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Case study for effects of pile installation on existing offshore facilities in brownfields

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ABSTRACT

New prop structures are meant to be used to strengthen existing ageing offshore platforms in brownfields. This technical note aims to assess the possible effects of pile installation of new prop structures on existing offshore facilities, specifically existing platform piles and nearby pipelines. Based on the case study presented, this work investigates the down-drag and interaction effects of installation-induced vibrations of new piles on the load-displacement response of existing piles, namely pile load-bearing capacity and settlement, as well as assessing vibration levels in the vicinity of existing pipelines.

1. Introduction

In offshore structures, it is common to extend the time use of an existing Well-Head Jacket (WHJ) platform by installing a new “prop” structure or small platform (see Figs. 1 and 2) to strengthen the adjacent existing ageing platform and provide new space for the jacket or topside of new facilities (e.g. risers, J-tubes, pigging, etc.). Pile driving of new props generates vibrations and shock waves that can cause physical damage to existing nearby piles and pipelines, thereby affecting the stability and support system of such facilities. Due to piling-induced vibrations, soil can undergo settlements due to densification or liquefaction and the potential magnitude of damage due to this process depends on several factors. These include the proximity of the existing piles/pipelines to the new pile installation site, the pile-driving method used, the size and type of pile being installed, the size and weight of the pile-driving equipment, the soil properties and conditions in the area, and the operating pressure of the existing piles/pipelines. The objective of this technical note is to assess the possible effects of pile installation of a new prop on the existing platform and facilities, including nearby piles and pipelines. This will be achieved in this work through a case study explained in detail in the next section.

2. Case study

As mentioned earlier, the installation effects of piles of new offshore props on existing offshore platforms and facilities will be investigated in this work through a case study. In the presented case study, a structural assessment of an ageing WJH (Fig. 1) was conducted to determine whether its life can be extended by additional 30 years. The assessment was performed to maximise the use of the existing WHJ platform and reduce the cost associated with the installation of a new WJH. The structural assessment concluded that a new prop structure is required for the following reasons: (1) there is no adequate space either on the jacket or the topside for the new facilities such as risers, J-tubes, pigging facilities, etc.; and (2) the jacket is not structurally sound due to insufficient pile strength/capacity and structural member/joint failures. Therefore, a prop structure was recommended to be installed adjacent to the existing platform to strengthen the existing ageing WHJ platform and provide space for the new required facilities. The new prop structure included 48-inch diameter piles with a wall thickness of 1.18 inches. The pile penetration length was 60 m for both new and existing piles. In addition, there were pipelines at a distance of 2 m away from the piles of the new prop (see Fig. 2). Due to the construction risk assessment associated with installing the new prop next to the existing WHJ platform, it was decided to ensure that there is a minimum separation distance of 5 m between the pile of the new prop and that of the existing

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Fig. 1. Typical example of WHJ installed back in 1985.

WHJ platform (at the seafloor). It should be noted that the existing piles are battered and the proposed new piles are vertical. The distance between the existing pile and the new pile increases with depth below the seafloor, thereby reducing the interaction between the piles. However, for a conservative approach, the analysis of the current work assumed that the existing and new piles are both considered to be vertical with a pile centre-to-centre distance of 5 m.

Most of the granular soil layers across the field are medium dense to dense, thereby soil settlements due to pile installation-induced vibrations can be smaller (neglecting earthquake-induced liquefaction effects). However, some of the soil profiles across the field have a thick cohesive (clay) layer. Pile installation-induced vibrations can induce soil settlements due to the consolidation of the clayey layers through the dissipation of excess pore water pressure generated during vibrations. This consolidation-induced soil settlement can impose negative skin friction or down-drag forces onto the pile foundation, eventually altering the capacity of pile foundations. It is appropriate to consider the worst soil profile in terms of inducing significant settlements and altering the response of existing piles. The Generalised Profile 4 (GP4) has the thickest clay layer seen across the three sites with clay extending from 21 m to 63 m depth (42 m thickness) and therefore is likely to suffer the greatest consolidation-induced settlement. For this assessment, an idealised soil profile consisting of 0 to 20 m carbonate sand, underlain by clay from 20 m to 65 m was considered in this study. Fig. 3 presents the adopted soil profile along with the associated geotechnical parameters. In this figure, the soil properties are given as follows: γ' is the submerged unit weight; q_c is the cone tip resistance; *PI* is the plasticity index; S_u is the undrained shear strength; and e_0 is the initial void ratio.

