

NOMENCLATURE

Symbol	Name	Units
D	Pile Diameter	m
I	Second Moment of Area of Pile	m ⁴
t	Thickness of Pile Wall	m
L	Length of Pile	m
L _e	Embedment Depth of Pile	m
m	Mass per unit Length of Pile	Kg/m
E	Young's Modulus of Pile	N/m ²
M	Mass Concentrated on Top of Pile	kg
M _p	Mass of the monopile	kg
M _o	Moment load on monopile	Nm
K _l	Lateral Stiffness	N/m
K _r	Rotational Stiffness	Nm/rad
K ₀	Coefficient of Earth Pressure	(-)
G	Soil Shear Modulus	N/m ²
I _D	Relative Density of Sand	(-)
G _s	Specific Gravity of Soil	(-)
e	Voids Ratio	(-)
ν	Poisson's Ratio	(-)
ρ	Density	Kg/m ³
γ	Unit Weight	N/m ³
σ'	Effective stress in soil	N/m ²
p'	Mean effective stress	N/m ²
γ	Shear Strain	(-)
z	Depth below Soil Surface	m
v _s	Shear Wave Velocity	m/s
p	Lateral Soil resistance	N/m
y	Lateral Displacement of Pile	m
f _{nat}	Natural Frequency	Hz

1. INTRODUCTION

1.1. Renewable Energy and Wind Energy Generation

The Earth is currently populated by approximately 7 billion people with high energy demands, which are continuously increasing as the population and use of technology increases. The depletion of fossil fuels, which have traditionally been the primary source of energy, and their effect on climate change have driven the attention of governments and environmental agencies to more sustainable solutions. Renewable energy generation and improvements in the technology that enables it have become imperative.

Wind turbines provide an attractive solution for renewable energy generation since they produce carbon-free energy. In Europe, there are currently 69 operating wind farms (EWEA 2014). UK, specifically, operates both onshore and offshore wind farms (fig. 1), including the largest offshore project in the world, the London Array Project (EWEA 2014). Offshore wind farms are more attractive to local people since they do not obstruct the view of the natural landscape. They also have the potential for more energy generation, since the winds out in the sea tend to be more consistent. Installing turbines in the sea rather than on land diminishes the concerns regarding ground borne vibrations and noise during installation and operation, which would disturb communities located nearby onshore wind farms. However, due to the fact that the foundation structure and the turbine are subjected to harsher working conditions, the construction and maintenance costs for offshore wind turbines increase compared to the ones onshore. Deciding where to construct a wind farm depends on the economic viability.



Figure 1: (a) London Array Project (Source: www.newsday.com) (b) Whitelee Wind Farm (Source: www.thetimes.co.uk)

UK's coastline provides an ideal environment for offshore wind farms, since the water is shallow. Therefore, investing in offshore wind turbines is the natural course towards achieving the goal of having 15% of UK's total energy consumption coming from renewable sources by 2020 (EWEA 2014). Figure 2 shows the current operational statistics for wind turbines in the UK, which make it the world leader in offshore wind energy, with more than 1000 turbines already installed and operating.



Figure 2: UK Wind Energy: Operational figures - At a glance (Figure: UKWED)

Although the capacity of installed wind farms is increasing, there is still a long way to go before UK meets its target, as shown in figure 3. Therefore, wind energy needs to become more cost effective and wind turbines more efficient, in order to make wind energy generation less expensive than energy generation from fossil fuels. Therefore, costs of construction and maintenance should decrease and be combined with government initiatives and incentives to encourage, support and help companies to invest in renewable energy (Huhne et al. 2011).

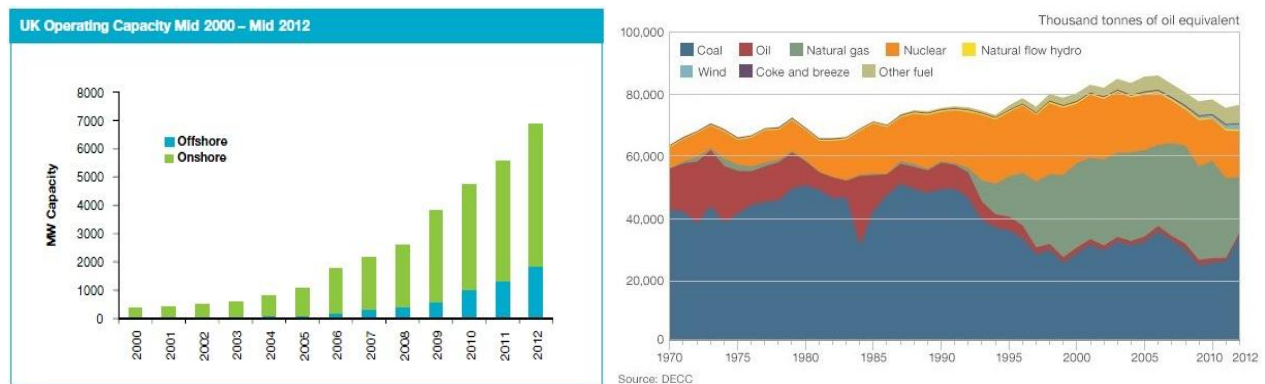


Figure 3: (a) UK Operating Capacity of wind turbines (RenewableUK, 2012), (b) Breakdown of electricity production in the UK

A large proportion of the construction cost of offshore wind turbines comes from the foundation's production, transportation on site and installation., accounting for 25-30% of the overall turbine installation cost (LeBlanc 2009). Therefore, it is essential for the design of the foundation to become more efficient in order to increase the economic viability of wind farms.

1.2. Offshore Wind Turbine Foundations - The Monopile Foundation

Design efficiency of turbine foundations depends on the understanding of the behaviour of the foundation structure under the loads it's subjected to. Offshore wind turbines are subjected to lateral wind and wave loading, which result in a high moment applied to the structure, as well as the vertical load from the turbine weight (fig. 4(a)). The foundation of the turbine needs to be able to withstand all the loads applied on the structure. There are a number of foundation solutions, such as the monopile, gravity base, tripod and jacket foundation (fig. 4(b)). Which one is used depends on the on-site conditions and design loads (Malhotra 2011).

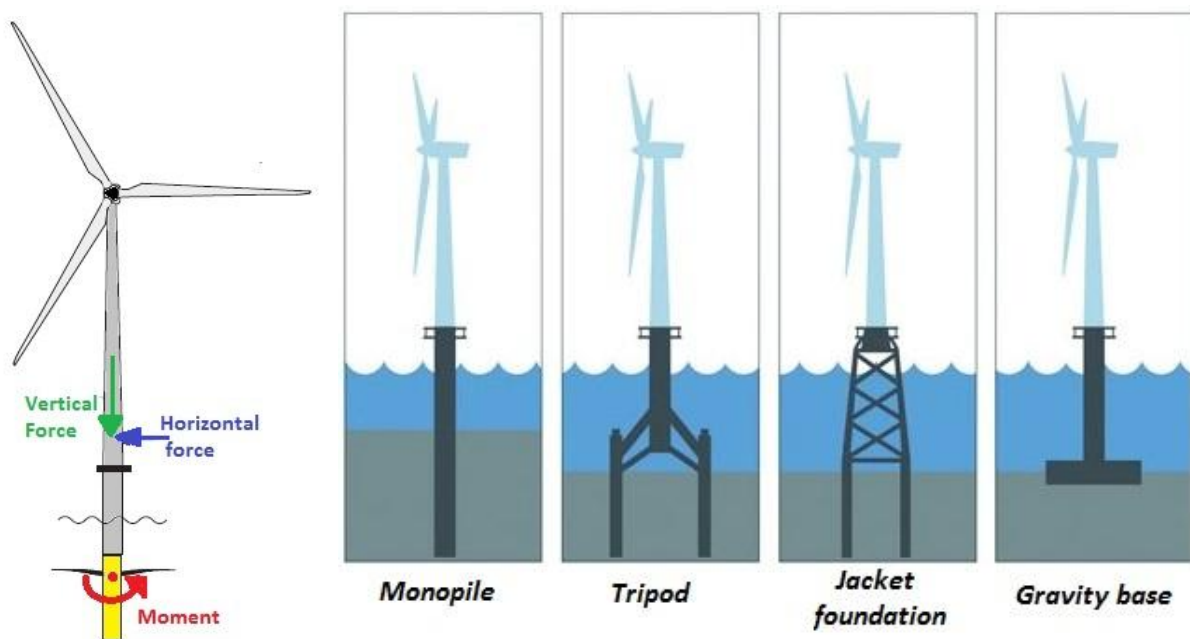


Figure 4: (a) Schematic of loads on a monopile foundation, (b) Typical foundation types for offshore wind turbines
(Source: www.theengineer.co.uk)

The focus of this project is on monopile foundations, since it's the most common foundation type for shallow water conditions (20-30m deep), such as in the case of the UK, used in 75% of turbine structures in Europe (EWEA 2014; Malhotra 2011). Their diameter, D , is typically 4-6m, with wall thickness, t , of up to 150mm (LeBlanc 2009; Malhotra 2011). The pile is installed either by hammer (impact or vibratory) or by grouting into shafts drilled in the ground, depending on soil conditions, into an embedment depth of 20-40m (LeBlanc 2009; Malhotra 2011). Figure 5(a) shows a schematic of turbine components on a monopile foundation, and figure 5(b) shows the installation of a monopile in the London Array Project.

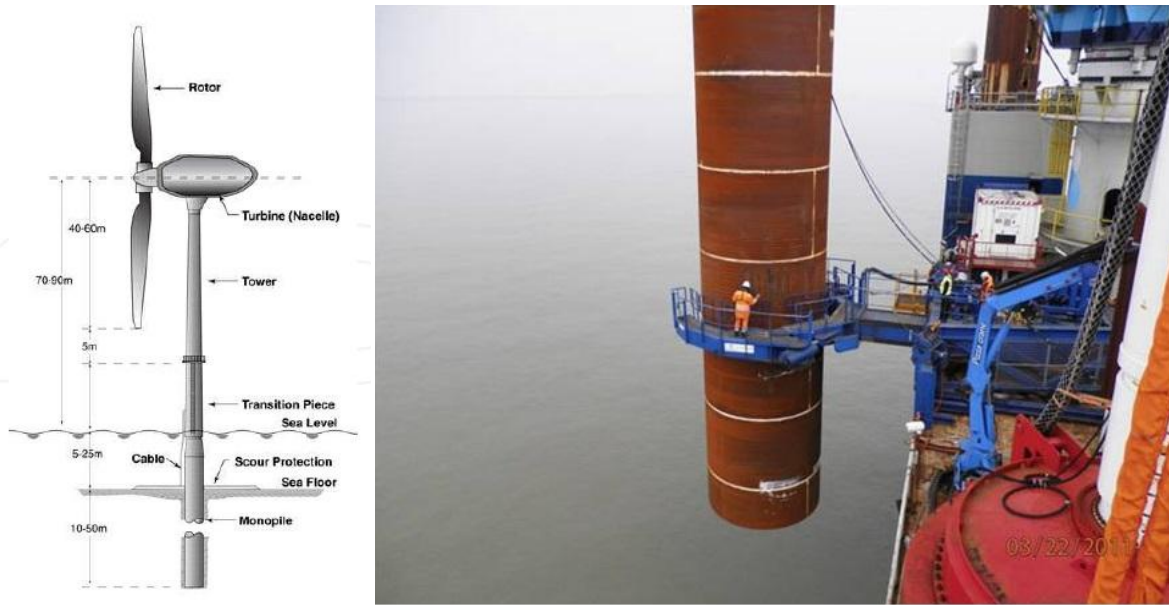


Figure 5: (a) Schematic of turbine structure components (Malhotra 2011), (b) Monopile installation by hammer in London Array Project (www.londonarray.com)

The monopile's dimensions, i.e. the large diameter and wall thickness, help the structure to withstand the large dynamic loading imposed on it. Different sources of loading force the structure to vibrate at different frequencies (fig. 6) (LeBlanc 2009; Malhotra 2011):

- Hydrodynamic (wave) and wind excitation occurs at frequencies between 0.05-0.14Hz.
- Mass imbalance or rotational speed variation in the rotating turbine causes an excitation frequency (1P) in the range of 0.17-0.33Hz.
- Blades passing in front of the tower structure also result in dynamic loading. For modern three-bladed 3.6MW turbines, this happens at around 0.5-1Hz, i.e. at about three times the 1P frequency, therefore denoted as 3P frequency.

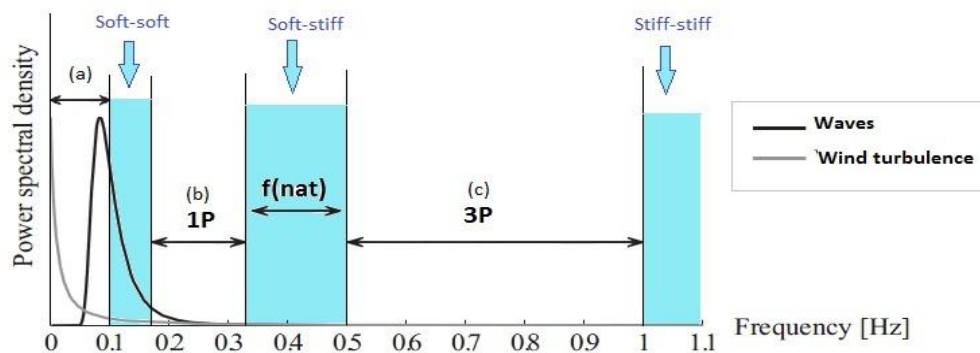


Figure 6: Forcing frequencies on turbines due to dynamic loading (Source: LeBlanc, 2009)

The areas in-between the forcing frequencies denote the range in which the natural frequency of the overall turbine system should lie, in order to avoid resonance and damage to the structure (Malhotra 2011). They refer to the different combinations of tower and foundation stiffness (Zania 2014). In current practice, the turbines are designed to have natural frequencies in the “soft-stiff” region. However, it is possible to design structures in the “soft-soft” region below the 1P frequency or “stiff-stiff” region above the 3P frequency; this will ultimately determine the pile’s stiffness. A soft structure will require less steel, and as the diameter of the monopile decreases, wind and wave loads decrease. This, however, might make fatigue or ultimate capacity the dominant failure mechanisms. Additionally, as turbines become larger, the rotation frequency 1P will decrease, merging with the hydrodynamic frequency range, diminishing the “soft-soft” region (LeBlanc 2009; Malhotra 2011). Bigger turbines will require stiffer foundations to support them, which means more material, such as steel, will be needed, increasing the cost of production and installation.

