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Original Article

Prediction Model of Soil Resistance to Driving for Large-Diameter Open-Ended Piles Under North Sea Conditions

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Abstract *- The prevailing industry methodology for predicting pile driving performance involves the analysis of the model of the hammer-pile-soil system. This model replicates the correlation between Soil Resistance to Driving (SRD) and blow counts. Traditional methods for predicting SRD are largely based on static pile capacity. Utilizing SRD in conjunction with a one-dimensional wave equation model helps forecast expected blow counts at varying depths for specific hammer energy settings. While this approach has primarily been fine-tuned for elongated flexible and small-diameter (2 to 3 m) piles in the oil and gas industry, efforts are underway to extend its applicability to compute results for relatively short, rigid, and large-diameter monopile foundations. Therefore, a new model named ε method for predicting SRD for Large-diameter Open-ended Offshore Pile (LOOP) was developed and presented in this paper. The soil model's direct correlation with CPT measurements eliminates the risk of individual misinterpretation of the data. Comparisons between predicted and recorded SRD show good agreement for case histories, with maximum deviation approx. 10% between recorded and predicted SRD deviate for pile tip penetration beyond 20m.*

Keywords *- Soil Resistance to Driving, Large-diameter Open-ended Offshore Pile, Pile driving, ε method.*

1. Introduction

Over the years, the authors have dedicated themselves to predicting the driveability of Large-diameter Open-ended Offshore Piles (LOOP) installed in diverse soil conditions across the North Sea sector.

In recent decades, the importance of renewable energy has grown significantly, intending to limit the global temperature by 1.5°C with respect to the period before industrialization. This has stimulated the development of competitive renewable energy sources, including offshore wind farms, to reduce the reliance on fossil fuels and control greenhouse gas emissions, Ahlinhan et al. (2019) [1].

Various foundation concepts have supported offshore wind turbines, including monopiles, jacket structures with four piles, tripod structures with three piles, suction caissons, gravity base foundations, tension leg foundations, and others.

Most currently used foundations are monopiles, which are Large-diameter Open-ended steel Offshore piles (LOOP). Monopile foundations represent about 75% of the foundations for offshore wind turbines, Ahlinhan et al. (2019) [1].

Monopiles, sturdy steel tubular piles with spacious diameter reaching 8 meters or beyond, are installed by driving them into the seabed at depths ranging from 30 to 45 meters, maintaining a length-to-diameter ratio between 4 and 7. Figure 1 illustrates a typical LOOP foundation that supports offshore wind turbines.

The driveability consists of analysing the model of the hammer-pile-soil interaction system. The soil is modelled with the Soil Resistance to Driving (SRD), which depends on the physical and shear parameters of the soil and the pile sizes.

The hammer is modelled with its particulars, and the impact of the hammer onto the pile top is described by a one-dimensional wave equation as per Smith (1960) [2]. This model of the hammer-pile-soil system replicates the correlation between Soil Resistance to Driving (SRD) and blow counts.

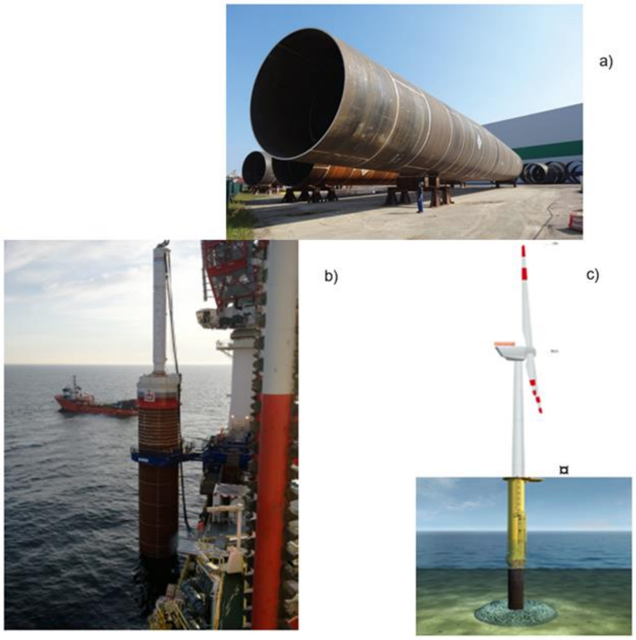

Fig. 1 Monopile foundation system, a) monopile structure, b) Monopile driving on construction site, c) monopile foundation supported offshore wind turbine.

Pile driveability is the pivotal element in offshore pile design, given that opting for an appropriate driving system accounting for initial accurate forecasts can markedly diminish the costs associated with pile installation. This is becoming increasingly important as the typical pile diameter rises to accommodate the heavy loads from increasingly larger wind turbine generators, deeper water depth and harsh environmental conditions. With respect to its diameter, monopile can be classified as regular monopile (5-6 m), XL monopile (6-8 m), XXL monopile (8-11 m), and mega monopile (over 11 m), Zhang et al. (2023) [3].

Analyses of pile driveability assess the pile's capability to be installed with an acceptable risk of early resistance. Ensuring it reaches the intended penetration depth as per design specifications. When early pile refusal occurs, the need to mobilize a larger hammer arises, resulting in both delays and increased costs for the project.

Additionally, when encountering low soil resistance to driving (SRD), unforeseen pile runs pose the risk of damaging the crane steel wires and could potentially result in the loss of both the pile and hammer into the sea if adequate precautionary measures are not implemented. Moreover, it's crucial to keep the number of hammer blows within a reasonable range to prevent overstressing of the steel that can cause fatigue damage to the pile, which could significantly reduce its predicted lifetime. Therefore, accurately assessing the soil resistance to driving (SRD) is crucial to ensure a safe installation process, Qin et al. (2022) [4], Qin et al. (2023) [5-7], Zhang et al. (2023) [3].