3. Methodology and analysis

In this section, the analytical methods used to investigate and analyse the impacts of installing new piles on existing nearby piles and pipelines are presented and the obtained results are discussed in the following section.

3.1. Influence on pipelines - determination of peak particle velocity

The mechanisms of energy transfer from driven piles into the ground were proposed by the Federal Highway Administration (FHWA) Synthesis 253 (Woods, 1997), as shown in Fig. 4 , for homogeneous soil. In an ideal pile-driving scenario, the pile tip can be considered the source of wave generation. As the pile is driven, body waves (P-waves) radiate from the pile tip in a spherical wavefront, while shear waves (S-waves) propagate from the pile shaft in a cylindrical wavefront. The interaction of these two wave types on the ground surface generates surface waves, previously believed to be Rayleigh waves (R-waves) (Hosking et al., 1988; Grizi et al., 2019). There are usually various criteria used to assess the damage that can happen to structures caused by ground vibrations from pile driving, many of which are derived from blasting experiments. Two types of empirical models for predicting ground motions have been established: (i) those that factor in the distance from the source; and (ii) those that factor in the scaled distance from the source. The distance used in such models is typically the horizontal distance from the source to the measurement point. However, when the source of vibration is not a point source, such as a pile, the actual distance from the measurement point to the source may not be horizontal (Grizi et al., 2018).

Eq. (1) is one of the earliest empirical correlations proposed by Bornitz (1931) for small amplitude motion at a long distance from an earthquake. However, Grizi (2018) concluded that Bornitz (1931) formulation exhibits a satisfactory correlation for soils consisting of sand and clay, and can be utilised in preliminary evaluations to ascertain the potential impact of pile-driving-induced vibrations. Nevertheless, Grizi (2018) emphasised that the soil attenuation process involves complex wave phenomena such as multiple reflections and refractions resulting from soil stratification, which are not encompassed by Bornitz (1931) method.

$$v_2 = v_1\left(\frac{r_1}{r_2}\right)^n \exp[-\alpha(r_2 - r_1)] \tag{1}$$

where:

v_1, v_2 = vibration amplitudes [m/s]
r_1, r_2 = distances [m]
n = coefficient depending on the wave type [m]
α = Attenuation coefficient [-]

Eq. (1) combines two forms of attenuation: the first part represents geometric damping with distance from the source, while the exponential part accounts for material damping as waves move through the soil. Values of the geometric damping coefficient, *n*, can be analytically determined from Gutowski and Dym (1976) and are shown in Table 1. The material damping coefficient, α, is frequency dependant and typical values as a function of the soil's standard penetration test (SPT) blow count (*N*) are shown in Table 2. It should be noted that Wiss (1981) has also provided some equations to determine the peak particle velocity but Bornitz (1931) equation was only considered in this paper, following the recommendations of Grizi (2018).

Heckman and Hagerty (1978) investigated pile driving cases and modified the scaled-distance relationship proposed by Wiss (1967) and Attewell and Farmer (1973), as follows:

$$v = K\left(\frac{D}{\sqrt{E}}\right)^{-1} \quad (2)$$

where:
v = peak particle velocity [m/s]
K = factor dependant on pile impedance ranging from 0.25 to 1.5 [-]
D = distance from the vibration source [m]
E = Impact energy transferred from the hammer to the pile [J]

Attewell and Farmer (1973) proposed a conservative value of $K = 1.5$. Wiss (1981) also suggested that it is convenient to have the distance and the energy from the source in a single expression, as shown in Eq. (3), as follows:

$$v = K\left(\frac{D}{\sqrt{E}}\right)^{-n} \quad (3)$$

where:
K = value of velocity at $D/\sqrt{E} = 1$ [m/s]
n = slope or attenuation rate [-]

