Simplified Design Procedure of Monopile Foundation for Offshore Wind Turbine in Gujarat, India

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Abstract: OWTs (offshore wind turbines) are currently considered as a reliable source of renewable energy. OWT support structures account for 20%-25% of the capital cost for offshore wind installations. Pre-feasibility studies involving estimation of preliminary dimensions of the wind turbine structure need to be performed for initial costing to arrive at the commercial viability of the project. The main objective of the paper is to obtain preliminary configuration for commercial viability and approximate sizing of the foundation pile. Design equations and nomograms are proposed for quick preliminary design of monopile founded wind turbines located offshore of Gujarat. Parametric studies are carried-out on various configurations of a hollow monopile by varying water depths and properties of sand. A nonlinear static analysis of substructure is performed considering aerodynamic forces and hydrodynamic forces for various structural and soil parameters. The sub-structure design of wind turbine is based on API (American petroleum institute) standards. A simplified design methodology for monopile support structure under extreme loading condition is presented based on multivariable linear regression analysis. The input variables for the regression analysis are hydrodynamic data, angle of internal friction of sand, and the output variables are length and outer diameter of monopile. This simplified methodology is applicable in pre-studies of wind power parks.

Key words: OWT, support structures, monopile, regression analysis, nomogram.

1. Introduction

An integrated perspective of energy use and environmental preservation can be achieved by the utilization of renewable energy resources. Renewable energy resources account for 12.2% of India’s installed capacity, with 70% contribution from wind sector [1]. Presently, the focus is shifting towards offshore wind energy due to favorable factors like higher wind speed, low visual intrusion and noise, easy transportation of higher capacity wind turbine, lesser transmission costs for coastal cities, conservation of utilisable land and over exploitation of potential onshore sites.

India currently ranks as the fifth largest onshore wind market in the world and is yet to develop its offshore wind potential. Countries like Denmark generate almost 40% of their electricity from wind farms, with around 1/4 contribution from offshore wind power [2]. For developing countries like India, reducing the cost to benefit ratio will help implement offshore wind farms and move towards sustainable energy production. The MNRE (ministry of new and renewable energy) has identified the offshore regions of Rameswaram and Kanyakumari in Tamilnadu, the gulf of Kutch and the gulf of Kambhat in the state of Gujarat, as potential zones in India for installation of OWTs (offshore wind turbines). The country’s first offshore wind farm is likely to come up in Gujarat. Feasibility studies show that the region has shallow water depths of 10 m to 20 m with predominant soil being silty sand. Monopile would be one of the ideal sub-structure solutions for these conditions [3]. Fig. 1
shows that historically, monopiles have dominated the offshore wind market. More than 75% of the OWTs in Europe (i.e., UK, Denmark, Germany, and Netherlands) are supported on monopiles [4].

A simplified mechanical model of a monopile founded OWT is shown in Fig. 2.
Real loads acting on the offshore structure are dynamic in nature and the exact dynamic loads are difficult to evaluate for analytical studies [6]. The load identification in structural dynamics is an ill-posed problem since the response is defined at only certain points of a continuous function. Hence no unique solution is guaranteed. In addition, the solutions are frequently found to be unstable where small perturbations of input data result in large changes in the calculated force magnitudes. To meet the quality and overcome the absence of a unique solution, an ESL (equivalent static load) is proposed by the researchers. The ESL is defined as the static load which generates the same displacement field as that under a dynamic load [6, 7]. Moreover, static analysis is more appropriate than dynamic analysis for pre-feasibility studies as it saves a lot of computational time and effort. This study investigates the sub-structure design of OWT using ESLs and nonlinear analysis. The novelty in the approach lies in relating important site parameters local to the individual mill to the preliminary dimensioning of the wind turbine foundation.

2. Methodology for Design of Monopile

A framework for obtaining the preliminary configuration and approximate sizing for monopile type sub-structure of OWT is shown in Fig. 3. The study
involves nonlinear static analysis using SACS (structural analysis computer system) and MLRA (multivariable linear regression analysis) using statistical tools like MS Excel and Minitab.

