing for adequate time for complete EPP dissipation between consecutive pushes of the jack handle.

For monopile installation, 1 *g* jacking was preferred in this study over wished-in-place installation to avoid the formation of a loose shadow zone around the monopile. Previous experimental works have shown that installation prior to sand pluviation results in a softer lateral response (e.g. Dyson and Randolph, 2001), an effect that has also been demonstrated numerically by Chaloulos *et al*. (2024). This may be attributed to the formation of a locally looser 'shadow' zone around the pile during sand placement (Sakellariadis *et al*., 2026).

Once the target penetration depth was achieved (Figure 11(d)), the alignment guides and the 1 *g* jacking frame were removed. The

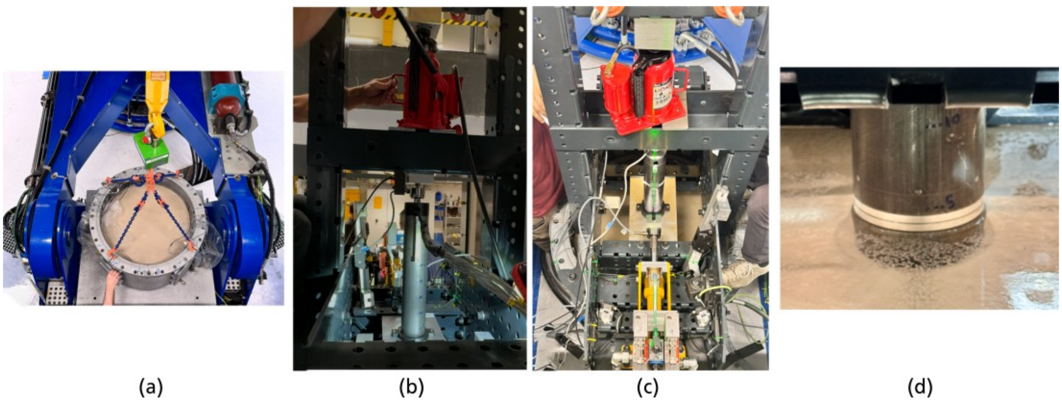

Figure 11. (a) Placement of soil model on the swing; (b) 1 *g* pile jacking; (c) connection to the actuator; and (d) embedded pile into the soil

actuator was then slowly repositioned to its starting location, and the sliding–hinge connection was established by inserting a pin through the roller-bearing hinges on both the actuator and the model pile head. With the mechanical setup completed, the centrifuge was accelerated to the target *g*-level, after which the prescribed loading sequences (Figures 3 and 4) were applied. All experimental stages (including 1 *g* pile installation, spin-up, cyclic loading and spin-down) were continuously monitored, and data from all sensors were recorded at a sampling rate of 2.5 kHz.

4. First results

Selected results are presented in this section to illustrate the key observed mechanisms and demonstrate that WESDOM successfully captured and explored the partially drained conditions due to cyclic dynamic and storm loading, while achieving high deformation accumulation, indicative of 'failure' states. Figure 12 depicts the load–displacement loops of the monopile at the seabed level, obtained from cyclic tests C1, C2 and C3, and plotted in colour code corresponding to the packages of loading sequences shown in Figure 3 for each test, respectively.

It is shown that the monopile displacements accumulate at a rate indicative of the loading amplitude, loading history and frequency content with a range of partially drained responses. This is better demonstrated by the recorded EPP ratios r_u, defined as the EPP over the vertical initial effective stress, for specific packages with similar load amplitudes in MN, selected from each test's load sequence (specifically, P5 for Test 1, P5 for Test 2 and P3 for Test 3). When applied at a short period (Test C1), this load level causes significant accumulation of EPP (r_u > 0.7) after a certain number of cycles, indicative of liquefaction occurrence, accompanied by development of large monopile displacement. However, when the same load level is applied for longer periods (Tests C2 and C3), both positive and negative EPPs develop, implying simultaneous generation and dissipation of EPPs at different degrees that may result in gradual accumulation of residual EPPs, depending on frequency and duration of the load (PPT B12, Test C2).