A comprehensive literature review on several methods/equations to determine the peak particle velocity due to pile installation-induced vibrations can be found in Gkrizi (2017) and include some of the most recent methods to assess the vibration magnitudes. According to Gkrizi (2017), the maximum particle velocity generated by the shear between the pile shaft and soil can be estimated using an equation proposed by Massarsch and Fellenius (2008) as follows:

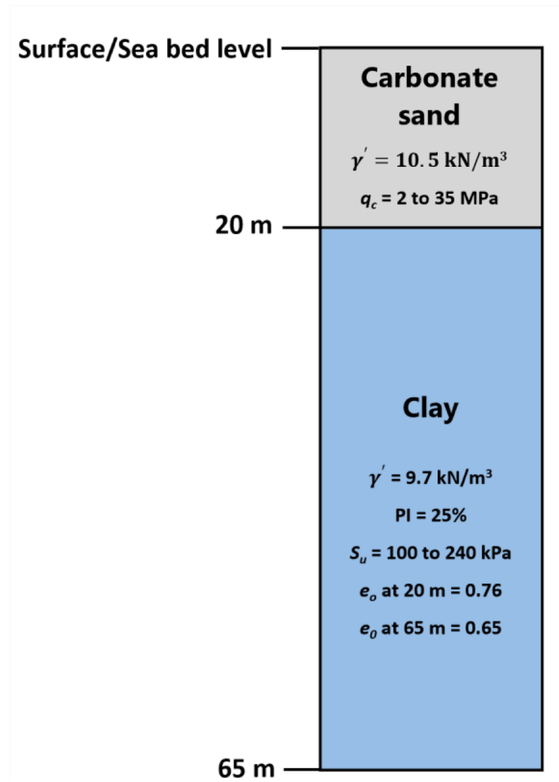

Fig. 3. Idealised soil profile and geotechnical parameters of the soil layers.

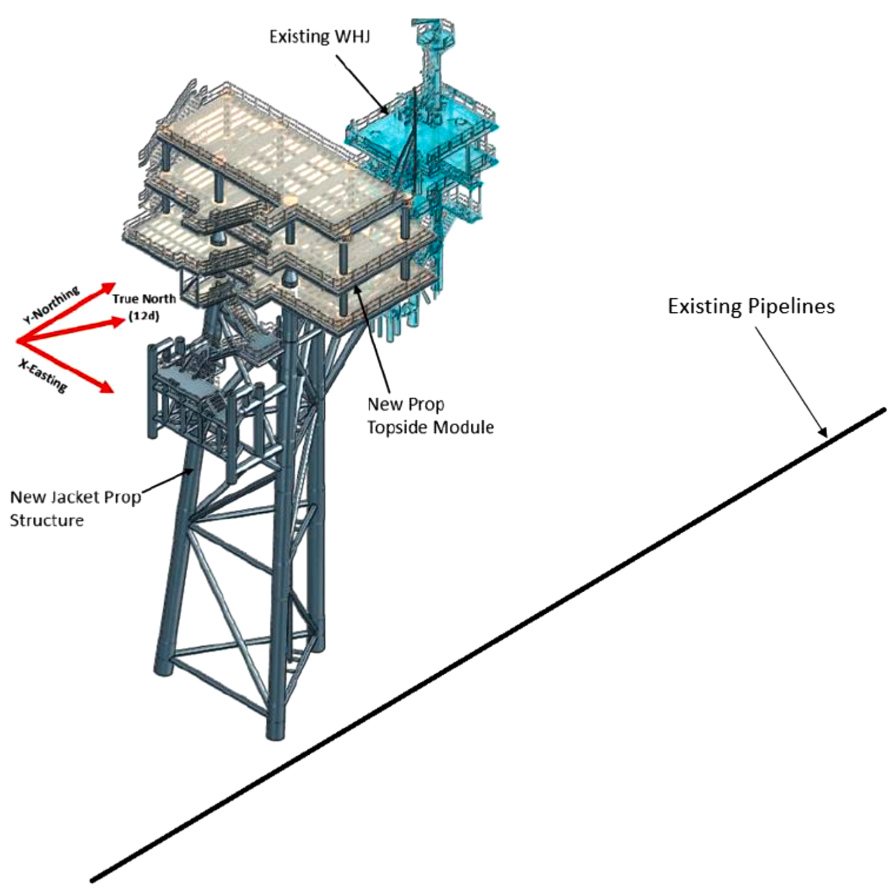

Fig. 2. Typical field scenario of new prop beside an existing WJH platform.