1.3. Vibrations in the Soil Surrounding the Monopile – Shear Wave Velocity

As discussed above, the dynamic loading on the turbines causes vibrations in the soil surrounding them as well. This might not be an issue for offshore wind turbines, since there are no buildings or residential areas nearby that these vibrations could affect, however they are important for onshore turbines. The soil properties determine how these vibrations are transmitted through the soil. Shear wave velocity is one way of characterising the soil.

1.4. Aims of the Project

This project is founded on the work done by two previous MEng students, Jeremy Burke (2012) and Jamie Munro (2013), on investigating the dynamic response of model monopile-soil systems. Burke carried out an evaluation of the different methods for finding the natural frequency of model monopiles in dry sand and then developed a simple model using an aluminium pile, which he embedded into sand and tested to measure its natural frequency, to compare his results with the different methods he discussed (Burke 2012). This model of the stiff aluminium pile placed in much less stiff sand is placed in the “soft-stiff” region of figure 6.

Munro used Burke's model and investigated the dynamic response of the aluminium model monopile under different types of loading. He focused on the effect of changing the relative density of the sand, and hence its stiffness, on the natural frequency of the aluminium model pile. He also investigated the propagation of shear waves in the sand (Munro 2013).

My project builds on these two projects by examining how changing the stiffness of the pile affects the natural frequency of the soil-pile system. This is done by using different materials for creating the model monopile, as was previously suggested by Munro (2013), while keeping the properties of the sand-model the same. The preferred material was high-density poly-ethylene (HDPE), since pipes of this material were readily available in the lab and could be cut at the desired length. It also means that I am investigating a soil-pile interaction that lies in the "soft-soft" region (see section 1.2), with a pile stiffness much lower than for the aluminium pile and therefore much closer to the soil stiffness. The specific aims of this project involve the following:

- Create model monopile foundations models using different materials (Aluminium and HDPE) to investigate contrasting soil-pile stiffnesses.
- Insert monopiles into soil models (dry Hostun sand) and perform 1-g tests under both impact and dynamic loading to obtain the natural frequency response of the monopile and investigate the ground borne vibrations.
- Using the results obtained from each test, compare the experimental natural frequency to the theoretical ones, as well as compare the results obtained for the two monopiles, in order to assess the effect of changing pile stiffness on the dynamic response of the system.

2. LITERATURE REVIEW

2.1. Pile design methods used currently in industry

Due to the lack of guidelines for any approved method for the design of piles with diameters 4-6m, p-y curves are used instead (Brødbæk et al. 2009; Malhotra 2011). These led to the development of the DNV standards for offshore wind turbines (DNV 2004), which most current designs are based on. The p-y curves have also been used as the basis for guidelines designed by the American Petroleum Institute (API), used already for 50 years in the design of offshore platforms for the oil and gas industry (Malhotra 2011; Brødbæk et al. 2009). The p-y were originally introduced by Reese and Matlock (1956) in an attempt to show the non-linear interactions between soil resistance, p , and pile lateral deflection, y , (Brødbæk et al. 2009; LeBlanc 2009). They were derived from full-scale tests on piles, using the Winkler's approach of modelling soil stiffness as decoupled springs, not taking into account soil continuity, but combining it with the pile stiffness (Brødbæk et al. 2009). This is shown in figure 7(a).

The p-y method is versatile and practical. However, since it was developed for piles in the oil and gas industry, it is valid for piles with small diameters up to 2m, and high slenderness (L_e/D) ratio (Brødbæk et al. 2009). In order to apply them to monopiles, they need to be extrapolated, which affects their accuracy in design process. Moreover, p-y curves consider piles as flexible beams, whereas in the case of monopile foundations, the piles may behave as rigid inclusions (fig. 7(b)) (Brødbæk et al. 2009). The effect of pile bending stiffness, EI , is not accounted for in the formulation of p-y curves (Brødbæk et al. 2009), neither is the soil-foundation interaction. This affects the expected failure mechanism, as it means the pile will rotate about a point below the ground surface (as shown in figure 7(b)) rather than by bending (Brødbæk et al. 2009). The p-y method also neglects parameters such as the pile-head rotation and the pile stiffness, which are important for serviceability limit state (SLS) requirement. The p-y curves are mostly designed and used for evaluating the ultimate limit state (ULS) lateral pile capacity and do not take into account the effect that long-term cyclic loading has on the stiffness of the pile (LeBlanc 2009) and therefore its natural frequency response (Lombardi et al. 2013). Finally, accumulated rotations and displacements due to cyclic loading are not considered by p-y curves, since the model doesn't account for the number of cycles the

structure is subjected to (Achmus et al. 2009). This changes the soil stiffness around the pile and might lead to underestimation of the pile displacements, breaching the SLS.

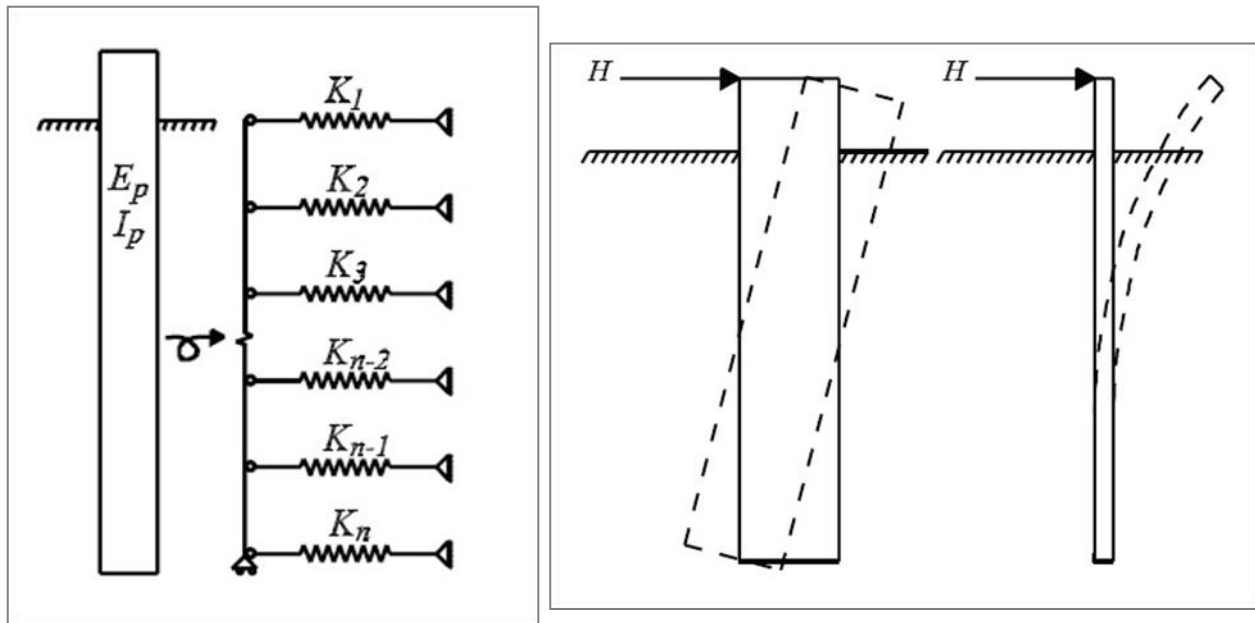


Figure 7: (a) The Winkler approach: The pile is modelled as an elastic beam and the soil stiffness is represented by a series of non-linear uncoupled springs; (b) Rigid versus flexible pile deflection under lateral load (Brødbæk et al. 2009)

A number of other limitations, assumptions and non-clarified parameters make the p-y (API) method questionable for the design of offshore wind turbine monopile foundations. Oil platforms are designed based on static calculations using a natural frequency well above the wave frequency. However, wind turbines are primarily subjected to dynamic loads from both wind and waves, and their designed natural frequency falls mostly in the range between 1P and 3P frequencies (Malhotra 2011). API doesn't give enough weight to the wind loads, which are the primary source of horizontal loading on wind turbines (Malhotra 2011). Furthermore, the p-y curves depend on the ultimate resistance of the soil. There are a number of ways in calculating this (Brødbæk et al. 2009), which in turn will affect the use of the curves. The soil ultimate resistance and soil-pile stiffness also depend on the horizontal earth pressure coefficient, K_0 , which in turn depends on the relative density or the angle of friction, depending on the type of soil, but the p-y method takes K_0 as constant, equal to 0.4 (Brødbæk et al. 2009). Finally, the p-y method depends on the initial stiffness of the p-y curves, which is only

depended on the relative density. In reality, however, this will also depend on pile stiffness and diameter as well, since the pile-soil interaction affects the overall stiffness of the system.

In summary, p-y curves suggested by the DNV standards are currently used in the design of wind turbine foundations. However, they need to be improved in order to apply to the larger diameter and higher rigidity of the monopile foundations, as well as account for the dynamic loading they are subjected to.

2.2. Predicting the natural frequency of the monopile structure

When designing the monopile foundation, it is essential that its natural frequency is accurately calculated, in order to avoid resonance, as aforementioned. The natural frequency depends primarily on the soil-foundation interaction, which is affected by the foundation stiffness, the soil stiffness and soil strength (Lombardi et al. 2013). The proposed equation (1) for finding the first natural frequency of the system, proposed by Tempel and Molenaar (2002), takes the foundation as a fixed cantilever with bending stiffness EI (as shown in figure 8(a)) but neglects the flexibility of the foundation and the stiffness softening effect due to the mass concentrated on top of the pile applying an axial load (Adhikari & Bhattacharya 2012).

$$f_{nat} = \sqrt{\frac{3.04EI}{(M+0.227mL)4\pi^2L^3}} \quad (1)$$

Adhikari and Bhattacharya (2012) suggested a simplified free-vibration analysis of the structure, in which they idealised the system and introduced assumptions, in an attempt to examine the overall dynamic behaviour of the system and provide a better understanding of the underlying principles. This incorporated the effect of the foundation flexibility and axial load, by considering the rotational and translational flexibility of the structure (Bhattacharya & Adhikari 2011), as shown below in figure 8(b).

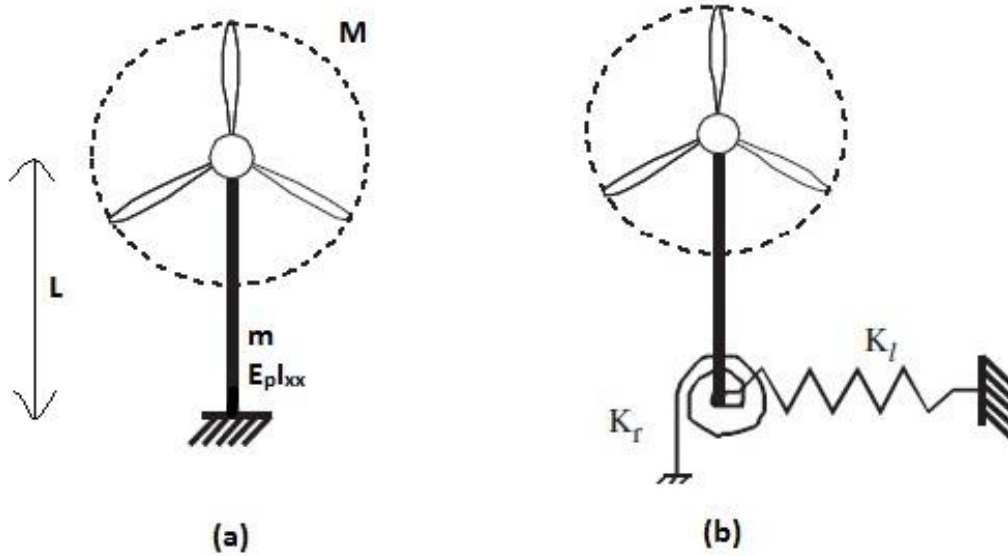


Figure 8: Models for estimating the natural frequency of the turbine structure using (a) the Tempel and Molenaar (2002) method and (b) the Adhikari and Bhattacharya (2012) method

Their analysis led to another equation (2) for the fundamental natural frequency of the pile (Adhikari & Bhattacharya 2012). Their experimental results showed that the natural frequency of the monopile changes depending on the type of soil and foundation (Bhattacharya & Adhikari 2011), proving that the soil-pile interaction is catalytic for the natural frequency.

$$f_{nat} = \frac{\Omega_1 c_0}{2\pi} \quad (2)$$

Where Ω is the non-dimensional frequency parameter and y is the lateral displacement:

$$\Omega^2 = y^2 \frac{mL^4}{EI} \quad (3)$$

And c_0 is the frequency scaling parameter obtained as:

$$c_0 = \sqrt{\frac{EI}{mL^4}} \quad (4)$$

However, it should be noted that the units in these equations seem to be inconsistent and therefore these equations were not used for this project.

