Structural Engineering and Mechanics, Vol. 36, No. 5 (2010) 599-624
DOI: http://dx.doi.org/10.12989/sem.2010.36.5.599

Basis of design and numerical modeling of offshore wind turbines

Francesco Petrini[1a], Hui Li[2b] and Franco Bontempi*[1]

[1]*School of Engineering, University of Rome "La Sapienza", Via Eudossiana 18, 00184 Rome, Italy*
[2]*Harbin Institute of Technology, P.O.B. 2537, 204 Haihe Road, Harbin, 150090, China*

(Received August 3, 2009, Accepted August 10, 2010)

Abstract. Offshore wind turbines are relatively complex structural and mechanical systems located in a highly demanding environment. In the present paper the fundamental aspects and the major issues related to the design of these special structures are outlined. Particularly, a systemic approach is proposed for a global design of such structures, in order to handle coherently their different parts: the decomposition of these structural systems, the required performance and the acting loads are all considered under this philosophy. According to this strategy, a proper numerical modeling requires the adoption of a suitable technique in order to organize the qualitative and quantitative assessments in various sub-problems, which can be solved by means of sub-models at different levels of detail, for both structural behavior and loads simulation. Specifically, numerical models are developed to assess the safety performances under aerodynamic and hydrodynamic actions. In order to face the problems of the actual design of a wind farm in the Mediterranean Sea, in this paper, three schemes of turbines support structures have been considered and compared: the mono pile, the tripod and the jacket support structure typologies.

Keywords: offshore wind turbines; systemic approach; numerical modeling; environmental actions; structural analysis and design.

1. Introduction

Offshore Wind Turbines (OWTs), which provide a renewable power resource (Hau 2006), are the result of an evolution of onshore plants, whose construction process is a relatively widespread and consolidated practice; in order to make the generated wind power more competitive than conventional exhaustible and high environmental impact sources of energy, the attention has been turned toward offshore wind power production (Breton and Moe 2008).

Besides being characterized by a reduced visual impact since they are placed far from the coast, OWT can take advantage from the more constant and intense wind force; this could increase the regularity and the amount of the productive capacity and could make such a resource more cost-effective, should the plant turn out to be durable and operating with minimum stoppage throughout

*Corresponding author, Professor, E-mail: franco.bontempi@uniroma1.it
[a]Ph.D., Associate Researcher, E-mail: francesco.petrini@uniroma1.it
[b]Professor, E-mail: lihui@hit.edu.cn

its life. This point is very demanding, being these structures located in a very harsh design environment.

From the general point of view, an OWT is formed by mechanical and structural elements. As a consequence, it is not a "common" civil engineering structure; it behaves differently according to its various functional phases (idle, power production etc.), and it is subjected to highly variable loads (wind, waves, sea currents etc.).

Moreover, since the structural behavior of offshore wind turbines is influenced by nonlinearities, uncertainties and interactions, this kind of structure can be defined a complex one (Bontempi 2006). In fact, it is now considered an emblematic example of structures on which new structural philosophies and smart technologies can be explored and applied (ASCE 2010).

These considerations highlight that, in Structural Engineering, a modern approach has to evolve from the idea of "Structure" as a simple device for channeling loads to the idea of a "Structural System" as "a set of interrelated components which interact one with the other in an organized fashion toward a common purpose" (NASA 2007): this systemic approach includes a set of activities which lead and control the overall design, implementing and integrating all the sets of interacting components (Bontempi *et al.* 2008a). In the present paper, the original NASA definition has been extended in such a way that the "structural system" contains also the actions; in this way, in what follows, the "set of interrelated components" is called simply "structure".

This general framework can handle all the design difficulties related to the different structural aspects; some of them can be referenced as:

- Foundation Design and Soil-Structure Interaction (Westgate and DeJong 2005, Zaaijer 2006),
- Marine Environment and Scour (Sumur and Fredsøe 2002, van der Tempel *et al*. 2004),
- Aerodynamic Optimization (Snel 2003),
- Structural Optimization and Fatigue Calculations (Veldkamp 2007, van der Tempel 2006),
- Vessel Impact and Robustness (Biehl and Lehmann 2006),
- Innovative Concepts and Possible Floating Support Configurations (Henderson and Patel 2003, Jonkman and Buhl 2007),
- Life Cycle Assessment (Gerdes 2006, Martìnez *et al.* 2008, Weinzettel *et al*. 2008),
- Standards Certification (API 1993, DNV 2004, GL 2002, GL-OWT 2005, BSH 2007, IEC 61400-3 2005).

This absolutely partial and biased toward authors' involvement list of problems and references gives a glimpse of what one means with problem complexity. Due to this reason, the design of these structures has to be carried out profitably under a Performance-Based Design philosophy: different aspects and various performances under several load conditions (referring to all possible system configurations that can be assumed by the blades and then by the rotor) have to be investigated for this type of structures (Bontempi *et al.* 2008a).

A noticeable amount of complexity comes from the lack of knowledge about the environment in which the turbine is located and from the pertinent modeling (Petrini *et al.* 2010). In particular, two main sources of uncertainty can be individuated: the stochastic nature of the environmental actions (aerodynamic and hydrodynamic actions, especially) and the possible presence of non linear interaction phenomena among different actions and among the actions and the structure.

