Numerical modelling of the influence of scour and scour protection on monopile dynamic behaviour

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Abstract

Scour of seabed sediments can occur around offshore foundations. For monopile-supported offshore wind turbine structures, the reduction in foundation stiffness due to scour presents certain operational challenges. In cases where scour causes the natural frequency of the structure to become, for example, close to the range of rotor loading frequencies then – due to the increased risk of fatigue damage - turbine support structures are at risk of reduced operation, or even premature decommissioning. In practice, scour protection and/or remediation systems are typically used to mitigate the development of scour. As well as preventing further erosion, scour remediation systems may have a restorative effect on the stiffness of the foundation. This paper describes a one-dimensional (1D) finite element model for the analysis of natural frequencies for monopile-supported turbine support structures with active scour process around the foundation. The model also incorporates procedures to model the influence of a rock fill scour remediation system on the foundation stiffness. The numerical model is calibrated and validated by comparison with a set of previously-described reduced-scale model tests conducted in a flume. The calibrated 1D model is applied to a field case study at a UK offshore windfarm site.

Keywords: Monopile; Scour; Wind Turbine; Sediment

Introduction

Scour processes (erosion of sediments) present various potential inter-related challenges for monopile-supported wind turbine structures. These include: a decrease in the structural natural frequency (which may cause the rate of fatigue damage to increase), an increase in the bending moments in the foundation (due to the larger loading lever arm) and a reduction in foundation stability.

Scour around offshore foundations can take the form of 'global scour' (in which lowering of the seabed occurs over spatial dimensions that are significantly larger than those of the foundation), or 'local scour' (in which scour holes develop around the foundation due to regional sediment transport gradients; this mechanism is also referred to as 'general scour'). Scour development is influenced by the local wave and current environment, the character of the seabed sediments, the local geology and the dimensions and layout of the structures and associated seabed infrastructure (e.g. Whitehouse, 1998 and Whitehouse and Draper, 2020).

Global scour is caused by large-scale processes such as migrating sand waves, sand bank or channel features. This occurs naturally in continental shelf environments due to variations in sediment transport causing erosion or deposition at a range of temporal and spatial scales (Couldrey et al. 2020; Velenturf et al. 2021). In contrast, local scour is associated with localised erosion caused by the action of flow-induced vortices around the foundation (e.g. Sumer et al. 1992). Scour holes caused by local scour processes tend to increase in depth with time until an equilibrium is reached such that the scour hole dimensions remain stable in the prevailing wave and current conditions. The equilibrium depth of scour holes typically scales with the diameter of the foundation; large diameter monopiles are therefore particularly susceptible to the effects of local scour.

A key concern around the influence of scour processes on the performance of monopile-supported offshore wind turbine structures is the potential for the rate of fatigue damage to be increased as a consequence of changes to the dynamic behaviour of the structure due to loss of foundation stiffness (e.g. Dingle et al. 2023). Offshore wind turbine support structures are typically designed such that the fundamental natural frequency lies between the rotor (1P) and blade (3P) frequencies, and above the frequencies of the wind and wave environmental excitation. If scour processes cause the fundamental natural frequency to drift downwards towards the range of significant excitations, then – for example – increased dynamic amplification of the cyclic stresses induced by imbalances in the rotor and drive train could have a damaging effect on the fatigue life of the structure. It is therefore of considerable importance – for the design and management of windfarms – to understand the influence of scour processes on the dynamic characteristics of offshore wind turbine support structures. In working offshore wind farms, it is common practice to either install rock armour scour protection during the initial foundation installation process or to employ a scour remediation system, after the wind farm has been commissioned, if scour damage is detected that exceeds specified design limits.

A key motivation for the installation of scour remediation systems is to add stiffness to the foundation to correct for any downward drift of the fundamental natural frequency that may have occurred. Previous studies (e.g. Prendergast et al., 2015, Qi et al. 2016, Lin and Jiang 2019) have confirmed that the loss of soil support due to scour can have a significant detrimental influence on the stiffness and stability of the foundation. The influence of scour protection on pile behaviour is intuitively expected to have a restorative effect on foundation performance (e.g. Yang et al. 2019); this aspect of scour protection systems has been the subject of recent studies (e.g. Kallehave et al. 2015, Stuyts et al. 2020, Winkler et al. 2023). To support the design and management of scour remediation procedures, estimates are needed of the likely improvements in foundation stiffness that will be achieved by the placement of scour remediation material.

The current paper describes a simplified one dimensional (1D) numerical model that has been developed to predict the influence of scour processes – and rock fill scour remediation – on the dynamic performance of a monopile-supported offshore wind turbine support structure. The 1D model is developed with reference to previously-described data on reduced-scale (1:20) tests on a monopile-supported wind turbine support structure (Mayall et al. 2020). These tests are described briefly in the current paper; for complete details of the experimental procedures reference should be made to Mayall et al. 2020. These previous tests were conducted in the Fast Flow Facility (FFF) flume at HR Wallingford, UK. To conduct the tests the monopile foundation was embedded in a prepared sand bed in the base of the flume. Active scour processes were induced in the sand bed by forced flow regimes. The experiments were designed to investigate the changes in the dynamic performance of the wind turbine support structure – quantified in terms of measured natural frequencies and mode shapes – caused by scour processes around the base of the foundation. The influence of scour protection and remediation systems on the dynamic performance of the support structure was also investigated.

The application of the proposed 1D modelling approach to working offshore wind turbine support structures is demonstrated in the current paper via a field case study on the influence of scour processes at a UK offshore windfarm. Scour protection was not incorporated during the foundation installation process when the wind turbine support structures were initially constructed. Instead, a scour management approach was adopted in which rock fill scour remediation was installed at selected wind turbine locations in response to the observed development of scour. It was found that the fundamental natural frequency of the support structures – inferred from data from accelerometers mounted in the nacelle – tended to drift downwards in response to global and local scour processes as observed from bathymetry data. Restoring the fundamental frequencies to their design values (in addition to the prevention of further scour) was a key purpose of the scour remediation system that was employed. Two of the wind turbine locations at the offshore site have been selected to provide representative case study data for the current paper.

The numerical model described in the current paper provides a means of extracting key information from a set of previously-described laboratory-scale experiments and then upscaling the findings to formulate a modelling procedure for the influence of scour and scour protection in the field case study.

The model described in the current paper relates to (selected) previous studies as follows. 1D models of an embedded monopile have been previously employed to incorporate the influence of scour holes on the soil reactions that act on the pile. In Prendergast et al. 2015, for example, the influence of local scour is represented in a 1D model by the removal of a soil layer (with thickness corresponding to the scour depth) from the model. The current work employs a similar 'soil layer removal approach' although an extended form of the model is developed to represent in more detail the local changes in vertical effective stress in the soil that occur due to loss of material when a scour hole develops. Procedures employed in the current work to incorporate these effective stress changes are consistent with the proposals – based on data from centrifuge experiments - in Qi et al. 2016. Separately, prior studies on the use of 1D models to represent the influence of scour protection on the dynamic performance of offshore wind turbine support structures have incorporated a corresponding approach in which an additional soil layer is added to the 1D model to represent the stiffness that is added to the system by the scour protection. Winkler et al. 2023, for example, describe a 1D model of this sort in which, consistent with the approach described in the current paper, soil reactions acting on the pile are represented using the PISA design model (Burd et al. 2020). In the Winkler et al. 2023 analysis the stiffness of the added soil layer is determined by seeking a direct match with data from a full-scale structure. The current paper describes an approach which differs from the Winkler et al. 2023 model in two respects. Firstly, a procedure is incorporated to represent the localised effective stress increases in the soil due to the weight of the added scour protection material (incorporating the possibility that scour protection might be used to mitigate a pre-existing scour hole). Secondly, parameters to define the stiffness of the scour protection for the analysis of full-scale wind turbine support structures are determined from prior reduced-scale model experiments (described in Mayall et al. 2020) rather than being adjusted to achieve a direct match with the measured performance of a full-scale structure.

