Dynamic Relocations of Seaward Wind Turbine Monopile Establishments Including the Impacts of Soil-Design Associations

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ABSTRACT

The old style technique for planning seaward monopile establishments comprises of utilizing static wave force hypothesis. With this strategy, the biggest wave not out of the ordinary in the region is chosen as the plan wave. By applying pertinent wave hypothesis, wave force coefficients to the chose wave, powers and minutes on the monopile establishment can be determined accepting static balance. While the methodology might appear to be coherent, it may not address reverberation. Reverberation may really happen when a more modest wave whose principal time of vibration combines close to the normal time of vibration of the monopile structure. This peculiarity might enhance rebellions and stresses causing critical dam-age. Utilizing the versatile properties of the monopile structure and its dynamic reaction to wave powers another technique is introduced for working out burdens and removals of a monopile establishment. Moreover, soil-structure associations of monopile establishments are explored. Various methodologies, for example, the p-y technique or the Winkler model have been utilized to plan monopile establishments exposed to sidelong static and dynamic stacking. Notwithstanding, another plan approach that considers other critical communication instruments between the heap and the dirt is talked about.

Keywords: Wave, monopile foundation, resonance, soil-structure interactions.

INTRODUCTION

As a result of growing energy crisis and environmental pollution, various renewable energy sources have been advocated. Wind energy is a clean and efficient type of renewable energy. O shore wind turbines (OWT) are exposed to environmental loadings which affect the OWT significantly. Dynamic analysis is required during the design phase of the structural components to ensure the safety of the turbine. The significance of the dynamic behavior for the design of the support structure has long been recognized and pertinent research has been carried out [1-4]. The prevailing support structure design for the o shore wind turbine is the monopile. This is mainly due to the fact that it is well suited to mass -fabrication and the installation method is based on conventional impact driving which is robust in most soil conditions [5]. For a monopile foundation, an accurate and detailed model will improve the precision of the dynamic analysis of the wind turbine. Studies have shown that proper modeling of o shore foundations is quite complex [6][7]. As a result, applying different foundation models such as the Winkler Finite Element Model (FEM), the stiffness matrix model, the distributed spring model and the effective fixity model often result in different structural responses [7]. The most common method of design of fixed offshore turbines located above the limit of wave motion is by means of a statical wave force analysis. With this method, the largest wave is selected as the design wave. Using applicable wave theory and suitable wave force coefficients, the maximum forces and moments are calculated assuming the structure is in static equilibrium [8]. However, recent studies have shown that the design wave may not result in maximum stress in the structure. A smaller wave whose fundamental period approaches the natural

period of vibration of the turbine structureweight of the structure, four main lateral loads may be critical for design due to largeact on o shore wind turbine (OWT) structures: amplification of defections and stresses nearwind, wave, 1P (rotor frequency), and 2P/3P resonance [9]. A new method of analysis that(blade passing frequency) loads [10]. These considers the elastic characteristics of theloads will be discussed in this paper. This structure and its dynamic response to wavepaper aims to present an engineering dynamic forces is discussed in this in this paper.analysis method and soil structure interactions Furthermore, the interaction of the monopileof o shore wind turbines. An analytical structure with the supporting soil ismethod for the dynamic analysis of monopile investigated in this study. O shore windfoundation acted upon by a train of oscillatory turbine structures are subjected to cyclic andsurface waves is developed. An equation of dynamic loadings which results in vibration ofmotion for the horizontal displacement of the the structure. The load transfer mechanism formonopile is presented. Wave force on the monopile wind turbines is through overturningmonopile based on the wave force theory is moments which results in the transfer of loadsdiscussed [11].
to the surrounding soil. In addition to the self-

Dynamic Analysis of Monopile Wind Turbine

The equation of motion of a monopile wind turbine foundation can be represented by the motion of a single degree of freedom, equivalent spring-mass system linear damping and restoring force subjected to a sinusoidal exciting force as shown in Figure 1. The horizontal displacement X_d, measured with reference to the neutral axis of the monopile is represented in Figure 1. The equation is given as follows [12]:

$$m_s \frac{d^2 X_d}{dt'^2} + C_s \frac{dX_d}{dt'} + K_c X_d = P(t') = P_a \sin f t' \quad (1)$$

where:

m_s = effective mass of the system
C_s = damping coefficient of structure
K_c = spring constant of structure
P_a = amplitude of harmonic exciting force
f = frequency of harmonic exciting force ($f = 2\pi/T$)
t' = time in the equivalent system

$$X_d(t') = \frac{P_a}{K_c \sqrt{\left[1 - \left(\frac{f}{f_n}\right)^2\right]^2 + \left[2 \frac{C_s f}{C_c f_n}\right]^2}} \cdot \sin(f t' - \phi) \quad (2)$$

where,

$$X_d(t') = \text{horizontal displacement due to } P(t')$$

$$f_n = \sqrt{\frac{K_c}{m_s}}, \text{undamped natural frequency of spring-mass system} \tag{3}$$

$$C_c = 2\sqrt{m_s K_c}, \text{critical damping coefficient} \tag{4}$$

$$\phi = \tan^{-1}\left[\frac{2\frac{C_s}{C_c}\cdot\frac{f}{f_n}}{1-\left(\frac{f}{f_n}\right)^2}\right] \tag{5}$$

The effective mass of the equivalent system (rotor, hub and nacelle) is concentrated at the top and the tower is considered to act as a cantilever springs of zero mass. The harmonic exciting force P (t) acts in the plane of the monopile and undergoes a translatory motion. The exciting force for the monopile F (t) is the resultant of the time dependent wave force distribution acting at a variable distance s from the mudline as shown in Figure 2.