Qin et al. (2023) [5] simulated the hammering process of a large-diameter tubular thin-wall pile in clay using large deformation finite element analysis in ABAQUS standard, that utilized the dynamic remeshing and interpolation technique with small strain, initially developed by Wang et al. (2015) [8]. Hereby, an elastoplastic-modified cam clay model is used in combination with user-defined subroutines to calculate pore water pressure accumulation and /or dissipation during pile driving. A prediction equation for SRD was proposed as a function of diameter and was found to reasonably predict SRD of LOOP, especially after penetration to a specific depth. However, this equation is only valid for clay used for the model verification, as the proposed SRD equation does not depend on shear strength and cone resistance. The common approach for SRD prediction is the function of the shear strength of soil (see section 4).

Zhang et al. (2023) [3] conducted back analysis of pile driving in two common stratified soil conditions encountered in offshore wind farms in the East China Sea are characterized by layers of clay interspersed with sand and layers of sand interspersed with clay, using the wave equation method. Their findings showed that the SRD results derived from Steven and Alm's methods were largely consistent with those from the back analysis. However, there were notable differences: The predicted value for the clay layer exceeded that of the back analysis, whereas the predicted SRD for the sand layer was lower than expected. To address these discrepancies, Zhang et al. (2023) [3] proposed a modified approach for determining unit end resistance in sand and unit friction in clay. Despite these improvements, the difference between the SRD obtained from the proposed method and the back analysis remained at approximately 20%.

In this context, the authors of this paper have developed a prediction model for Soil Resistance to Driving (SRD). This prediction model was named the ε method for LOOP. The model is correlated to cone resistance (CPT) measurements, which eliminates individual misinterpretation of measured data. The paper presents the framework for the SRD, the model development, and the application of the ε method for two case histories. It's important to highlight that the model has been rigorously tested and applied to LOOP installed in diverse soil conditions across the North Sea region for decades.

Developing a reliable and accurate SRD prediction model has significant implications for offshore pile foundation design and construction. It can help reduce the risks and costs associated with pile driving and increase the safety and longevity of offshore structures. This paper contributes to the ongoing efforts to advance the state of the art in large-diameter pile foundation engineering. It provides a valuable resource for researchers and practitioners in the field.

2. Materials and Methods

The review of existing SRD methods shows that these methods were developed and validated for small pile diameters ranging from 2 to 3 m. Therefore, SRD methods for LOOP are required. For that, SRD methods used in practice design have been closely scrutinised, and the adaptability of some principles and equations has been examined (section 3). Then, a back analysis of driving records of LOOP with a diameter ranging from 6 to 8 m driven in the sand under North Sea conditions was performed to assess the SRD in the sand.

The back analysis has been carried out by means of the one-dimensional wave software GRLWEAP (2010), which enables the linking of SRD to blow counts. In GRLWEAP, the wave equation, according to Smith (1960) [2], is implemented.

The hammer-pile-soil system is segmented into multiple interconnected elements (element length of about 1m) for calculating stress wave propagation during pile driving. Each element consists of a mass block and a spring (Figure 2). The driving process is subdivided into discrete time intervals Δ, t, within which the physical quantity remains constant.

Based on results from the back analysis of driving records, a new SRD for LOOP named the ε method was developed. Finally, two case histories demonstrating the use and applicability of the new ε method are presented, although this ε method has been applied for decades to LOOP in the North Sea.

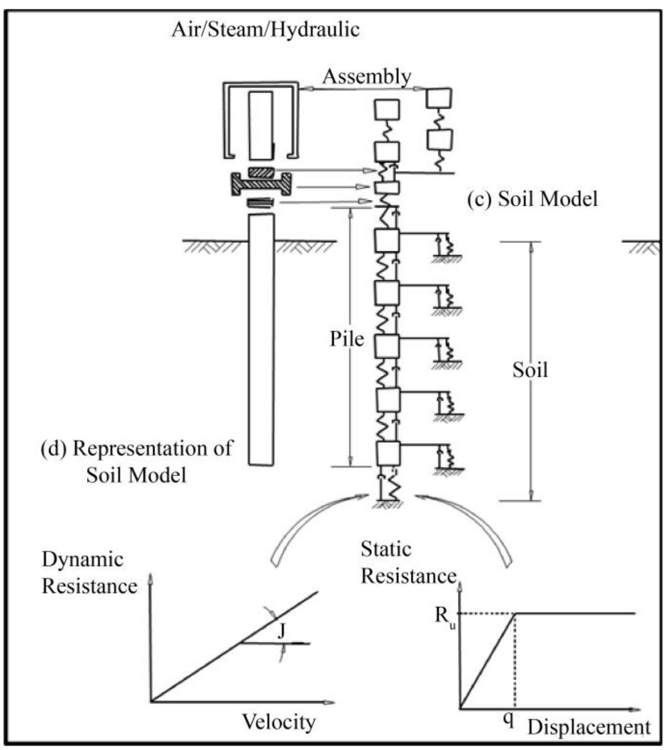

Fig. 2 System of pile-soil-hammer

3. SRD Approaches used in Practice Design

The soil resistance to driving SRD accounts for the decrease of the static soil resistance at the time of driving. For the prediction of the SRD, methods that used direct CPT data without any interpretation, which may lead misinterpretation, are more accurate, Alm and Hamre (2001) [11], Alm and Hamre (1998) [10], Alm et al. (1989) [9], Ruiter and Beringen (1979) [12], Heerema (1979) [13, 14], Heerema (1981) [15], Toolan and Fox (1977) [16]. The methods that used soil strength parameters, including the angle of internal friction φ and cohesion c, which have been derived from the CPT data and or the results of laboratory tests, may lead to inaccurate results, Schertmann (1978) [17], Stevens et al. (1982) [18]. The SRD is the sum of the pile tip resistance SRD_{tip} and the shaft friction resistance $SRD_{f,shaft}$ during driving and can be expressed as follows:

$$SRD = SRD_{tip} + SRD_{f,shaft} \tag{1}$$

SRD is influenced by the plugged or unplugged performance of the pile-soil system, which affects the pile driveability, Qin et al. (2023) [6]. Indeed, soil movement inside the open-ended characterized the plugged or unplugged performance. In the context of the driving process, the "no-plug" state refers to a condition in which the soil inside and outside the open-ended pile exhibits minimal relative movement. In this case, the soil level within the open-ended pipe pile remains nearly equal to the surrounding ground level. The internal soil is pushed downward relative to the external soil for the partial plug state. If the internal soil stays relatively stationary with the pile, the situation of the full plug develops. Two quantitative parameters, the Incremental Filling Ratio (IFR) and the Plug Length Ratio (PLR), are commonly used to characterize the state of soil plugging. IFR describes the height of the internal soil during driving, and PLR describes the height after driving, Fattah et al. (2016) [21]. For details regarding plug behaviour in open-ended piles driven by impact hammer, reference is made to Qin et al. (2023) [6], Guo et al. (2025) [22], Ko et al. (2014) [23], Ko et al. (2016) [24], Zhang et al. (2022) [25].

Since monopile for offshore wind energy usually has a pile diameter larger than 4 m, no plug state may be expected, Zhang et al. (2023) [3]; Ahlinhan et al. (2019) [1]; Heerema (1979) [13]. Therefore, SRD_{tip} and $SRD_{f\ shaft}$ can be written as follows:

$$SRD_{tip} = q_{dyn,CPT} \cdot A_{ann} \tag{2}$$

$$SRD_{f,shaft} = SRD_{f,int} + SRD_{f,out} \tag{3}$$

Here, $q_{dyn,CPT}$ is the unit tip resistance to driving derived from the CPT data and is the function of the cone tip resistance q_c, A_{ann} is the pile annular area ($A_{ann}=(\pi/4)(D_{out}^2-D_{int}^2)$), $SRD_{f,int}$ is the internal friction resistance to driving and $SRD_{f,out}$ is the outer friction resistance to driving.

Several approaches exist for calculating the unit tip resistance $q_{dyn,CPT}$ to driving, derived from the CPT data.

Alm and Hamre (1998) [10] carried out a back calculation of SRD based on data from 10 different pile installations performed at 8 different locations. The soil consists of very dense sand and highly over-consolidated clay with undrained shear strength between 400 and 600 kN/m^2. A substantial segment of the database primarily consists of piles with diameters spanning from 1.80 meters (72 inches) to 2.40 meters (96 inches). Notably, the deepest recorded pile penetration stands at 75 meters, except for the Oseberg conductors, which were driven to an impressive depth of 115 meters. A total number of 70 support piles and 14 conductor piles were evaluated. Based on the back calculation of these data, Alm and Hamre (1998) [10] developed a soil model for calculating SRD values for continuous driving of piles in typical North Sea soil conditions. Alm and Hamre (1998) [10] stated that this model, without additional factors, is appropriate for predicting best-estimate curves. For the prediction of upper-bound resistance, the effect of soil variability should be considered. Alm and Hamre (1998) [10] found a best-fit value for the unit tip resistance for dense sand to driving as follows:

$$q_{dyn,CPT} = 0.4 \cdot q_c \tag{4}$$

Where q_c is cone tip resistance. Based on an updated reanalysis of the above database, Alm and Hamre (2001) [11] found that the best-fit value for the unit tip resistance to driving depends on the overburden pressure as follows:

$$q_{dyn,CPT} = 0.15 \cdot q_c \cdot \left(q_c/p_0^{'}\right)^{0.2} \tag{5}$$

Where p'_0 is effective overburden pressure. With the above formulation (Equation 5), the unit tip resistance will increase with increasing q_c and sand density. It will be in the range typically from 0.35 to 0.55 times the cone resistance when sand density ranges from loose to very dense, Alm and Hamre (2001) [11]. These values are lower than the value of 1.0 proposed by Heerema (1980) [14] and Toolan and Fox (1977) [16]. According to Alm and Hamre (2001) [11], the basis of the value of 1.0 was the q_c value with a reduction factor due to different shape factors for a circular cone and a pile steel annulus. The reason for this lower value can be explained by the effect of wedged pile tips, where the actual tip area in practice design is often reduced to approximately 50 %. The ratio of the end bearing of statically loaded closed-ended displacement piles to the cone tip resistance of CPT (q_b/q_c) is often considered to vary from 0.6 to 1, White and Bolton (2005) [26], Xu et al. (2008) [27]. Based on the above information, the unit tip resistance to driving for large monopile in dense sand is set to:

$$q_{dyn,CPT} = 0.6 \cdot q_c \tag{6}$$

The major contribution to SRD is due to side friction. Approaches exist for calculating the unit fiction resistance to driving and are derived from the CPT data accounting for the fiction fatigue phenomenon. The friction fatigue or friction degradation is the decay of the pile unit friction at a given elevation during driving as the pile tip progresses into the soil. Observations that demonstrate this effect have been reported by Vesic (1970) [28] and Heerema (1979) [13]. Alm et al. (1989) [9] based on monitoring from the Oseberg B conductor installations. Effects of friction fatigue have also been verified by signal-matching procedures performed by both Randolph (1993) [29], Colliat et al. (1996), White and Lehane (2004) [30], Gavin and O´Kelly (2007) [31].