Another parameter that affects the soil stiffness and hence the natural frequency of the pile is the number of cycles that the soil is loaded to. Scale tests under 1-g conditions showed that

for loose to medium dense sand, strain-hardening causes the soil stiffness to increase with increasing numbers of loading cycles, leading to an increase in the natural frequency of the system. The opposite is observed for strain-softening clay sites (Bhattacharya et al. 2013). Achmus et al. (2009) introduced the degradation stiffness model, a numerical modelling based on the finite element method and results from drained triaxial tests, that accounts for the non-linearity of soil properties due to cyclic loading. His theory was tested against scaled-model tests and was found to agree to a high extent. Examples of his results are shown in figure 9, where the lateral deflection of the pile increases with number of cycles and depth from seabed. Comparing two piles with embedment length 20m (fig. 9(a)) and 40m (fig. 9(b)), it is obvious that lateral deflection decreases with increasing embedment and the pile fails by rotation, forming a toe kick near its base. Finally, the rate of lateral displacement at soil surface is higher for a pile of shorter embedment length, as shown in figure 9(c).

These models have the potential of being developed further, in which case they will replace the p-y method for the design of monopile foundations for offshore wind turbines, since they account for parameters such as number of cycles of loading and soil-stiffening that p-y method doesn't.

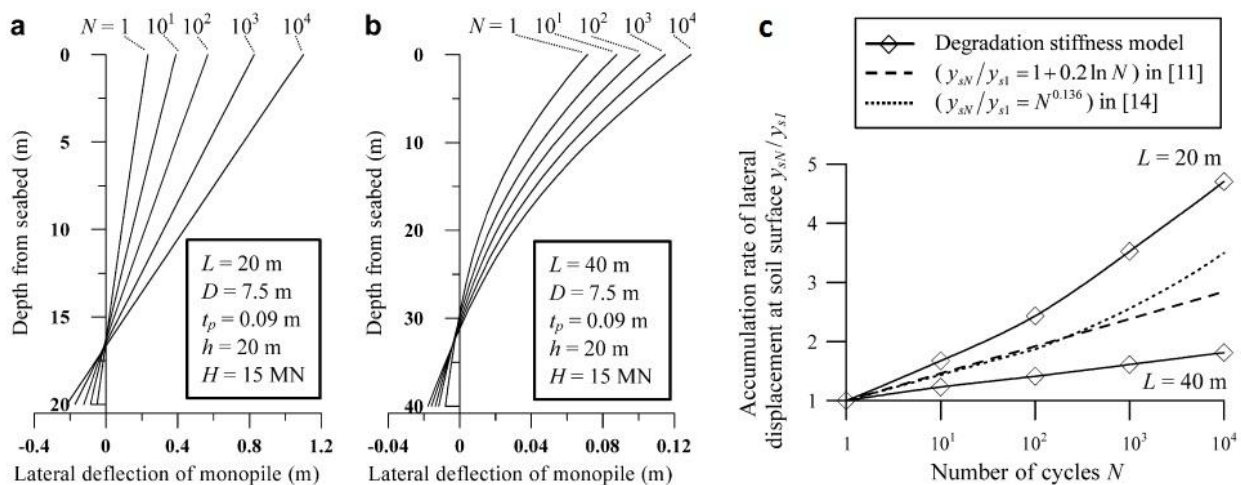


Figure 9: Lateral deflection of monopile for embedment depth of (a) 20m and (b) 40m and (c) the rate of accumulation of lateral displacement (Achmus et al. 2009)

2.3. Experimental modelling using dimensionless groups

Bhattacharya et al. (2013), Muir Wood et al. (2011) and Byrne et al. (2010) carried out tests on model-monopiles at 1-g conditions, using different materials for the pile and soil. They derived and agreed on six dimensionless groups for the applied loads and moments on the piles, its dimensions and the strain levels in the soil, which are needed for small-scale tests. These are listed in table 1. Muir Wood et al. (2011) stress the importance of maintaining the soil strain next to the pile, in order to have similar stiffness degradation between model and prototype. He also states that the ratio between forcing frequency and natural frequency should be maintained in order to model the dynamics of the system correctly.

Table 1: Dimensionless groups for scaled monopile models

Moment loading, M_o	$\left[\frac{M_o}{L^3 D \gamma'} \right]$
Vertical force, V	$\left[\frac{V}{L^2 D \gamma'} \right]$
Horizontal force, H	$\left[\frac{H}{L^2 D \gamma'} \right]$
Load eccentricity, e'	$\left[\frac{M_o}{HL} \right]$
Aspect ratio, η	$\left[\frac{L}{D} \right]$
Soil strain level, ϵ	$\left[\frac{H}{GD^2} \right]$

2.4. Shear wave velocity in soils under cyclic loading

Shear wave velocity can be calculated using equation (5), as known from wave propagation theory (Santos & Gomes Correia 2000):

$$v_s = \sqrt{\frac{G_o}{\rho_{sand}}} \quad (5)$$

Experiments analysing the shear wave velocity are used in order to form empirical relationships for soil shear modulus, G , (Rahman et al. 2012; Brennan et al. 2004). These have shown that the shear modulus varies linearly with the voids ratio, e , and the mean effective stress, p' (Rahman et al. 2012). At small strain levels ($\gamma \approx 10^{-6}$), the stress-strain in the soil

displays a linear relationship governed by the initial shear modulus, G_0 (Santos & Gomes Correia 2000). The equation for G_0 for sandy soils is (Hardin & Drenvich 1972):

$$G_0 = 100 \frac{(3-e)^2}{1+e} (p')^{0.5} \quad (6)$$

As the strain level increases to medium ($\gamma \approx 10^{-6} - 10^{-2}$), the stress-strain behaviour becomes non-linear, with the shear modulus changing in relation to shear strain. This degradation in shear modulus causes the degradation of soil stiffness, which, as explain above, will affect the dynamic behaviour of the overall system. The change in shear modulus is shown in figure 10, on a graph created by Bolton and Oztoprak (2013) by collecting the data of 454 tests carried out on sands of different drainage conditions.

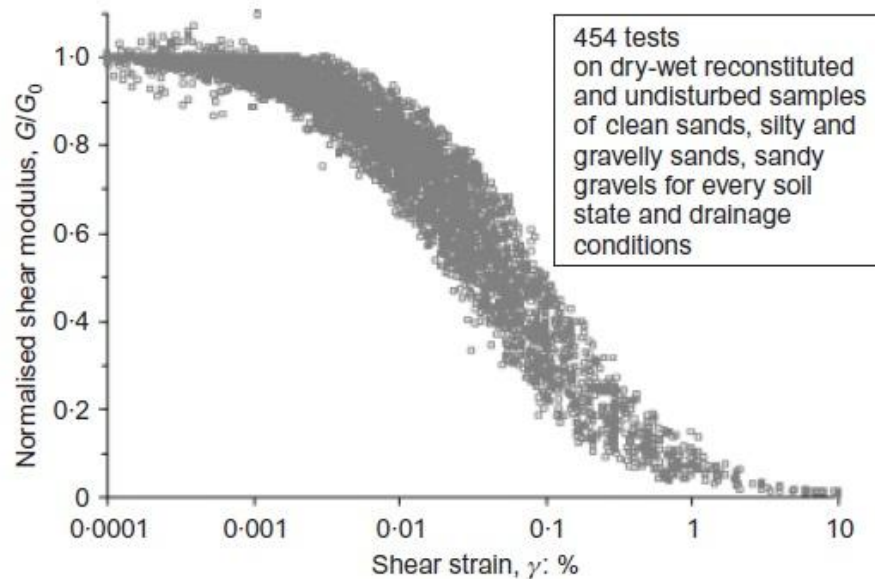


Figure 10: Change in shear modulus for with respect to shear strain for sands (Bolton & Oztoprak 2013)

Finally, as aforementioned, the number of load cycles affects the soil stiffness, which is not taken accurately into account in the current design methods. Degrading the p-y curves is the most common way of accounting for the degradation of soil stiffness. However, tests on model piles in dry sand have shown that cyclic loading can sometimes increase the system stiffness (Byrne et al. 2010), since the soil undergoes strain-hardening (Bhattacharya et al. 2013).

2.5. Conclusions

The current design method for offshore wind turbine monopile foundations is based on research for offshore platforms for the oil and gas industry, using modified the p-y curves as suggested by the DNV standards. The applicability of this method for turbine foundations is questioned by many researchers, due to the difference in foundation characteristics, such as the diameter of the piles used, and the difference in the loads that each structure is subjected to. It also fails to capture the effect of number of load cycles on the soil stiffness, the system's natural frequency and the accumulated displacement and rotation. Research concentrates on improving the current methodology by trying to incorporate these parameters in the current design code.

This project focuses on the dynamic response of the monopile in sandy soils, aiming at investigating its natural frequency for different soil-pile stiffnesses under different loading modes. The models used were adopted from previous MEng students, Burke (2012) and Munro (2013) who had incorporated dimensional analysis in designing their aluminium monopile and sand models. Therefore, no dimensional analysis was used in this project. The HDPE monopile was chosen to have the same dimensions as the aluminium pile previously used by the two students, in order to investigate only the effect of material stiffness on the dynamic response of the monopile.

The following section on experimental set-up describes how the tests were designed and carried out.

3. EXPERIMENTAL SET-UP

3.1. Model monopiles and instrumentation

All the tests were carried out on the same scaled-down, open-ended model piles made from dural aluminium alloy (Al) and high-density polyethylene (HDPE), which have the dimensions and characteristics shown in table 2. The experiments were carried out at 1-g conditions, since the project does not aim at predicting the response of the pile in field conditions but rather focuses on providing a better understanding of the effect of pile stiffness on the dynamic response of the monopile.

Table 2: Properties of the model piles used in the experiments

	Aluminium	HDPE
Pile length, L (mm)	450	452
Outer diameter, D (mm)	76	68.5
Wall thickness, t (mm)	1.7	2.0
Young's modulus, E (GPa)	70	1.9
Second moment of area, I (mm ⁴)	2.74x10 ⁵	2.31x10 ⁵
Mass of monopile, M _p (kg)	0.475	0.245
Mass concentrated on top of pile (top-plate and actuator), M (kg)	0.645	0.645

Micro-electro-mechanical (MEM) accelerometers of 1.7-g and 35-g were attached on each pile in order to measure the acceleration of the pile itself during testing, which would then allow us to find the natural frequency. Since MEMs can only measure acceleration in one direction, three were installed along the side of the pile in the direction of loading. One was also installed at right angles to the loading direction, in order to check that the vibration was indeed unidirectional. The two model piles used, with the attached MEM accelerometers, are shown in figure 11. MEM accelerometers have very sensitive wiring, therefore it was essential to handle them with care and make sure to attach them securely on the pile so as the wires wouldn't break during pile installation into the soil model.



Figure 11: (a) Aluminium model pile with installed MEMs; (b) HDPE model pile annotated with installed MEMs

Four more 1.7-g MEM accelerometers were inserted in the soil during the impact and harmonic loading tests, as shown in figure 14, to enable the measurement of shear wave velocity. Since they are very small and light instruments, they are not expected to have affected the results in any way. MEMs are also relatively cheap and easily replaceable.

All 1.7-g accelerometers operate with a power supply of 5V and have a DC offset when no acceleration is applied to them, which was removed during data analysis by averaging the values of voltage measured when no load was applied on the pile. The measured voltage output would change when they are subjected to some acceleration, and they can be calibrated in the range of -1g to 1g to find the actual acceleration measured in relation to the voltage output. For the purpose of this project, no calibration was needed since we were not interested in the value of the measured acceleration but rather the relative acceleration measured (proportional to the voltage output) and where maximum acceleration occurred.

Data was recorded using DasyLab 9.0 software via a junction box at a sampling rate of 50kHz. This needed to be sufficiently high in order to be able to capture the shear wave velocity

and account for the small distances between the MEMs, but not too high as it would cause problems with the memory capacity of the computer. MATLAB was then used to analyse the data, converting data from time domain to frequency domain using the fast Fourier transform function, *fft*(signal). The *Butter* function was used to remove all high frequency noise and was combined with the *filtfilt* function, which minimised phase distortion resulting after filtering.

3.2. Initial impact tests on model monopiles

Both the aluminium and the HDPE piles were tested as fixed cantilevers, clamped at the base, in order to determine the validity of equation (1) by measuring the natural frequency of the pile when it's fixed at its base. An impact load, by the means of a hammer, was applied to the top of the monopile, shown in figure 12. Attention was paid to hitting the pile along the axis in which MEM accelerometers record acceleration, and trying to always hit on the same spot at the top of the pile. The top 1.7-g MEM was used to determine the experimental natural frequency of the monopile alone, without the influence of the soil's stiffness.