### 2.1 The Monopile Supported OWT Model

The reference wind turbine used in the study is 5 MW NREL wind turbine configuration, as the data are available from the published literature [8] and the turbine capacity is similar to those adapted in the site. The modeling, analysis, and design are carried-out using SACS. Fig. 4 shows the SACS model of 5 MW NREL wind turbine. Homogeneous layer of silty sand is considered up-to a depth of 60 m from the mudline. A density 7,850 kg/m³ for steel pile and transition piece is used in the study. Tables 1 and 2 show the properties of 5 MW NREL wind turbine structure.

### 2.2 Nonlinear Static Analysis

Wind is the dominant load on fixed OWT structures. Calculation of aerodynamic force at the rotor level requires information related to the wind field characteristics, including the wind shear and the mean wind speed [9]. The aerodynamic loads on turbine are obtained using FAST developed by NREL (national renewable energy laboratory) based on BEM (blade element momentum) theory, for standard 5 MW NREL wind turbine. The predominant frequency for the aerodynamic thrust force is obtained through FFT (Fast Fourier Transform) for various wind conditions (Fig. 5). Relevant information is taken from the design standards IEC 61400-1(2005) and IEC 614000-3(2009) [10, 11].

The aerodynamic load in time-domain is converted to ESL using DAF (dynamic amplification factor). DAF is a dimensionless number which describes the number of times the deflections or stresses that should be multiplied to the deflections or stresses caused by the static loads when a dynamic load is applied to a structure [12]. Since a dynamic load can have a significantly larger effect than a static load of the same magnitude due to the structure’s inability to respond quickly to the loading, the increase in the effect of a dynamic load is accounted in terms of the DAF. For an

![Fig. 4 SACS model of 5 MW NREL wind turbine.](image-url)
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Fig. 5 Fast Fourier transform.

Fig. 6 DAF for damped SDOF system subjected to harmonic force [12].

SDOF (single degree of freedom) system subjected to harmonic vibration with viscous damping, DAF is calculated based on Eq. (1). The graphical representation of DAF vs. frequency ratio for damped SDOF system excited by harmonic force is shown in Fig. 6.

\[
DAF = \frac{x_{dyn}}{x_{st}} = \frac{1}{\sqrt{[1-(\frac{w}{w_n})^2] + (2\xi(\frac{w}{w_n})^2]}}
\]

(1)

where, \(x_{dyn}\) = maximum dynamic displacement, \(x_{st}\) = maximum static displacement, \(w\) = forcing frequency, \(w_n\) = natural frequency of the structure, \(\xi\) = structural damping ratio.

The frequency plot for extreme coherent gust with direction change at rated wind speed (ECD+R) is shown in Fig. 7. The first modal frequency of the wind turbine structure (\(\omega_n\)) is used for obtaining DAF since it lies in the range of 75% to 120% of the excitation frequencies and as a result, dynamic amplifications of responses are expected [6]. Thus, the dynamic load is converted to ESL using DAF. The maximum thrust force of 1.2 MN at the rotor level of the turbine of self-weight 3,434 kN is obtained for ECD+R wind condition.

Extreme wind speed value of 50 m/s for gulf of Kambhat and gulf of Kutch regions is obtained using IS 875-part 3-1987 code provision [13]. The total wind force on the tower is calculated using SACS software tool based on API standards.
For wind load calculation, the structure is broken into elements of different geometrical shapes and the wind load on the geometrical shapes can be calculated using Eq. (2):

\[ F = \frac{1}{2} \rho \times C_s \times U^2 \times A \]  

(2)

where, \( \rho \) = mass density of air, \( C_s \) = shape coefficient, \( U \) = wind speed, \( A \) = area of object.

It is necessary to obtain wind speed vertical distribution \( U(z) \) and the value of the shape coefficient in Eq. (2). This coefficient is determined by experimental methods and its value depends on: (1) the shape of the cross-section of the frame subjected to the wind, (2) the flow Reynolds number, (3) the roughness of the body, and (4) shielding effects. The variation of wind intensity is generally in time and space. Statistical wind properties (like mean speed and standard deviation of speed) taken over durations of the order of one hour vary with elevation.