Monopile Features

The harmonic exciting force P (t) has to be related to the actual exciting force F (t) of the system. The actual exciting force F (t) consists of hydrodynamic drag and inertia components. Therefore, it cannot be represented by a single term of form F sin t. It can be represented by a series of sine terms using Fourier approximation. By using the method of influence fractions, each F term in the series can be related to a term through the method of influence fractions. The equation of motion is linear as shown in equation (1) is linear, therefore the displacements due to the individual exciting force terms (P sin t) in the Fourier series can be added to determine the total monopile displacement as a function of time resulting in equation (6) as follows:

$$X_{tot}(t) = \sum_{m=0}^{m} x_m(t) \tag{6}$$

By using the normal procedure for structural analysis, the displacements (strains) can be related to stresses. The static defection of the monopile foundation can be obtained from cantilever beam theory. The maximum defection of the monopile structure for an applied force F at height y (see Figure 2) can be expressed as:

$$[X_{\text{max act.}}] = \frac{F\bar{y}^2}{EI}\left(\frac{\ell}{4} - \frac{\bar{s}}{6}\right) \tag{7}$$

Applying a force P at the hub of the monopile tower, the deflection is obtained by substituting into equation (7) and letting s = l as follows:

$$[X_{\text{max equiv.}}] = \frac{P\ell^3}{12EI} \tag{8}$$

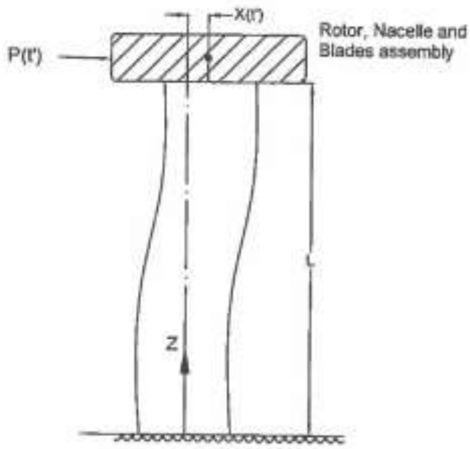

Fig. 1. *Equivalent spring-mass system*

Where:
E = Modulus of elasticity of the steel monopile

I = moment of inertia the monopile
l = hub height

The influence fraction, f which relates F and P can be obtained by equating the static defection of the equivalent system under force P to the actual defection under force F . Therefore, by equating equation (7) to (8) and setting the ratio of P=F = f, we have:

$$\frac{P}{F} = f = 3\left(\frac{\bar{y}}{\ell}\right)^2 - 2\left(\frac{\bar{y}}{\ell}\right)^3 \tag{9}$$

The spring constant by definition is given by:

$$K = \frac{P}{[X_{\text{max equiv.}}]} \tag{10}$$

From equation (4),

$$K = \frac{12EI}{\ell^3} \tag{11}$$

DYNAMIC DISPLACEMENTS OF OFFSHORE MONOPILES AND SOIL-STRUCTURE INTERACTIONS 5

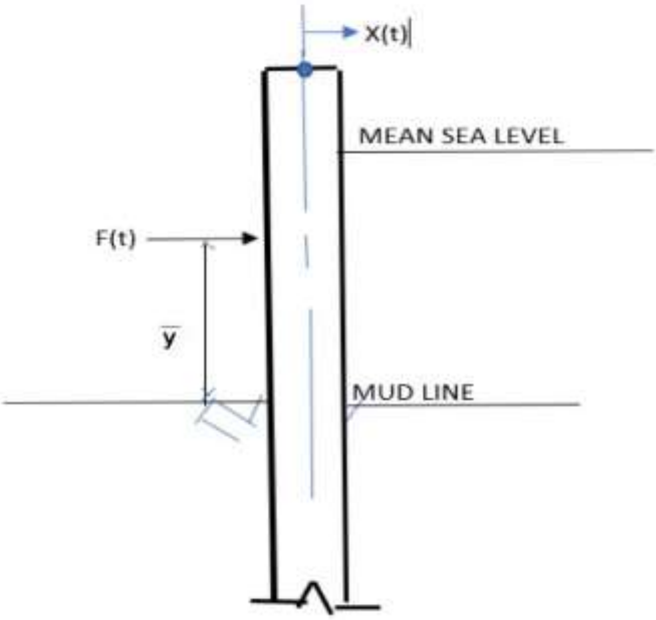

Fig. 2. *Exciting force on Monopile foundation*

The natural frequency of the monopile can be determined using static defection curve to calculate the kinetic and potential energies using Rayleighs Energy Method. The defection of the monopile at any section z (Figure 1) is given in terms of X_{max} by the influence fraction (equation (9)):

$$\frac{X(z)}{X_{max.}} = 3\left(\frac{z}{\ell}\right)^2 - 2\left(\frac{z}{\ell}\right)^3 \tag{12}$$

maximum kinetic energy

$$[K.E.]_{max.} = \frac{W}{g}\frac{f_n^2 X_{max}^2}{2} + \int_0^{\ell} \frac{w}{g}\frac{f_n^2 X(z)}{2} dz$$

Substituting $X(z)$ from equation (7) results in:

$$[K.E.]_{max.} = \frac{f_n^2 X_{max}^2}{2g}(W + \frac{13}{35} w\ell) \tag{13}$$

where,

W = weight of turbine (rotor, hub and nacelle)
w = weight per foot of the monopile tower