Structural Dynamics Testing in the FFF Tests

A detailed description of the experimental program on reduced scale physical model tests in the FFF facility is given in Mayall et al. 2020. Features of these tests that are especially relevant to the current work on the development and calibration of the 1D model are reviewed below; for further details – including a discussion on potential model and scale effects - the reader should refer to Mayall et al. 2020.

The tests employed the configuration in Fig. 1. A reduced-scale monopile (1:20 scale) manufactured from glass fibre reinforced polymer was installed by means of a percussion pile driving system into a compacted sand bed (comprising Bathgate psf sand, $d_{50}=0.161\,mm, G_s=2.65$) in the FFF flume. The tower was modelled as an aluminium tube; an adjustable set of brass discs, each with mass 1.107 kg, was mounted at the top of the tower to simulate the mass of the nacelle and rotor. (No attempt was made to model the rotational inertia of the rotor blades). The materials for the physical model of the pile-tower system were selected for similarity of geometry, pile relative stiffness (Poulos & Davis, 1980; Abadie et al., 2015) and tower relative stiffness (Arany et al., 2016), further details are described in Table 3, and related text, in Mayall et al. 2020. Accelerometers were mounted at key locations inside the tower. Strain gauges were installed on the monopile foundation.

Six tests were conducted as specified in Table 1. The sand bed was prepared using a consistent protocol and the structural characteristics of the monopile-tower system were the same for each test. Dynamic characteristics of the monopile-tower system were investigated by applying a finger tap to a force sensor at the top of the tower and recording the structural response via the accelerometers and strain gauges. It was possible to obtain highly repeatable data on natural frequencies and mode shapes using this relatively simple procedure. The damping ratio determined from these finger tap tests was generally in the range between 0.5% and 2.0%. This is broadly comparable to reported values for full scale structures (e.g. Arany et al. 2016). These relatively low damping ratios are significant since they demonstrate that damping in the system is sufficiently small for its effect on the natural frequencies to be negligible. Damping is therefore not incorporated in the analyses presented in this paper.

Scour processes (both global bed lowering and local scour around the pile) were induced in the flume by the action of forced cyclic flow regimes. Target values of local and global scour depths (S_L and S_G) are listed in Table 1. Finger tap tests were conducted at various stages in the development of scour; this procedure provided data on the natural frequencies and mode shapes as the scour processes developed.

A key purpose of the physical model tests was to investigate the influence of scour protection systems on the dynamic performance of the structure. In Test 1 (Table 1) scour protection systems were absent; this test therefore provided a baseline for comparison with the other configurations. In Test 2, scour processes were initially induced in the sand bed; remedial scour protection in the form of 'tyre-filled nets' (Whitehouse et al. 2011; Fazeres-Ferradosa et al. 2021) was then installed (see Mayall et. al. 2020). Data from these tests indicate that the effectiveness of tyre-filled nets in adding stiffness to the foundation is minimal; the performance of tyre-filled nets is therefore not further considered in this paper (although data from Test 2 prior to the installation of the tyre-filled nets are employed to support the calibration of the 1D model for scoured conditions). Tests 3, 4 and 6 adopted a protocol in which a remedial rock fill scour mitigation system was deployed after scour had developed in the sand bed; these tests are representative of the remedial scour protection system employed at the case study offshore windfarm site. Test 5 adopted a different approach. In this test, rock armour was installed around the monopile before installation (i.e. before the development of scour); this procedure was intended to model the case where rock armour scour protection is installed as part of the initial construction phase for an offshore wind turbine support structure. This test is not relevant to the current work (which is concerned with the performance of remedial (rather than pre-installed) scour protection). Moreover, data from Test 5 indicated that the dynamic behaviour of the structure remained relatively unchanged during the test procedure; data from Test 5 are therefore not considered in the current paper.

Test 1 was concerned entirely with the influence of scour processes on the dynamic performance of the system; Tests 2, 3, 4 and 6 all involved the development of scour prior to the installation of remedial scour protection systems. Tests 1, 2, 3, 4 and 6 therefore provide a data set on the influence of scour processes on the dynamic performance of the model structure; these data are employed in the current work to refine the procedures employed in the 1D model to represent the scour-induced stress and stiffness changes that occur in the soil around the pile. Separately, data from Tests 3, 4 and 6 are used to refine and demonstrate the procedures employed in the 1D model to represent the influence of remedial rock-fill scour mitigation on the structural behaviour of the model structure; relevant data from these tests are listed in Table 2.

Formulation of the 1D Numerical Model

Overview of the Model

An idealised form of the problem at full scale is illustrated in Fig 2a; this idealised configuration forms the basis of the formulation of the proposed 1D model. A wind turbine support structure is mounted on a monopile foundation via a transition piece. Global scour is presumed to lower the global seabed level by distance S_G. Additional local scour processes cause a scour hole of depth S_L to develop. Loading applied to the foundation, via the tower and transition piece, is considered to induce distributed lateral load soil reactions, p, along the embedded length of the monopile. Additionally, vertical shear tractions are presumed to act on the sides of the pile; these shear tractions are considered to apply a distributed moment, m, along the embedded length of the monopile. A horizontal force H_B and moment M_B are considered to act at the pile base.

The components of the 1D model are illustrated in Fig 2b. This model is based on the PISA design model, specified in Byrne et al. 2020 and Burd et al. 2020a; this model is suitable for the analysis of monopile foundations subjected to monotonic loading. In this analysis procedure the embedded monopile is modelled as a beam. Specially-formulated soil elements are employed to represent the soil reactions that act along the embedded length of the monopile; the formulation of these elements adopts the Winkler assumption (i.e. the distributed lateral load and moment soil reactions are considered to be functions of the *local* pile lateral displacement, v and rotation, ψ, i.e. spatial coupling via the soil is not incorporated in the model). Similarly, the horizontal force H_B and moment M_B reactions at the base of the pile are functions only of the base lateral displacement, v_B and rotation ψ_B. The relationships between the soil reactions and the local displacements/rotations are referred to in Byrne et al. 2020 and Burd et al. 2020a as 'soil reaction curves'. The current model employs the initial, linear, portion of these (nonlinear) curves. To reflect the linear nature of the current model, the term 'stiffness coefficient' is employed in the current paper to define the (linear) relationships between the various load and displacement soil reaction components that are present in the model.

The current numerical implementation extends the PISA design model as follows. A structural dynamics modelling procedure is employed to determine the natural frequencies and mode shapes of the structure via a linear eigenvalue analysis (e.g. Williams 2016). (The current model does not consider the response of the structure for detailed loading time histories; this extension is beyond the scope of the current paper). The influence of the mass of water inside, and in proximity to, the monopile/transition piece is incorporated via additional 1D 'water mass' elements (e.g. DNV 2010). The mass and stiffness of the soil plug is incorporated in the model with the same beam element formulation that is employed for the monopile and tower. Importantly, the displacement interpolation functions adopted for the soil plug elements and the water mass elements are consistent with the Hermitian shape functions employed for the Euler-Bernoulli beam elements that are used to model the monopile and the tower.