Based on the database described above, Alm and Hamre (1998) [10] and Alm and Hamre (2001) [11] formulated friction fatigue along piles during driving as follows:

$$f_s = f_{sres} + (f_{si} - f_{sres}) \cdot e^{k(d-p)} \tag{7}$$

Where f_s is actual pile side unit friction, f_{si} is initial pile side unit friction, f_{sres} is residual pile side unit friction found to be 20% of initial pile side unit friction f_{si}, k is shape factor for degradation, d is depth to actual layer, p is pile tip penetration. The shape factor for degradation has been established as effectively described by a unified formula applicable to both clays and sands, as expressed by the following relation:

$$k = \frac{\left(\frac{q_c}{p_0^{'}}\right)^{0.5}}{80} \tag{8}$$

For sands, the initial friction can be described as the basic static friction formulation:

$$f_{si} = K \cdot p'_0 \cdot \tan\delta \tag{9}$$

Where K is the coefficient of earth pressure, p'_0 is overburden pressure, δ is the interface friction angle of the soil-pile interface. Jardine and Chow (1996) [32] linked lateral stress coefficient K to cone resistance as follows:

$$K \cdot p_0^{'} = 0.0132 \cdot q_c \cdot \left(\frac{p_0^{'}}{p_a}\right)^{0.13} \tag{9}$$

Therefore, Equation 9 can be rewritten as follows:

$$f_{si} = 0.0132 \cdot q_c \cdot \left(\frac{p_0^{'}}{p_a}\right)^{0.13} \cdot \tan\delta \tag{10}$$

However, when using the above relation (Equation 11) for the lateral stress coefficient, it is important to note that this equation was developed under the assumption that friction occurs only on the outside of the pile wall, as stated by Alm and Hamre (2001) [11]. Therefore, this assumption must also be considered when determining the sand friction. This can be done either by including only the outside friction in the calculation or, more conveniently, by reducing the unit friction to 50% of the original value and applying it to both the inside and outside of the pile wall. Hence, Equation11 can be rewritten as follows:

$$f_{si,out} = f_{si,int} = 0.50 \cdot f_{si} = 0.0066 \cdot q_c \cdot \left(\frac{p_0'}{p_a}\right)^{0.13} \cdot \tan\delta \quad (11)$$

Since the product of the earth pressure coefficient to the effective overburden pressure depends on several factors, including the cone tip resistance, the monopile size (diameter, length, and wall thickness), etc., the CPT-based approaches such as Simplified ICP-05, Offshore UWA-05 and Fugro-05 can be applied for the determination of the initial friction $f_{si,out}$ and $f_{si,int}$. The unit skin friction (f_{si}) for open-ended steel pipe piles for recommended CPT-based methods (Simplified ICP-05, Offshore UWA-05 and Fugro-05) API RP (2017) [33] can be calculated as follows:

$$f_{si} = u \cdot q_c \cdot \left(\frac{\sigma_{v0}'}{p_a}\right)^a \cdot A_r^{\,b} \cdot \left[max\left(\frac{L-z}{D}, v\right)\right]^{-c} \cdot [\tan\delta_{cv}]^d \cdot \left[min\left(\frac{L-z}{D}\frac{1}{v}, 1\right)\right]^e \quad (12)$$

Recommended values for parameters a, b, c, d, e, u and v for compression and tension are given in Table 1.

Here, A_r is pile displacement ratio $A_r=1-(D_{int}/D_{out})^2$, D_{out} is pile outer diameter, D_{int} is pile inner diameter, $D_{int}=D_{out}-2WT$, WT is pile wall thickness at pile tip (including driving shoe).

Table 1. Unit skin friction parameter values for driven open-ended steel pipes (Simplified ICP-05, Offshore UWA-05 and Fugro-05 Methods), API RP (2017) [33]

Method	Parameter						
	a	b	c	d	e	u	v
Simplified ICP-05	0.1	0.2	0.4	1	0	0.023	$4\cdot A_r^{0.5}$
Offshore UWA-05	0	0.3	0.5	1	0	0.030	2
Fugro-05	0.05	0.45	0.90	0	1	0.043	$2\cdot A_r^{0.5}$

The above-presented methods for SRD prediction were originally developed for offshore open-ended piles in the oil and gas industry, with diameters ranging from 2m to 3m.

Reference is made to Kortsch and Kirsch (2018) [34] and Zhang et al. (2023) [3] for other methods for the determination of the SRD that were established for small-diameter open-ended pipe piles. Therefore, a new approach predicting SRD named ε-method was developed for LOOP foundation in the sand in the North Sea conditions.

4. New Approach to SRD Prediction: ε-Method

A new approach called the ε- method for the SRD prediction was developed for LOOP in sand. This ε-method is in line with the method described in De Ruiter and Beringen (1979) [12] and Toolan and Fox (1977) [16], Alm and Hamre (2001) [11]. It is a semi-empirical method to convert static soil resistance into soil resistance to driving (SRD). Assuming no plug state for LOOP during driving (see section 4), the total soil resistance to driving can be described as follows:

$$SRD = SRD_{tip} + SRD_{fout} + SRD_{fint} = SRD_{tip} + \varepsilon \cdot (Q_{fout} + Q_{fint})/2 \quad (13)$$

Where SRD_{tip} is the soil resistance at the pile tip during driving, i.e. steel pile wall, SRD_{fout} is outside friction resistance on the pile during driving, SRD_{fint} is inside friction resistance on the pile during driving, ε is calibration coefficient accounting for friction fatigue, Q_{fout} is static outside friction resistance, Q_{fint} is static inside friction resistance.