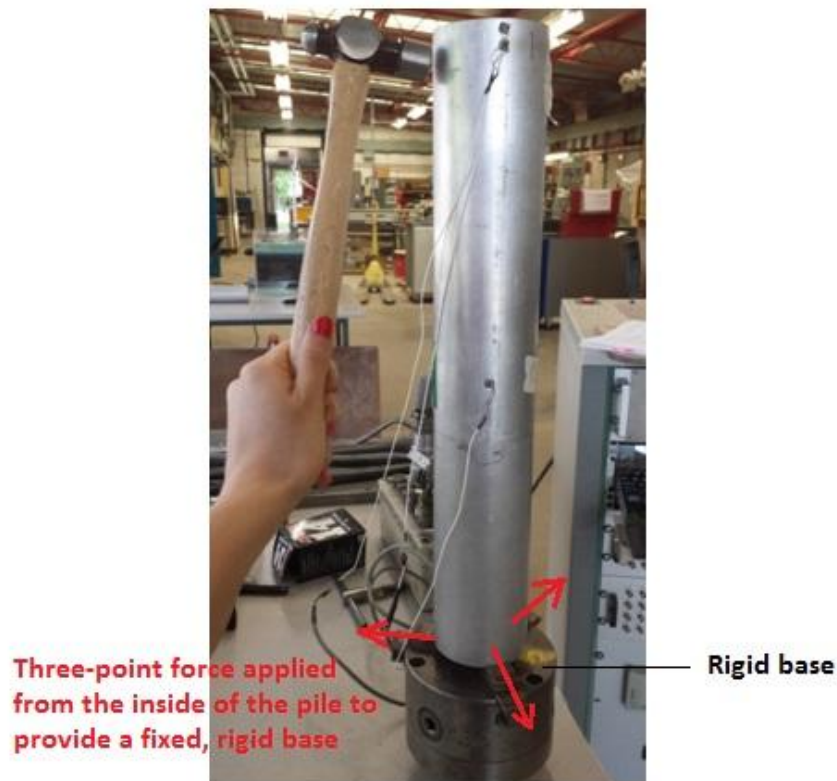


Figure 12: Test on the aluminium model pile fixed at its base

3.3. Preparation of soil and experimental model set-up

The model piles were installed in dry Hostun sand for the experimental tests. It was decided to use this type of sand as it was readily available in the lab. Sand was used instead of clay since it would allow for more tests to be carried out in the time available for the project. Finally, dry sand was used since it makes it quicker and easier to prepare the model. The properties of Hostun sand (Amat 2007) are shown in table 3.

Table 3: Properties of Hostun sand (Amat 2007)

Specific gravity, G_s	2.65
Poisson's ratio, ν	0.25
Maximum voids ratio, $e_{\max}$	1.041
Minimum voids ratio, $e_{\min}$	0.648

The sand was poured into an 850mm diameter tub using the sand hopper. The hopper was calibrated by using pouring samples, in order to determine the flow rate required for the desired relative density. When filling the tub, the sand was poured in layers to achieve a homogeneous isotropic soil, by moving the hopper in a zig-zag motion first from top to bottom and then from right to left, alternating between layers. This is schematically shown in figure 13.

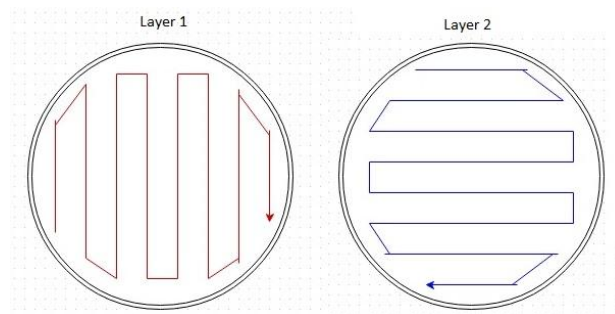


Figure 13: Filling in the tub

The tub was filled with sand to a height of 190mm, at which point the pile was driven in, to a depth of 170mm, creating a soil plug to resemble real pile installation. Then, the tub was filled with sand until a depth of 310mm was reached, when the four 1.7-g MEMs were placed in the soil radially outwards, at distance increments of 0.5D (where D is the diameter of the pile), at 38mm, 76mm, 114mm and 152mm. Finally, the tub was filled to a total depth of 350mm, leaving the pile with an embedment depth of 330mm. The top-plate on which the actuator is

connected was at 120mm above the soil surface. The arrangement of the soil-pile model is shown schematically in figure 14, while figure 15 shows the actual prepared model for each pile, with the actuator attached by using the top-plate in order to apply a harmonic load.

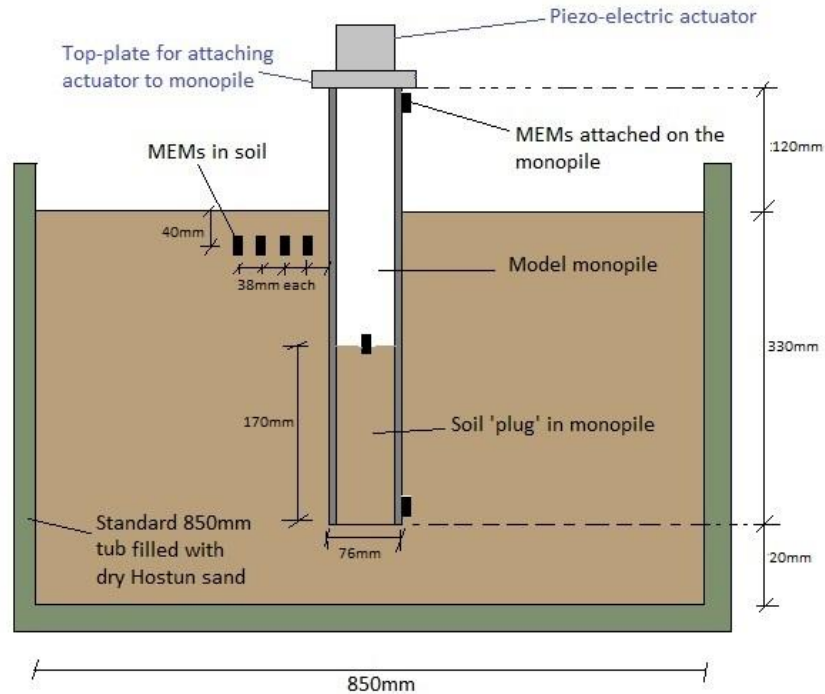


Figure 14: Schematic of experimental model set-up

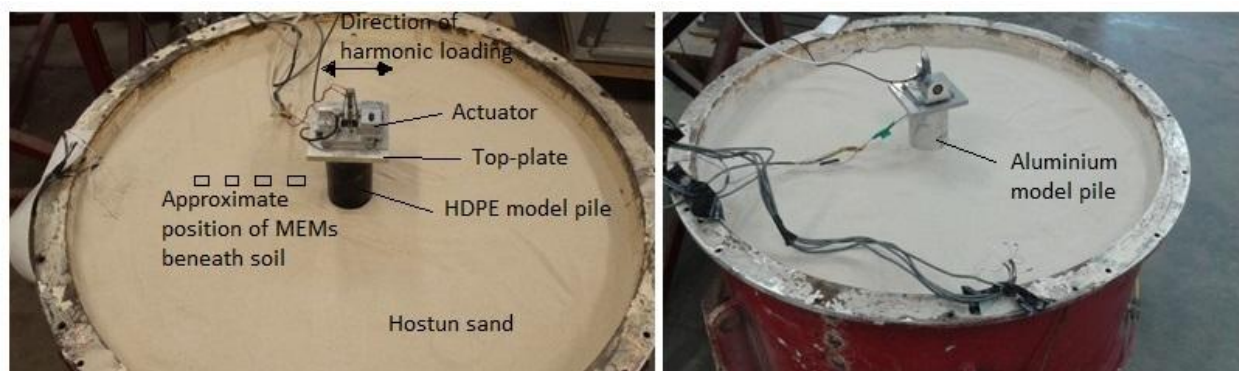


Figure 15: Soil-pile models ready for testing

3.4. Harmonic loading tests on model monopiles in sand

The harmonic loading tests were carried out to resemble the dynamic loading applied on turbine monopile foundations installed offshore. Cyclic loading was applied on the monopile by

using a piezo-electric actuator attached to the top of the pile by a square top-plate. The actuator and top-plate are shown in figure 16. The loading was applied only in one direction, which needed to be aligned with both the MEMs on the pile and the MEMs in the soil.

A sinusoidal input was given to the actuator through a signal generator and an amplifier, which operated at a DC-offset and had a peak-to-peak input voltage of 5V. The signal generator allowed for a range of forcing frequencies to be applied to the pile, adjusted manually to the desired frequency input. Each frequency load was recorded and analysed separately, using the top 1.7-g MEM to find the natural frequency of the pile-soil system.

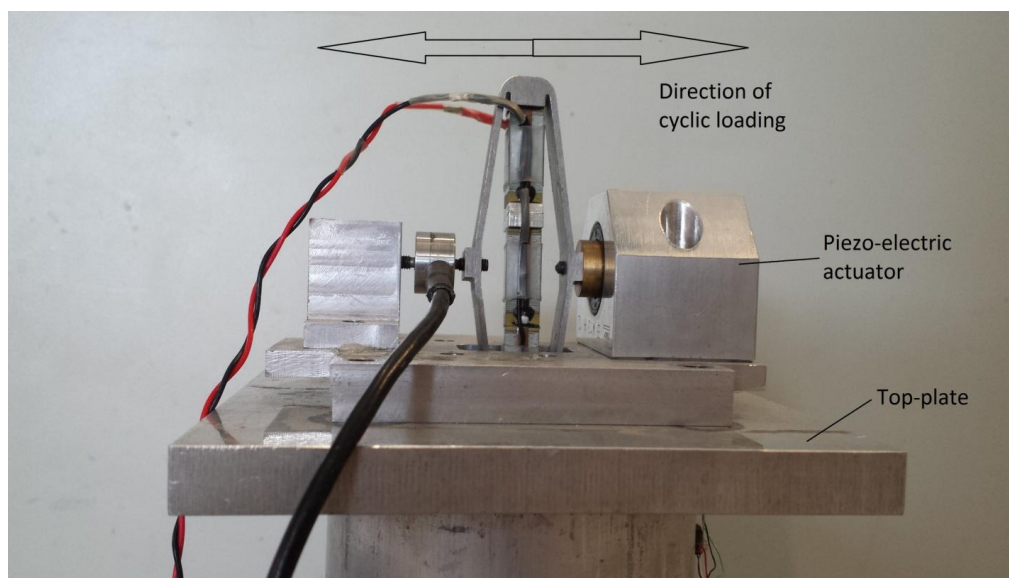


Figure 16: Top-plate and piezo-electric actuator on top of Al-pile

3.5. Impact tests on model monopiles in sand

Impact tests were conducted on both aluminium and HDPE models, using the same set-up as with harmonic loading shown in figure 15. A hammer was used to apply an impulse at the top of the pile. The top plate and actuator were kept on the pile in order to be consistent between impact and harmonic tests, since the added mass on top of the monopile would affect the overall response of the system. A number of impulses were applied on each model, to improve reliability of the results and minimise error. Each impulse was recorded and analysed separately, using the top 1.7-g MEM to calculate the natural frequency of the soil-pile system under impact loading, and the MEMs in the soil to measure the wave shear velocity.

4. EXPERIMENTAL RESULTS AND DISCUSSION

4.1. Investigating the natural frequency of the soil-pile system

4.1.1. Tests on model monopiles fixed as cantilevers

As aforementioned, impact tests were conducted on each monopile when it was fixed as a cantilever, by clamping its base rigidly. Figure 17 shows the response of each MEM accelerometer attached on the HDPE model monopile for a single impulse test. Comparing the graphs displaying the response of MEM1 (1.7-g) and MEM 35g we can see that they measure the same acceleration more or less. Therefore, we can assume that the acceleration doesn't exceed the levels which can be measured by the 1.7-g. At later tests, MEM 35g had some wiring problems that affected its readings. Therefore, for consistency purposes, MEM1 was used instead of MEM 35g during the analysis of all the tests, when trying to determine the natural frequency of the structure. Moreover, from figure 17(c) we can see that MEM2, which was attached in the middle of the pile and at a perpendicular orientation to the direction of the load, gives a smaller acceleration with approximate half the magnitude than at the top of the pile measured by MEM1. This was expected, since the MEM accelerometers only measure acceleration in one direction. In fact, MEM2 should have given zero acceleration. The acceleration measured indicates that the impulse load was not applied perfectly in line with the MEMs we aimed (MEM1, MEM3 and MEM 35g), which is attributed to human error. Finally, figure 17(d) shows the response of the MEM attached at the bottom of the pile. It shows accelerations of the same magnitude as the ones measured by MEM1 but in opposite direction. This is because the vibrations are transferred through the body of the model pile, causing the bottom part to vibrate too and therefore experience the accelerations. However, they are in the opposite direction since, as we hit the top of the pile from one side, and the pile is displaced in the direction of the impulse load, the bottom is pushed in the opposite direction.