The 1 hour means wind speed \( U(z) \) (ft/s) at height \( z \) (ft) above the sea level which is calculated based on Eqs. (3) and (4):

\[ U(z) = U_o \times [1 + C \times \ln\left(\frac{z}{32.8}\right)] \]  

(3)

\[ C = 5.73 \times 10^{-2} \times (1 + 0.0457 \times U_o)^{1/2} \]  

(4)

where, \( U_o \) (ft/s) is the standard wind which is 1 hour means wind speed at reference height 32.8 ft (10 m) above sea level.

Wind load is calculated using Eq. (2) for the wind speed as per IS 875-part 3-1987.

The hydrodynamic load due to extreme waves of period 12 s and breaking wave height of 0.78 times the water depth at the location is calculated based on Morison’s equation (Eq. (5)). This equation expresses the wave force as the sum of an inertia force proportional to the particle acceleration and a non-linear drag force proportional to the square of the particle velocity.

\[ F = C_M \frac{\rho n v^2 a}{4} + C_D \frac{\rho |v|^2}{2} \]  

(5)

where, \( F \) = wave force per unit length on a circular cylinder (N), \( n \), \( |v| \) = water particle velocity normal to the cylinder, calculated with the selected wave theory at the cylinder axis (m/s), \( a \) = water particle acceleration normal to the cylinder, calculated with the selected wave theory at the cylinder axis (m/s²), \( \rho \) =
water density (kg/m³), $D =$ member diameter (m), $C_D, C_M$ are drag and inertia coefficients, respectively.

The fluid velocities and accelerations used in Morison’s equation are calculated based on Airy’s wave theory, chosen from API chart using the wave steepness and relative depth parameters (Fig. 8). Both wave and current loads are applied along the same direction for maximum loading. A $360^\circ$-wave is applied with $10^\circ$ phase increments. As the monopile is axi-symmetric, any direction for the load could be considered along with the corresponding wind effects to produce the maximum effect on the monopile.

Current forces are calculated using power law as shown in Eq. (6).

$$U_c(z) = U_{co} \left( \frac{z+h}{h} \right)^{\alpha}$$

where, $U_c(z)$ is the current speed at elevation $z$, $z = 0$ at the surface, $U_{co}$ is the surface current speed, $h$ is the water depth, $\alpha$ is an exponent, typically $1/7$. The

![Fig. 8 Regions of applicability of different wave theories [14].](image-url)
different forces applied on the wind turbine for a specific case is shown in Fig. 9.

PSI (pile structure interaction) in SACS, analyzes the behavior of a pile supported structure subjected to one or more static load conditions. Finite deflection of the piles (“P-delta” effect) and nonlinear soil behavior along directions both longitudinal and transverse to the pile axis are accounted for. The program uses a distributed spring type flexible foundation model and employs finite difference technique to solve the pile model which is represented by a beam column on nonlinear elastic foundation. Beam-column elements model structural members to resist both axial and bending actions. The structure resting on the piles is represented as a linear elastic model. In general, soil exhibits nonlinear behavior for both axial and transverse loads, therefore an iterative procedure is used to find the pile influence on the deflection of the structure [15]. The analysis is generally based on the theory of subgrade reaction. The pile is discretized into segments with nodes at each end of the segments, and the soil around the pile is replaced by a series of discrete springs. Fig. 10 shows a typical pile-soil model.

The equation governing the pile-soil interaction for an elastic pile (based on Bernoulli-Euler beam theory) is given in Eqs. (7) and (8).

\[ EI \frac{d^4 y}{dx^4} + P_x \frac{d^2 y}{dx^2} - p - W = 0 \]  
and substituting,

\[ p = -K_y \]

where, \( y \) is the lateral deflection of pile, \( x \) is the pile length along “x” direction, \( EI \) is the flexural rigidity of the pile, \( P_x \) is the axial load, \( p \) is the lateral soil reaction per unit length, \( K \) is modulus of subgrade reaction, \( W \) is the distributed load along the length of pile.

