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DESIGN OF MONOPILE AND TRIPOD FOUNDATION OF FIXED OFFSHORE WIND TURBINES VIA ADNANCED NUMERICAL ANALYSIS

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Abstract. *Aiming to contribute towards development of offshore wind turbines, in this paper a comparative investigation of two different types of foundation of fixed offshore wind turbines, by means of a tripod or a monopile, is carried out in terms of their structural behavior and design procedure under different loading conditions, as defined in the IEC61400-3 Standards. For this purpose, the superstructure of the turbine is first simulated under wind and wave dynamic loading using public domain software FAST, developed by the National Renewable Energy Laboratory (NREL) and the National Wind Technology Center (NWTC). The coupled aero-hydro-servo-elastic behavior is simulated by FAST and its modules AeroDyn and HydroDyn and time histories of internal actions are obtained for the pylon, considering it as fixed at its base, as the foundation cannot be modeled in this environment. Then, the pylon's foundation is modeled in the finite element software ADINA, modeling the piles with beam elements and taking into account the foundation-soil interaction by means of nonlinear equivalent soil springs. Proper sets of action effects, as obtained from FAST, are then imposed at the top of the pile as static loads. Iterative analyses of this type are utilized to provide the required pile lengths and cross-sections for both monopile and tripod options. Moreover, stiffness-equivalent, free-standing single cantilever piles are obtained, which are then used in FAST as a fictitious pylon part below the actual pylon, employing the so called "apparent fixity model" and a new set of dynamic analyses are carried out for comparison of the effect of foundation on the overall response.*

1 INTRODUCTION

Offshore wind turbines are configured as either fixed or floating, depending on the depth of the seabed. For depths greater than 50m fixed offshore wind turbines are considered ineffective, or even not feasible, so floating turbines are constructed instead, employing buoyancy for balance and stability of the structure. The most common structural system for fixed wind turbines is the monopile, where the pylon is extended up to the seabed and is then driven into the soil as a common pile, to support the whole structure. Depending on the depth and soil quality a tripod or jacket support structure may also be used, as can be seen in Fig.1. The main advantage of the monopile is its design simplicity and constructability. However, this simplicity usually requires a large pile diameter ranging from 3.5m to 6m and a wall thickness up to 150mm. As a result, water pressure, hence hydrodynamic loads acting on the cylindrical pile, affect the offshore wind turbine a lot more than in the case of the tripod or jacket, which are constructed with tubular cross section of significantly smaller diameters. To that effect, greater depths, more than 30m, require a stiffer support structure to respond to the higher imposed wind and wave loads.

Offshore wind turbines are complex structures subjected to environmental cyclic loads, such as wind and waves. Their analysis is a complex procedure that includes the "site specific" determination of the environmental loads as well as the detailed simulation of the wind turbine, including the support body, the foundation and the foundation-soil interaction. The study - construction of such projects was originally based on regulatory frameworks and texts from three sources: (i) regulations governing the construction of conventional onshore wind turbines, (ii) regulatory texts from the oil extraction industry and (iii) texts of compliance issued by international certification bodies, and have the form of Design Guidelines - Recommended Practice or Specifications- Technical Requirements. Since 2009 available regulations and standards have been also published specifically for the design and construction of offshore wind turbines. The various loads acting on offshore wind turbines and have to be taken into account in the design are classifieded into the following categories according to international standards: permanent loads, variable loads, accidental and mainly environmental loads.

To simulate the response of the turbine specialized software has been created by several companies and research centers. In this article a monopile and a tripod offshore wind turbine are simulated based on the advanced wind turbine simulation models proposed by the Natural Renewable Energy Laboratory (NREL) and the Natural Wind Technology Center (NTWC) in the United States [1]. FAST software fully simulates the coupled aero-hydro elastic response of an offshore wind turbine through a number of sub-codes, such as AeroDyn and HydroDyn, which are executed in parallel with FAST. These sub-codes exchange input/output data, so that FAST derives a final output file showing results for wind loads, wave loads and internal forces for structural members (blade and pylon), reactions, displacements etc. that are then used for the design of the wind turbines.

The objective of this paper is to present in a comparative way the analysis and design procedure of both monopile and tripod structural systems using FAST software and other specialized finite element software such as ADINA and SAP2000. For this purpose, two different load cases described in IEC standards 61400-3[2] "Design requirements of offshore wind turbines" are imposed in the offshore wind turbine simulation in FAST. In the second part, design and check of the superstructure and substructure members of monopile and tripod offshore wind turbine is carried out based on EN1993-1-6 [3] for shell structures.