The maximum potential energy can be expressed as:

$$[P.E.]_{max} = \frac{KX_{max}^2}{2} \quad (14)$$

Equating the kinetic energy to potential energy (i.e. equations (13) and (14)) results in the natural frequency as follows:

$$f_n = \sqrt{\frac{K}{\frac{1}{g}(W + \frac{13}{35}w\ell)}} \quad (15)$$

(15)

Comparing the frequencies obtained in equations (3) and (13), the resultant mass of the monopile is,

$$m_s = \frac{1}{g}(W + \frac{13}{35}w\ell) \quad (16)$$

In equation (16), the factor 13=35 represents an added mass due to vibration in water and it's considered negligible. The critical damping coefficient C_c is a function of m and K and can be derived from equation (4). Studies have shown that a convenient way to determine the magnitude of the structural damping of an o shore structure is to measure the decay of the amplitude of oscillation under the action of a single impulse force in either the actual structural member or a model [13]. Research also showed that if X_1 and X_2 are two successive displacements, the ratio of C_s=C_c is proportional to logarithmic decrement as follows [13]:

$$\frac{C_s}{C_c} = \frac{1}{2\pi}(1n\frac{X_1}{X_2}) \quad (17)$$

In general, the value of viscous damping due to vibration in water is small compared to the structural damping [13]. The frequency of the harmonic exciting force is given by,

$$\sigma = \frac{2\pi}{T}$$

where T is the period of the wave motion. The phase angle may be calculated from equation (5) as it is a function C, C_c, and $_n$.

Therefore, the equation of motion, equation (1) can be solved by knowing all the parameters discussed above.

Wave Force Theory

The total wave force F on a vertical cylinder is based on wave motion shown in figure 3 [9]. Using the notations shown on figure 2.2, the total wave force is given by:

$$F = F_C \sin^2\theta + F_I \cos\theta \quad 0^\circ \le \theta \le 180^\circ \quad (18)$$

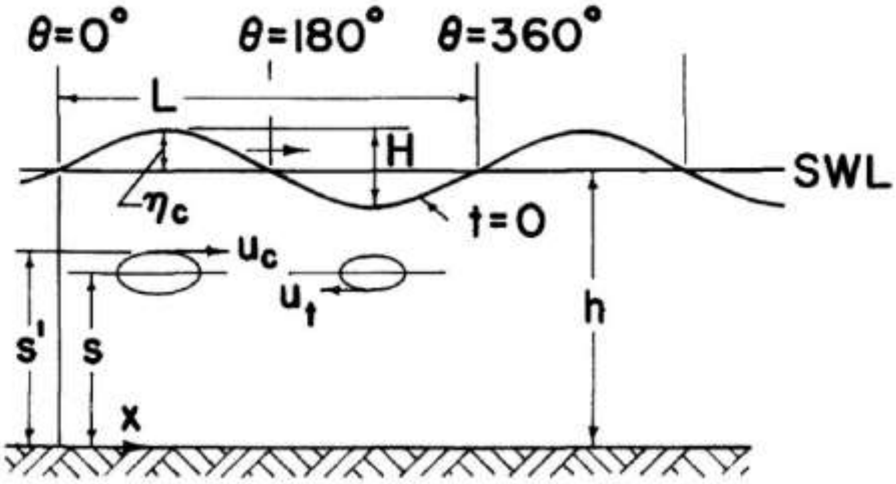

Fig 3. *Definition of geometry of wave motion on a vertical cylinder. Adapted from: D.R.F. Harleman, and W.C. Shapiro \Experimental and Analytical Studies of Wave Forces on O shore Structures: Part i, Results for Vertical Cylinders": Tech. Report No. 19, M.I. T. Hydrodynamics Laboratory, May 1955.*

$$F = F_T \sin^2\theta + F_I \cos\theta \quad 180^\circ \leq \theta \leq 360^\circ \tag{19}$$

F_c = hydrodynamic drag force at crest ($\theta = 90^\circ$)
F_T = hydrodynamic drag force at trough ($\theta = 270^\circ$)
F_I = inertial force at $\theta = 0^\circ$ and 180°
θ = phase angle = $(\sigma t - kx)$
k = wave number = $2\pi/L$
L = save length

The hydrodynamic drag forces F_c and F_T are given in terms of drag coefficients C_D by [9]:

$$F_c = \int_0^{h+\eta_c} C_{D_c}\rho\frac{u_c^2}{2}D ds' = C_{D_c}\frac{\gamma D h^2}{2}\cdot A \tag{20}$$

$$F_T = \int_0^{h+\eta_c-H} C_{D_T}\rho\frac{u_T^2}{2}D ds' = C_{D_T}\frac{\gamma D h^2}{2}\cdot B \tag{21}$$

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Journal of Environmental Engineering and its Scope
Volume 1 Issue 3

u_c = particle velocity under the wave crest
u_T = particle velocity under the wave trough
ρ = density of water
γ = specific weight of water
C_D = drag coefficient for cylinder
s' = instantaneous elevation of particle
D = cylinder diameter
η_c = wave amplitude at crest
h = depth to s.w.l.