For the best estimate soil profile, SRD_{tip} for the pile tip is based on the Alm and Hamre (2001) [11] method described above, whereas SRD_{fout} for the pile outside and SRD_{fint} for inside shaft are based on the approach described in Tolan and Fox (1977) [16], and De Ruiter and Beringen (1979) [12]. Hence, SRD_{tip}, Q_{fout} and Q_{fint} can be expressed as follows:

$$SRD_{tip} = q_{dyn,CPT} \cdot A_{tip} = 0.15 \cdot q_c \cdot (q_c/p_0')^{0.2} \cdot A_{tip} \quad (14)$$

$$Q_{fout} = q_{dyn,CPT} \cdot A_{out} = (q_c/300) \cdot A_{out} \quad (15)$$

$$Q_{fint} = q_{dyn,CPT} \cdot A_{int} = (q_c/300) \cdot A_{int} \quad (16)$$

Where A_{tip} is the area section of the pile annulus, A_{out} is the outside area section of the pile $=\pi\cdot D_{out}\cdot\Delta L$, A_{int} is the inside area section of the pile $= \pi\cdot D_{int}\cdot\Delta L$, D_{out} is the pile outside diameter, D_{int} is the pile inside diameter, ΔL is layer thickness of soil.

Table 2. Quake and damping parameters used for the calibration

		Sand	Clay
Quake (mm)	Side	2.5	2.5
	Tip	2.5	2.5
Damping (s/m)	Side	0.16	0.65
	Tip	0.50	0.03

For SRD prediction based on the new ε-approach proposed in this paper, the ε function needs to be known. Therefore, the ε function in equation 14 was calibrated to the results of the driving records for LOOP-supporting offshore wind turbines in the North Sea. The soil consists mainly of medium to very dense sand. The post-analysis for the calibration purpose was carried out as follows:

Step : 1 Divide the driving distance, i.e. foundation soil as much as possible, into calculation layers based on soil profile (CPT data) and actual driving results. It should be noted that each such layer (between top and bottom) should have an almost linear change in cone resistance from CPT, in impact energy and blow counts (e.g., bl./0.25m). It is even better when these values are constant. This would allow for the working of average values in each layer (CPT, energy, and blow count).

Step : 2 Calculate the soil resistance to driving for the pile tip SRD_{tip} based on equation 15.

Step : 3 Calculate static friction resistance outside and inside of the pile Q_{fout} and Q_{fint} according to equations 16 and 17, respectively.

Step : 4 Keep the parameters such as triangular friction distribution, quake, and damping constant for the shaft and tip (see Table 2).

Step : 5 For the top and bottom of each layer, calculate blow count by means of pile driving software (e.g. GRLWEAP) while keeping impact energy at its actual value during driving. Vary ε such that the calculated blow counts fit the actual blow count given in the driving records (Figure 3).

Figure 4 presents the back calculation of the ε function for the LOOP for predominantly sand in the North Sea. The back-calculation was carried out according to the methodology described above. The ε function decreases with increasing pile tip penetration. That can be explained by the friction degradation during pile driving. The friction degradation or friction fatigue is the phenomenon by which the horizontal effective stress, σ'_h (and hence local shaft friction τ_s), acting on the pile shaft at a given soil horizon decreases as the pile tip penetrates to deeper levels, Heerema [13], Toolan and Fox [16] (see also section 4).

The trend curve for the ε function can be expressed as follows:

$$\varepsilon = 4-3.05 \cdot tanh(3p \cdot \pi/180) \tag{17}$$

Where p is the depth of pile tip penetration with respect to seabed level, the ε function decreases exponentially with increasing depth of pile tip penetration. It tends asymptotically to the value of 1 (Figure 5), which reflects the friction fatigue described in section 4 and expressed with equation 7.

To apply the ε method in this paper, two case histories for driven piles installed in the North Sea have been presented below.

For case history, A, an open-ended pipe pile with an outer diameter of 2.43 m and wall thickness ranging from 40 to 60 mm, was driven into the soil ground in the North Sea.

Before the pile was driven, the pile vibrated up to 10 m by means of the vibrator type Mueller MS 240-HHF, Kortsch and Kirsch (2018) [34]. The soil consists mainly of Holocene glacial dense sand (Figure 6).

Case history B consists of a monopile of diameter ranges from 6.5 m to 7.8 m and thickness from 65 mm to 90 mm, which was driven in dense sand in the North Sea.

Hereby, the hydraulic hammer IHC S-4000 was used for the driving operation (Table 4). The water depth is about 40 m concerning the Lowest Astronomical Tide (LAT).

Table 3 presents the detailed geometry for the pile of case history A (Kortsch and Kirsch, 2018) and case history B. The data of the hammers used are presented in Table 4.