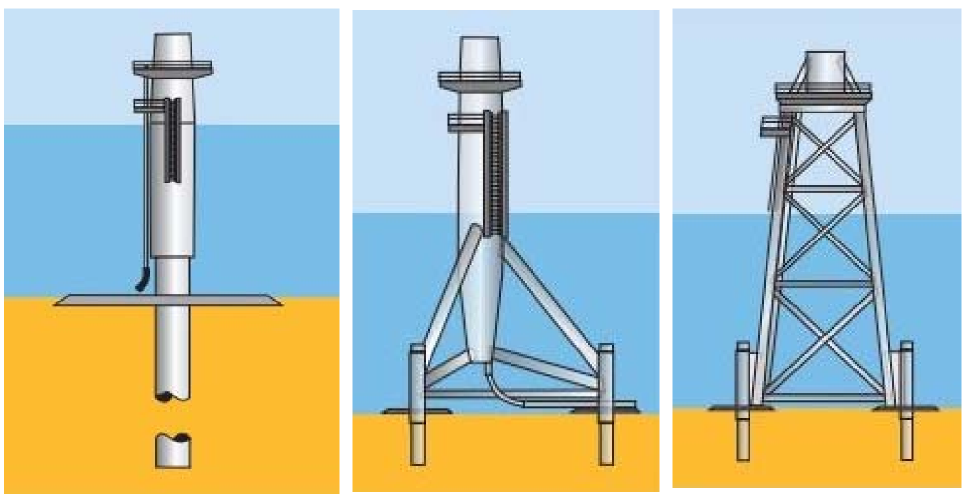

Figure 1: Monopile, Tripod and Jacket Foundation [Source: EWEA]

2 PROBLEM DEFINITION

2.1 Properties of the reference offshore wind turbines

The same generic offshore wind turbine is used for the analysis of both monopile and tripod foundation for comparative reasons. Thus, the parts of pylon and turbine above the mean sea level are exactly the same, and the two options are differentiated in terms of the foundation, corresponding to different sea depths. This prototype is the one developed by NREL known as NREL 5MW- Baseline Offshore Wind Turbine [4]. This is a conventional three bladed, upwind variable speed variable blade-pitch-to-feather-controlled turbine, with technical and operational characteristics presented in Table 1. The steel tower is of cylindrical variable cross section with diameter varying from 6m at the base to 3.87m at the top and wall thickness varying from 0.027m to 0.019m, respectively. The tower has a total height of 77.60m and is linked to the support structure 10m above mean sea level (MSL) through a transition piece of circular cross section with 6m diameter and thickness equal to 0.06m.

Rotor , hub diameter	126m, 3m
Hub height	90m
Cut-in, rated, cut-out wind speed	3m/s, 11.4m/s, 25m/s
Cut-in rated rotor speed	6.9rpm, 12.1rpm
Rated tip speed	80m/s
Rotor mass	110000kg
Nacelle mass	240000kg
Tower mass	347500kg

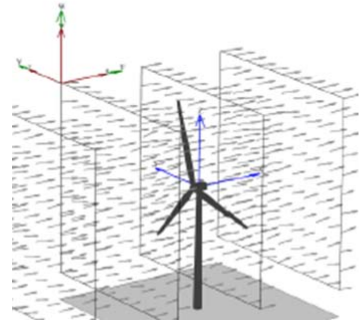

Table 1: Technical and operational properties of NREL 5MW- Offshore Wind Turbine and coordinate system

2.2 Location of the reference wind turbines- Soil properties

As regards the theoretical location of the wind turbines, the depths and environmental conditions chosen for this study are encountered in the Northern part of the Aegean Sea. However, the monopile structure is applied in the smaller depths of 28m whereas the tripod is preferred for the depth of 45m for reasons of efficiency and constructability, as explained above. The foundation soil is non-cohesive with internal friction angle $\varphi = 35^\circ$, while a uniform soil profile is considered for the penetration depth of the piles. The soil-pile interaction is simulated using nonlinear springs mounted in both transverse directions. In the vertical direction of the pile no springs are placed, as it is assumed that lateral friction can be incorporated into the peak resistance of the pile base and, moreover, due to relatively small developing compressive force, the settlements will be minimal. The stiffness of nonlinear springs depends on the geometrical characteristics of the pile and the properties of the soil layers. For non-cohesive soils, the calculation of nonlinear stiffness is based on the nonlinear lateral load-displacement "p-y" curves, which are obtained from guidelines for offshore platforms provided by the American Petroleum Institute [5], as:

$$p = Ap_u \tanh\left(\frac{kz}{Ap_u} y\right) \tag{1}$$

where p is the lateral load per meter of pile length in kN/m, y is the lateral displacement of the pile in m, k is the modulus of subgrade reaction in kN/m^3, z is the depth in m, p_u is the limiting value of lateral resistance per unit length in kN/m, while a=0.9 for cyclic loading [5]. The value of the limit lateral resistance p_u varies with the distance from the soil surface. Examples of these curves are shown in Figure 2.