The term A in equation (20) is expressed as:

$$A = \int_0^{1+\eta_{c/h}} \frac{u_c^2}{gh} d(s'/h) \tag{22}$$

The term B in equation (21) is expressed as:

$$B = \int_0^{1+\frac{\eta_c - H}{h}} \frac{u_T^2}{gh} d(s'/h) \tag{23}$$

The above terms are evaluated using equations for particle velocity under wave crest (u_c) and under wave trough (U_T). The equations for u_c and U_T were proposed by Stokes [14] as third approximation for large amplitude waves in water of finite depth. Others [14] have provided wave equations and numerical tables. The drag coefficients in the crest and trough regions are computed from a standard steady state plot of C_D versus Reynolds number. The Reynolds numbers are given as follows [16].

$$R_c = \frac{\sqrt{\bar{u}_c^2 D}}{\nu} = \frac{D}{\nu}\sqrt{ghA} \tag{24}$$

$$R_T = \frac{D}{\nu}\sqrt{ghB} \tag{25}$$

The distance from the bottom to the line action of the crest wave F_c, designated as s_c is given by [9]:

$$\frac{\bar{s}_c}{h} = \frac{1}{A}\int_0^{1+\eta_{c/h}} \frac{u_c^2}{gh} \cdot \frac{s'}{h} \cdot d(s'/h) \tag{26}$$

For the trough wave F_T, the distance from the bottom to the line of action is represented as S_T and given by [9]:

$$\frac{\bar{s}_T}{h} = \frac{1}{B}\int_0^{1+\frac{\eta_c - H}{h}} \frac{u_T^2}{gh} \cdot \frac{s'}{h} \cdot d(s'/h) \tag{27}$$

Studies have shown that s_c can be assumed constant over a range of $0° \leq \theta \leq 180°$ and S_T can be assumed constant over a range of $180° leq\theta \leq 360°$. [9].

From the Newtonian equation of motion for an object in an accelerating flow field, the inertia force is given by:

Journal of Environmental Engineering and its Scope
Volume 1 Issue 3

$$F_I = C_M \rho \frac{\pi D^2}{4} \int_0^h a_{x_o} ds \tag{28}$$

Where,

C_M = added mass coefficient

a_{x_o} = horizontal acceleration at $\theta = 0°$

The integral can be evaluated using Stokes third order tables [14]. Research suggests CM could be in the order of 2.0 9. Therefore, substituting into equation (28), we have,

$$F_I = \frac{\gamma \pi D^2 H}{4} \tanh kh \tag{29}$$

The line of action of the inertia is given as,

$$\frac{\bar{s}_I}{h} = \frac{1 + kh \sinh kh - \cosh kh}{kh \sinh kh} \tag{30}$$

This assumed constant over all ranges of θ.

Fourier series can be used to approximate the wave functions given in equations (18) and (19) as follows:

Let the resultant wave force F on the monopile tower be given as,

$$\begin{aligned} F = a_o + \sum_{n=1}^{\infty}(a_n \cos n\theta + b_n \sin n\theta) &= F_c \sin^2\theta + F_I \cos\theta (0° \leq \theta \leq 180) \\ &= -F_T \sin^2\theta + F_I \cos\theta (180° \leq \theta \leq 3) \end{aligned} \tag{31}$$

Where,

$$\begin{aligned} a_o &= \tfrac{1}{2\pi}\int_0^{2\pi} F d\theta \\ a_n &= \tfrac{1}{\pi}\int_0^{2\pi} F(\cos n\theta) d\theta \end{aligned}$$

Where

$$\begin{array}{cc} a_0 = \frac{F_c - F_T}{4} & b_n = \frac{1}{\pi}\int_0^{2\pi} F(\sin = \theta) d\theta \\ a_1 = F_I & b_1 = \frac{4}{3\pi}(F_c + F_T) \\ a_2 = -\left(\frac{F_c - F_T}{4}\right) & b_2 = 0 \\ a_3 = 0 & b_3 = -\frac{4}{15\pi}(F_c + F_T) \end{array}$$

From which,

$$\begin{aligned} F &= \frac{F_c - F_T}{4} + F_I \cos\theta + \frac{4}{3\pi}(F_c + F_T)\sin\theta \\ &\quad - \left(\frac{F_c - F_T}{4}\right)\cos 2\theta - \frac{4}{15}\pi (F_c + F_T)\sin 3\theta \ldots \end{aligned} \tag{32}$$

The last two terms contribute very little to the total force and can be ignored. Noting that $\cos\theta = \sin(\theta + 90°)$, we can rewrite equation (32) as,

$$F = \frac{F_c - F_T}{4} + F_I \sin(\theta + 90^\circ) + \frac{4}{3\pi}(F_c + F_T)\sin\theta \tag{33}$$

The sine terms are valid for $\theta \le \theta \le 360^\circ$.

2.3. **Monopile Displacement.** The influence fraction given as equation (9) can be written for crest wave as,

$$f_c = \frac{P_c}{F_c} = \left(\frac{\bar{s}_c}{\ell}\right)^2 - 2\left(\frac{\bar{s}_c}{\ell}\right)^3 \tag{34a}$$

For trough wave, it is written as,

$$f_T = \frac{P_T}{F_T} = 3\left(\frac{\bar{s}_T}{\ell}\right)^2 - 2\left(\frac{\bar{s}_T}{\ell}\right)^3 \tag{34b}$$

And in terms of inertia force as,

$$f_I = \frac{P_I}{F_I} = 3\left(\frac{\bar{s}_I}{\ell}\right)^2 - 2\left(\frac{\bar{s}_I}{\ell}\right)^3 \tag{34c}$$

Appropriate values of $\bar{s}/l$ can be calculated from equations (26), (27) and (28).

The Fourier series for the forcing function at the hub is obtained from the wave force equation (33).