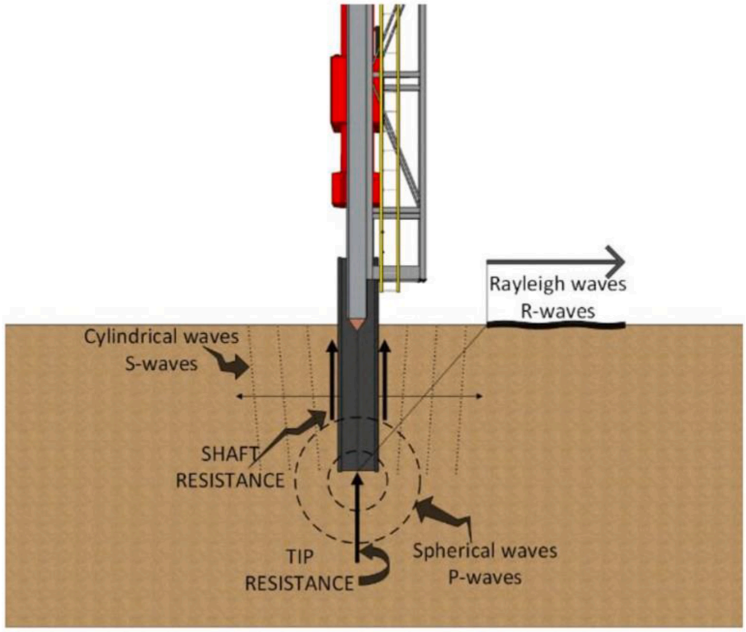

Fig. 4. Mechanisms of energy transfer from pile to soil (Grizi et al., 2016).

Table 1
Geometric attenuation coefficient (after Gutowski and Dym, 1976).

Source Type	Wave	Source Location	n
Line	Rayleigh	Surface	0.0
Line	Body	Surface	1.0
Point	Rayleigh	Surface	0.5
Point	Body	Surface	2.0
Buried Line	Body	Interior	0.5
Buried Point	Body	Interior	1.0

Table 2
Proposed classification of earth materials by attenuation coefficient (after Woods and Sharma, 2004).

Class	Attenuation coefficient, α at 5 Hz (1/m)	(1/ft)	Description of material
I	0.01–0.033	0.003–0.01	Weak or soft soils ($N < 5$)
II	0.0033–0.01	0.001–0.003	Competent soils ($5 < N < 15$)
III	0.00033–0.0033	0.0001–0.001	Hard soils ($15 < N < 50$)
IV	< 0.00033	< 0.0001	Hard competent rock ($N > 50$)

$$\dot{Z}_{shaft} = \frac{\tau}{\rho_s V_s^*} = \frac{\tau}{\rho_s R_s V_s} \tag{4}$$

where:

$\dot{Z}_{shaft}$ = peak particle velocity in the soil at the pile-soil interface [m/s]
τ = shear strength of soil [kPa]
ρ_s = mass density of the soil [g/cc]
V_s^* = shear wave velocity of soil at contact with the pile [m/s]
R_s = reduction factor [-]
V_s = shear wave velocity at low strains [m/s]

$$\dot{Z}_{tip} = 2R_R \frac{Z_s}{Z_p} (E_0)^{0.5} \cos\theta \tag{5}$$

where:

R_R = dimensionless correction factor accounting for soil compaction in granular soils and remoulding in cohesive soils [-]. It is equal to 2 for loose to medium-dense sand and ($0.2 < R_R < 0.5$) for normally consolidated to overconsolidated clay.
Z_s = impedance of soil at pile-tip [-] = $A_c \rho_s V_{sp}^*$
Z_p = impedance of pile at tip [-] = $A_c \rho_p V_p^*$
A_c = contact area between pile and soil [m^2]
V_{sp}^* = velocity of Biot wave of the second kind in soil [m/s]
V_P = compression wave velocity in pile [m/s]
E_0 = 0.5 times the rated energy of the hammer [kJ]
Θ = Angle between any ray of spherical wave and vertical [radians]