The dimensions and definitions used to define the 1D model are illustrated in Fig. 3.

Beam Element Formulation

The pile-tower structure and the soil plug are modelled with conventional four degree-of-freedom Euler-Bernoulli beam elements employing the conventions in Fig. 4. The element stiffness matrix k_{beam} and element mass matrix $mass_{beam}$ are determined from,

$$k_{beam}=\int_{z_1}^{z_2} B_b^T(EI)B_b\,dz \tag{1}$$

$$mass_{beam}=\int_{z_1}^{z_2} B_\psi^T(\rho I)B_\psi+B_v^T(\rho A)B_v\,dz \tag{2}$$

where B_v is a vector containing Hermitian shape functions, and B_ψ and B_b are vectors containing appropriate shape function derivatives. The cross-section area and second moment of area are A and I respectively; E is the Young's modulus and ρ is the mass density of the material.

To model the soil plug, the Young's modulus of the plug, E_{plug}, is related to the small strain shear modulus of the soil by,

$$E_{plug}=2G_0(1+v) \tag{3}$$

where the depth variation of the small strain shear modulus, G_0, is assumed identical to the variation of shear modulus in unscoured soil. Since the soil is confined, a value of Poisson's ratio of $v=0.5$ is assumed, corresponding to undrained behaviour.

Soil Distributed Stiffness Elements

The soil reactions $p=(p,m)^T$ are related to the local pile displacement and rotation $v=(v,\psi)^T$ by,

$$p = D_s v \tag{4}$$

where the local soil stiffness matrix is,

$$D_s = G_o \begin{pmatrix} k_p & 0 \\ 0 & D^2 k_m \end{pmatrix} \tag{5}$$

and k_p, k_m are dimensionless stiffness coefficients. D is the outside diameter of the pile. Values of the stiffness coefficients – determined via three dimensional finite element calibration procedures – are determined from the data in Byrne et al. 2020 and Burd et al 2020a,b as described in the Appendix.

The displacement/rotation v at a point in the soil distributed stiffness element is related to the element nodal lateral displacements/rotations V^e by,

$$v = B_s V^e \tag{6}$$

where, consistent with the displacement interpolation adopted for the beam elements (Eqs. 1 and 2), the vector B_s incorporates appropriate Hermitian shape functions to define the lateral soil displacement v and Hermitian shape function derivatives to define the rotation ψ. The distributed stiffness matrix $k_{soil,d}$ is obtained from,

$$k_{soil,d} = \int_{z_1}^{z_2} B_s^T D_s B_s \, dz \tag{7}$$

At the base of the pile the soil reactions are represented by the lumped stiffness model,

$$\begin{pmatrix} H_B \\ M_B \end{pmatrix} = G_o \begin{pmatrix} D k_H & 0 \\ 0 & D^3 k_M \end{pmatrix} \begin{pmatrix} v_B \\ \psi_B \end{pmatrix} \tag{8}$$

where v_B, ψ_B are the displacement and rotation at the base of the pile and k_H, k_M are dimensionless stiffness coefficients (Byrne et al. 2020, Burd et al 2020a,b) determined as defined in the Appendix.

Scour remediation material, when present, is incorporated in the model by additional soil distributed stiffness elements with an appropriate depth variation of shear modulus. Scour remediation materials are assigned a lateral dimensionless stiffness k_p as defined in the Appendix.

Lumped Nodal Mass Elements

The turbine rotor and nacelle, and other attachments, are modelled as rigid bodies with appropriate values of mass and moment of inertia assigned to the topmost node in the finite element mesh.

Water Mass Elements

The water mass elements incorporate mass effects due to lateral translation of the water inside and in proximity to the pile/transition piece, but rotational inertia effects are excluded. The cross-section area of the internal water is $A_{w,\int ¿=\pi D_i^2/4 ¿}$ where D_i is the inner diameter of the pile. The cross-section area of the external water is $A_{w,ext}=C_a \pi D^2/4$ where D is external diameter of the pile and C_a = 1 is an assumed added mass coefficient (DNV 2010). The mass matrix $mass_w$ for the water mass elements is,

$$mass_w=\int_{z_1}^{z_2} B_v^T \rho_w ¿¿ \tag{9}$$

where the density of water is taken as ρ_w = 1000 kg/m^3 and, consistent with Eq. (2), B_v is a vector containing Hermitian shape functions.

Eigenvalue Problem

The homogeneous equation of motion for the discretised system with assembled mass matrix M and assembled stiffness matrix K is,

$$M\ddot{V}+KV=0 \tag{10}$$

where V is a vector comprising the displacements/rotations at all nodes in the model. The natural frequencies ω_m (where the subscript $'m'$ denotes the mode number) and the mode shapes V_m are determined from the solution to,

$$\left(K-\omega_m{}^2 M\right)V_m=0 \tag{11}$$

1D Modelling Procedures for the FFF Tests

Soil Stiffness Coefficients

In previous work (Burd et al. 2020a) a soil reaction curve calibration was conducted for monopiles embedded in a coarse-grained material known as Dunkirk sand ($D_{50}=0.28\,mm$¿. The characteristics of the Bathgate sand employed in the FFF tests are considered to be broadly similar to those of Dunkirk sand (e.g. the values of maximum and minimum void ratio, and the critical state friction angle, are comparable for the two sands, see Mayall 2019). The soil stiffness coefficients determined from the Generalised Dunkirk Sand Model (GDSM) are therefore adopted in the current application (see Appendix).

Small Strain Soil Shear Modulus

The small strain soil shear modulus, G_0, for the sand bed depends on the local values of the effective stress and void ratio (as well as the grading and the detailed mineralogy and shape of the particles). The small strain modulus for the Bathgate sand employed in the FFF tests is represented by the Hardin and Richart 1963 correlation,

$$G_0=\frac{B\,p_{ref}}{0.3+0.7\,e^2}\left(\frac{p'}{p_{ref}}\right)^{0.5} \tag{12}$$

where B is an empirical coefficient, e is void ratio, p' is mean effective stress and $p_{ref}=100\,kPa$. Data from bender element tests (Mayall et al. 2020) indicate that $B=478$ is appropriate for the

Bathgate sand; this value is therefore adopted in the current model. In the absence of information on the lateral effective stresses in the prepared sand bed, the coefficient of lateral earth pressure, K_0, was assumed, straightforwardly, to be $K_0=1$ for the purpose of determining p'; this gives $p'=\sigma'_v$ where σ'_v is the vertical effective stress.

Data on relative density, D_R for the five prepared sand beds considered in the current analyses, determined by cone penetrometer tests (Mayall et al. 2020), are plotted in Fig. 5a. Also shown (Fig 5b) are inferred depth distributions of submerged unit weight, γ' (determined on the basis of minimum and maximum void ratios of 0.502 and 0.753 respectively for the Bathgate sand, together with the inferred D_R values) and (Fig 5c) G_0, determined using Eq. (12).

When the surface of the sand is level and scour remediation materials are absent, the vertical (and horizontal) effective stresses adjacent to the pile are estimated straightforwardly. In such cases, depth-wise profiles of G_0 can be developed as shown in Fig. 5c. Scour holes around the pile, however, cause the effective stresses in the soil adjacent to the pile to depart from the level bed case – especially near to the soil surface – with a consequential variation in G_0. When scour holes are present, therefore, certain assumptions are needed to estimate the depth variation of the effective stresses adjacent to the pile for the purpose of estimating local values of G_0 from Eq. (12).