Therefore,

$$P = \frac{F_c f + c - F_t f_T}{4} + j\frac{4}{3\pi}(F_c f_c + F_T f_T)\sin\theta + F_I f_I \sin(\theta + 90^\circ) \tag{35}$$

In simplifying the notation, let

$$\begin{aligned} P_1 &= \tfrac{F_c f_c - F_T f_T}{4} \\ P_2 &= \tfrac{4}{3\pi}(F_c f_c + F_T f_T) \\ P_3 &= F_I f_I \end{aligned}$$

Noting that $\theta = (\sigma t - kx)$ and substituting in equation (35) we have,

$$P = P_1 + P_2 \sin\sigma(t - \frac{kx}{\sigma}) + P_3 \sin\sigma(t - \frac{kx}{\sigma} + \frac{90}{\sigma}) \tag{36}$$

where

$$\begin{aligned} t' &= t - \tfrac{kx}{\sigma} \\ \text{and } t'' &= t - \tfrac{kx}{\sigma} + \tfrac{90^\circ}{\sigma} \end{aligned}$$

$$P = P_1 + P_2 \sin\sigma t' + P_3 \sin\sigma t'' \tag{37}$$

Each of the term in the above equation gives rise to a displacement X given in equa- .on (2). The $P1$ term is given by,

$$X_1 = \frac{P_1}{K} \tag{38a}$$

Journal of Environmental Engineering and its Scope
Volume 1 Issue 3

Simplifying the notations, let the denominator of equation (2) be expressed as:

$$G = \sqrt{\left[1 - \left(\frac{\sigma}{\sigma_n}\right)^2\right]^2 + \left[2\frac{C\sigma}{C_c\sigma_n}\right]^2} \tag{38b}$$

$$\begin{aligned} t' &= t \\ t'' &= t + \frac{90^\circ}{\sigma} \end{aligned}$$

Therefore, from equation (2), the instantaneous displacement due to the forcing term P_2 in equation (37) is given by,

$$X_2(t) = P_2/KG[\sin(ot - \phi)] \tag{39}$$

Similarly, the instantaneous displacement due to the forcing term P_3 is given by,

$$X_3(t) = P_3/KG[\sin(ot + 90^\circ - \phi)] \tag{40}$$

Following equation (6), we can write the total displacement of the monopile at any instant of time as,

$$X_{tot.}(t) = X_1 + X_2(t) + X_3(t) \tag{41}$$

Soil-Structure Interactions

Monopile wind turbines are subjected to a combination of environmental load that are cyclic and dynamic in nature. The main function of the foundation is to distribute these loads safely to the supporting soil. In addition, In addition, the behavior of saturated soils under cyclic and dynamic loading is quite complex and not well comprehended making the design of the foundation for monopile structures very challenging. Besides gravity loads due to the self-weight of the turbine, the three primary lateral loads acting on o shore wind turbine (OWT) structures are: wind, wave, and vibration (caused by the mass and aerodynamic imbalances of the rotor). The vibration load (forcing function) has a frequency equal to the rotational frequency of the rotor and it's often referred to as the 1P loading. 1P is a frequency band between the frequencies associated the lowest and the highest revolutions per minute. The 2P/3P loading refers to vibrations in the tower caused by the blade shadowing effects. In this situation, the blades of the wind turbine rotating in front of the tower cause a shadowing effect which results in the loss of wind load on the tower. Since this is a dynamic load, the resulting frequency for a three bladed wind turbine is equal to three times the rotational frequency of the turbine (3P). While the resulting frequency for a two bladed turbine is equal to two times the rotational frequency of the turbine (2P). The (2P/3P) loading is a frequency band and its obtained by multiplying the limits of the 1P band by the number of turbine blades [10]. Figure 4 shows a diagram of the main frequencies of the loads described above for a 3 bladed 3 MW turbine. The figure shows the 1P and 3P frequency bands. To avoid resonance, and fatigue damage, the design engineer has to select a system frequency (global frequency of the entire wind turbine and the foundation) which lies outside these frequencies. Based on the natural frequency f_o of the wind turbine structure, three types of design are possible:

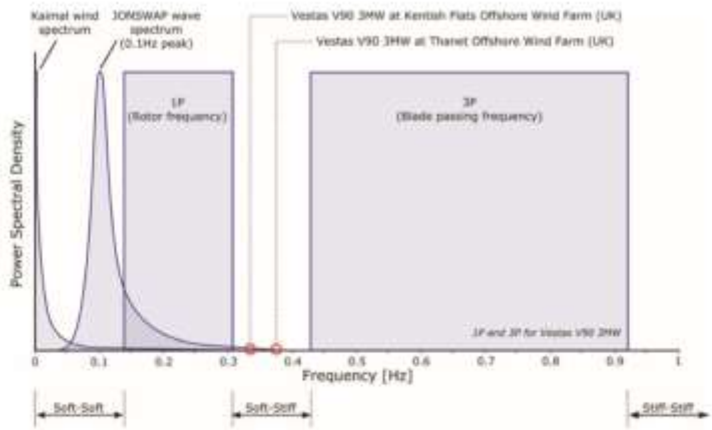

Fig. 4. *Frequency diagram for the Vestas V90 3 MW wind turbine. The frequency diagram includes Wind Spectrum, Wave Spectrum, 1P and 3P frequency bands. Note: The amplitudes are normalized to unity to focus on the frequency content of the loading. Adapted from S. Bhattacharya, G. Nikitas, L. Arany, N. Nikitas Soil structure interactions for o shore wind turbines IET Eng Technol Ref (2017), 10.1049/etr.2016.0019*

- Soft-Soft design the natural frequency is placed below the 1P frequency range. Designing in this range will result in a very flexible structure and quite impossible for a grounded system.
- Soft-Stiff design - in this design, the natural frequency or the resonant frequency is placed close to the upper end of 1P (frequency matching the rated power of the turbine) and lower bound of the 3P (cut-in wind speed of the turbine). Contemporary design strives to locate the natural frequency of the entire turbine structure in this range.
- Stiff -Stiff design the natural frequency is higher than the upper limit of the 3P band resulting in a very stiff support structure.