V^* is the velocity of the Biot wave of the second kind. This wave velocity is slightly lower than the primary wave velocity in the soil. Gkrizi (2017) recommended using the primary wave velocity (*Vsp*), which will be utilised to determine the strain resulting from pile penetration at the tip in the surrounding soil area at any given point, and (*p*) can be calculated as follows:

$$V_{sp} = kV_s \tag{6}$$

$$k = \left[\frac{2(1-\vartheta)}{(1-2\vartheta)}\right]^{0.5} \tag{7}$$

$$V_{sp}^* = R_s V_{sp} \tag{8}$$

where:

V_{sp} = primary wave velocity in the soil [m/s]
V_s = shear wave velocity at low shear strains [m/s]
k = dimensionless ratio [-]
υ = Poisson's ratio of soil [-]. It is equal to 0.2 for granular soils, yielding $k = 1.63$ and 0.45 for cohesive soils, yielding $k = 3.32$.
V_{sp}^* = reduced primary wave velocity in the soil based on strain amplitude [m/s]
R_s = Dimensionless reduction factor [-]

Gkrizi (2017) proposed Eq. (9) to determine the particle velocity amplitudes at different distances from the source, as follows:

$$\dot{z}_2 = \dot{z}_1 \left(\frac{r_1}{r_2}\right)^n \exp[-\alpha(r_2 - r_1)] \tag{9}$$

where:

$\dot{z}_2$ = particle velocity amplitude at point 2 [m/s]
$\dot{z}_1$ = particle velocity amplitude at point 1 [m/s]
R_1 = distance from source to Point 1 [m]
R_2 = distance from source to Point 1 [m]
n = power exponent depending on wave type [-]. It is equal to = 0.5 for a cylindrical wave coming from the pile shaft and 1 for a spherical wave coming from the pile tip
α = coefficient of attenuation [-]

The distance $r_1 = 1$ inch (≈ 0.1 ft) represents the first point where the maximum amplitude of soil motion right next to the pile shaft for cylindrical waves and below the pile tip for spherical waves is estimated (Gkrizi, 2017).

3.2. Influence on pile foundations - determination of pile down-drag effects

As discussed earlier, cohesive soil layers can undergo consolidation settlements due to the dissipation of excess pore water pressure generated during new pile installation. Under stable conditions, the soil-pile frictional resistance acts against the loading direction (frictional resistance acts upward under compressive loads and vice-versa). However, due to soil consolidation, the soil next to the pile generates resistance in the downward direction, which in turn reduces the compression capacity of the pile. This phenomenon is referred to as the pile "down-drag" effect. The soil consolidation around a pile foundation can have implications on its performance and stability, including:

- ***Pile compression:*** In some cases, soil consolidation can lead to compression of the pile itself. This compression can reduce the stiffness of the pile head load-deflection response and cause it to deform under load, affecting the stability of the structure supported by the pile.
- ***Pile settlement:*** Soil consolidation can result in the settlement of the pile, which can cause an increase or decrease in the load on the pile and potential damage to the structure supported by the pile as the load distribution in the structure is altered.

Therefore, it is essential to determine the possible soil consolidation settlements due to the new pile installation so that the influence of consolidation settlements on the existing pile can be evaluated. Consolidation settlements (Sc) can be evaluated using Terzaghi's one-dimensional compression equation, as follows:

$$S_c = \frac{C_C H}{1+e_o}\log\left(\frac{\sigma_o^- + \Delta\sigma^-}{\sigma_o^-}\right) \tag{10}$$

where:
C_C = compression index of soil [-] = 0.009 ($LL - 10$)
LL = liquid limit of soil [%]
H = thickness of soil layer [m]
E_o = initial void ratio [-]
σ_o^- = initial effective vertical stress [kPa]
$\Delta\sigma^-$ = change in effective vertical stress [kPa]