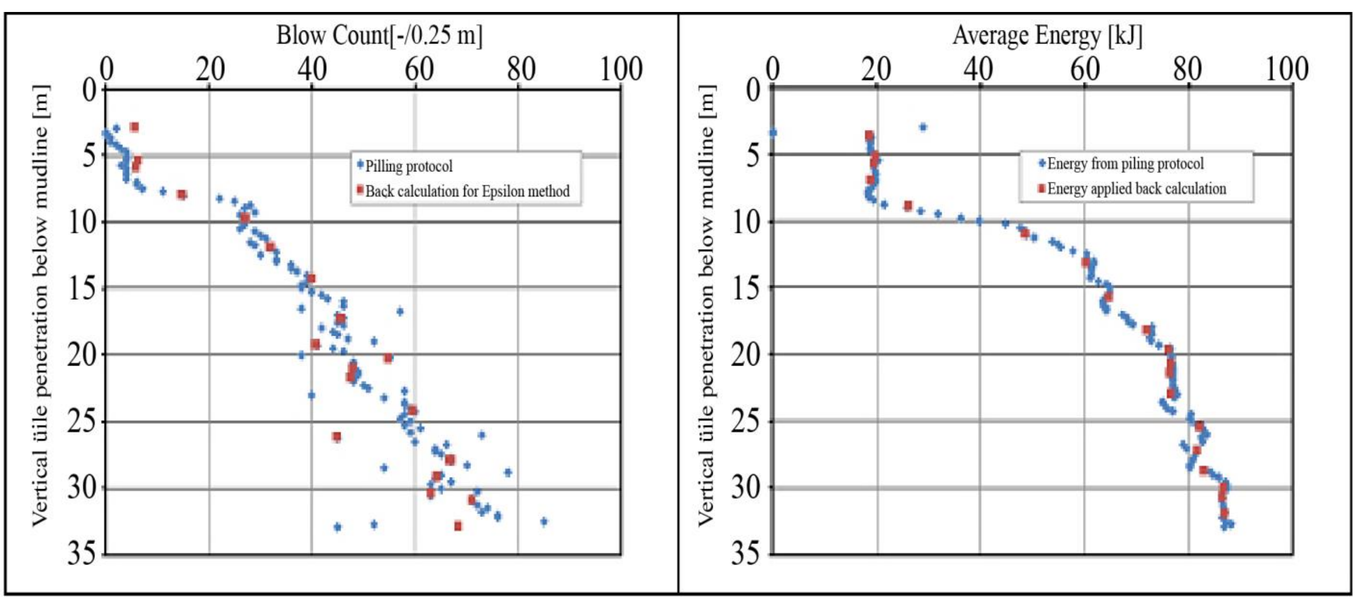

Fig. 3 Comparison between recorded and applied blow count and average energy for calibration of ε function