Since, wind velocity and wave height on the sea are both variables, they are best treated statistically using Power Spectral Density (PSD) functions [10]. This means loads produced from each case are analyzed in the frequency domain range such that the contribution of each frequency to the total power in wind turbulence and ocean waves is accounted for [10]. Another procedure is to construct representative wind and wave spectra using Discrete Fourier Transform from site specific data [16][17]. DNV design standard states that the Kiamal spectrum for wind and the JONSWAP (Joint North Sea Wave Project) spectrum for waves in o shore wind turbines may be used.

Practically speaking, it is obvious from Figure 4 that the natural frequency of an offshore wind turbine needs to be fitted within a narrow band (between 1P and 3P). Since OWT design is sensitive to the prediction of the first natural frequency, it would seem logical that the safest solution is to place the natural frequency of the wind turbine above the 3P range. This would lead to a stiffer design which may be cost prohibitive. Therefore, a softer structure is more desirable from an economic point of view. Current trend in the design of o shore wind turbine is the “soft-stiff” approach. As a result of the sensitive nature of the design to the first natural frequency, the consideration of dynamic amplification and to potential change in system frequency due to cyclic/dynamic loading is imperative. A change in the natural frequency will enhance the dynamic amplifications which will increase the vibration amplitudes and may cause fatigue damage of on the turbine.

Model for Soil Structure Interaction

Several factors contribute to the loading response from the soil on an o shore wind turbine foundation. One mechanism is the linear response from the surrounding soil, a situation where soil reaction is linearly dependent on the displacement of the foundation. In case of a small displacement, the soil response can be approximated by a linear model. How-ever, for large displacements, non-linearities and soil deformations can be cogent. Linear models also fail to account for the dynamic behavior of the soil. Soil-structure interaction models for monopile foundations will be addressed in the following section.

Linear Foundation Models

The response of the soil at a single point at the mudline is represented by a stiffness matrix. The response from the soil at the mudline, F is given as [18]:

$$F = \begin{bmatrix} H \\ M \\ V \end{bmatrix} = \begin{bmatrix} k_{uu} & k_{u\theta} & 0 \\ k_{\theta u} & k_{\theta\theta} & 0 \\ 0 & 0 & k_w \end{bmatrix} \begin{bmatrix} u \\ \theta \\ w \end{bmatrix} \qquad (42)$$

Where U is the horizontal displacement, w is the vertical displacement and is the angular displacement. The corresponding forces and moments are given by H, M and V. The stiffness coefficients for each displacement are represented by k. This method is quite easily implemented in a finite element analysis program. Several methods exist for calculating the stiffness coefficient and are discussed below.

Stiffness Coefficient by Effective Fixity Length

This approach replaces the soil with an extended pile length, fixed below the mudline at a length l. The extended length represents the soil response since the pile is free to rotate at the mudline. A pile length of four times pile diameter provides the best result [18]. Other studies have suggested pile length ranging from 3.3 to 8 times pile diameter depending on soil conditions [19]. The stiffness coefficient is expressed in terms of a sti ness matrix as follows:

$$K = \frac{3EI}{\ell^3} \begin{bmatrix} 6 & -3\ell & 0 \\ -3\ell & 2\ell^2 & 0 \\ 0 & 0 & \frac{A\ell^2}{2I} \end{bmatrix} \qquad (43)$$

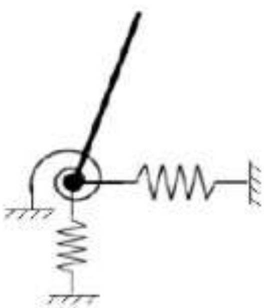

Fig. 5. *Uncoupled springs model Adapted from \Foundation models for the dynamic response of o shore wind turbines," M.B. Zaaijer. In: Proceedings of MAREC 2002, UK, September 2002.*

Where:
E = modulus of elasticity of the pile

A = cross-sectional area of the pile I = moment of inertia of the pile

Stiffness Coefficient by Uncoupled Springs
The stiffness of the soil-pile system may be represented as a set of uncoupled springs at the mudline as shown in Figure 5 below. The uncoupled model only has stiffness coefficients along the diagonal matrix since the degrees of freedom do not interact. The stiffness coefficients of the springs can be determined by static analyses using a reference model such as finite element model of the soil-structure system. The spring stiffness can be found by using the displacements at the mudline. It is obtained by solving the following equation:

$$\begin{bmatrix} H \\ M \\ V \end{bmatrix} = \begin{bmatrix} k_{uu} & 0 & 0 \\ 0 & k_{\theta\theta} & 0 \\ 0 & 0 & k_w \end{bmatrix} \begin{bmatrix} u \\ \theta \\ w \end{bmatrix} \qquad (44)$$

Stiffness Coefficient by Coupled stiffness matrix from a reference model
For a coupled model, the stiffness matrix has additional coefficients compared with uncoupled model. Using different load combinations from a reference model, a coupled matrix can be developed. Stiffness coefficients can be obtained from the following equation [1]:

$$\begin{bmatrix} F_1 \\ M_1 \\ F_2 \\ F_3 \end{bmatrix} = \begin{bmatrix} x_1 & \theta_1 & 0 & 0 \\ 0j & x_1 & \theta_1 & 0 \\ x_2 & \theta_2 & 0 & 0 \\ 0 & 0 & 0 & k_z \end{bmatrix} \begin{bmatrix} k_{xx} \\ k_{x\theta} \\ k_{\theta\theta} \\ k_z \end{bmatrix} \qquad (45)$$