For pile installation-induced settlements, the excess pore water pressure (Δu) during pile installation is equal to Δσ'. The Δu during pile installation can be determined from the cylindrical cavity expansion theory, which assumes that during pile installation, soil displacement occurs similarly in areas away from both the pile tip and ground surface as it does in areas adjacent to an expanding cylindrical cavity. The Simple Cavity Expansion Theory, when applied to the soil with elastic and perfectly plastic properties, characterised by a shear modulus (G) and an undrained shear strength (s_u), generates an excess pore pressure distribution (Gibson and Anderson, 1961 and Randolph, 2003), as follows:

$$\frac{\Delta u}{s_u} = \ln\left(\frac{\rho_a G}{s_u}\right) - 2\ln\left(\frac{r}{R}\right) = \ln\left(\frac{G}{s_u}\right) - 2\ln\left(\frac{r}{r_{eq}}\right) \geq 0 \tag{11}$$

where:
ρ_a = area ratio = 1 − − $(r_{inner}/R)^2$ [-]
r = radial distance [m]
R = radius of the pile [m]
R_{eq} = radius of an equivalent solid pile that gives the same volume of displaced soil [m]
R_{inner} = inner radius of a tubular pile [m]

Eq. (11) does not consider the variations in mean effective stress that occurs as the soil is sheared and remoulded. These variations may be accounted approximately for lightly over-consolidated clays by adjustment of the rigidity index, $Ir = G/s_u$ (Randolph, 2003). Once the possible consolidation settlements are evaluated, the load-settlement behaviour of the pile with and without the down-drag effect can be evaluated. The load-settlement behaviour can be evaluated using a column and non-linear Winkler spring foundation analysis, where the pile is modelled as the column and the soil surrounding the pile is modelled as a non-linear Winkler foundation. In this analysis, the pile is supported by the soil springs, which represent the soil's capacity to carry the load and resist deformation. The non-linear behaviour of the soil springs is considered, allowing the analysis to account for soil stiffness that changes with increasing load. This type of analysis can be used to evaluate the load-bearing capacity and behaviour of pile foundations under different loading conditions and to design pile foundations that are better able to resist deformations and failure. If the consolidation settlements are large, a more comprehensive finite-element/finite-difference method needs to be employed to understand the effect of down-drag on existing piles.

4. Results and discussion

4.1. Installation effects on pipelines (at 2 m distant from installation)

As explained earlier in Section 3.1, various methods have been used in the literature to evaluate the vibration magnitudes at the pipeline level for the case study presented in Section 2. Some empirical solutions have specific dimensionless constants that are highly dependant on the site conditions. Therefore, for a reliable determination of vibration magnitudes, Bornitz (1931) method and Gkrizi's (2017) method were considered in this work, and Tables 3 and 4 give a summary of the input parameters used in both methods, respectively, and the obtained vibration magnitudes at a distance of 2 m from the new pile installation. Based on the results obtained, vibration magnitudes at the pipeline levels were in the range of 69.0 mm/s to 97.2 mm/s. British Standard (BS 7385- 2:1993) states that allowable vibration magnitudes for masonry/brick structures are in the range of 6 mm/s to 100 mm/s. Considering that pipelines are mostly made of steel, the maximum determined vibration magnitude of 97.2 mm/s indicates that the effect of new pile installation on existing pipelines is minimal.

Table 3
Vibration magnitude from Bornitz (1931) method.

Entity	Symbol	Value
Vibration amplitude at the source [m/*sec*]	v_1	1944.4
Distance 1 [m]	r_1	0.1
Distance 2 [m]	r_2	2.0
Coefficient	n	1.0
Attenuation coefficient	α	0.00033
Particle velocity at 2 m distance [mm/s]	v_2	97.2

Table 4
Vibration magnitude obtained from Gkrizi's (2017) method.