Fig. 6 illustrates the 'bilinear approach' that is incorporated in current design standards (e.g. API 2011) to estimate the apparent vertical effective stress $\sigma'_{v,A}$ acting in the soil adjacent to the pile when a scour hole is present. In this approach, an 'overburden reduction depth', Δz_O, is chosen. At depths greater than Δz_O below the global bed level the vertical effective stresses are assumed equal to the 'global effective stresses' $\sigma'_{v,G}$ (i.e. the effective stress that correspond to a flat seabed at the global bed level). At shallow depths (i.e. less than Δz_O below the global bed level) the vertical effective stress is influenced by the 'local effective stress' $\sigma'_{v,L}$, (i.e. the effective stress corresponding to a flat bed at the level of the base of the scour hole) and the apparent stress is given by,

$$\sigma'_{v,A} = \alpha_L \sigma'_{v,G} + (1-\alpha_L)\sigma'_{v,L} \tag{13}$$

where the local scour influence factor, α_L, is,

$$\alpha_L = \frac{z_L}{\Delta z_O - S_L} \tag{14}$$

This bilinear approach is adopted in the current work. The recommendations in API 2011 suggest that the overburden reduction depth is specified as $\Delta z_O = \text{¿}6D$. In the analyses of the FFF tests, however, it is found that the overall fidelity of the 1D model is improved when a smaller value of Δz_O is adopted.

When scour protection material is present the local vertical effective stresses $\sigma'_{v,L}$ are calculated with respect to the level of the scour protection material adjacent to the pile using a similar bilinear approach, as illustrated in Fig. 7. In the configuration in Fig. 7b the local bed level (i.e. the level of the scour protection material adjacent to the monopile) is higher than the global bed level by a distance h_L. In this case the apparent vertical stress adjacent to the pile is given by Eq. (13) but with the local scour influence factor defined by,

$$\alpha_L = \frac{z_L}{\Delta z_O + h_L} \tag{15}$$

Small Strain Shear Modulus for Rock Fill Scour Remediation Materials

To estimate the stiffness introduced by rock fill scour protection, a simplified form of the Hardin and Richart 1963 correlation is employed,

$$G_0 = B_{SP}\, p_{ref} \left(\frac{p'}{p_{ref}} \right)^{0.5} \tag{16}$$

where B_{SP} is a (non-dimensional) scour protection stiffness coefficient. In the current work it was not feasible to determine values of B_{SP} for rock fill scour protection by direct measurement. Instead, representative values of B_{SP} were inferred from natural frequency data obtained from the FFF tests. Values of G_0 to represent the scour protection material are determined from Eq. (16). The corresponding value of the stiffness coefficient k_p is determined as described in the Appendix.

1D Finite Element Mesh

Nodes are generated at the elevation of structure geometry changes, point masses, soil layer boundaries including scour protection layering, scour depths, water levels, and plug levels. To achieve a suitably refined mesh, if the spacing between adjacent nodes exceeds a specified value then additional nodes are incorporated in the model.

In the current analyses a total of about 200 elements was used. This number of elements was chosen on the basis of convergence studies of the frequency of the first three vibration modes changing less than 0.1% as the mesh was refined.

Analysis of the FFF Tests

Overall Modelling Procedures

Tables 3 and 4 list the adopted beam element and nodal mass/inertia properties respectively. The mass of the cables (for accelerometer, force sensor, and strain gauge instrumentation) is not incorporated in the model. The top mass is incorporated in the 1D model as a beam element with a solid circular cross-section, where the length depended on the number of top-mounted brass discs, N_M, employed in the tests and the density, ρ, was selected to give the correct total mass, M_{top}. The transition piece (i.e. the h_{TP} region indicated in Fig. 8) is modelled using beam elements with the same stiffness properties as the tower. The mass of the pile material within the transition piece is incorporated into the transition piece mass (as indicated in Table 4); this is implemented as a nodal mass located at the mid-height of the transition piece.

Analysis of Unscoured Cases

With the exception of Test 5 (which incorporated pre-installed scour protection), all of the tests provided the opportunity to investigate the monopile-tower system under flat bed conditions without scour. Natural frequencies and mode shapes for unscoured conditions were therefore measured in five of the six tests. In each case four separate top mass configurations (N_M = 0, 1, 3, 6) were employed. These tests provided an extensive data set for analyses using the 1D model.

Natural frequency data from Test 1 (taken as an example case) are plotted in Fig. 9. These data indicate that the natural frequencies f_m^{Eig} computed using the 1D model conform closely to the measured values f_m^{Meas} for the first three modes. Also shown are the computed natural frequencies for separate cases where full fixity at seabed level and fixity at the pile base are imposed on the model. As expected, the measured and computed natural frequencies lie between the fixed seabed and fixed pile base analyses.

Equivalent data from Tests 2, 3, 4 and 6 indicate a similar pattern to the Test 1 data in Fig. 9 (see Mayall 2019). For the complete set of analyses and measured data on unscoured conditions, the computed natural frequencies are generally slightly higher than the measured data for all three modes. On the basis of data from all of the tests, the natural frequencies are overpredicted in the 1D model by a mean of 1.9% for mode 1, 2.7% for mode 2 and 3.4% for mode 3.

Analysis of the Influence of Scour on the Natural Frequencies

With the exception of Test 5 (in which pre-installed scour protection was incorporated) all of the tests incorporated phases in which un-remediated local and/or global scour was present. Analyses were conducted using the 1D model for each of these cases. The bathymetry analysis for Test 1 suggested that sand backfilling (i.e. deposition of sand into the local scour hole) occurred during one part of this test where a reduction in depth was observed adjacent to the pile. There was a corresponding slight increase in the observed natural frequency during this part of the test but, since the effect was small, this component of the test was excluded from the current analysis. It is possible that sand backfilling could also have occurred – undetected – in some of the other tests.

The influence of scour on the natural frequency is quantified in terms of the change in natural frequency (Δf_m^{Meas} for the measured data and Δf_m^{1D} for data computed with the 1D model) where,

$$\Delta f_m^{Meas} = f_m^{Meas} / f_{m,0}^{Meas} - 1 \tag{17}$$

$$\Delta f_m^{1D} = f_m^{1D} / f_{m,0}^{1D} - 1 \tag{18}$$

In Eq. (17), f_m^{Meas} refers to the measured natural frequency in the presence of scour and $f_{m,0}^{Meas}$ refers to the measured natural frequency for the same pile-tower configuration in the initial (unscoured) case. Similarly, in Eq. (18), f_m^{1D} and $f_{m,0}^{1D}$ signify the computed natural frequencies with, and without, the presence of scour.

Global scour is incorporated straightforwardly in the model by adjusting downwards the seabed level (Fig 3a). To incorporate a local scour hole in the analysis, the bilinear approximation of apparent vertical effective stress (Eqs. 13 and 14) was employed, together with the shear modulus correlation in Eq. 12, to specify the depth variation of shear modulus in the 1D model. Since the appropriate value of the overburden reduction depth Δz_O is uncertain, an approach was adopted in which the non-dimensional parameter,

$$\delta_L = \Delta z_O / S_L \tag{19}$$

was adjusted to provide a match between the experiment and the model. The change in the first natural frequency Δf_1 was used to provide independent best-fit values of δ_L for each scour depth and top mass configuration. The median value obtained from this process was δ_L = 1.49; this value was employed in subsequent analyses. The calibrated bilinear approach with δ_L = 1.49 produces a faster recovery of stress with depth than the API 2011 recommended specification.