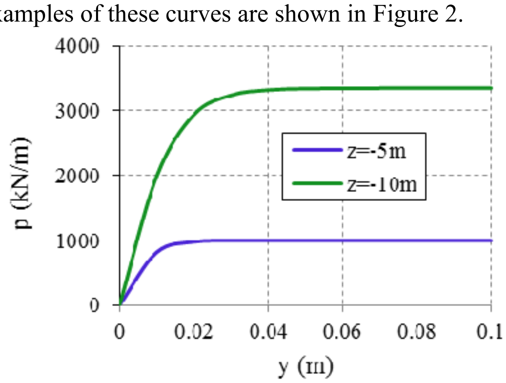

Figure 2: Load- displacement curves for depths z=-5m and z=-10m

2.3 Load Cases

Calculation of design loads for offshore wind turbines is a very complex procedure and is typically performed as a joint effort between manufacturers and designers. As regards wind climate and waves, there is a basic distinction between Normal and Extreme conditions, related to a certain return period that is for example 1year for Normal or 50 years for Extreme conditions.

To cover all possible design situations, International standards include a very large number of Design Load Cases (DLCs). Each environmental load case comprises a specific combination of environmental loads, depending on the operating condition of the wind turbine (power generation, planned or emergency stop, stop due to repair etc.). In this section, the two design load cases used in the analysis are presented (Table 2). In both cases the turbine is operating in mean winds between cut in and cut out wind speeds and is therefore in a power producing state. DLC 1.3 combines an Extreme Turbulent Wind Model (ETM) with Normal Sea State (NSS) whereas DLC 1.6 combines Normal Turbulent Wind conditions (NTM) with Severe Sea State (SSS), as can be seen in Table 2. Both load cases account for Ultimate Limit State rather than Fatigue.

DLC	Wind Condition	Wave Condition	Wind/Wave directionality	Type of analysis
1.3	ETM	NSS	Co-directional	ULS
1.6a	NTM	SSS	Co-directional	ULS

Table 2: Design Load Cases

For the determination of the artificial wind field, for each load case, a range of reference wind velocities (V_{hub}) is tested between V_{in} and V_{out} using TurbSim subcode. TurbSim produces a three dimensional wind field time history for every reference velocity and for each DLC simulated. Figure 3 below, shows indicative wind speed time series in x, y, z directions at the reference hub height. Time series were produced using time step equal to 0.05s and analysis time equal to 610s. It should be noted that the first 10sec of the analysis were excluded in order to minimize spurious response encountered during startup. As can be observed from the graphs, it is verified that DLC 1.3 simulating Extreme Turbulent wind gives a more turbulent wind profile with higher peaks. A statistical summary comparing these DLCs is presented in Table 3.

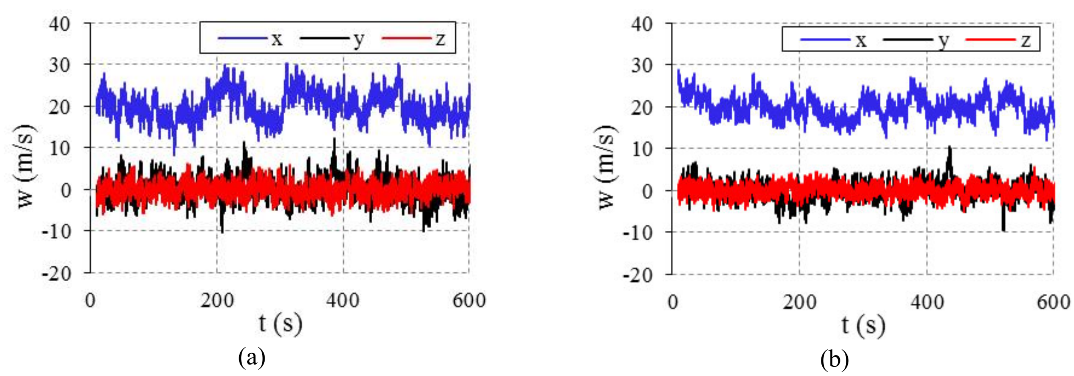

Figure 3: Wind velocity (m/s) time histories in three directions for DLCs 1.3(a) and 1.6a(b) with V_{hub}=20m/s

DLC	Mean w_x (m/s)	Max w_x (m/s)	Min w_x (m/s)	Sigma (m/s)	I %
1.3	20.07	30.38	8.10	3.30	16
1.6a	19.87	28.76	12.13	2.58	13

Table 3: Statistical Summary of w_x time series

The determination of the characteristic normal wave for DLC 1.3 was based on predicting the significant wave height and return period employing SPM-JONSWAP method [6]. Apart from average wind speed, wave height and period are also affected by fetch, which was taken equal to 113.62km as an adverse case according to [7]. For the Severe Sea State to be applied in DLC 1.6, a statistical processing was carried out in order to find the wave height with return period of 50 years. Approximately, the severe wave height was taken equal to H_{sss}=4.5m, while T_p is calculated for every mean wind speed.