Entity	Symbol	Value
Pile outer diameter [m]	D	1.22
Wall thickness [m]	t	0.03
The contact area between pile and soil [m2]	A_c	0.112
The mass density of soil [g/cc]	ρ_s	1.8
The mass density of the pile [g/cc]	ρ_p	7.86
Compression wave velocity in pile [m/s]	v_P	5200
Energy induced by hammer [kJ]	E_h	300
$E0 = 0.5* Eh$ [kJ]	E_0	150
Shearing strength of soil [kPa]	τ	70
Shear wave velocity [m/s]	V_s	100
Reduction factor	R_s	0.3
shear wave velocity of soil at contact with the pile [m/s]	V_s^*	30
Dimensionless ratio	k	3.32
Reduced primary wave velocity in the soil based on strain amplitude [m/s]	V_{sp}^*	54.66
Dimensionless correction factor accounting for soil compaction in granular soils and remoulding in cohesive soils	R_R	0.3
Peak particle velocity in the soil at the pile-soil interface [mm/s]	$\dot{z}_{shaft}$	1944.4
The vertical component of particle velocity in the soil at the pile tip [mm/s]	$\dot{z}_{tip}$	5134.9
Coefficient of attenuation	α	0.00033
Particle velocity at 2 m distance (shaft) [mm/s]	$\dot{z}_2$	69.0
Particle velocity at 2 m distance (tip) [mm/s]	$\dot{z}_1$	6.5

4.2. Installation effects on pile foundation (at 5 m distant from installation)

The possible generation of the excess pore water pressures at a distance of 5 m from the new pile installation was determined using the Cavity Expansion Theory (Eq. (11)). For the analysis, G/s_u ratios of 1200 and 2000 were considered based on limited shear modulus (G) data available across the field. To account for larger strains imposed by pile driving, a strain correction factor of 0.7 was applied to the G/s_u ratio in Eq. (11). The 40 m clay layer was discretised into finite layers to compute the consolidation settlement in each layer using Eq. (10). The consolidation settlement over the 40 m clay layer was computed as 0.3 m and 1 m with G/s_u ratios of 1200 and 2000, respectively. The computed consolidation settlement in each layer was applied as a down-drag force on the existing pile to understand the consequences of down-drag on the existing pile behaviour. The finite element method was used to evaluate the load-settlement response of the existing pile with and without the down-drag effect. The down-drag settlements, which were utilised as input in the finite element, were calculated using the methodology outlined in Section 3.2. The finite element employed a 1D non-linear Winkler analysis, where the pile was represented as a column and the soil surrounding the pile was represented as a non-linear Winkler foundation. Fig. 5 shows the effects of down-drag on the load-displacement ($P - \delta$) response obtained from the finite element. The difference highlights the changes in the performance of the existing pile due to the new pile installation. As Figure indicates, with the inclusion of a down-drag for a G/s_u ratio of 1200, the pile initial response becomes softened but remains able to carry the existing applied load but with a