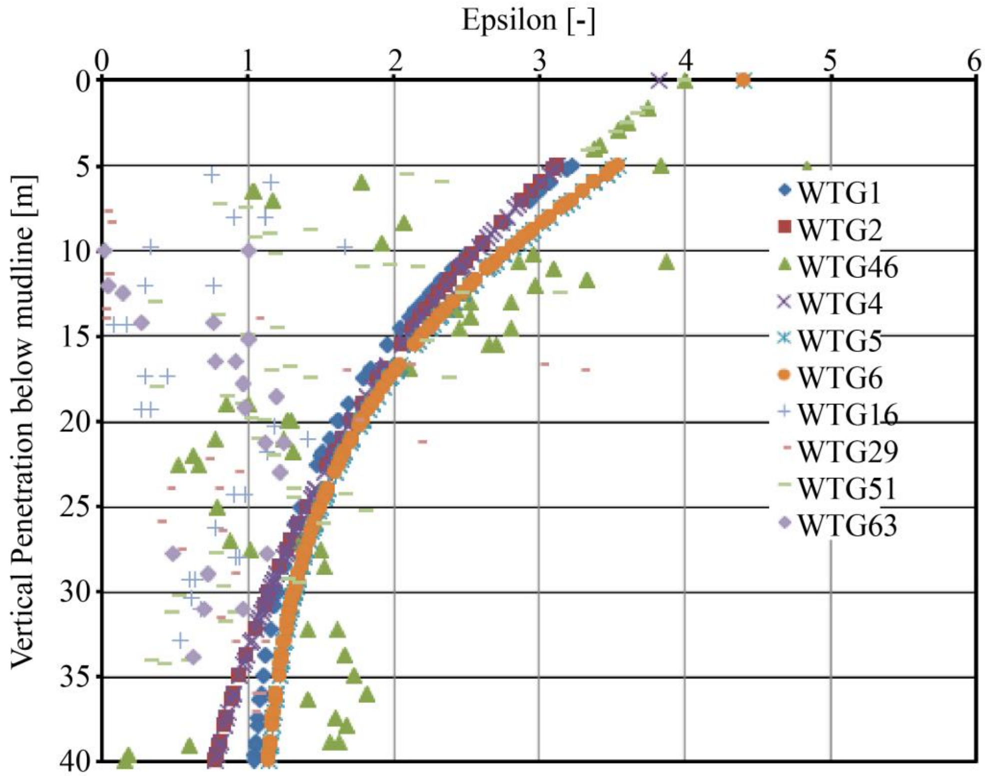

Fig. 4 Back calculation of epsilon for monopile with large diameter

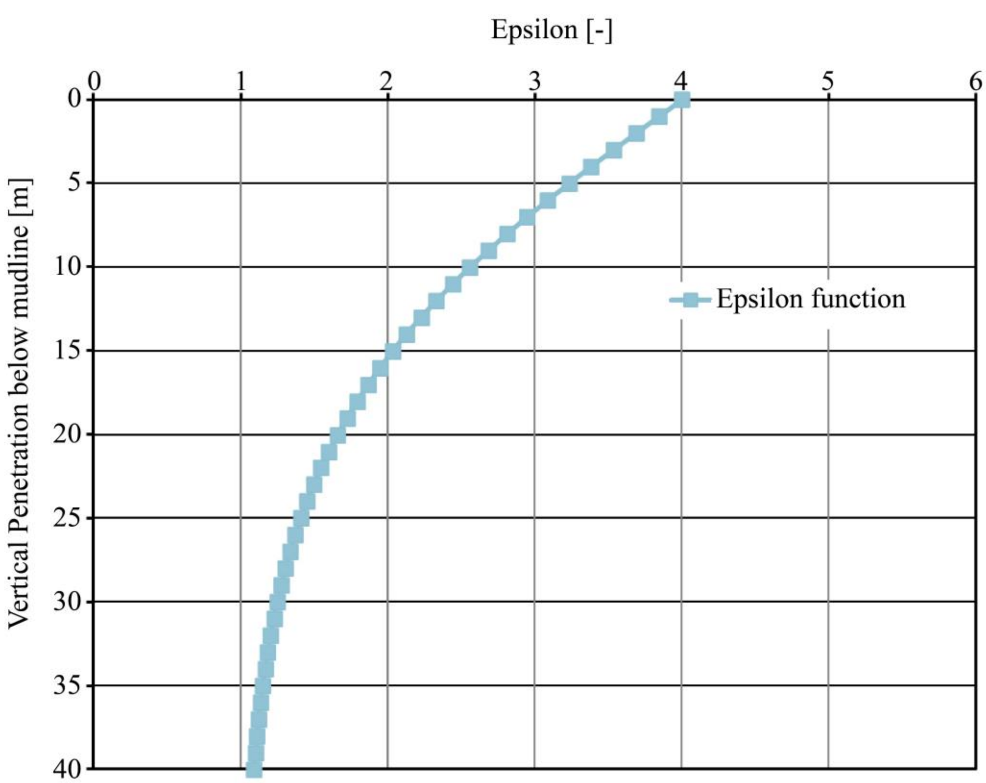

Fig. 5 Graph of ε function with vertical pile tip penetration

Table 3. Monopile data for the two case histories

	Case history A			Case history B		
Pile segments	**Outer diameter [m]**	**Length [m]**	**Wall thickness [mm]**	**Outer diameter [m]**	**Length [m]**	**Wall thickness [mm]**
1	2.43	3.35	50	*flange	0.310	90
2	2.43	5.5	60	6.5	4.69	90
3	2.43	3	50	6.5	6.6	80
4	2.43	30.1	40	7.017	11.4	68
5	2.43	1	40	7.80	11.51	80
6				7.80	2.6	90
7				7.8	20.5	96
8				7.8	3	85
9				7.8	3	75
10				7.8	7.59	65
11				7.8	2.5	75
12				7.8	2.5	85
Pile tip penetration [m]	33.6 to 35.5			33		

Table 4. Hammer data for the two case histories

		Case history A	**Case history B**
Parameter	Unit	IHC S-1200	IHC S-4000
Max. Energy	[kJ]	rd. 1.200	4000
Ram weight	[kN]	rd. 600	2000
Anvil weight	[kN]	221	2300
Stroke	[m]	2.02	2.02

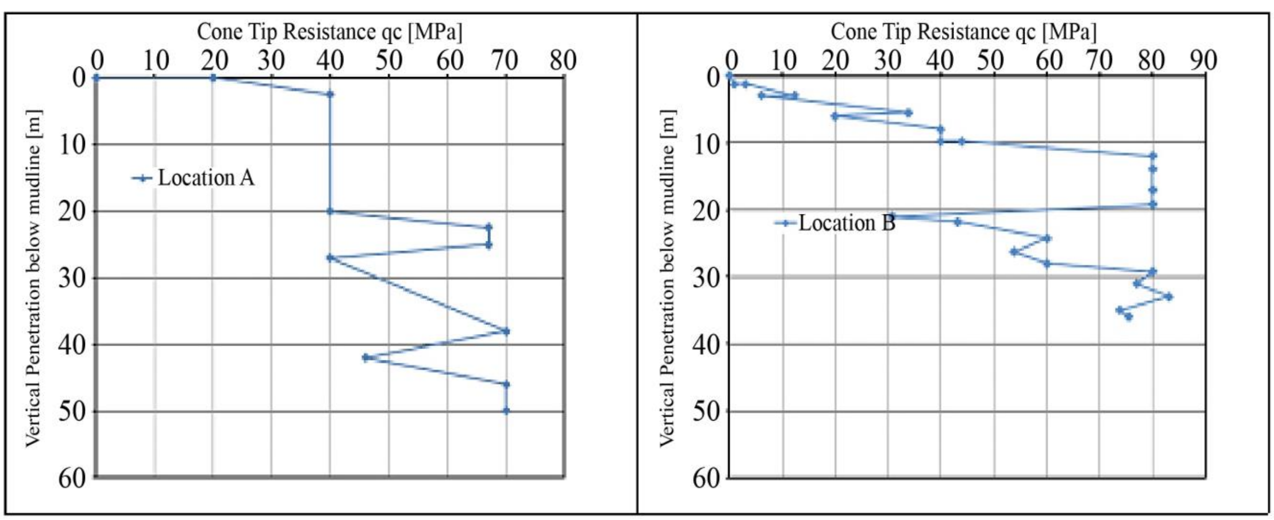

Fig. 6 Cone resistance for locations A and B

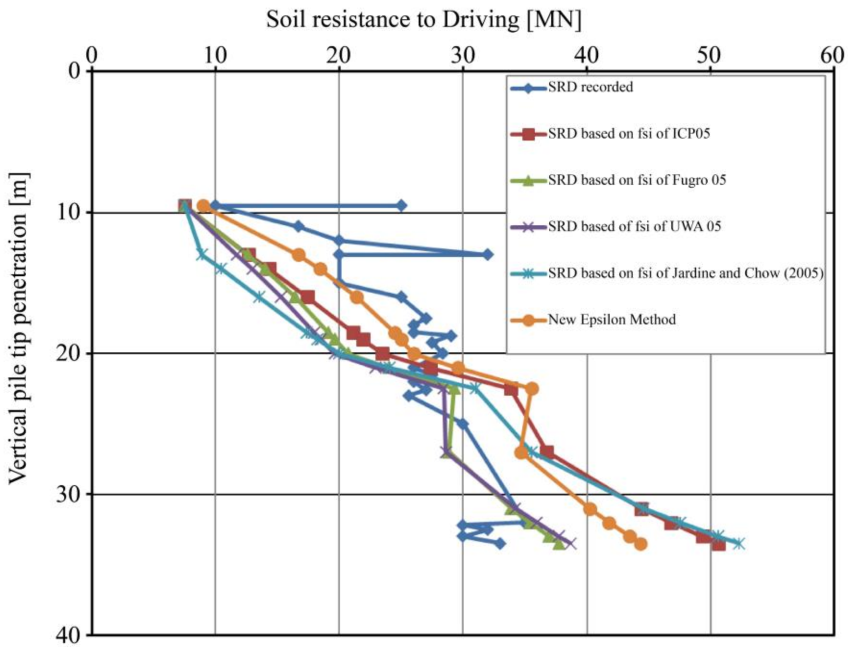

Fig. 7 Comparison between predicted SRD with new epsilon method and common methods for location A