3 DESIGN OF MONOPILE

3.1 Superstructure

In this section the analysis and design procedure of the monopile is presented. For each Load Case the following steps are followed:

1. First, the monopile is simulated in FAST for a range of wind speeds V_{in}< V_{hub}< V_{out} with a step of 2-3m/s, as stated in IEC standards. For each mean wind speed V_{hub}, 10 different simulations are carried out changing RandomSeeds parameters during the stochastic calculation of the artificial wind time series generated through TurbSim.
2. Sub-codes AeroDyn, HydroDyn running through FAST calculate wind and wave loading acting on the

monopile, both on pylon and blades. The response of the pylon and the support structure is then calculated through ElastoDyn and SubDyn, obtaining time series of internal forces (F_x, F_y, F_z, M_x, M_y, M_z) for nine output stations along the height of the pylon.

3. Then, from the time series of each internal force, maximum values are isolated along with the concurrent values of the other internal forces. In this way a final set of 6×6 forces is determined for every output station. Figure 4 illustrates results of the response of the structure F_x (kN), which is the shear force in the along wind direction and M_y (kNm), which is the moment in the transverse direction, at the transition piece level, for the range of reference wind speeds tested and for its 10 simulations, including the average result. It is noted that the most extreme response occurs for wind speed V_{hub}=20m/s, which is lower than the cutout wind speed V_{out}=25m/s, as the pitch control system rotates the blades for safety reasons when wind speeds approach V_{out}.

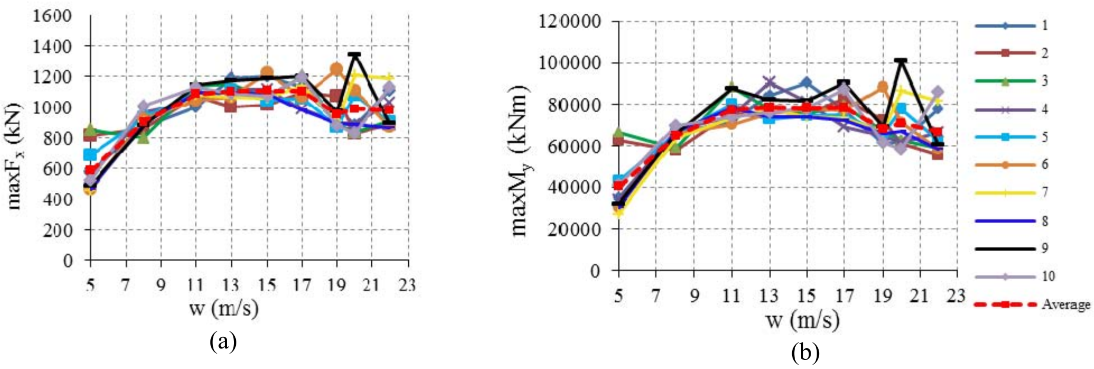

Figure 4 : DLC 1.3-Maximum actions M_y, F_x at the transition piece level for different mean wind speeds V_{hub} (m/s) and for 10 simulations

4. Finally, checking of the pylon in ULS is performed at each output station according to Eurocode EN1993-1-6 for shell structures [1]. In the examined case the most critical actions were found to take place for DLC 1.3, as can also be seen in Fig. 5(a) and for mean wind speed V_{hub}=20m/s. For this given wind speed the acting wave has the following characteristics according to SPM-Jonswap method [6]: H_s=2.63m and T_p=6.34sec. Analyses of the pylon also showed that for DLC 1.3 the maximum exploitation ratio equals r=0.54 (Figure 5(b)).

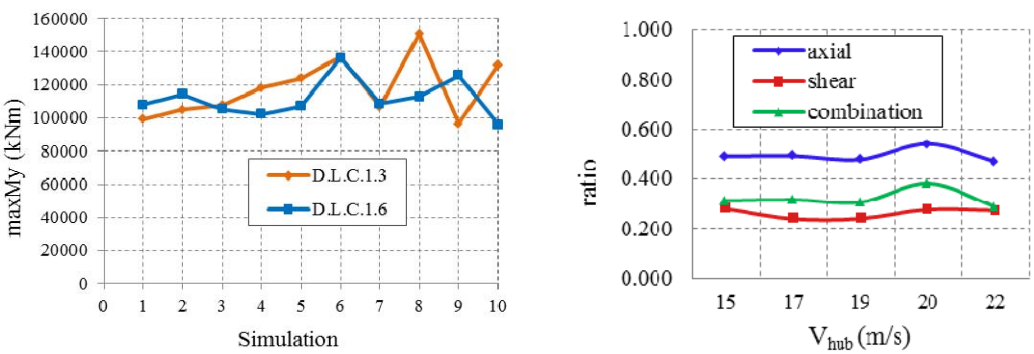

Figure 5: Maximum value of bending moment M_y for DLC 1.3 and DLC 1.6 for V_{hub}=20m/s and exploitation ratio of the most unfavorable output station for different velocities V_{hub}