Accounting for Energy Dissipation by Adding Damping Matrix
To account for energy dissipation, a damping matrix obtained at the mudline can be added to account for energy dissipation in the soil. The dynamics at the mudline will then be represented by:

$$\begin{bmatrix} H \\ M \\ V \end{bmatrix} = \begin{bmatrix} k_{uu} & k_{u\theta} & 0 \\ k_{\theta u} & k_{\theta\theta} & 0 \\ 0 & 0 & k_w \end{bmatrix} \begin{bmatrix} u & \\ \theta & w \end{bmatrix} + \begin{bmatrix} c_{uu} & c_{u\theta} & 0 \\ c_{\theta u} & c_{\theta\theta} & 0 \\ 0 & 0 & c_w \end{bmatrix} \begin{bmatrix} \dot{u} \\ \dot{\theta} \\ \dot{w} \end{bmatrix} \qquad (46)$$

Nonlinear Foundation Models
Extreme loads can lead to permanent soil deformation which can also result in stiffening/softening of the soil surrounding the foundation. These may significantly affect the eigen frequencies of the monopile structure. Therefore, for higher loads, a nonlinear model will be a better representation. A common non-linear model for determining the dynamic response of OWT considering soil-structure interaction is the p y curve method. The stiffness of the soil is modeled with a series of independent nonlinear springs along the pile length as shown in Figure 6 [20]. The relationship between the lateral displacement of the pile and the soil reaction determines the behavior of the springs. The standard beam column equation is used to describe the response of the pile, with the spring representing the resistance of the

soil. The beam column equation is given by [21]:

$$\frac{d^2}{dx^2}\left(E_p I_p \frac{d^2 y}{dx^2}\right) + P_x\left(\frac{d^2 y}{dx^2}\right) - p(y) - W = 0 \tag{47}$$

Where:
P_x = Axial load on the pile
y = Lateral deflection of the pile along the pile length p = Resistance per unit length
W = Distributed load along the length of the pile
E_p = Elastic modulus of the pile
I_p = Area moment of inertia of the pile

The springs are described by nonlinear functions (p y curves) to de ne the soil reaction p, at a given depth as a function of the lateral displacement, y. The p y approach was originally based on the results of field tests on long slender piles with diameters around 610 mm and an aspect ratio of 34. However, current monopile diameters are significantly larger than those for which the p y method is based. Furthermore, the design conditions for the o shore monopiles are different from the oil and gas structures for which the p y curve was originally based. The fatigue design and dynamic performance of the monopole is different from the slender piles of the oil and gas industry for which the p y approach was originally based. Therefore, a new design method tailored to for o shore wind turbine has been developed [22].
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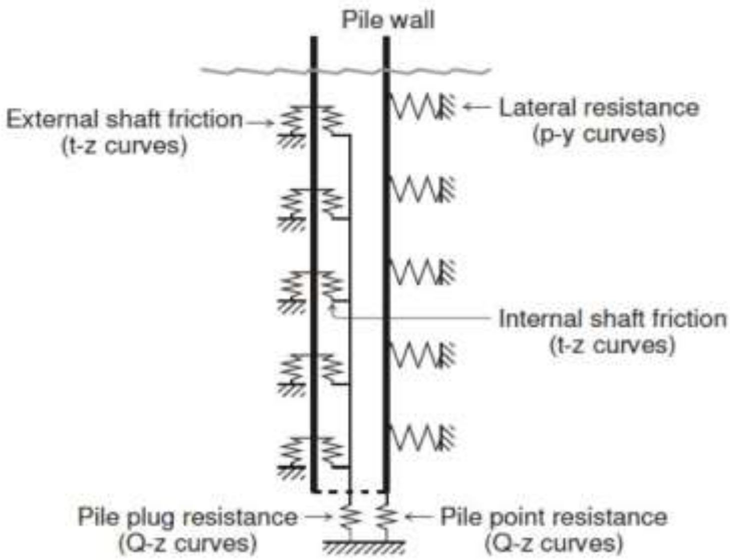

Fig. 6. *Spring model for pile-soil interaction. Adapted from \Sensitivity analysis for foundations of o shore wind turbines," by M.B. Zaaijer. Section Wind Energy WE 02181, Delft.*

New Design Approach
A new design approach has been developed by a consortium of the Pile Soil Analysis (PISA) industrial group and an Academic Work Group (AWG). The design method pro-posed is based on an extension of the p-y approach. The new method considers other significant interaction mechanisms between the pile and the soil for short, large diameter monopiles. Four components of soil reaction (also known as soil reaction curves) are considered in the proposed model as shown [22]:

Distributed load curve represents the relationship between the distributed lateral load, p, applied to the pile and the lateral displacement v. This is essentially the same as the conventional p y curve. Distributed moment curve establishes the relationship between the distributed moment, m, applied to the pile and the pile cross-sectional rotation. The distributed moment is a function of the vertical shear stress developed on the pile shaft. Base shear curve denotes the relationship between the base shear force, S and the lateral displacement of the pile toe.