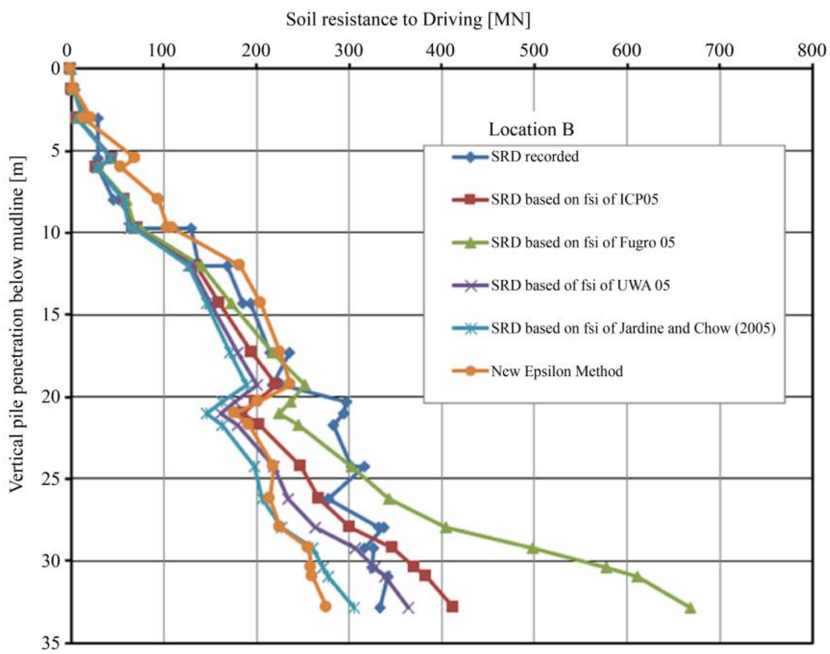

Fig. 8 Comparison between predicted SRD with the new epsilon method and common methods for location B.

Figure 7 and Figure 8 present the Soil Resistance to Driving (SRD) determined according to the new ε method for case history A and B, respectively. Hereby, the initial friction resistance of ICP05, Fugro 05, UWA 05, Jardine et al. (2005), reported in API RP 2A WSD [34], was applied. Predicted SRD applying the new ε method are generally consistent with the recorded SRD, particularly for pile penetration depth up to 20 m. Beyond this depth, the recorded and predicted SRD deviate slightly from each other by approx. 10% that covers the range from the best estimated and lower estimated or upper estimated soil profile used in practice design. This slight deviation of 10% can be explained by the applied initial friction resistance and the friction fatigue. However, this deviation for the present analysis is lower than that of 20% reported by Zhang et al. 2023 [3] and confirms the accuracy of the new ε method.

5. Conclusion

This paper has presented a new ε method for predicting Soil Resistance to Driving (SRD) for Large-diameter Open-ended Offshore Piles (LOOP) in North Sea conditions. The model has been verified by comparing predicted and recorded SRD for various locations, demonstrating its reliability and accuracy.

While the paper presents only two case histories, the model has been extensively tested and applied for decades for different soil conditions in the North Sea, and the results have been very promising. The results of case histories show that the new ε method can accurately predict the SRD for LOOP. When using the ε method, a deviation of 10% between recorded and predicted SRD is possible for pile tip penetration beyond 20m.

Developing a reliable SRD prediction model significantly impacts offshore pile foundation design and construction. Accurate predicting pile driveability can help reduce risks and costs associated with pile driving and increase the safety and longevity of offshore structures. This new model is a valuable contribution to the field of pile foundation engineering. It provides a more reliable tool for researchers and practitioners to use in their design and analysis of offshore structures. Future work is ongoing and focuses on expanding the application of this model to other offshore regions with similar soil conditions, as well as investigating the effects of other parameters such as pile diameter, length, length-to-diameter ratio, over-consolidation and shape on pile driveability. In summary, this paper highlights the importance of accurate pile driveability predictions. It demonstrates the effectiveness of the new SRD prediction model for large-diameter open-ended offshore piles in the North Sea.

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List of symbols

D_R	=	Relative density (-)
γ'	=	Effective unit weight of foundation soil (kN/m^3)
φ'	=	Effective angle of internal friction of soil (°)
ν	=	Poisson´s ratio (-)
q_c	=	Cone tip resistance from CPT (MN/m^2)
t	=	Monopile wall thickness (m)
f_s	=	Pile side friction (kN/m^2)
f_{si}	=	Initial pile side friction (kN/m^2)
f_{sr}	=	Residual pile side friction (kN/m^2)
d	=	Depth to actual clay layer (m)
p	=	Pile tip penetration (m)
k	=	Shape factor for degradation (-)
q_{TIP}	=	Unit pile tip resistance (kN/m^2)
q_T	=	Total cone tip resistance from CPT (kN/m^2)
p_o’	=	Effective overburden pressure (kN/m^2)
p_a	=	Reference pressure = 100 kN/m^2
δ	=	Constant volume friction angle (degrees)
LOOP	=	Large-diameter Open-ended Offshore Pile

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