3.2 Foundation

The interaction between the support structure and the soil cannot be taken into account in the current version of FAST, where the turbine can only be considered fixed at its base. The design of the pile is therefore carried out outside FAST environment, through simulation in finite element software ADINA. Soil data and modeling are described in section 2.2. For the worst loading condition analysis and design steps are outlined below:

1. Maximum actions at the bottom of the monopile (seabed level) are shown in Table 4, as obtained from dynamic analysis of the fixed offshore wind turbine in FAST.

No Set	M_x (kNm)	M_y(kNm)	M_z (kNm)	F_x (kN)	F_y (kN)	F_z (kN)
1	612.70	150500	-2049	1374	44.85	-9423
2	506.70	132000	-1079	2026	27.26	-9354

Table 4: Two sets of actions at the top of the pile

2. A model is created in ADINA simulating the pile with beam elements and the soil with nonlinear spring elements, as explained in section 2.2. The pile length was initially chosen equal to 30m, having a section with diameter D=6m and wall thickness t=0.06m.

3. Static analysis is carried out and the results are evaluated. In Figure 6 the deformed shape of the pile, and the axial and shear forces and bending moments along the pile are illustrated. It is deduced that a length of 30m is adequate, judging from the smooth deformed shape and very small values of actions at the bottom.

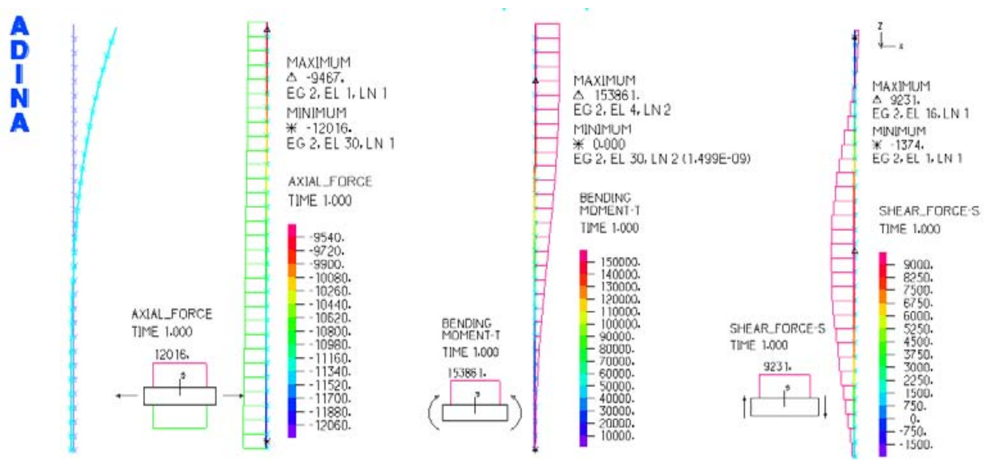

Figure 6: Deformed shape, axial and shear forces and bending moments for DLC 1.3-MaxMy-L=30m

4. The lateral carrying capacity of the soil is automatically taken into account by the nonlinear analysis, as shown in Figure 7, where nonlinear spring reactions F_x, F_y (kN) are plotted along with the ultimate soil strength P_u (kN). It is evident that for the upper 4m of the pile a plasticity zone is created and that reactions in y direction are almost zero, as expected.

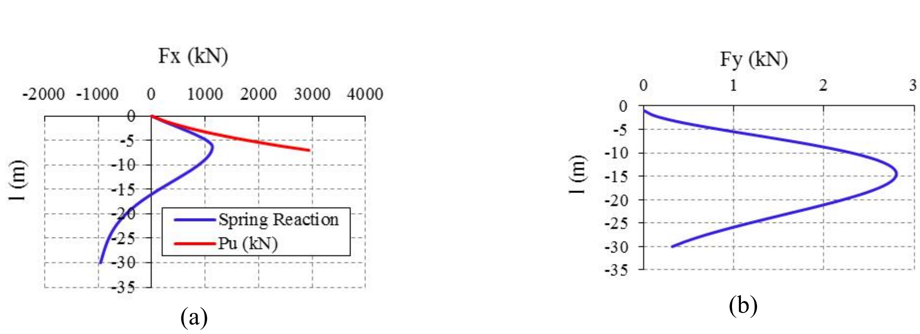

Figure 7: Spring reactions F_x (kN) and F_y (kN)

5. Finally, the pile's cross section is checked as a shell, according to EN1993-1-6. Regulations include check of the meridional, shear and circumferential shell buckling, if applicable [3]. The results showed that the ratios of exploitation of the pile's section are quite low, with maximum ratio equal to r=0.42. However, a decrease in diameter or thickness cannot be recommended at this point, as there is no evidence that the DLCs checked are the most critical for the structure.