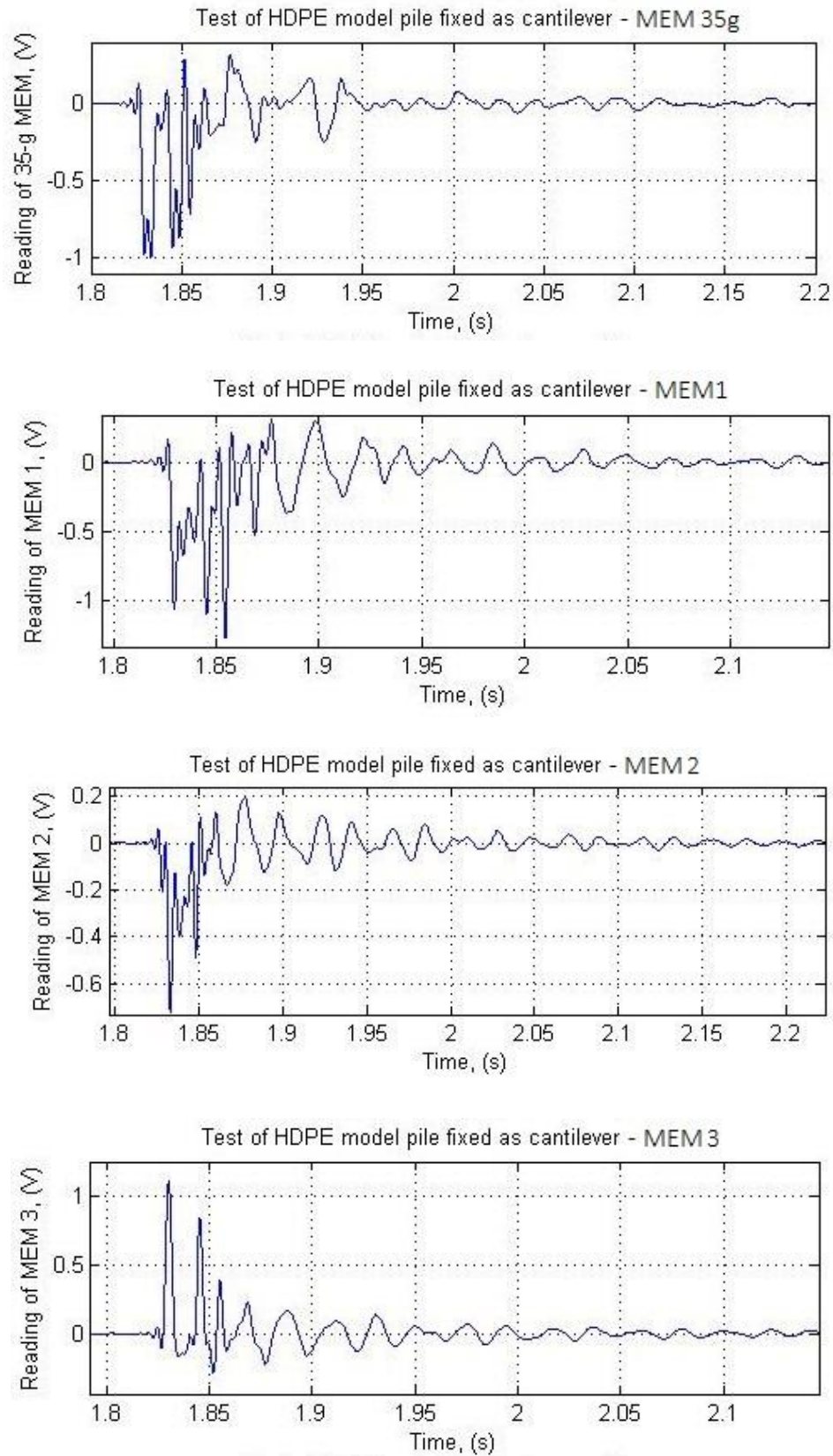


Figure 17: Response of each MEM on the HDPE pile when fixed as a cantilever and subjected to an impulse load

Several impulses were applied on each pile. The data for the 1.7g MEM attached on the top of the pile (MEM1) was analyzed and plotted in MATLAB, an example shown in figure 18. The peak with highest acceleration occurs at 124.2Hz, marking the natural frequency of the Al-pile.

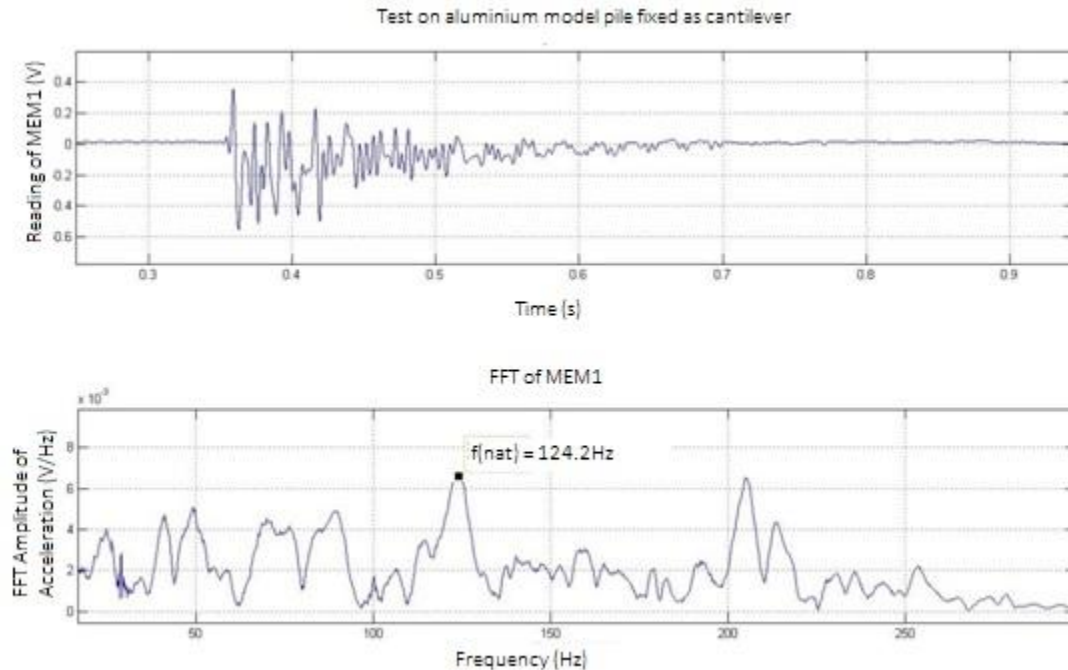


Figure 18: Example of an impact test on the aluminium pile fixed as a cantilever

The results for all the impulse loads on both model piles are shown in table 4. The average natural frequency found from the experiments is compared against the theoretical natural frequency, calculated using equation (1).

From the results, we can see that the theoretical value differs from the experimental results. What is surprising is the fact that for the Al-pile, the experimental f_{nat} is lower than the one predicted by equation (1) by about 14%, whereas for the HDPE-pile it is higher by 41%. The reason for this might come from the way the pile was clamped, which was done by applying a triangular point-force from the inside of the pile's base, shown in figure 12. The rigid base might have not held the Al-pile as rigidly as it was desired, leading to the lower natural frequency. Contrastingly, it might have stretched the base of the HDPE, since it is a much softer material, increasing its stiffness and therefore leading to a higher natural frequency than the one

expected. Maybe if we had the piles fixed in concrete, the experimental result would be closer to the theoretical calculation of f_{nat} .

Table 4: Results for the natural frequency of the model piles

Natural Frequency (Hz) of:	Aluminium pile	HDPE pile
Impulse test 1	124.2	31.1
Impulse test 2	131.8	34.5
Impulse test 3	124.2	30.2
Impulse test 4	123.9	28.5
Impulse test 5	130.9	32.0
Impulse test 6	126.6	34.9
Impulse test 7	127.0	33.7
Impulse test 8	118.4	34.3
Average f_{nat}	125.9	32.4
Theoretical value of f_{nat}	147.0	22.9

The big difference between the natural frequencies of the two piles is because of the different stiffness of the material they are made of. The natural frequency of the Al-pile is about 3.9 times larger than the natural frequency of the HDPE pile. This agrees closely with the prediction of the natural frequency of the Al-pile being about 4.4 times higher, as calculated using the fact that natural frequency is proportional to square root of stiffness (EI) divided by mass. Since the second moment of area of the two piles is very similar, it has very little effect on the natural frequency and can be ignored in the calculation. The calculation is shown below:

$$\frac{f_{nat(Al-pile)}}{f_{nat(HDPE-pile)}} \propto \sqrt{\frac{(EI/M_p)_{Al}}{(EI/M_p)_{HDPE}}} = \sqrt{\frac{(E/M_p)_{Al}}{(E/M_p)_{HDPE}}} = \sqrt{\frac{70GPa/0.475kg}{1.9GPa/0.245kg}} \approx 4.4 \quad (7)$$

The experimental results for the piles fixed as cantilevers seem to agree with the theory to some extent, but could be improved if the design of the experiment was better. We could have clamped the pile at the point where the embedment depth would be, so as to provide more accurate comparison of natural frequencies between the fixed-base pile and the soil-pile system. For that, a different way of fixing the pile would be required.

4.1.2. Impact loading of soil-pile models

A number of impact loading cycles were applied on each monopile, when embedded in Hostun sand. A set of tests was carried out on Al-pile in sand of relative density, $I_D=0.4$, and two separate sets of tests on the HDPE-pile, first embedded in sand of relative density $I_D=0.4$ and then in sand of relative density $I_D=0.55$. In every case, the response of the top 1.7-g MEM accelerometer (MEM1) was used to find the natural frequency of the monopile. As mentioned in section 3.1, MATLAB was used to convert the signal from time domain into frequency domain using fast Fourier transform (FFT). The peak in the graph of frequency domain denotes the natural frequency of the system. Figure 19(a) shows the response of MEM1 on the HDPE pile in sand of relative density 0.55 in the time domain, while figure 19(b) shows the frequency response once the fast Fourier transform function in MATLAB was applied. From the plot we can see that the fundamental frequency of the soil-pile system is at about 40.7Hz.

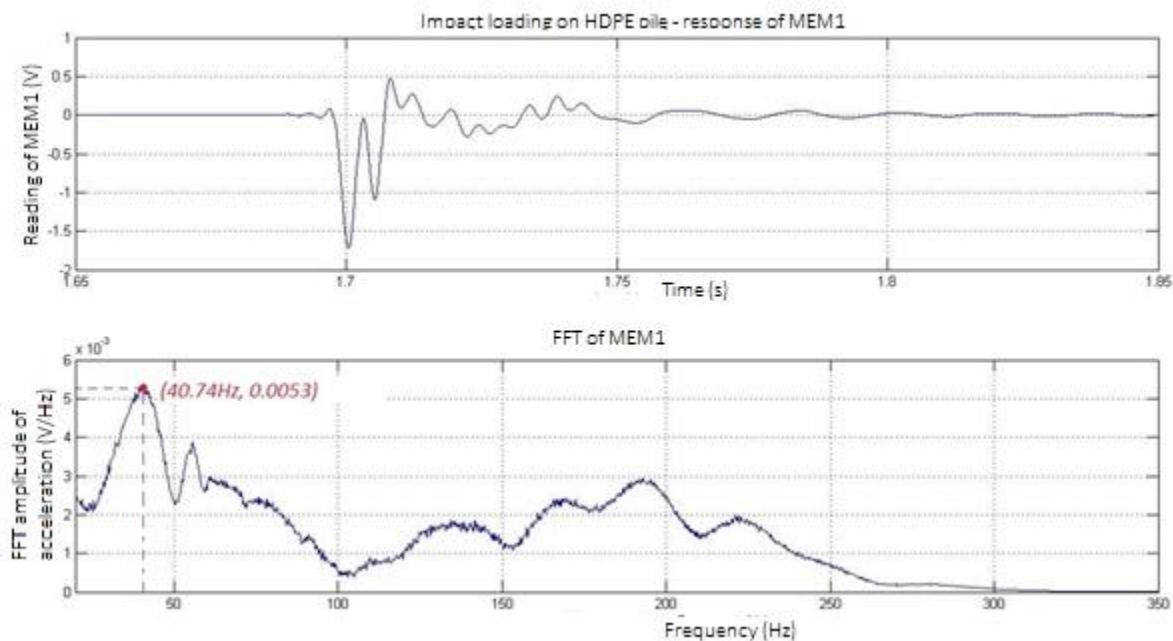


Figure 19: HDPE pile in sand of relative density 0.55. (a) Response of MEM1 in time-domain; (b) FFT response of MEM1

From the graphs of MEM1 in figures 17(b) and 19 (a), we can see that the vibrations from the impact load last much longer when the pile is fixed as a cantilever than when it is embedded in the sand. More specifically, for the fixed-base tests on the HDPE pile, the impulse last for about 0.35s (fig. 17(b)) while for the soil-pile system it lasts only about 0.05s (fig. 19(b)).

The same trend is observed for the aluminium model pile. This was expected since the soil increases the damping of the vibrations, shortening their duration.

Each impact loading was analyzed separately, and the natural frequency of the soil-pile system was determined using the FFT graph generated by MATLAB. Table 5 shows the results for each test carried on each monopile.

Table 5: Results for the natural frequency of each soil-pile model

Impact test No.	Fundamental Frequency, f_{nat} (Hz)		
	Al-pile ($I_D=0.4$)	HDPE-pile ($I_D=0.4$)	HDPE-pile ($I_D=0.55$)
1	159.4	37.5	41.5
2	150.1	38.4	40.6
3	173.5	37.8	40.3
4	156.5	38.5	40.9
5	160.5	36.0	39.4
6	156.9	39.2	38.6
7	154.4	39.1	39.5
8	152.2	38.8	42.4
Average	157.9	38.2	40.4
Standard deviation	7.18	1.05	1.23

The results for HDPE pile seem to be in high agreement, with an average $f_{nat}=38.2\text{Hz}$ for the pile in loose sand and $f_{nat}=40.4\text{Hz}$ for the pile in denser sand. The results for the aluminium pile show a slightly higher spread, but average at $f_{nat} = 157.9\text{Hz}$. The higher spread seems to be due to the result of impact test no.3, which is much higher than the other results. Ignoring this outlier, the remaining results are closely agreed.

Both the Al-pile and the HDPE-pile have higher fundamental frequency when embedded in soil compared to the cantilever tests and the theoretical value for f_{nat} (section 4.1.1.). This is explained by the fact that the soil increases the stiffness of the system due to its own stiffness and by decreasing the effective length of the pile. The soil stiffness also increases with increasing relative density, leading to an increase in the natural frequency. This is shown by the increase in the fundamental frequency of the HDPE pile in sand of relative density $I_D=0.55$ compared to the result of the pile embedded in looser sand. This relationship between relative density and natural frequency was examined and proven by Jamie Munro (2013).