The differential equation in Eq. (7) is solved by subdividing the pile into discrete elements and expressing the equation in difference forms. A finite difference is a technique by which derivatives of functions are approximated by differences in the values of the function between a given value of the independent
variable, say $x_0$, and a small increment $(x_0 + h)$. It provides an alternative way to the conversion of continuum field equations into relationships between discrete numerical values [18]. With the FDM (finite difference method) used, the pile is discretized into $n$ segments of length $h$ (Fig. 11). Each segment contains imaginary nodes which are used to obtain solutions. The beam-column equation is applied at each node to solve for pile deflection. The initial modulus of subgrade reaction ($k_o$) is obtained based on the API chart on $k_o$ as function of the angle of internal friction of soil, proposed by Reese et al. [19]. Boundary
conditions are employed at pile head and at the toe of the pile. Any of the two boundary conditions from shear \((V)\), moment \((M)\), slope \((S)\), rotational stiffness \((M/S)\), and deflection \((Y)\) are employed at the pile head. The two boundary conditions that are imposed at the toe of the pile are based on moment and shear.

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\[ \frac{dy}{dx_{=n}} = \frac{y_{n-1} - y_{n+1}}{2h} \]  

\[ \frac{d^2y}{dx^2_{=n}} = \frac{y_{n-1} - 2y_n + y_{n+1}}{h^2} \]  

\[ \frac{d^3y}{dx^3_{=n}} = \frac{y_{n-2} - 2y_{n-1} + 2y_{n+1} - y_{n+2}}{2h^3} \]  

\[ \frac{d^4y}{dx^4_{=n}} = \frac{y_{n-2} - 2y_{n-1} + 6y_n - 4y_{n+1} + y_{n+2}}{6h^4} \]  

Eqs. (9)-(12) are the difference equations to evaluate the differential terms. These are substituted in Eq. (6) and solved iteratively using computer. The finite difference equations are solved for deflection of pile from point-2 through points till \(t+2\) (Fig. 11). The advantage of the FDM is that it usually takes a shorter computational time in comparison with the Finite Element method. After obtaining the pile deflection, the shear, bending moment and slope of the pile are calculated using the following Eqs. (13)-(15).

\[ EI \frac{d^4y}{dx^4} + P_x \frac{dy}{dx} = V \]  

\[ EI \frac{d^2y}{dx^2} = M \]  

\[ \frac{dy}{dx} = S \]  

The PSI module requires the input parameters such as pile dimensions, angle of internal friction of soil, coefficient of lateral earth pressure and submerged density of soil. The coefficient of lateral earth pressure \((K)\) can be calculated as the average of active \((K_a)\), passive \((K_p)\) and neutral \((K_o)\) earth pressures (Eq. (16)) since the soil around a driven pile is compressed during construction and the lateral earth pressure of this soil acting on the pile skin is greater than the earth pressure at rest (given by coefficient \(K_o\)) and smaller than the maximum earth pressure (passive earth pressure given by coefficient \(K_p\)) [20]:

\[ K = \frac{K_a + K_p + K_o}{3} \]  

where,

\[ K_a = \frac{1 - \sin \phi}{1 + \sin \phi}, K_p = \frac{1 + \sin \phi}{1 - \sin \phi}, K_o = 1 - \sin \phi, \]

\( \phi \) = angle of internal friction of soil.

The axial resistance of the soil is provided by a combination of load transfer along the sides of the pile and end bearing resistance at the pile tip [14]. The relationship between mobilized soil-pile shear transfer and local pile deflection at any depth is described using a \(t-z\) curve (Fig. 12). Similarly, the relationship between mobilized end bearing resistance and axial tip deflection is described using a \(Q-z\) curve (Fig. 13).

The ultimate lateral bearing capacity for sand is calculated using API formulae as (Eqs. (17) and (18)):

\[ P_{ux} = (C_1 \times H + C_2 \times D) \times \gamma \times H \]  

\[ P_{ud} = C_3 \times D \times \gamma \times H \]  

where, \(P_u\) = ultimate resistance (force/unit length), lbs/in. (kN/m) \((s = shallow, d = deep)\), \(\gamma\) = effective soil weight, lb/in\(^3\). (KN/m\(^3\)), \(H\) = depth, in. (m), \(\phi\) = angle of internal friction of sand, deg., \(C_1, C_2, C_3\) = coefficients (determined from Fig. 14a) as function of \(\phi\), \(D\) = average pile diameter from surface to depth, in. (m).