5. Conclusions

This paper presented a novel centrifuge experimental campaign of an OWT monopile foundation in saturated sand, subjected to load-controlled, cyclic dynamic loading, aiming to explore the effect of

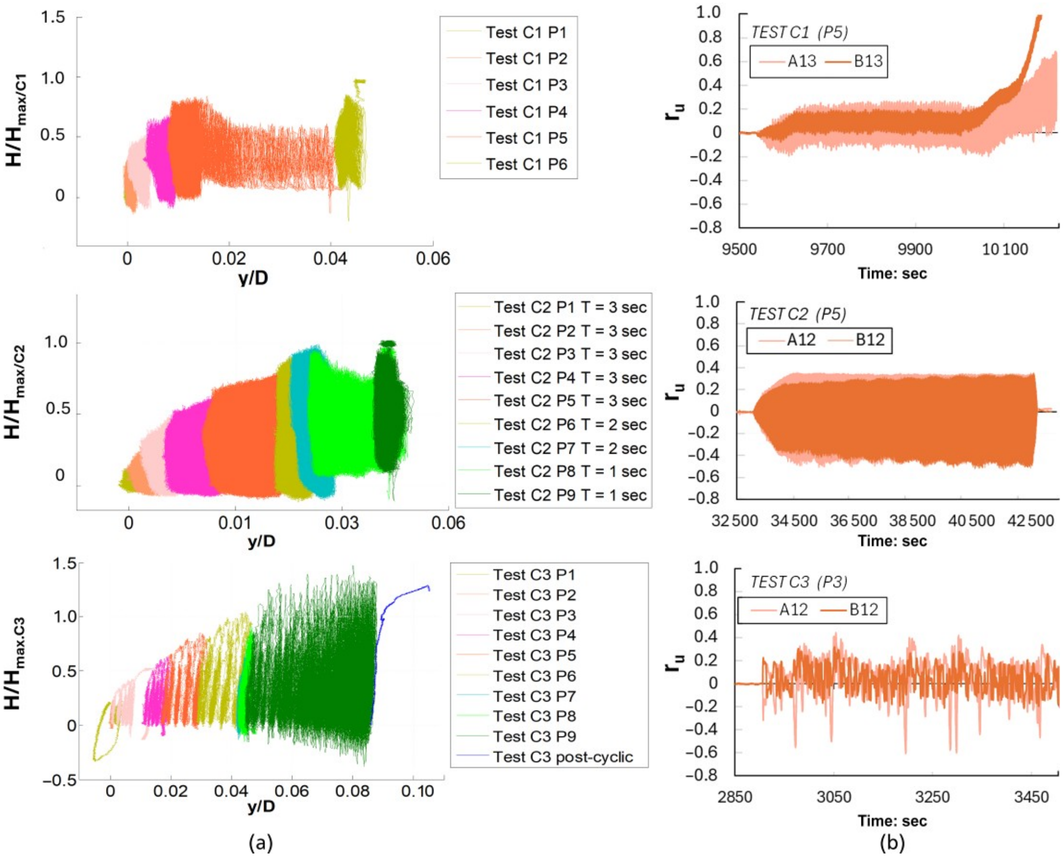

Figure 12. (a) Cyclic and monotonic monopile response in terms of applied load levels and monopile displacement at the seabed y over the diameter D; and (b) recorded time histories of excess pore pressure ratio r_u for selected packages with similar load levels

partial drainage on system response. A newly developed dynamic actuator permitted a first-of-its-kind application of realistic storm loading time histories to an OWT-monopile foundation, in addition to sinusoidal constant-amplitude time histories of varying frequencies. The facility equipment including the data acquisition system, combined with the availability of a large strong box and multiple PPT sensors facilitated the detailed instrumentation, ensuring robust documentation of deformations and failure mechanisms of the OWT-monopile system, which is greatly affected by drainage conditions. A thorough experimental procedure was established for realistic simulation of OWT monopile foundations under storm loading, paving the way for many more experiments to follow. The WESDOM test campaign provided valuable insights into partial drainage effects on system response together with a detailed high-quality dataset – an indicative part of which is presented herein.

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