Comparing the natural frequency of the soil-pile system between the Al-pile and the HDPE-pile embedded in sand of the same relative density ($I_D=0.4$), shows that the fundamental frequency for the Al-pile is about 4.1 times higher than the f_{nat} for the HDPE-pile. This difference in the values of f_{nat} is governed by the difference in the stiffness of the pile alone, since the soil stiffness is kept the same between the two tests by keeping the relative density of the soil the same. This difference in magnitude agrees with the expected difference for f_{nat} , calculated in equation (7). However, more tests are needed in order to say confidently that the relationship between natural frequencies of piles made from different materials (provided that they are embedded into soil of similar stiffness) can be calculated accurately and reliably using the ratio of bending stiffness over mass. Moreover, in real conditions, it is very unlikely that the soil stiffness will be similar between two different sites, or even within the same site.

Finally, the percentage difference between the impact test on the piles embedded in soil and the theoretical value of f_{nat} for the piles fixed as cantilever differs greatly between the Al-pile and the HDPE-pile. For the Al-pile, there is an increase of approximately 7.4%, while for the HDPE-pile the increase in natural frequency is more than 66%. This can be explained by the relationship between the pile's stiffness and the soil's stiffness. For the Al-pile, the stiffness of the pile is much greater than the stiffness of the soil (stiff-soft combination); therefore the soil stiffness has a small effect on the natural frequency of the combined soil-pile system. The opposite happens with the HDPE-pile, which has a much more similar stiffness to the soil (soft-soft combination); therefore when the pile is embedded in the sand, the sand increases the stiffness of the soil-pile system to a larger extent.

It is important to note that calculating the relative density of the sand was quite inaccurate, due to the nature of the equipment (hopper, tub etc) and the large volumes of sand involved. A much better method of making the sand model would be to use the mechanical sand hopper, which fills up the tub accurately to the desired relative density, creating an isotropic and homogeneous model. However, this was not accessible for this project, due to the fact that it takes time to fill up the tub and priority was given to the PhD students to use it for their experiments. Also, more tests could be carried out in different soil-pile models of the same relative density to improve reliability of results and minimise errors. Finally, tests of piles (both

Al and HDPE, or even different material) in sand of different relative density could also be contacted to provide more data for comparison between piles of different materials. However, there was no time to complete these for this project.

4.1.3. Harmonic loading of soil-pile models

Once the impact tests on each soil-pile model were completed, the pile was subjected to a dynamic loading produced by the piezo-electric actuator. Discrete frequencies in the general range of 30-200Hz were applied, each loading cycle incrementing the forcing frequency by 10Hz. This was analyzed using MATLAB in the same way as in the previous tests, producing graphs that look like the ones in figure 20. Figure 20(a) shows the response of MEM1 when the HDPE pile, embedded in sand of relative density 0.4, is loaded at 50Hz, first in time domain and then in frequency domain once the FFT function in MATLAB was applied. Similarly, figure 20(b) shows the response of MEM1 at 60Hz and figure 20(c) shows the response of MEM1 at 70Hz. If we see the graph for the FFT of MEM1 in figure 20(c), we clearly see the first harmonic at about 70Hz, which was the forcing frequency. This makes sense, since the monopile will vibrate at the same frequency as its input load. The acceleration amplitude of the first harmonic recorded by MEM1 was noted for each forcing frequency, indicated by the points on the graphs in figure 20, in order to observe the change in acceleration amplitude with the change in forcing frequency. The results for each pile were plotted in figure 21.

For the HDPE piles, the first peak signifies the fundamental frequency. The tests show a natural frequency of about 50Hz at loose soil and 70Hz at dense soil. In the case of the Al-pile, the natural frequency is believed to be about 170Hz, with the peaks at lower frequencies denoting excitation of the equipment. This is because we expect the natural frequency of the system under dynamic loading to be higher than the respective natural frequency under impact loading, which is explained by the difference in the nature of the loading. Harmonic loading applies a lower strain on the soil resulting in a higher shear modulus (as shown in figure 10), increasing soil stiffness and therefore the natural frequency of the pile embedded in it. The opposite happens with impact loading.

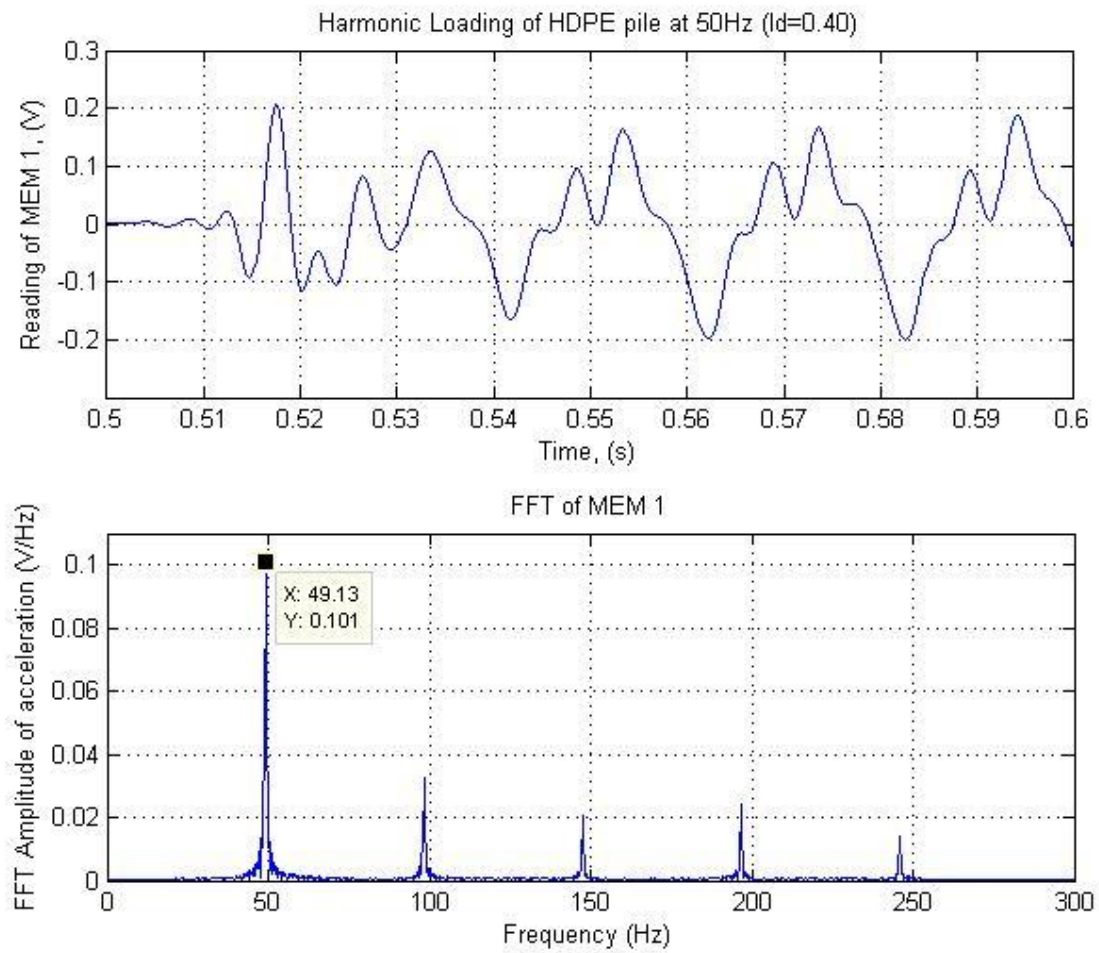


Figure 20: (a) Frequency response of the HDPE-pile in sand ($I_D=0.4$) at a forcing frequency of 50Hz

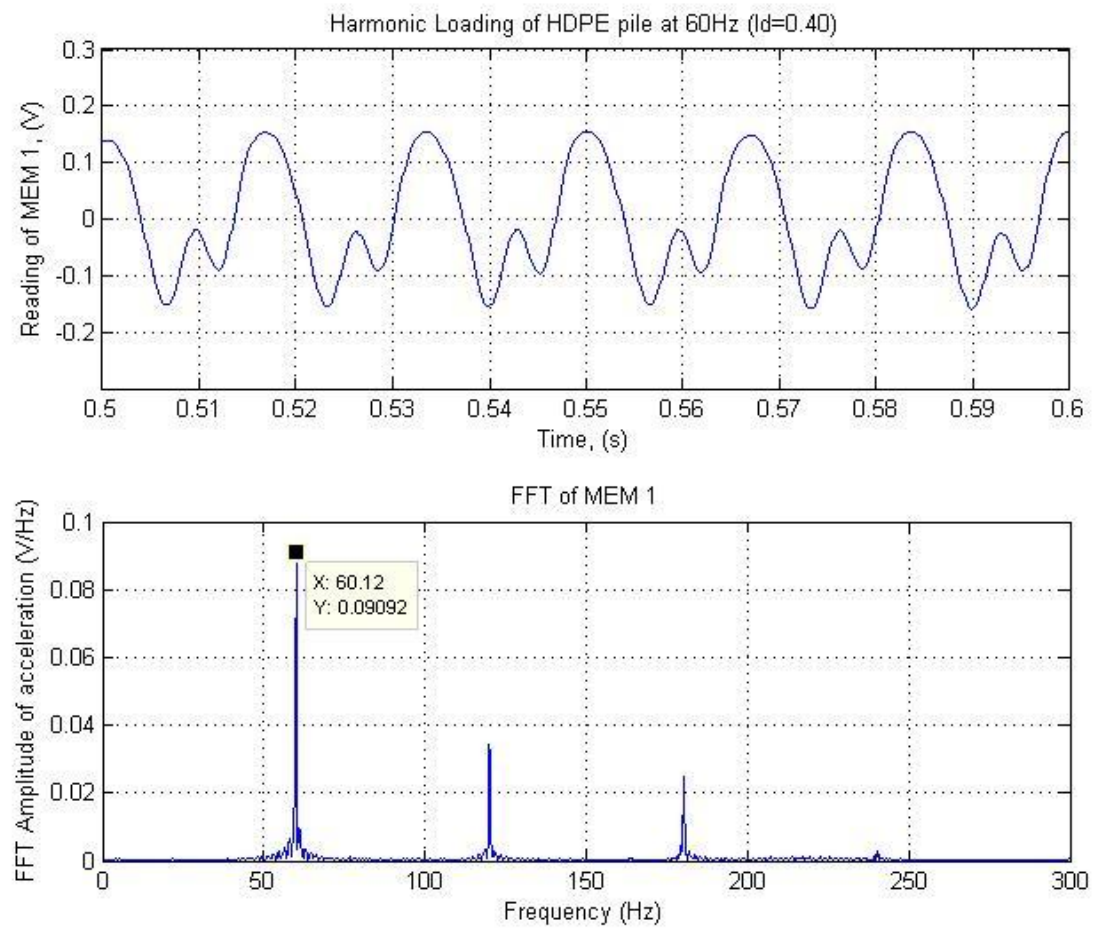


Figure 210: (b) Frequency response of the HDPE-pile in sand ($I_D=0.4$) at a forcing frequency of 60Hz

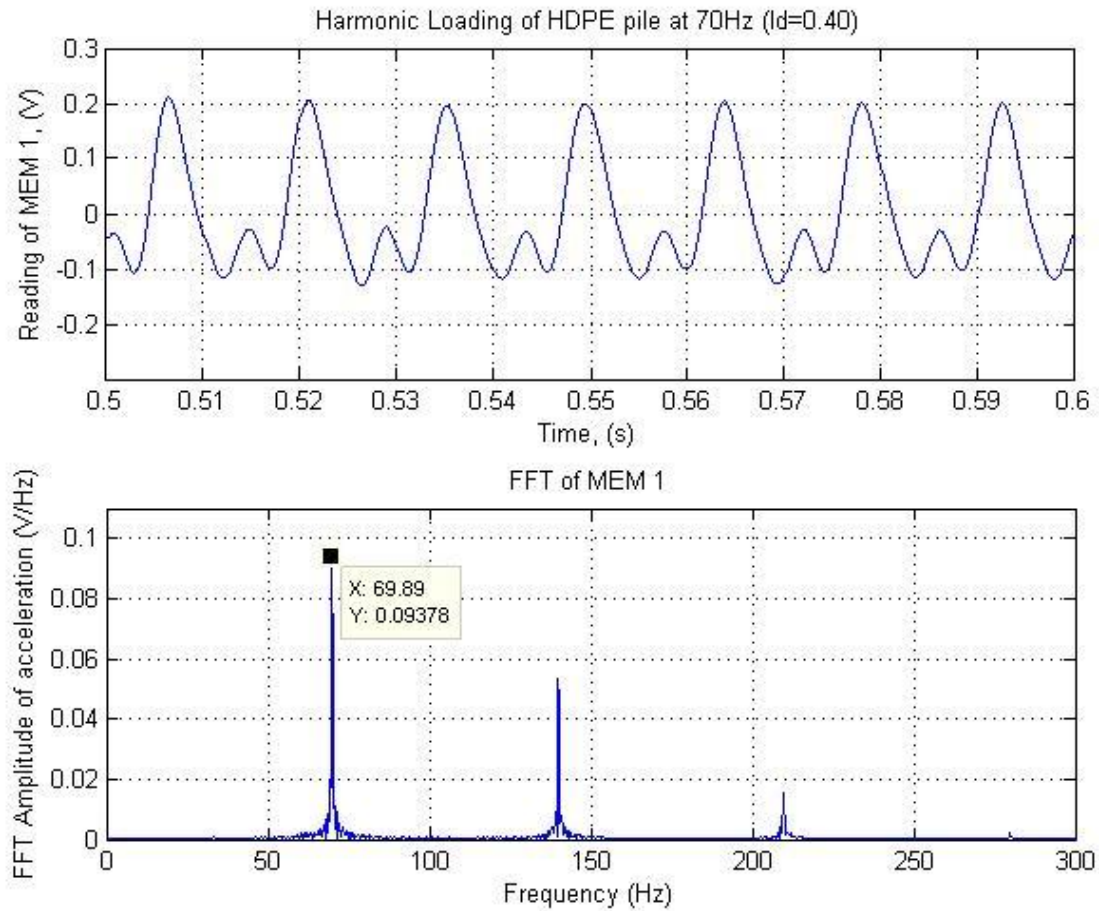


Figure 220: (c) Frequency response of the HDPE-pile in sand ($I_D=0.4$) at a forcing frequency of 70Hz