3.3 Apparent Fixity Model

As discussed in the previous section, for the particular loading and soil data a pile length of 30m with a 6m diameter and 0.06m wall thickness has been selected. In addition, analysis of the pile in ADINA resulted in a horizontal displacement δ=0.0215m and rotation φ=0.00247rad at the pile top. Given the above, a fictitious pylon part is proposed to be inserted in FAST below the actual pylon, adopting the so called "apparent fixity model (AFM)", in order to substitute and efficiently simulate the soil- pile interaction in FAST modeling (Figure 8). For the calculation of the equivalent pylon part, the principle of virtual work is employed. For the given δ and φ, and the force F and moment M acting on the pile head, the length L and flexural stiffness EI of a stiffness-equivalent cantilever pile are obtained. An apparent fixity length L_{eq}=17m and a corresponding stiffness EI=1114792531kNm^2 (diameter D_{eq}=6m and thickness t_{eq}=0.065m) are obtained. Then, dynamic analyses of the AFM are carried out again in FAST in order to compare the resulting time histories with those of the initial fixed model (FM). It is deduced that for the same wind excitation (Fig.9(a)), the two models AFM and FM give slightly different results, as shown indicatively in Fig.9(b), where the maximum values of the AF model are larger than the ones of the FM by approximately 22%.

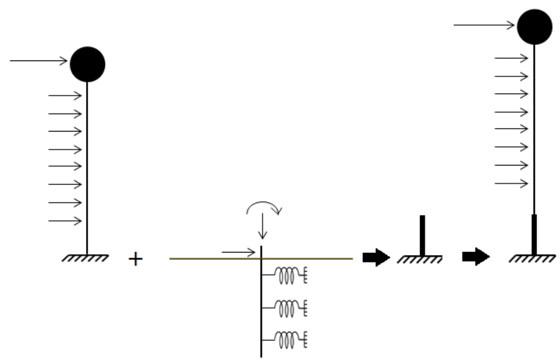

Figure 8: Procedure of analysis of the offshore wind turbine and determination of the AFM

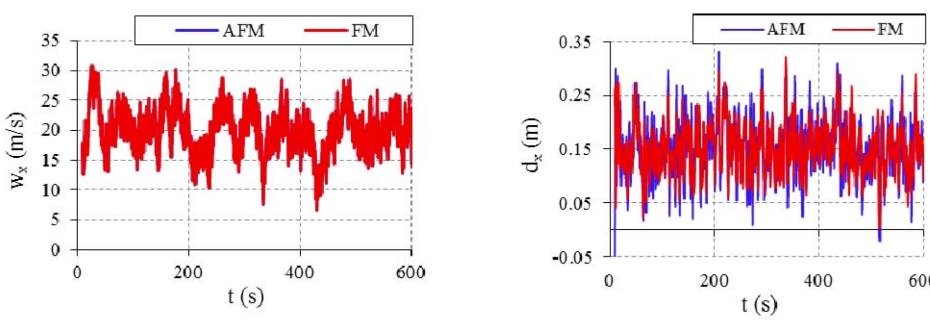

Figure 9: Wind speed w_x (m/s) time series and deformation d_x (m) at the top of the turbine for Apparent Fixity (AFM) and Fixed Model (FM)

4 DESIGN OF TRIPOD

4.1 Superstructure

In this section the analysis and design of the tripod support structure is presented. The geometry and characteristics are adopted from the reference tripod structure of the NREL laboratory [1]. For each one of the Load Cases of section 2.3 the following steps are applied:

1. As described in section 3.1 through FAST's dynamic analysis, the actions at the transition piece level of the offshore wind turbine are isolated, in order to be exerted at the top of the tripod as acting loads. It is

noted that FAST cannot yet produce internal forces of the tripod members, which are necessary in order to perform their structural design. The most critical sets of actions at that level are shown in Table 4, as obtained from dynamic analysis of the fixed turbine in FAST for DLC 1.3 and V_{hub}=20m/s.

No Set	M_x (kNm)	M_y(kNm)	M_z (kNm)	F_x (kN)	F_y (kN)	F_z (kN)
1	2610	101000	-676.30	1210	47.70	-5859
2	2610	-101000	-676.30	-1210	47.70	-5859
3	-1702	100000	2094	1341	-39.61	-5873
4	-1702	-100000	2094	-1341	-39.61	-5873

Table 5: Critical sets of actions at the top of the tripod

2. A model of the tripod support structure is created in finite element software SAP2000 in order to facilitate the design of its members based on EN1993. The dimensions of the tripod cross sections are then slightly modified in order to ensure that all secondary members are at least of section class 3. As can also be seen in Figure 10, beam elements are used for the simulation and fixity is applied at the bottom. As regards hydrodynamic loading, for the given characteristic wave H_s=2.63m and T_p=6.34s wave loads were approximately calculated based on Stokes 2nd Order Theory [8]. In Figure 11 indicative results of the analysis of the tripod are illustrated. All secondary structural members of the tripod are at very low levels of exploitation varying from 0.10 to 0.30. The main pylon has section class 4 and was checked separately, outside SAP2000, having a maximum exploitation ratio r=0.45.