The lateral soil resistance-deflection \((p-y)\) relationships for sand are non-linear (Eq. (19)):

\[ P = A \times P_u \times \tanh \left( \frac{kHy}{Ap_u} \right) \]  

where,

\(A\) = factor to account for cyclic or static loading condition and evaluated by:

\(A = 0.9\) for cyclic loading.

\(A = (3 - 0.8H/D) \geq 0.9\) for static loading.

\(P_u\) = ultimate bearing capacity at depth \(H\), lbs/in. (kN/m), \(k\) = initial modulus of sub-grade reaction, lb/in\(^3\). (kN/m\(^3\)) (determined as function of angle of internal friction, \(\phi\) from Fig. 14b), \(y\) = lateral deflection, inches (m). \(H\) = depth, inches (m).

Fig. 15 shows the schematic illustration of \(p-y\) curves for analysis of lateral capacity of pile and the
Fig. 12  Soil axial load transfer (t-z) curve from SACS.

Fig. 13  Tip load-displacement (Q-z) curve from SACS.
Fig. 14  (a) Coefficients as function of $\phi^*$, (b) initial modulus of sub-grade reaction as function of $\phi^*$ [14].

Fig. 15  Schematic illustration of $p$-$y$ curves for analysis of lateral capacity of pile [21].

diagram showing relation between lateral soil resistance and pile deflection is shown in Fig. 16. The $p$-$y$ curve model in API was established based on the results of field tests (Reese et al. 1974, Matlock 1970, and Reese & Welch 1975) and adjusted mathematically using empirical parameters to extrapolate beyond the soil’s specific field test conditions [22].

Thus, a combined wind-wave-current static analysis is performed for extreme loading conditions along with soil parameters using PSI module in SACS.
2.3 Structural Design

The analysis and design are done for 80 different combinations of water depth, current velocity and angle of internal friction of soil. The input variables used in the analysis include water depth varying from 10 m to 30 m, current velocity varying from 0.5 m/s to 2 m/s and angle of internal friction of soil varying from 25° to 40° (Table 3). A homogeneous layer of soil is considered throughout the length of the pile for simplicity. The output variables include diameter \(D\), thickness \(t\), and minimum length of the pile \(L\). Thickness of the pile is obtained as a function of diameter, based on API standard [14].

Serviceability criteria is defined based on the tolerance requirements for the operation of the wind turbine and is often described as “turbine manufacturer requirements”. Ideally, these should be turbine specific, that is, size and the hub height, gear boxed on direct drive. Typically, these tolerances are specified in some codes of practice or a design specification supplied by the client that may be dictated by the turbine manufacturer. Maximum allowable rotation at pile head after installation (as per DNV code) is specified with 0.25° limit on “Tilt” at the nacelle level [6]. The maximum rotation at the pile head is limited to 0.22° and the UC (unity check) ratio is limited to 0.8.

3. Statistical Analysis for Monopile Configuration

MLRA (Multivariable Linear Regression Analysis) is a statistical analysis used for prediction of unknown parameters. It explains the linear relationship between
one continuous dependent variable (Y) and two or more independent variables (X). MLRA generally uses LSM (least squares method) to estimate the parameters. The LSM finds its optimum when the sum (S) of squared residuals is a minimum (Eq. (20)). A residual is defined as the difference between the actual value of the dependent variable (Y) and the value predicted by the model (Y’).

\[ S = \sum_{i=1}^{N}(Y_i - Y'_i)^2 \]  

(20)

The prediction of unknown dependent variable is accomplished by the following equation (Eq. (21)):

\[ Y'_i = b_0 + b_1X_{1i} + b_2X_{2i} + \ldots + b_KX_{Ki} \]  

(21)

The “b” values are called regression weights or regression coefficients. There are K predictor variables and K + 1 regression weights must be estimated, one for each of the K predictor variable and one for the constant (b0) term.

For the study, MLRA is performed on a sample size of 80 to obtain the desired equations for pile diameter and pile length. A sample size is a part of the population chosen for the study. As the sample size increases (n ≥ 30), the shape of sampling distribution is approximately normal irrespective of the population distribution [23]. It is recommended that the minimum number of samples for multiple regression analysis be at least ten times the total number of variables involved in the study [24].