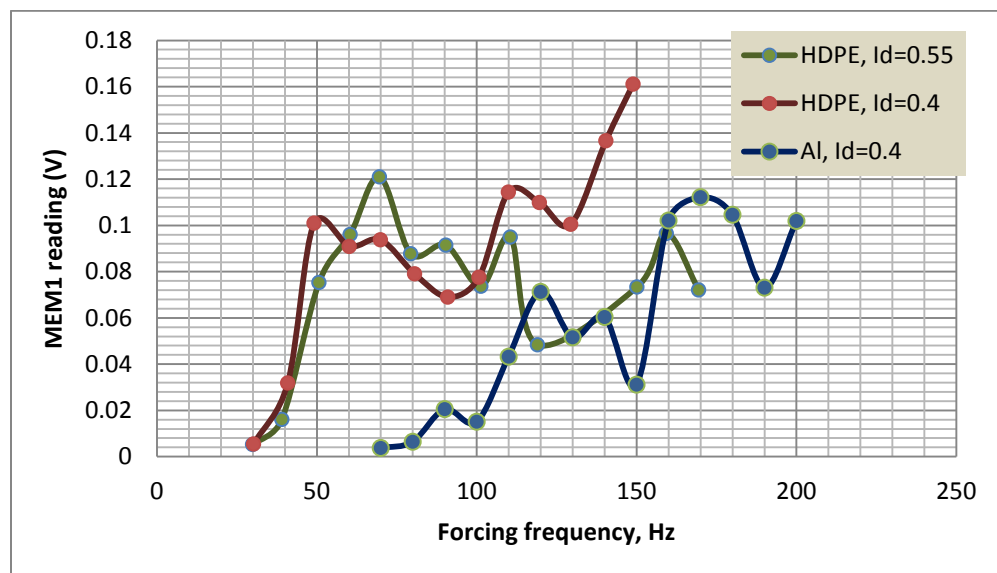


Figure 231: Response of MEM1 under harmonic loading at different forcing frequencies

Moreover, as expected, the natural frequency of the HDPE pile increases with the increased relative density of the soil since soil stiffness increases. Also, it can be seen that, in general, the magnitude of the acceleration (proportional to the voltage reading of MEM1) increases with higher forcing frequencies. This is natural, since higher frequencies mean that the actuator shakes faster, making the same displacement motion in less time. Finally, in this case, the natural frequency of the Al-pile is only 2.4 times bigger than the one for the HDPE-pile. This differs a lot from the prediction of 4.4 but more investigation needs to be done in order to disregard this method for predicting the natural frequency of piles with different material stiffness.

It should be noted that at lower frequencies (10-30Hz), the actuator couldn't provide a smooth sinusoidal input and electrical noise was high, resulting in bad readings which were disregarded in the analysis. At too high frequencies (>170Hz) the accelerations became too high for the MEMs to measure. These, however, do not affect the dynamic response of the piles, since the natural frequencies we were expecting were neither too low nor too high.

A better experiment would be to load the pile continuously rather than in discrete frequencies. This would give a better idea of the natural frequency since it would peak exactly where f_{nat} occurs. Also, gathering discrete data means that the graph plotted above is governed by the number of points, and might lose peaks in-between the points we had. However, this was not possible with our actuator as there was a time lag between input and output which resulted in huge amounts of data and a lot of noise.

4.2. Shear wave velocity

4.2.1. Experimental results and comparison with theory

The response from the MEM accelerometers in the soil, placed in a horizontal array as shown in figure 14, is used to calculate the shear wave velocity in the sand. This was done by measuring the time difference between two peaks between two MEMs. Since the distance between the two MEMs is known, the velocity of the wave could be calculated. An example of this is shown in figure 22. The distance between MEM6 and MEM7 is 38mm, while the time

difference between the peaks at the two MEMs is about 0.6ms. Therefore, the shear velocity is found to be 63.3m/s.

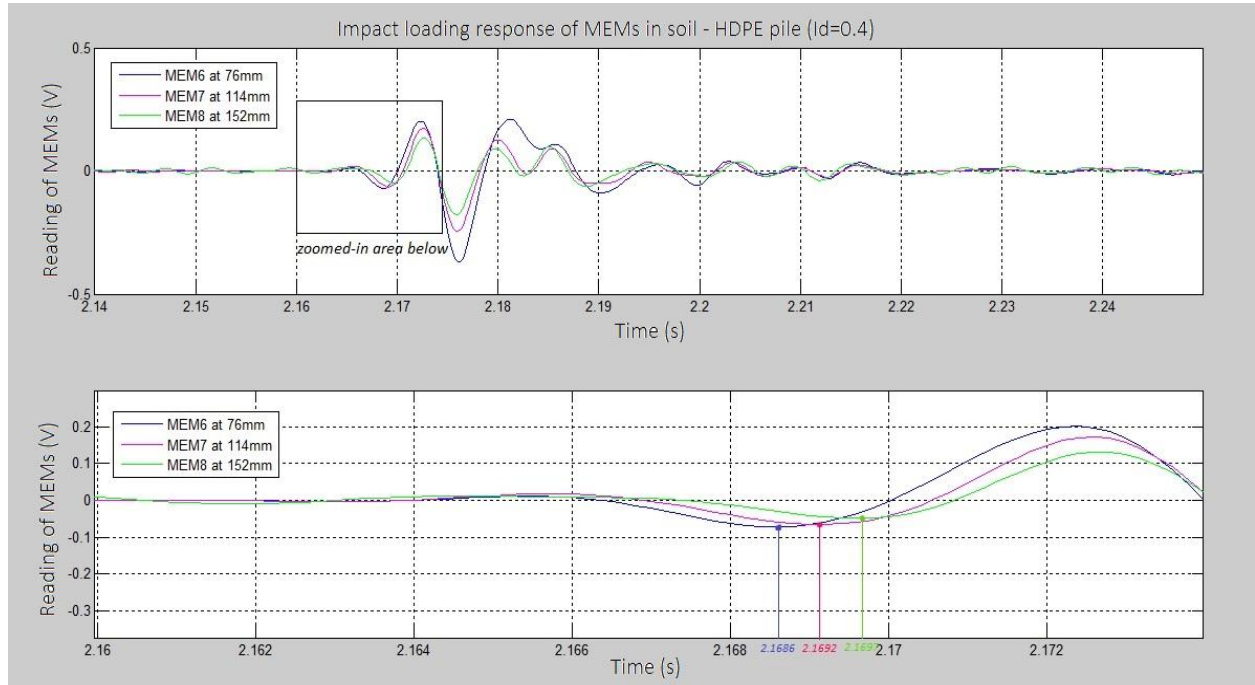


Figure 242: Measuring the time difference between the first peak at each MEM

This method of analysing the data was carried out for all the different soil-pile models. The results, using MEM6 and MEM8, are shown in table 6. The theoretical value of the shear velocity was calculated using equations (5) and (6), revised here:

$$v_s = \sqrt{\frac{G_0}{\rho_{sand}}} \quad (5)$$

$$G_0 = 100 \frac{(3-e)^2}{1+e} (p')^{0.5} \quad (6)$$

Voids ratio, e , was calculated using the equation for relative density, I_D :

$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}} \quad (8)$$

Mean effective stress, p' , was calculated using the following equations:

$$p' = \frac{\sigma'_v + 2\sigma'_h}{3} ; \sigma'_v = \frac{G_s \gamma_w z}{1+e} ; \sigma'_h = K_0 \sigma'_v ; K_0 = \frac{v}{1-v} \quad (9)$$

where: $v=0.25$, $G_s=2.65$, $z=0.04m$, $\gamma_w=9.81kN/m^3$.

Table 6: Results for shear wave velocity

	Shear Wave velocity in sand (m/s) from:		
Impact test No.	Aluminium pile test ($I_D=0.4$)	HDPE pile test ($I_D=0.4$)	HDPE pile test ($I_D=0.55$)
1	57.0	31.7	76.0
2	45.6	42.2	54.3
3	63.3	54.3	58.5
4	60.0	63.3	43.7
5	57.0	31.7	63.3
6	54.3	54.3	50.0
7	60.0	76.0	40.0
8	57.0	29.2	55.9
Average	56.8	47.8	55.2
Standard deviation	5.28	16.97	11.35
Theoretical	55	55	59

The Al-pile causes the highest shear wave velocity, at 56.8m/s. The relative stiffness between the pile and the soil can play a role in the shear wave velocity. In the case of the aluminium pile, the relative stiffness is high and therefore it leads to smaller soil strains. At smaller strains, the shear modulus degradation is small (shown by figure 10) leading to higher shear wave velocities. The opposite happens in the case of the HDPE pile, where the relative stiffness is small and therefore the strains are larger, leading to a higher degradation of the stiffness modulus and smaller shear wave velocities. The experimental results confirm this theory. However, it should be noted that the high spread of the results for the HDPE pile make the average value doubted as to its validity. The results for the Al-pile, on the other hand, are in high agreement, with a standard deviation of 5.28.

The HDPE-pile causes a higher shear wave velocity when embedded in denser sand, which is expected since higher relative density means the soil particles are closer together and therefore can transfer vibration faster. The shear velocity resulting from the denser soil is closer to the theoretical value than the one in loose soil, when an impact load is applied to the HDPE pile. The spread is also smaller for the denser sand model.

4.2.2. Propagation of shear waves through the soil

Looking at figure 22, we can clearly see that under impact loading, the further away we move from the pile, the smaller the amplitude of the response measure by the MEMs. This is natural, since the soil attenuates the shear waves and dissipates their energy.

Examining the MEMs' response under harmonic loading (fig. 23), we can see that the MEM closest to the pile ($R=38\text{mm}$) peaks close to the natural frequency of the system. This graph also has the same shape as the one in figure 21, showing that this MEM is heavily affected by the pile. However, the other three MEMs peak at around 110-120Hz. This might be due to superposition of the waves from the pile and the waves reflected from the wall of the tub, which is reinforced by the fact that the relative amplitudes of the MEMs compared to one another change through the range of frequencies applied.

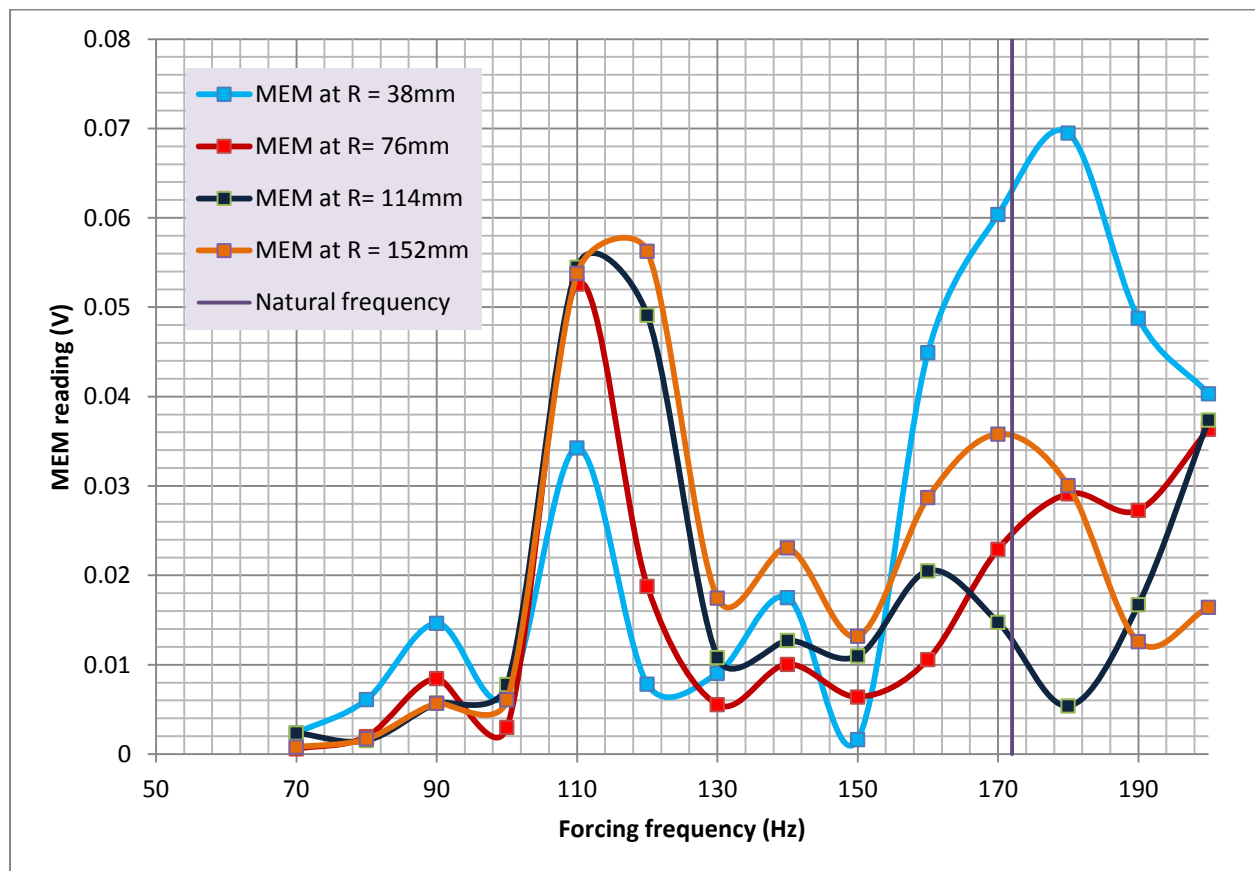


Figure 253: Variation of acceleration amplitude (proportional to MEM reading) within the soil due to changing forcing frequencies

4.2.3. Limitations of the model

The main limitation of the model was that tests at 1-g acceleration level do not impose as high confining stresses as one would find in the field. Ideally, the tests should have been carried out in the centrifuge at higher 'g' levels, but that was beyond the scope of this project.