Analysis of Rock Fill Scour Protection on Natural Frequencies

Table 2 presents an overview of the scour protection properties and associated measured scour depths at key stages in the flume experiments for Tests 3,4 and 6. Consistent with data in An et al. 2014, sand accretion was observed in the rock scour protection matrix in the flume experiments. For the numerical model, the scour protection was assumed to contain accreted sand throughout its thickness after any post-installation flume flow had occurred. The bulk unit weight of scour protection with accreted sand was estimated as,

$$\rho_{bulk} = \rho_w \frac{G_{S,SP}\left(1+e_{sand}\right)+G_{S,sand}\, e_{SP}+e_{SP}\, e_{sand}}{\left(1+e_{SP}\right)\left(1+e_{sand}\right)} \tag{20}$$

where $G_{S,SP}$ and $G_{S,sand}$ are the specific gravity of the rock and sand respectively, and e_{SP} and e_{sand} are the void ratio of the rock and sand matrix respectively (see Fig. 10). To estimate e_{sand} a simplified approach was adopted in which the relative density of the accreted sand was assumed to be D_R = 50%.

1D analyses were performed to determine the influence of scour protection systems on the natural frequencies of the system. The shear modulus of the scour protection materials is estimated using Eq. (16). In the absence of independent data on the scour protection stiffness parameter B_{sp}, a procedure was adopted in which values of B_{sp} were estimated by comparing the 1D model with the experimental data on the first natural frequency. In these analyses the vertical effective stresses in the soil adjacent to the pile were determined with the bilinear approach with Δz_0¿$\delta_L S_L$ (see Fig. 7a) with $\delta_L = 1.49$. In instances where the global scour level dropped below the scour protection level at the pile wall (i.e. $S_T < S_G$, Fig. 7b) the bilinear approach was modified assuming Δz_0¿$\delta_L h_L$ with $\delta_L = 1.49$.

Analyses were conducted on a total of fifty six natural frequency measurements from fourteen scour protection geometries with four top mass configurations (N_M = 0, 1, 3, 6) drawn from Tests 3,4 and

6. The scour protection geometries comprise three as-installed conditions plus eleven geometries measured after further flume flow.

Fig. 11 shows values of Δf_m^{1D}(i.e. change in natural frequency computed with the 1D model) for a range of values of B_{SP} for the conditions in Test 3, for the different values of N_m that were employed in the test. A polynomial fit to these data is also shown. Results are shown in Fig11a for the configuration immediately after installation of the scour protection and in Fig11b at the end of the test. Also shown are the values, Δf_m^{Meas}, actually measured in the tests. These data sets were used to infer a set of independent values of B_{SP} as the value from the polynomial fit corresponding to the measured frequency change Δf_m^{Meas}. The data in Fig. 11 indicate that the inferred values of B_{sp} depend to some extent on N_M. This dependency on N_M (which does not have a physical basis) is considered a consequence of the simplifications and approximations employed in the 1D model.

Fig. 12 presents the variation of the inferred B_{SP} values at certain stages in Tests 3, 4 and 6 determined using the procedures outlined in the previous paragraph. Measurement number 1, refers to the 'as installed' condition. The other measurements relate to cases where flow had occurred in the flume; these measurements are therefore potentially affected by further scour and/ or sand accretion within the scour protection material. The inferred B_{SP} values in Fig. 12 vary with measurement number in all cases; the measurement numbers correspond to testing windows after subsequent flow regimes and scouring processes (as described in Mayall et al. 2020). The increase of B_{SP} in Tests 3 and 6 after the initial flow stages is presumed to be caused by sand accretion in the scour material. There is a lack of evidence in the literature on whether sand accretion occurs in field conditions, such accretion would likely only occur at sites with live bed conditions and suspended sediment in the flow. It is possible that this accretion mechanism is responsible for the observations of frequency gains for wind turbines with scour protection at Greater Gabbard offshore wind farm (Hucker et al. 2019). The decreases of B_{SP} in Test 4, and in Test 3 and 6 (after a peak), are presumed to be due to reductions in the effective stresses around the pile caused by further global scour. The simplified scoured effective stress model and scour protection stiffness model employed in the current work may be insufficiently detailed to capture the full detail of the scour protection stiffness contribution for the more complex three-dimensional geometries encountered in Tests 4 and 6.

Table 5 presents the average inferred values of B_{SP} for the rock fill remedial scour protection material for the 'as-installed' and 'post flow' conditions. Table 5 also shows the standard deviations in the inferred values of B_{SP}; these relatively high standard deviations reflect the wide range of fitted B_{SP} values, as indicated in Fig. 12. The median values in Table 5 are considered to provide appropriate values of B_{SP} for the rock fill scour protection material. The 'as-installed' median value ($B_{SP}=259¿$is relevant to cases where the rock fill is in its initial state. The higher 'post flow' median value ($B_{SP}=500$) is considered appropriate for cases where sand accretion is has occurred.

It is instructive to compare the stiffness of the rock fill material with the stiffness of the native Bathgate sand. To facilitate this comparison an 'equivalent stiffness parameter' $B^{¿}$ is defined for the sand such that,

$$B^{¿}=\frac{B}{0.3+0.7\left(e^{¿}\right)^{2}} \tag{21}$$

where $e^{¿}$ is a representative void ratio for the in situ sand. This definition of equivalent stiffness parameter facilitates a direct comparison of the stiffness of the Bathgate sand (defined by Eq. 12) and the scour protection material (defined by Eq. 16). For comparison purposes the loose state $e^{¿}=e_{max}=0.753$is selected. For the case where $B=478$, this gives $B^{¿}=686$. On this basis the as-installed stiffness of the rock fill material is about 40% of the stiffness of the native sand, rising to about 73% of the native sand stiffness as sand accretion occurs.

Offshore Windfarm Field Case Study

The case study considers an application of the numerical model to two full-scale wind turbines at a UK offshore windfarm where remedial rock fill scour mitigation was employed in response to the development of scour around the monopile foundations. Data on the details of the bathymetry data and scour protection systems relevant to the case study were supplied by HR Wallingford. Data on measured natural frequencies were supplied by E.ON. The case study is concerned with two locations – referred to below as Pile 1 and Pile 2. A water density ρ_w = 1000 kg/m^3 is assumed for the model.

Case Study Description

Fig. 13 provides an overview of the structural details (based on the original confidential design reports) of the wind turbine structures at the offshore windfarm. Vertical coordinates are generally presented as elevation above the local chart datum (z_{CD}) which corresponds approximately to the lowest astronomical tide. The monopile-tower-nacelle structures comprise a monopile with outside diameter 4.3 m and variable wall thickness. The tower, which has variable diameter and thickness is connected by a grouted joint at the transition piece.

The geotechnical conditions comprise layered sand and clay. A mudstone bedrock is present at depth. The original site investigation did not include measurements of G_0; the depth variation of G_0 therefore was estimated for the current study. The sand layers comprise fine sand e_{min} = 0.5868, e_{max} = 1.0385, and G_s = 2.65. The clay layers typically comprise a slightly sandy clay. Data from oedometer tests suggest that the clay layers are normally consolidated or lightly consolidated.