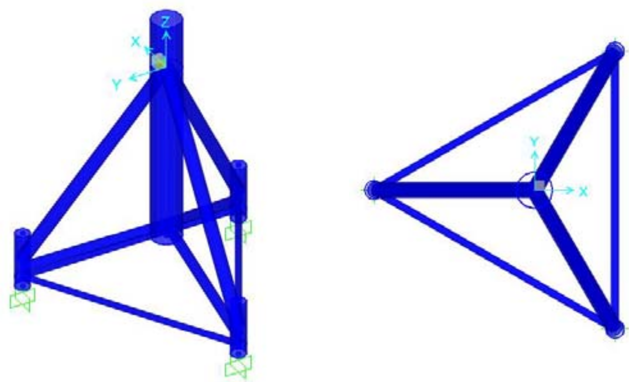

Figure 10: Simulation of the fixed tripod in SAP2000

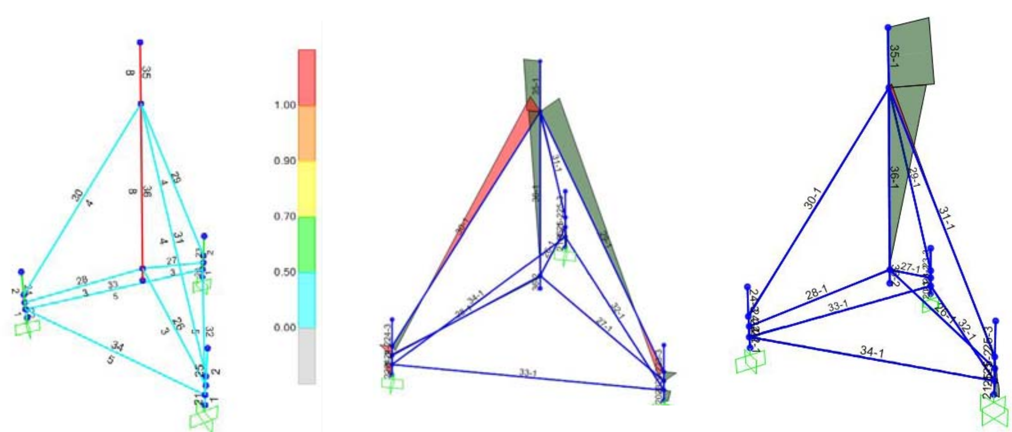

Figure 11: Ratios of exploitation and Bending Moments M_x, M_y for the worst loading case

4.2 Foundation

For the foundation of the tripod three identical piles are placed below the tripod's legs. After further analysis and testing, the length of the piles was selected equal to L=20m, with diameter D=2m and thickness t=0.02m. In this section the analysis and design procedure of the foundation of the tripod is presented.

1. A third numerical model is created in finite element software ADINA, including both superstructure

and foundation piles equipped with nonlinear soil springs, as shown in Figure 12. Loading imposed in this simulation is the same as in the model in software SAP2000 presented above.

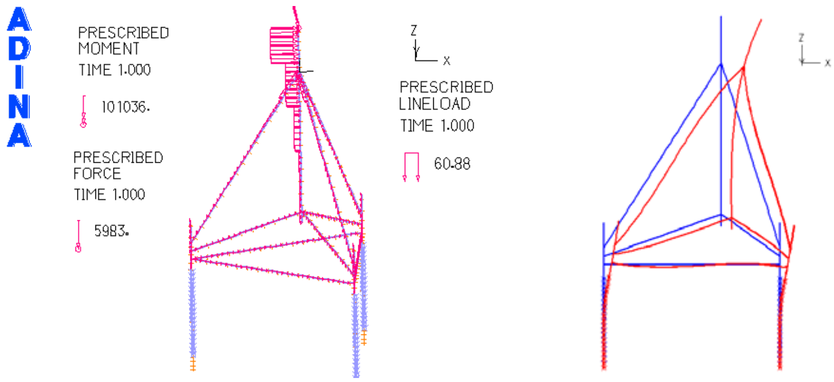

Figure 12: Modeling in ADINA and deformed geometry

2. Static analysis is carried out in order to obtain actions along the piles and reaction forces F_x, F_y of nonlinear springs. As in the monopile case, the adequacy of pile's length is evaluated through axial and shear force as well as bending moment plots, as shown in Figure 13. It is deduced that a length of 20m is adequate. The predominantly axial rather than flexural function of the piles in the tripod case is evident. As regards soil capacity, it is shown that a plasticity zone is created only in the upper 3m of the foundation (Figure 14).