One of the limitations for this project was the fact that the pouring of the sand in the tub was done manually. This decreases accuracy with which the sand is laid, which might result in non-homogenous, non-isotropic soil. It was also difficult to ensure that the relative density of the sand was the same throughout the sand model, or that it's exactly what we wanted, especially since it was impossible to measure the relative density of the sand after it was poured without disturbing the model. Calculating the relative density was done by filling a container of known volume with a mass of sand that we measure, and by knowing the Poisson's ratio and the specific gravity we could calculate the relative density. This method assumed that the sand poured in the container was uniform, and the mass and volume of sand were accurately measured. Using the automatic sand pourer would have improved the model, since it ensures a constant relative density throughout the model. However, as mentioned above, it was more time consuming than the manual sand pourer.

Another issue was instrumentation. MEM accelerometers had to be placed near the surface of the sand, since here the accelerations of the pile were higher. However, this made them very easy to move, either by accidentally touching their wires or due to the movement of the pile since the nearest one only 38mm away. Another problem was that they were placed at very close intervals. Since the distance between them is so small, and the shear wave velocities are relatively high, any small displacement of the MEMs would change the measured speed. However, it was not possible to place the MEMs further apart from each other, as the acceleration in the soil die quickly and the MEMs would not be able pick them up. Also, further away, noise and filtering would become dominant since the accelerations would be smaller.

The final concern involves the tub. The tests are assumed to be carried out in a semi-infinite plane, which is not exactly the case since the tub is only 850mm in diameter. However, MEM8 is 235mm from the tub boundary. This is far enough from the boundary, to assume that

reflections of the waves off the tub boundaries do not reach this MEM is the timeframe we are using to find the shear wave velocity.

5. CONCLUSIONS

The aim of the project was to examine the dynamic behaviour of monopile foundations for offshore wind turbines. More specifically, the natural frequency of two piles made from different material, aluminium and high-density poly-ethylene (HDPE), was to be investigated under both impact and harmonic loading. The piles were tested alone, having their base rigidly fixed so as to act as cantilevers, and embedded into a sand model. All tests were conducted at 1-g acceleration conditions.

Firstly, the piles were tested on their own, fixed as cantilevers. MEM accelerometers were attached on them to measure the acceleration of the pile. The top MEM was used to find the natural frequency of the pile, using the fast Fourier transform method. The experimental results showed a small discrepancy from the theoretical value of the natural frequency. This was thought to be due to the clamping method used to fix the base of the pile. However, the ratio of the natural frequencies between the aluminium and HDPE pile was as expected by the theoretical calculation.

Then the piles were embedded into Hostun sand. They were first tested under impact loading, from which the natural frequency of the soil-pile system was examined. The results for both piles conveyed higher fundamental frequencies, which were expected since the sand adds to the stiffness of the pile. Moreover, higher relative density of the sand increased the natural frequency of the system, since it increases the stiffness of the soil.

The final tests involved harmonic loading of the piles. Different forcing frequencies were applied to the top of the pile using a piezo-electric actuator. The acceleration at each forcing frequency was calculated and then they were plotted against each other to determine the fundamental frequency. The soil-pile system for both piles has a higher natural frequency, since the harmonic loading applies a lower strain on the sand, increasing the shear modulus and therefore the stiffness of the soil. Furthermore, this confirms that increased relative density increases the natural frequency. However, under harmonic loading, the ratio between the two piles' natural frequencies is decreased compared to the previous tests, possibly because the equation of natural frequency doesn't take the soil stiffness into account.

The shear wave velocity was finally calculated to characterize the soil. It was calculated using the impact tests' data, by measuring the time delay between the maxima between two MEMs of known distance in the soil. For the aluminium pile, the calculated shear wave velocity was in agreement with the theoretical value, while for the HDPE piles it was slightly lower. Another observation is that the relative density affects the shear wave velocity: as relative density increases, shear wave velocity also increases.

These results provide an insight into the behaviour of monopiles used for wind turbines under dynamic loading using simplified models. They could not be used for designing the monopiles, since in reality there are many complications and variations between, and even within, sites. Moreover, the effect of one monopile on another needs to be considered when wind farms are designed. However, they are ideal for isolating different conditions and examining separate variables independently. They provide the stepping stone for more in-depth, analytical research that will be able to combine results from different tests to provide a more holistic understanding of monopiles used as turbine foundations.

6. FURTHER RESEARCH

Similar tests to the ones conducted in this project could be carried out to complement and reinforced the results presented here. Firstly, more tests could be carried out on both piles embedded in sand of different relative densities, in order to establish the relationships suggested here. Then, other properties of the soil-pile model could be modified. For example, tests could be carried out in fully saturated sand, since this is the condition that will be faced in reality. Clay could also be investigated and then compared to sand results, in order to determine the applicability of monopiles in the two soil types. And then, the effect of having layers of soil of different properties, whether relative density or type of soil, could contribute to the research and deepen the understanding of monopile behaviour in real situations, where there is very likely to have strata of soils of different properties.

Moreover, it would be interesting to examine the effect of total and embedded length on the natural frequency response of the monopile. As turbines are getting larger, they will require longer monopiles to support them, therefore a study of how the increased length would change the dynamic response of the soil-pile system would enhance the design process. Additionally, testing the interaction between multiple monopile foundations at 1-g acceleration levels is another option. The optimal distance needs to be found between them, where vibrations caused by a turbine do not have an adverse effect on another.

An issue indicated by the literature review is the fact that the current design method does not include the effect that the number of cycle-loading has on the turbine behaviour. Testing this response will help improve the design methodologies, making them more efficient and ultimately decreasing the cost of the turbine foundations, making the whole structure more cost effective and economically viable.

Finally, more 'realistic' models can be tested. A geotechnical centrifuge can be used to simulate field stresses and strains on scaled models, predicting the real-life response of the structure and therefore being able to use the results directly in the design of full-size monopile foundations. Moving outside the lab, monopiles of offshore wind turbines that are being installed currently can be instrumented and monitored in order to provide a direct view of the

behaviour of the turbine under the loading it is subjected to throughout its lifetime, and examine the ground borne vibrations around it. These results can also aid in designing more targeted laboratory experiments to investigate a specific behaviour.

REFERENCES

- Achmus, M., Kuo, Y.-S. & Abdel-Rahman, K., 2009. Behavior of monopile foundations under cyclic lateral load. *Computers and Geotechnics*, 36(5), pp.725–735. Available at: <http://linkinghub.elsevier.com/retrieve/pii/S0266352X08001596> [Accessed May 17, 2014].
- Adhikari, S. & Bhattacharya, S., 2012. Dynamic analysis of wind turbine towers on flexible foundations. *Shock and Vibration*, 19, pp.41–60.
- Amat, A.S., 2007. *Elastic stiffness of Hostun sand*. University of Bristol.
- Bhattacharya, S. et al., 2013. Observed dynamic soil–structure interaction in scale testing of offshore wind turbine foundations. *Soil Dynamics and Earthquake Engineering*, 54, pp.47–60. Available at: <http://linkinghub.elsevier.com/retrieve/pii/S026772611300167X> [Accessed May 17, 2014].
- Bhattacharya, S. & Adhikari, S., 2011. Experimental validation of soil–structure interaction of offshore wind turbines. *Soil Dynamics and Earthquake Engineering*, 31(5-6), pp.805–816. Available at: <http://linkinghub.elsevier.com/retrieve/pii/S0267726111000054> [Accessed May 17, 2014].
- Bolton, M.D. & Oztoprak, S., 2013. Stiffness of sands through a laboratory test database. *Géotechnique*, 63(1), pp.54–70. Available at: <http://www.icvirtuallibrary.com/content/article/10.1680/geot.10.P.078>.
- Brennan, A.J., Thusyanthan, N.I. & Madabhushi, S.P.G., 2004. *Evaluation of Shear Modulus and Damping in Dynamic Centrifuge*,
- Brødbæk, K.T. et al., 2009. *Review of p-y relationships in cohesionless soil*,
- Burke, J., 2012. *The dynamic response of model monopiles*. University of Cambridge.
- Byrne, B.W., Leblanc, C. & Houlsby, G.T., 2010. Response of stiff piles in sand to long-term cyclic lateral loading. *Géotechnique*, 60(2), pp.79–90. Available at: <http://www.icvirtuallibrary.com/content/article/10.1680/geot.7.00196> [Accessed May 8, 2014].
- DNV, 2004. *Offshore Standard: Design of offshore wind turbine structures*, Hellerup, Denmark.
- European Wind Energy Association (EWEA), 2014. *The European offshore wind industry - key trends and statistics 2013*,
- Hardin, B.O. & Drenvich, V.P., 1972. Shear modulus and damping in soils: design equations and curves. *Journal of Soil Mechanics and Foundation Division*, 87(7), pp.667–691.

- Huhne, C. et al., 2011. *UK Renewable Energy Roadmap*,
- LeBlanc, C., 2009. *Design of Offshore Wind Turbine Support Structures*. Aalborg University, Denmark.
- Lombardi, D., Bhattacharya, S. & Muir Wood, D., 2013. Dynamic soil–structure interaction of monopile supported wind turbines in cohesive soil. *Soil Dynamics and Earthquake Engineering*, 49, pp.165–180. Available at: <http://linkinghub.elsevier.com/retrieve/pii/S0267726113000171> [Accessed May 17, 2014].
- Malhotra, S., 2011. Selection , Design and Construction of Offshore Wind Turbine Foundations. In Dr. Ibrahim Al-Bahadly, ed. *Wind Turbines*. InTech, pp. 231–264. Available at: <http://www.intechopen.com/books/wind-turbines/selection-design-and-construction-of-offshore-wind-turbinefoundations>.
- Muir Wood, D., Lombardi, D. & Bhattacharya, S., 2011. Similitude relationships for physical modelling of monopile-supported offshore wind turbines. *International Journal of Physical Modelling in Geotechnics*, 11(2), pp.58–68. Available at: <http://www.icevirtuallibrary.com/content/article/10.1680/ijpmg.2011.11.2.58> [Accessed May 17, 2014].
- Munro, J., 2013. *Modelling the dynamic response of monopile foundations for wind turbines*. University of Cambridge.
- Rahman, M.M., Cubrinovski, M. & Lo, S.R., 2012. Initial shear modulus of sandy soils and equivalent granular void ratio. *Geomechanics and Geoengineering: An International Journal*, 7(3), pp.219–226.
- Reese, L. & Matlock, H., 1956. Nondimensional solutions for laterally loaded piles with soil modulus assumed proportional to depth. In *8th Conference of Soil Mechanics and Foundation Engineering*. Austin, University of Texas, p. 29.
- Santos, J.A. & Gomes Correia, A., 2000. *Shear modulus of soils under cyclic loading at small and medium strain level*,
- Tempel, J. Van Der & Molenaar, D., 2002. Wind Turbine Structural Dynamics – A Review of the Principles for Modern Power Generation , Onshore and Offshore. *Delft University, Wind Engineering*, 26(4), pp.211–220.
- Zania, V., 2014. Natural vibration frequency and damping of slender structures founded on monopiles. *Soil Dynamics and Earthquake Engineering*, 59, pp.8–20. Available at: <http://linkinghub.elsevier.com/retrieve/pii/S0267726114000086> [Accessed May 7, 2014].

APPENDIX 1: RISK ASSESSMENT RETROSPECTIVE

At the beginning of the project, a risk assessment was carried out to identify the potential risks involved with the project and suggest possible ways to manage them. The greatest concerns involved:

- Inhalation of dust and sand particles during pouring of sand, since the room in which the preparation of the models took place was small and didn't have enough ventilation – this was minimised by wearing a ventilated safety visor.
- Manual transferring and handling of heavy Hostun sand bags (25kg each) could cause back problems – the technicians helped me empty the bags into a large container from which I would take smaller amounts of sand and pour into the hopper. All tests were carried out while the technicians were in the lab.
- Typical lab hazards, such as tripping over wires – this was minimised by keeping the lab organised and neat, keeping equipment to its designated position.

The only issues that were overlooked were the implications concerning the amount of time spent using the computer while running the experiment, analysing the data and writing up the reports and presentations. Extensive exposure to the computer's screen light could cause problems with the eyes and headaches. This was eliminated by taking short breaks to rest my eyes. Also, assuming correct sitting position in front of the computer avoided neck and back problems.

In general, the risk assessment should be a continuous process throughout the lifetime of a project, since the project details such as experimental set-up and objectives might change during the completion of the project. This project ran smoothly from beginning to end, with no serious risks or dangers at any point in time.