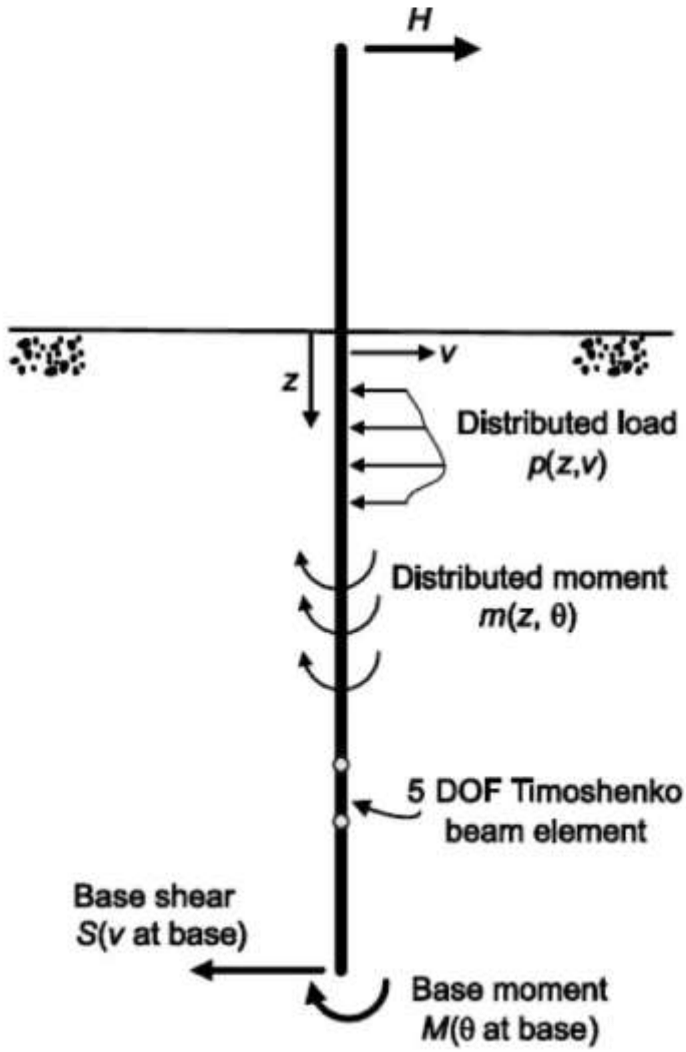

Fig. 7. *New 1D nite model for monopile foundations. Adapted from B. W. Byrne et al., New design methods for large diameter piles under lateral loading for o shore wind applications, in Frontiers in O shore Geotech-nics III - Proceedings of the 3rd International Symposium on Frontiers in O shore Geotechnics, ISFOG 2015, 2015.*

Base moment curve describes the relationship between the base moment, M and the rotation of the pile toe.

The model shown in Figure 7 was performed in the 1D finite element frameworks. The pile was modeled using Timoshenko beam theory. The 1D finite element model study suggested that the influence of the distributed moment, base shear and base moment are small for long piles but quite significant for small piles. Therefore, for piles with large diameter to height ratio, the diameter effect associated with the aforementioned parameters is significantly high

[22]. To determine appropriate soil reaction curves for use in the 1D model, a detailed 3D finite element parametric study was conducted [23]. Two ground conditions likely to be encountered in the North Sea, a stiff over consolidated clay and dense sand were selected for analyses. 3D finite element analyses of test piles were performed for the two sites. Results indicated a decrease in load capacity with increasing height above the mudline and an increase in pile load capacity with increasing length of embedment. A rigid body deformation of shorter piles and longer piles behaving in a flexible manner was observed. The shorter piles also showed large displacement at pile base. Therefore, additional soil reaction components must be considered. The analyses results showed that the likely mechanisms of monopile/soil interaction for short piles in particular cannot be fully rationalized by the conventional p y methodology.

Two practical ways to apply the new design method would be, one to serve as a rule of thumb. In this case soil reaction curves based on pre-defined soil functions would be used in determining strength and stiffness parameters of the soil. The second approach known as numerical-based method would involve conducting a detailed site investigation and laboratory testing to establish strength and nonlinear parameters of a given site. Soil reaction curves could then be determined using 3D finite element analysis from a set of laterally loaded monopiles. The soil reaction curves would be extracted and incorporated within the 1D modeling frameworks.

CONCLUSION

The analytical procedure for the dynamic displacement of monopile foundation has been presented. The dynamic displacement is commonly used in the stress analysis of the entire structure. The common practice in the design of o shore structures is to use static analysis based on the on the period of the highest wave. This approach may be correct because the period of the highest wave may be large compared with the natural period of the monopile structure. However, situations may occur in which smaller waves with lesser period may approach resonance with the monopile structure. In which case stresses under near resonance condition may exceed stresses of the statical design wave. Therefore, proper consideration should be given to the dynamic behavior in the design of monopile structures.

Based on the development of the numerical modeling of o shore wind turbine monopiles using 3D finite element analysis, the development of a new design method and field testing, it has been revealed that the conventional method used for piles under lateral loading, known as the p-y method is not suitable for short and stubbier piles. Results showed the influence of other significant interaction mechanisms such as distributed moment, base shear and base moment on the total response.

REFERENCES

1. M. Zaaijer, *Foundation modelling to assess dynamic behaviour of o shore wind turbines,"* Applied Ocean Research, vol. 28, pp. 45{57, September 2006.
2. M. Kuhn, *Modal analysis of the nedwind 40 tower - comparison of measurements and calculations for the sites oostburg and lely*," Delft: Institute For Wind Energy, 1995.
3. M. Zaaijer, *Sensitivity analysis for foundations of o shore wind turbines*," Section Wind Energy, vol. WE 02181, Delft 2000.
4. L. Reese and S. Wang, *Verification of computer program lpile as a valid tool for disign of a single pile under lateral loading."* [Online].
5. Available: http://www.ensoftinc.com/main/products/pdf/Lpile-Validation Notes.pdf