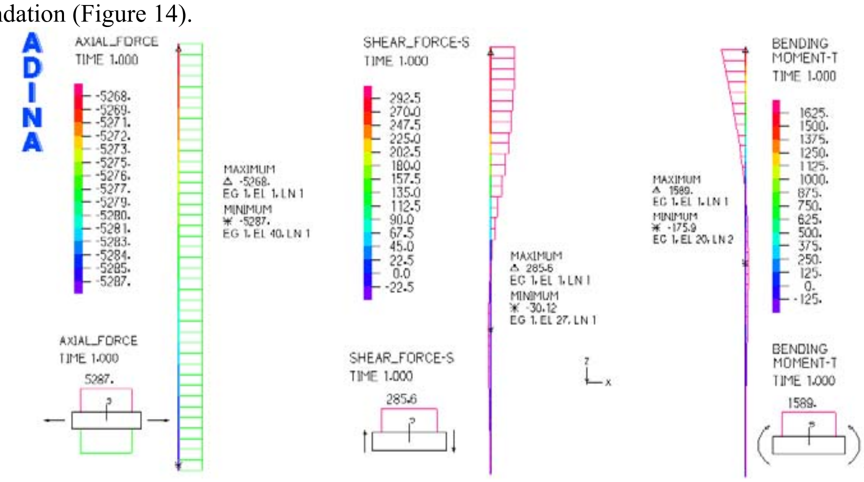

Figure 13: Pile 1- Axial, Bending and Shear Forces for DLC 1.3-MaxMy-L=20m

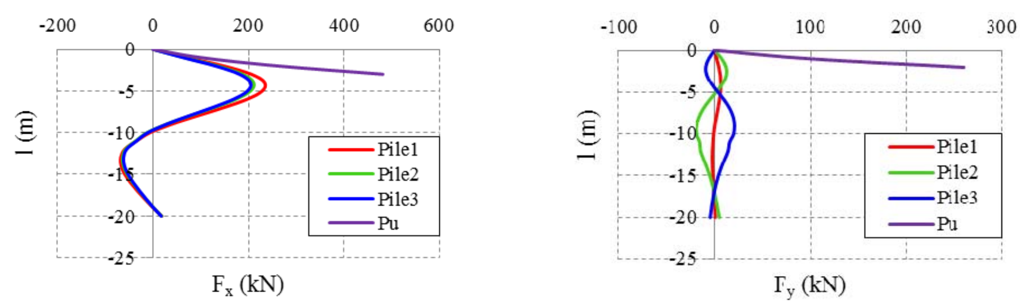

Figure 14: Spring reactions F_x (kN) and F_y (kN) for the three piles

4.3 Apparent Fixity Model

As discussed in section 3.3, the true tripod foundation and the surrounding soil are replaced by three cantilever stiffness-equivalent members that are fixed not at the original mudline but at lower depth, whose location is derived as a point of apparent fixity. For every loading condition maximum F, M actions and corresponding rotation φ and displacement δ at the top of every pile are obtained. Then, the apparent fixity length and section are calculated as described in section 3.3 for each pile. Finally, a single equivalent pylon part is adopted for all piles, calculated as the average of the three individual ones. The apparent fixity length is calculated equal to L_{eq}=9.50m and the thickness t_{eq}=0.048m, retaining the same external diameter D=2m. Results of the corresponding analysis of the tripod in FAST are similar to the case of the monopile and are omitted for reasons of brevity.

5 CONCLUSIONS

A proposed design methodology of both monopile and tripod support structures for offshore wind turbines has been presented. Due to the complexity of the determination of pertinent loads and actions, as well as the difficulty to efficiently simulate the response of the turbine's mechanical and structural parts and the interaction between their foundation and the soil, different numerical analyses are carried out, either dynamic or static, in different software environments. The monopile and tripod are analyzed considering the same design load combinations, the same soil properties and the same, generic offshore wind turbine developed at the National Renewable Energy Laboratory (NREL) that is characteristic of utility-scale offshore wind turbines being manufactured today. As regards simulating design load cases, analyses showed that numerous 10 minute simulations should be carried out for each wind speed tested and that DLC 1.3, simulating extreme wind conditions, was proven more critical than the one with simulating extreme wave conditions. This is closely related to the environmental conditions governing the assumed area of installation. As regards the design of both structures, it was shown that for these specific cases examined ratios of exploitation of the cross sections were very low implying that other design load cases would result in more critical results, as was initially predicted. Finally, the apparent fixity model exhibited slightly different peak results compared to the fixed model, although the mean values remained the same.

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