Rock fill remedial scour protection was installed 7.76 years after pile installation at Pile 1 and 7.72 years after pile installation at Pile 2. The rock fill material has specific gravity G_s = 3.15. The median particle mass is 22 kg, which corresponds to d_{50} = 509 mm for a spherical particle assumption. The total mass of installed scour protection was measured by the installation contractors (3433 tonnes at

Pile 1; 3166 tonnes at Pile 2). The *in situ* installed bulk unit weight of the rock fill has not been assessed.

Bathymetry Data

Bathymetry surveys, typically covering the full windfarm extent, were conducted approximately annually after wind farm had been remediated. These data were collected using Multibeam Echo Sounder surveys as part of the management process for the wind farm. Example bathymetry data, for Pile 2, indicating the bathymetry both before and after scour protection installation, are shown in Fig 14.

The bathymetry data were employed to determine representative seabed level data for incorporation in the 1D model. Following the approach in Mayall et al. 2020 local bed levels were taken as the median value of data points close to the pile wall ($r/D \leq$ 0.75) where r is the radial distance from the pile centre; global bed levels were assessed using data points beyond the scour hole extent (7.5 $\leq r/D \leq$ 15). Fig. 15 shows the inferred scour depths determined in this way; Table 6 lists the maximum inferred scour depths.

The scour depths in Fig. 15 show temporary reductions at approximately 1.2 and 6.4 years, caused by natural backfilling of the scour holes. Since the site bathymetry is only conduced periodically, it is unclear for how long this backfill was present.

The installation of scour protection, indicated in Fig. 15 manifests as a reduction in local scour depth. By coincidence at Pile 1 there was a sudden increase in global scour approximately 7.5 years after pile installation (Fig. 15a) shortly before the scour protection was installed. At Pile 2 (Fig. 15b) the global scour continued after installing the scour protection and the global bed level at the end of the data set is lower than the top of the scour protection (i.e. $S_G > S_T$ as indicated in Fig. 7b).

Natural Frequency Data

Fundamental natural frequencies of the monopile-tower system were determined from the peak of the power spectral density of acceleration data obtained from accelerometers mounted in the nacelle. Accelerations were measured during rotor stop tests and standstill periods.

There are some uncertainties in the acceleration data as follows. Natural frequency data are only available from four years after turbine installation (corresponding to the end of the turbine warranty). It is therefore not possible to track the complete evolution of the structural characteristics of the system. Additionally, the frequency resolution of the data (approximately 0.012 Hz, corresponding to roughly 4% of the fundamental natural frequency) is relatively low. Measurements are relative to the nacelle orientation (which is a rotating reference frame); frequencies are reported in the downwind (DW) and crosswind (CW) directions. The orientation of the nacelle was not analysed in the current work. It is therefore not possible to determine whether the scatter in the measured frequencies is caused by scour asymmetry or rotor inertia effects.

Fig. 16 shows the natural frequency measurement data; a measurement trend is plotted as the mean of five values centred on each measurement. At Pile 1 (Fig. 16a) the natural frequency was stable before scour protection was installed (4 years to 7.76 years), and the scour depths (Fig 15a) are also relatively stable within the same period. At Pile 2 (Fig. 16b) the natural frequency reduces over the monitored period before scour protection was installed (4 years to 7.72 years); the total scour depth (Fig. 15b) increased over the same period leading to a reduction in pile embedment depth.

Consistent with the data from the FFF tests, the measured natural frequencies tended to rise immediately after the installation of the scour protection at both pile locations. The FFF results also show that the natural frequencies after installing scour protection tend to drift upwards due to sand accretion, or to drift downwards in cases of continued global scour. It is unclear whether these effects are fully captured in the measured natural frequency field data due to the discrete nature of the natural frequency data but the general correlation with seabed lowering and scour protection installation look to be plausible.

Numerical Model

A 1D model representing the monopile, transition piece and tower was generated employing data on mass, diameter and wall thickness supplied by E.ON. Items attached to the structure (e.g. rotor, nacelle, ladders, flanges, platforms, etc.) were modelled as lumped nodal masses and moment of inertia.

Temporal variations in the water depth (e.g. due to tides) are likely to induce small changes to the natural frequencies of the structure (due to the influence of water added mass). In the current model, however, a single water depth, corresponding to be the mean water level $z_{CD}=4.38\,m$, was adopted to specify the water mass elements.

The soil stratigraphy (illustrated in Fig. 13) comprises interbedded layers of sand and clay. Following the procedures in Burd et al. 2020a the small strain shear modulus, G_0, for the sand layers was calculated using Eq. (12) with $B=875$. The soil stiffness coefficients were determined from the GDSM (Burd et al. 2020a) as described in the Appendix. The clay layers are reported as being approximately normally consolidated. Shear modulus distributions with depth, and associated soil stiffness coefficients were therefore based on the 'Bothkennar clay' model described in Burd et al. 2020b. (Bothkennar clay is an approximately normally consolidated clay; the PISA design model calibration for this soil is therefore considered to be applicable to the current case). The relationships$G_0=500\,p'$ where p' is mean effective stress, employed in the calibration process for Bothkennar clay, as described in Burd et al. 2020b), is therefore adopted for the current analysis. The stiffness coefficients adopted for the sand and clay layers are defined in the Appendix.

Variations with depth of relative density, submerged unit weight γ' and G_0 for both pile locations are plotted in Figs. 17 and 18. The soil plug length is unknown and was assumed to be equal to the installed pile embedment length.

The scour protection unit weight (Table 7) was calculated using an assumed voids ratio representative of the flume experiments (void ratio e = 0.95), and the rock fill stiffness was based on the analysis of the FFF experiments (Table 5). The ‘as-installed’ value of median scour stiffness parameter $B_{sp}=259$ was used to model the natural frequency change immediately after scour mitigation installation. The ‘post-flow’ median value $B_{sp}=500$ was used in subsequent analyses; this is based on the assumption that sand accretion occurs in the rock fill material, as observed in the FFF tests.

1D Model Results

The natural frequencies of Pile 1 and 2 were calculated using the 1D model for each of the measured scour depths in Fig. 15c to produce a time series of predicted natural frequencies. Two scour bilinear effective stress models were analysed: (i) ‘FFF Calibrated’ model from the flume experiments with δ_L = 1.49 and (ii) ‘API’ (2011) with $\Delta z_O/D$ = 6.

Fig. 19 shows the computed change in fundamental natural frequency with reference to a common initial unscoured natural frequency, inferred from the 1D model. These data are presented alongside the changes in natural frequency determined from the measurements. The data in Fig. 19 indicate that the 1D model provides a realistic representation of the evolution of the fundamental natural frequency for both piles. Interestingly the results are insensitive to the precise assumption on the bilinear stress distribution adjacent to the pile; the ‘FFF Calibrated’ and ‘API’ data are similar for scour only but the API formulation cannot include the effect of the scour protection.

Conclusions

A simplified 1D model is shown to provide high fidelity predictions of the fundamental natural frequency of a model wind turbine support structure – tested in the FFF flume - for a range of scour and scour mitigation conditions.

Results from the 1D model for level-bed conditions in the FFF tests (i.e. in which scour and scour protection are absent) indicate that – in comparison with test results - the model is able to provide consistently reliable estimates of the natural frequency. This outcome supports the modelling choices that are incorporated in the 1D model.