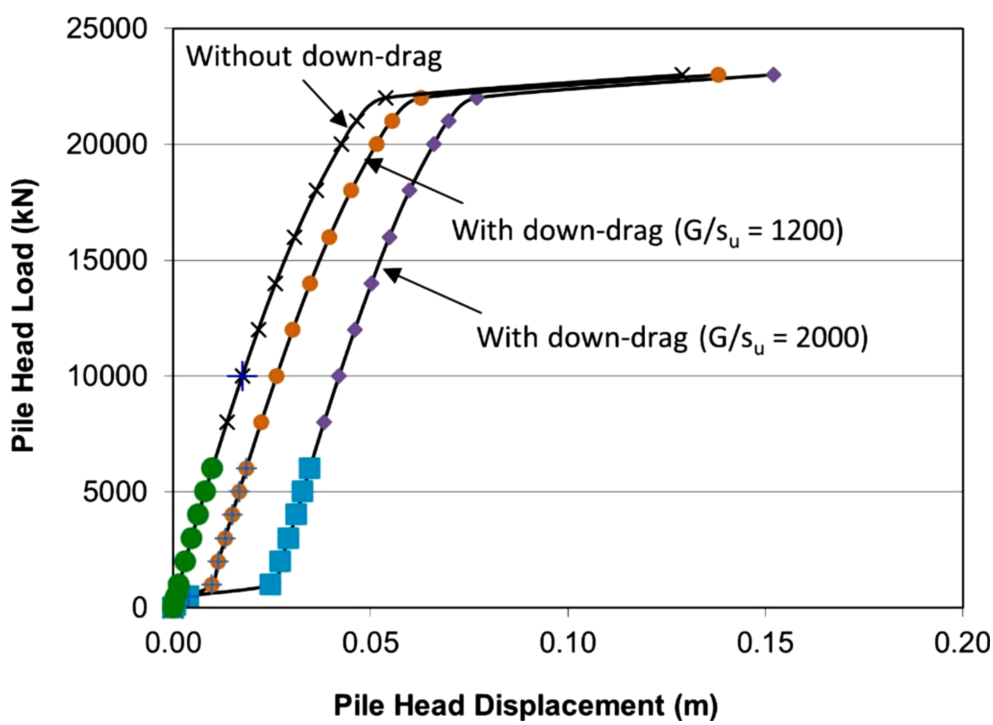

Fig. 5. Load-displacement response of existing pile with and without down-drag.

small additional pile settlement of the order of 10 mm. Similarly, the pile settlement due to down-drag could be around 25 mm for a G/s_u ratio of 2000. Therefore, due to the installation of a new pile at a 5 m distance, the existing pile underwent a settlement of 10 mm (with a G/s_u ratio of 1200) to 25 mm (with a G/s_u ratio of 2000) but continued to support the existing applied load.

It should be noted that the preliminary analysis conducted above is based on a conservative scenario, where the pore-water pressure dissipation during pile driving was not taken into account. Additionally, the evaluation of the potential soil settlements resulting from pile driving considered only the virgin soil consolidation characteristics. If such a conservative analysis gives rise to any indications of possible structural problems, it is recommended to conduct a more thorough and rigorous analysis that incorporates the cyclic load effects caused by continuous pile driving and accounts for the true soil behaviour, including possible pore-water pressure dissipation during driving, as well as a combination of unloading-reloading and virgin consolidation characteristics.

5. Conclusions and recommendations

The effects of pile driving-induced vibrations on adjacent existing nearby pile foundations and pipelines have been investigated for a case study of vertical 48-inch diameter open-ended piles with a wall thickness of 1.18 inches and 60 m pile tipping depth in a soil profile of 0 m to 20 m of carbonate sand, underlain by a 45 m thick clay layer (from 20 m to 65 m depth). The following conclusions can be drawn from the analysis of this case study.

For existing pile foundations:

- The assessment considered the conservative case of two vertical piles – one new and one existing at a 5 m separation. Conservatively, the existing pile is loaded with static loading with a factor of safety of 2 or greater.
- The analysis indicates that the predicted soil settlement due to the consolidation of a 40 m clay layer is between 0.3 m and 1 m, for the G/su ratio ranging from 1200 to 2000.
- The results show that the vertical displacement at the pile head of the existing pile will be of the order of 10–25 mm downwards (for a G/su ratio ranging from 1200 to 2000) due to the down-drag effect. This will cause load shedding through the jacket structure, but the ultimate pile capacity will not be adversely affected.
- In the situation where the two piles are battered with a minimum separation of 5 m the effects of down drag (and consequently the vertical displacements at the pile head of the existing pile) would be even less.
- In summary, the analysis indicates that the new pile installation has minimal effects on the existing piles' facilities. However, it is recommended that the load-settlement response of existing pile foundations should be monitored for critical cases. After pile installation is complete, regular inspection and maintenance of existing pile foundations should be performed to monitor for any signs of damage or instability.

For existing pipelines:

- The peak particle velocity (PPV) from different theories indicates a range of about 69.0 mm/s to 97.2 mm/s at a distance of 2 m from the new pile installation. For masonry/brick structures, British Standard (BS 7385–2:1993) allowable vibration magnitudes in the range of 6 mm/s to 100 mm/s. Considering the pipelines are mostly made of steel, the determined peak vibration magnitude of 97 mm/s indicates that the effect of new pile installation on existing pipelines is minimal.
- Apart from monitoring the existing facilities during the new pile installation, there is no need to investigate anything further.

Overall, based on the case study presented in the current work for pipelines located at a 2 m distance and piles at a 5 m distance from the piles of an existing offshore platform, the current analysis concludes that the driven installation of the new piles has minimal effects on the existing facilities. Thus, apart from monitoring the existing facilities during and after the installation of new piles, there was no need to investigate anything further.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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