The determination of influencing parameters for the dependent variables (i.e., pile diameter and pile length) is based on its p-value. A predictor that has a low p-value is likely to be a meaningful addition to the model because changes in the predictor’s value are related to changes in the response variable. Conversely, a larger (insignificant) p-value suggests that changes in the predictor are not associated with changes in the response. In other words, p-value indicates the probability of insignificance of a variable in the prediction model. A maximum p-value of 5% is allowed in the model. MLRA mainly involves the following steps:

1. Based on the data set, establish a mathematic model to estimate the unknown parameters. LSM is a common method of estimation. In the LSM, the unknown parameters are estimated by minimizing the sum of the squared deviations between the data and the model.

2. Credibility of the obtained relations is tested by p-value of predictors in F-test of the overall significance.

3. Introduce predictors with significant influence into the model, and eliminate those with no significant influence.

4. Predict the pile outer diameter and pile length using the derived relations.

The use of more variables in fitting regression equations reduces equations’ stability, and results in low reliability and precision in using these equations for prediction. Adoption of variables with less influence on dependent variable will cause shift and inconsistency in the estimated value. Therefore, optimal regression equation shall be taken into account where the equations involve all variables with significant influence on Y, but not those with insignificant influence [25]. Water depth and current velocity are found to be the influencing parameters for determining pile diameter. The influencing parameters for pile length are angle of internal friction of soil and MI (moment of inertia) of the cross-section. The prediction model is validated with eight different cases.

4. Results and Discussions

4.1 Multivariable Linear Regression Analysis

The regression analysis is performed on the sample data using MS Excel’s data analysis tool and the design equations are obtained. These equations are tested with new samples.

The equation for obtaining the optimum outer diameter (D) of monopile (Eq. (22)) with coefficient of determination, $R^2$ of 94% is:

\[ D = 4.25 + 0.072 \times WD + 0.112 \times C \]  

(22)
where,

WD: water depth (m)
CV: current velocity (m/s)

Figs. 17a and 17b show scatter plot and surface plot of pile outer diameter, respectively. A nomogram for obtaining the pile outer diameter for a given water depth and current velocity is shown in Fig. 17c. Minimum thickness of the pile is based on the API formula (Eq. (23)):

\[ t = 6.35 + \left( \frac{D}{100} \right) \] (all the variables are in mm) (23)

The equation for obtaining the minimum (min.) pile length \( L \) of monopile (Eq. (24)) with \( R^2 \) of 87% is:

\[ L = 56.52 - 1.397 \times \theta A + 1.138 \times MI \] (24)

where,

\( \theta A \): angle of internal friction of soil (deg.)
\( MI \): moment of inertia (m\(^4\))

Figs. 18a and 18b show scatter plot and surface plot of pile length, respectively. A nomogram for obtaining the minimum pile length for a given angle of internal friction of soil and moment of inertia of pile cross-section is shown in Fig. 18c.

### 4.2 Validation

The predicted model is validated using eight different cases. Tables 4 and 5 show the various cases considered for validation and the corresponding results from SACS and predicted design equations for pile length and pile diameter.

![Fig. 17](image)

(a) Scatter plot of pile outer diameter, (b) surface plot of pile outer diameter, (c) multi-variable chart for pile outer diameter.
Fig. 18  (a) Scatter plot of min. pile length, (b) surface plot of min. pile length, (c) multi-variable chart for min. pile length.

Table 4  Validation for pile diameter prediction model.

<table>
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<tr>
<th>No.</th>
<th>WD</th>
<th>CV</th>
<th>SA</th>
<th>PD1</th>
<th>PD2</th>
<th>Er1</th>
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<td>1.7</td>
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</table>

WD: water depth (m)
CV: current velocity (m/s)
SA: angle of internal friction of soil (deg)
PD1: pile diameter using equation (m)
PD2: pile diameter from SACS (m)
Er1: error in pile diameter (%)
Table 5  Validation for pile length prediction model.