For cases where local scour holes develop around the base of the monopile, the influence of stress changes in the soil (due to the surface erosion) is represented with a bilinear model as proposed in API design guidance (API 2011). The API standard recommends that the value of overburden reduction depth, Δz_0, in the bilinear model is set to Δz_0=6D. It was found, however, that the quality of the fit between the data and the measurements on the reduced-scale model structure in the FFF tests could be improved by adopting a smaller value of Δz_0. After some numerical experimentation, the correlation $\Delta z_0 = 1.49 S_L$ was found to provide an optimum fit between the test results and the 1D model (for the first mode natural frequency). This is consistent with other research that found stresses in the soil to be higher than the API approach (e.g. Lin and Jiang 2019). Applications of the 1D model to the field case study piles indicate that both assumptions (Δz_0=6D and $\Delta z_0 = 1.49 S_L$) provide a close representation of the natural frequency measurements.

Independent stiffness data on the rock fill material employed in the FFF tests and at the offshore site are not available. Indeed, the significant practical difficulties that would be associated with these measurements mean that obtaining test data for such materials is unlikely to be practical. In the current paper, an alternative approach is employed in which the stiffness characteristics of the rock fill scour mitigation material is determined indirectly on the basis of fitting an overall structural model to data obtained from the FFF tests. This appears to provide a successful approach to approximating the natural frequency of the complete system, although the procedure was found to deliver values of B_{SP} that are subject to significant scatter, as quantified by the relatively large standard deviations in the inferred values. A key aspect of the inverse modelling employed in the study is that relatively small changes in observed natural frequency lead to large changes in inferred values of B_{SP}; variations in B_{SP} are therefore inevitable when employing this approach.

The analyses of the FFF tests reported in the paper suggests that rock fill remediation is capable of restoring some measure of stiffness to the foundation, but consistent with the earlier experimental work in Mayall et al. 2020 the stiffness contributed to the system by rock fill scour remediation does not fully reverse the losses due to the scour processes.

The calibrated 1D model is shown to provide a close representation of the evolution of the fundamental natural frequencies observed for the field case study piles. The 1D model is also able to represent the recovery in the natural frequency after scour mitigation rockfill has been installed. This supports the application of the 1D model to the assessment of the influence of scour and scour protection on the fundamental natural frequency for full scale offshore monopile-supported wind turbine support structures.

Appendix: Soil Stiffness Coefficients for Incorporation in the 1D Model

The soil stiffness coefficients are determined from the soil reaction curve calibrations developed for the PISA design model (Byrne et al. 2020, Burd et al. 2020a,b) as described below.

Soil and Rock fill Stiffness Coefficients for the FFF Tests

The coefficients adopted for the Bathgate sand are based on the general Dunkirk sand model (GDSM) described in Burd et al. 2020a. A particular feature of the sand framework on which the GDSM is based is that the distributed moment reaction, m, scales with the current value of the net distributed lateral load reaction, p. Since in the initial state the net distributed lateral load is zero, the initial stiffness coefficient k_m is also zero. The dimensionless lateral load stiffness coefficient k_p (Burd et al. 2020a) is,

$$k_p = 8.731 - 0.6982\, D_R - 0.9178 \times \frac{z_L}{D} \tag{22}$$

where z_L is distance below the level of the soil surface adjacent to the monopile, D_R is relative density of the sand expressed as a percentage and D is the external pile diameter. The values of $k_m = 0$ and k_p defined above are employed in the local stiffness matrix, D defined in Eq. (5).

For the stiffness model at the base of the pile, Eq. (5), the dimensionless stiffness coefficients specified in the GDSM (Burd et al. 2020a) are,

$$k_H = 6.505 - 2.985\, D_R + \frac{z_L}{D}\left(-0.007969 - 0.4299\, D_R\right) \tag{23}$$

$$k_M = 0.3515$$

The lateral load stiffness coefficient for the remedial scour protection is determined from Eq. (22) with $z_L = 0$ and D_R = 67.5% (representing the mid-point of the relative density calibration for the GDSM, Burd et al. 2020b). Consistent with the model employed for the sand soil, the moment stiffness coefficient is $k_m = 0$.

Stiffness Coefficients for the Field Case Study

The sand layers employ the GDSM with values of k_p, k_H and k_M as defined in Eqs. (22) and (23), with $k_m = 0$.

The clay layers employ the Bothkennar model specified in Burd et al. 2020b. The stiffness coefficients in this case are:

Distributed stiffness coefficients

$$k_p = 12.05 - 1.547 \times \frac{z_L}{D} \tag{24}$$

$$k_m = 1.698 - 0.1576 \times \frac{z_L}{D} \tag{25}$$

Base stiffness coefficients

$$k_H = 3.008 - 0.2701 \times \frac{L}{D} \tag{26}$$

$$k_M = 0.3409 - 0.01995 \times \frac{L}{D} \tag{27}$$

It is noted that the original embedment lengths, L, of some piles at the site (e.g. $L/D = 6.93$ at Pile 1 are outside the calibration space of the PISA soil reaction models ($2 \leq z_L/D \leq 6$). For cases where $\frac{z_L}{D} > 6$ or $\frac{L}{D} > 6$ the stiffness coefficients were determined from Eqs. (24) to (27) by putting $\frac{z_L}{D} = 6$ or $\frac{L}{D} = 6$.

Data Availability Statement

The data plotted in Figs. 5, 9, 11, 12, 15, 16 and 19 are available from the corresponding author upon reasonable request.

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Table 1. Schedule of tests conducted in the FFF flume (see Mayall et al. 2020 for further details). Water depth h_w, local scour depth S_L, and global scour depth S_G, are defined in Fig. 2. The data indicate 'target values'; relevant values actually recorded during the tests are listed in Table 2

Test	**Initial condition**		**Local scour phase**	**Global scour phase**	
	Target pile embedment L/D	Target water depth h_w/D	Target local scour S_L/D	Scour protection system	Target global scour S_G/D
1	4.5	4.5	1.5	None	1.5
2	4.5	4.5	1.5	Tyre-filled nets (remedial)	1.5
3	4.5	4.5	1.5	Rock fill (remedial)	1.5
4	4.5	4.5	0.75	Rock fill in partial scour hole (remedial)	1.5
5	4.5	4.5	0.0	Pre-installed rock armour	1.5
6	5.5	3.5	1.5	Rock fill (remedial)	2.5

Table 2. Data determined from the FFF tests for Tests 3, 4 and 6 . The local scour thickness S_L and scour protection thickness t_{sp} are defined in Fig. 7

Test	Local scour	Scour protection 'as-installed' condition			Scour protection end of test condition			
	S_L [m]	t_{SP} [m]	γ [kN/m^3]	e	t_{SP} [m]	γ [kN/m^3]	ΔS_T [m]	S_G [m]
3	0.294	0.161	18.85	0.953	0.155	23.70	0.006	0.059
4	0.025	0.105	18.99	0.923	0.065	23.77	0.117	0.072
6	0.297	0.201	18.60	1.008	0.188	23.60	0.013	0.352

Table 3. Beam element data for finite element models of the monopile – tower system for the FFF tests

Beam element type	Material	D	t_w	E	ρ
		[m]	**[m]**	**[GPa]**	**[kg/m^3]**
Top mass	Brass	0.0945	0.04725	407	8529
Tower	Aluminium	0.1016	0.0016	69	2700
Pile	GFRP	0.197	0.0035	31.235	1855

Table 4. Nodal mass properties for finite element models of the monopile – tower system for the FFF tests. R_{gyr} signifies radius of gyration

Nodal mass description	z **[m]**	**Mass [kg]**	R_{gyr} **[m]**
Force sensors and mounting	5.47	0.296	0.050
Accelerometers and mounting	5.38, 4.13, 3.53, 3.18, 2.83, 2.33	0.070	0.025
Transition piece plus mass of pile	2.38	3.667	0.075

Table 5. Values of B_{SP} for rock fill remedial scour protection inferred from measurements in the FFF tests

Scour protection condition	Median	Mean	Standard deviation
As-installed	259	407	357
Post flow	500	562	527

Table 6. Maximum observed scour depths in the field case study

Location	Installed pile embedment L/D	Maximum global scour S_G/D	Maximum total scour S_T/D	Maximum local scour S_L/D	Installed Scour protection thickness t_{sp}/D
Pile 1 (median)	6.93	1.58	3.14	1.80	1.40
Pile 2 (median)	7.21	2.30	3.14	1.85	1.24

Table 7. Scour protection parameters employed for the field case study

Scour protection condition	γ **[kN/m^3]**	B_{SP}
As-installed	20.63	259
With accreted sand	25.47	500

Figure Captions

Fig. 1. General test arrangement adopted in the FFF tests showing indicative dimensions; detailed dimensions are given in Fig. 8. The tower comprises an aluminium tube with external diameter 0.1016m. The monopile is constructed from a filament wound glass fibre reinforced polymer, outside diameter $D=$¿ 0.197 m. The target pile embedment length, L, is 0.9m (Tests 1 to 5) and 1.1m for Test 6. Further details are provided in Mayall et al. 2020.

Fig. 2. 1D finite element model configuration for monopile eigenvalue analysis: (a) idealised physical configuration for a full-scale wind turbine system; (b) 1D numerical model representation. The vertical coordinate z is used to formulate and define the 1D model; z_{BSF} is used to specify and interpret relevant soil conditions; z_L is used to interpret the influence of scour on the soil effective stresses.

Fig. 3. Dimensions and definitions used to define the 1D model; (a) after scour processes have occurred but before installing scour protection; (b) after installing scour protection.

Fig. 4. Definitions for the elements used to represent the beams, soil stiffness elements, soil plug elements and water mass elements in the 1D model. Nodes 1 and 2 have coordinates $z=z_1$ and $z=z_2$. The lateral displacement and rotation at nodes 1 and 2 are V_1, V_2 and Ψ_1, Ψ_2 respectively. The lateral displacement and rotation at an arbitrary point in the element are v and ψ.

Fig. 5. Inferred soil parameter profiles for initial flat bed configurations in the FFF tests: (a) relative density, D_R; (b) submerged unit weight, γ'; (c) small strain shear modulus, G_0. Data in (a) and (c) are reproduced from Fig. 3 in Mayall et al. 2020. Further explanation of these data is given in Mayall et al. 2020.

Fig. 6. Soil vertical effective stress profiles when scour is present: (a) Idealised local scour hole geometry; (b) Vertical effective stress profiles indicating the 'bilinear approach'. z_{BSF} is the depth below the initial bed level (i.e. the bed level at the time of pile installation).

Fig. 7. Proposed soil vertical effective stress profiles when scour and scour protection is present; (a) scour protection recessed in scour hole, $S_T > S_G$; (b) scour protection raised above global bed level, $S_T < S_G$. (i) Idealised scour protection geometry; (ii) assumed vertical effective stress profiles. The coordinate z_{BSF} is the depth below the initial seabed level (i.e. the seabed level at the time of pile installation).

Fig. 8. Numerical model representation of the monopile – tower system in the FFF tests. The left figure shows the structural idealisation that is employed. The right figure illustrates the numerical model.

Fig. 9. Data on the first three natural frequencies computed using the 1D model and from measurements. '1D model' signifies computed data from the 1D model and 'Measured data' signifies corresponding data from the FFF tests. '1D model; fixed at pile base' and '1D model; fixed at seabed' signify separate 1D calculations to explore the likely bounds of the behaviour of the system.

Fig. 10. Grain structures and phase diagrams for scour protection rock without and with accreted sand.

Fig. 11. Data for the conditions in Test 3 showing values of Δf_1^{1D} (see Eq. 18) computed using the 1D model for the different values of N_m employed in the test, for a range of assumed values of B_{SP}: (a) data collected immediately after installation of scour protection; (b) data collected at the end of the test. Hollow circles indicate data for Δf_1^{1D}; dotted lines indicates a polynomial fit for Δf_1^{1D}. Also shown, indicated as *, are the values of Δf_1^{Meas} determined using data from the test using Eq. 17.

Fig. 12. Inferred B_{SP} data for rock fill remedial scour protection: (a) Test 3 (rock fill in a full scour hole); (b) Test 4 (rock fill in a partial scour hole); (c) Test 6 (rockfill in a full scour hole with falling apron development). Measurement number 1 corresponds to the first scour protection measurement. Increasing measurement numbers correspond to testing windows after additional flow had occurred (see Mayall et al. 2020).

Fig. 13. Characterisation of offshore wind turbine structures and soil stratigraphy at the field case study Piles 1 and 2. The tower, transition piece and monopile are made from steel with assumed values of 210 GPa for the Young's modulus and 7850 kg/m^3 for the density .

Fig. 14. Example bathymetry measurements before and after scour protection installation for field case study Pile 2: (a) 0.6 years after pile installation; (b) 7.7 years after pile installation; (c) 8.3 years after pile installation (0.6 years after scour protection installation); (d) 10.2 years after pile installation (2.5 years after scour protection installation). Coordinates x and y are the distance east and north from the pile centreline, respectively; coordinate z is vertical distance of the bed level above the pile tip.

Fig. 15. Time series of measured scour depths for the field case study: (a) total and global scour depths at Pile 1, scour protection installed at 7.76 years (red dashed line); (b) total and global scour depths at Pile 2, scour protection installed at 7.72 years (red dashed line); (c) local scour depths, calculated from median total and global scour in (a)&(b). Notation; pXX indicates XXth percentile, i.e. p50 indicates the median value. In Figs. (a) and (b) $t_{sp}$is the thickness of scour protection adjacent to the pile wall.

Fig. 16. Time series of measured natural frequency at Pile 1 and 2 for the field case study: (a) natural frequency measurements at Pile 1: (b) natural frequency measurements at Pile 2. DW and CW indicate measurement in the downwind and crosswind direction respectively. The legend 'SP installed' denotes the time at which rock fill remedial scour protection was installed. 'Measurement Trend' is a five-point moving average.

Fig. 17. Field case study soil parameter profiles at Pile 1; z_{BSF} is the depth below seabed level at the time of pile installation. These data relate to the design parameters that were adopted during the original design process for the wind farm.

Fig. 18. Field case study soil parameter profiles at Pile 2; z_{BSF} is the depth below seabed level at the time of pile installation. These data relate to the design parameters that were adopted during the original design process for the wind farm.

Fig. 19. Comparison of natural frequency results computed with the 1D model and the measured data from the field case study: (a) change in natural frequency at Pile 1; (b) change in natural frequency at Pile 2. The legend 'SP installed' denotes the time at which rock fill remedial scour protection was installed. A 'bilinear' approach is used to define the depth variation of vertical effective stress in the soil adjacent to the monopile; the legend 'FFF calibrated' refers to the bilinear model developed in the current paper with $\delta_L = 1.49$ and 'API' refers to the bilinear stress model as specified in API (2011).