<table>
<thead>
<tr>
<th>No.</th>
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<th>SA</th>
<th>MI</th>
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<th>PL2</th>
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<td>14.24</td>
<td>15</td>
<td>-5.1</td>
</tr>
<tr>
<td>7</td>
<td>14</td>
<td>2</td>
<td>26</td>
<td>3.83</td>
<td>24.55</td>
<td>27</td>
<td>-9.1</td>
</tr>
<tr>
<td>8</td>
<td>23</td>
<td>1.8</td>
<td>30</td>
<td>5.84</td>
<td>21.25</td>
<td>19</td>
<td>11.8</td>
</tr>
</tbody>
</table>

WD: water depth (m)
CV: current velocity (m/s)
SA: Angle of internal friction of soil (deg)
MI: moment of inertia (m⁴)
PL1: min. pile length using equation
PL2: min. pile length from SACS
Er2: error in pile length (%)

The variation of predicted values for pile outer diameter and pile length from the mathematical model with respect to the original values from SACS analysis is represented graphically in Figs. 19 and 20.

It is seen that the predicted pile diameter values are very much close to the original values with a maximum error of 2.6%. The predicted values for minimum pile length are found to be slightly deviating with that of the original results from SACS analysis with a maximum error of 13%. This may be due to the following reasons:

The assumption that the soil layer throughout the depth of the pile has the same uniform properties introduces a certain amount of error in the model. However, consideration of various soil type layers in the analysis increases the complexity of the problem by involving several parameters.

The analysis and design of monopile for OWT involves larger pile diameters. Since API method underestimates the stresses and deflections in the design of larger diameter monopile (> 2 m) for OWT, it is more suitable to set up a finite element modelling in order to generate the $p$-$y$ curves [26]. These generated curves when used in the design will positively impact the predicted model in comparison with the existing API formula.

4.3 Worked-Out Example

Consider a location where water depth is 12 m, current velocity of 1 m/s and medium sand with angle of internal friction of 35°.

By substituting the values in the proposed design equations, outer diameter of pile ($D$) and minimum pile length ($L$) are obtained.

$$D = 4.25 + 0.072 \times WD + 0.112 \times CV$$
$$D = 4.25 + 0.072 \times 12 + 0.112 \times 1$$
$$D = 5.23 \text{ m}$$

$$L = 56.52 - 1.397 \times SA + 1.138 \times MI$$
$$MI = \left(\frac{\pi}{64}\right) \times (D^4 - d^4)$$
$$MI = \left(\frac{\pi}{64}\right) \times (5.23^4 - (5.23 - 2 \times 0.059)^4)$$
$$MI = 3.2 \text{ m}^4$$
$$L = 56.52 - 1.397 \times 35 + 1.138 \times 3.2$$
$$L = 11.3 \text{ m}$$

Hence, a pile of outer diameter 5.23 m, thickness 5.9 cm and minimum length of 11.3 m is required for the given site conditions. The length mentioned here is the minimum desired length to satisfy the rotation conditions.
However, the actual length of the monopile can be increased beyond the minimum length based on the in-situ soil conditions.

5. Conclusions

This paper proposes equations and nomograms for quick preliminary design of offshore monopile foundation for wind turbine using simplified design procedure. This study is based on extreme environmental conditions offshore of Gujarat, India. However, these equations are applicable for any other location having similar environmental conditions and turbine specifications.

The proposed design equations give a good estimate for the initial study on the offshore monopile design for a given location. Hence, the time and effort required for prefeasibility study are minimized by this simplified design methodology. The response of monopiles under cyclic/dynamic load is not well understood and there is a lack of guidance in codes of practice. If cyclic design is incorrect, monopile can tilt in the long term. If the tilt is more than the allowable limit, the turbine may need a
shutdown.

Monopile design is usually carried out using API design procedure calibrated for flexible pile design where the pile is expected to fail by plastic hinges. Future studies include development of prediction model by considering multi-layers of soil along the pile length and finite element modeling to generate p-y curves for large diameter piles which are to be further included in the monopile design for developing the prediction model. The new set of p-y curves should take into account the rigid behavior of large-diameter OWT monopiles. The large-diameter calibrated p-y curves formulations for clays and sand under monotonic and cyclic conditions will contribute substantially to reducing conservatism in OWT substructure design.

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References