Generally speaking, the uncertainties can be subdivided into three basic typologies: the aleatoric uncertainties (arising from the unpredictable nature of the magnitude, the direction and the variance of the environmental actions), the epistemic uncertainties (deriving from both insufficient information and errors in measuring the previously mentioned parameters) and the model uncertainties (deriving from the approximations in the models).

With regards to the wind model, for example, an aleatoric uncertainty is the one affecting the mean wind speed; an epistemic uncertainty is the one affecting the values of the aerodynamic coefficients of the structure, measured by wind tunnel tests, and, finally, considering the turbulent wind velocity field like a Gaussian stochastic process, a model uncertainty is related to the very hypothesis of the Gaussian character introduced.

While for the sake of simplicity, the epistemic uncertainties are not considered in the present paper; it is important to remark that the aleatoric uncertainties can be treated by carrying out a semi-probabilistic (looking for the extreme response) or a probabilistic (looking for the response probabilistic distribution) analysis while a possible way to reduce the model uncertainties is given by the differentiation of the modeling levels. This can be carried out for the structural models but also for both action and interaction phenomena models. For this last reason, various model levels have been adopted in the paper.

Generally speaking, the uncertainties can spread themselves during the various analysis phases which are developed in cascade; a malicious alignment of uncertainty sources could produce an unacceptable level of unquantifiable risk. For this reason, a suitable tool to govern the complexity is given by the *structural system decomposition* which is represented from the design activities related to the classification and the identification of the structural system components and of the hierarchies (and the interactions) among the components. The decomposition regards the structure, the actions and the performances, and it is the subject of the first part of the paper. In the second part, these ideas are applied to explore and assess different structural configurations for offshore wind turbines design, located in a wind farm project in the Mediterranean Sea.

2. Structural system decomposition

As previously stated, the decomposition is a fundamental tool for the design of complex structural systems (Simon 1998). It has to be done both for the structure and the design environment, and can be carried out focusing the attention on different levels of detail: the decomposition, usually, starts from a macro-level vision and goes on toward the micro-level details which, in the case of the structure, regard the connections level. Finally, the same process must be applied to the performances.

2.1 Structure decomposition

Following the previously introduced philosophy, the OWT structure is organized hierarchically, considering the structural parts categorized in three levels:

- *Macroscopic* (Macro level), related to geometric dimensions comparable with the whole construction or with a general role in the structural behavior; the parts so considered are called macro-components; one has essentially three components, as shown in Fig. 1:
  - the main structure which has to carry on the main loads;
  - the secondary structure, connected with the structural part directly loaded by the energy production system;
  - the auxiliary structure, related to specific operations that the turbine can normally or exceptionally face during its design life: serviceability, maintainability and emergency.

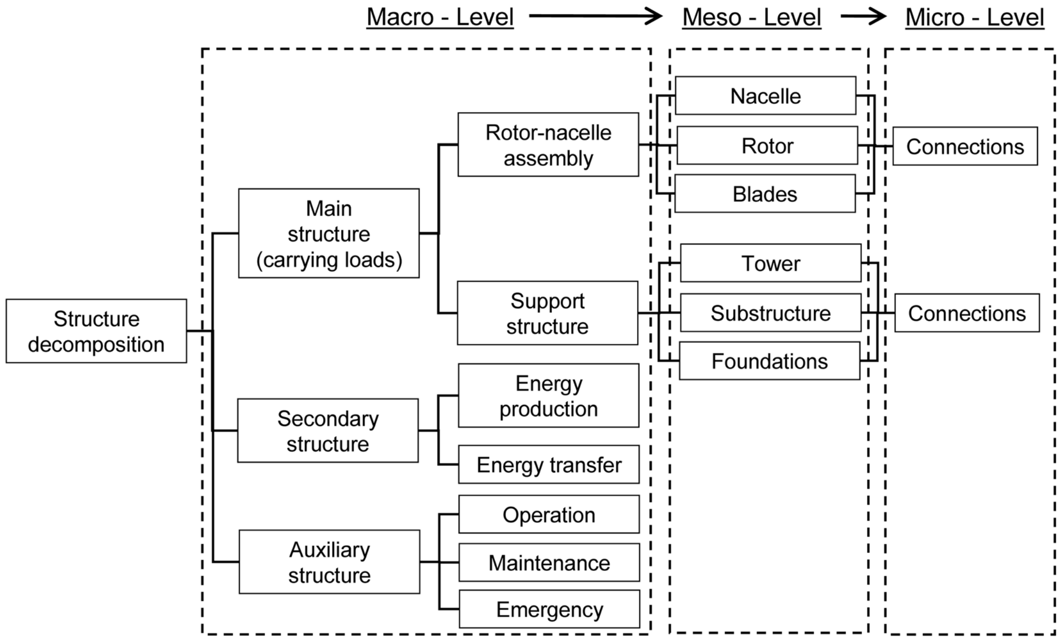


Fig. 1 Scheme for structural decomposition of an offshore wind turbine

Focusing the attention on the main structure, in general, the following segments can be further identified:

a. support structure (foundation, substructure and tower), which is the main subject of this paper;
b. blades-rotor-nacelle assembly.

- *Mesoscopic* (Meso level), related to geometric dimensions still relevant if compared to the whole construction but associated with a specialized role in the macro components; the parts so considered are called meso-components. In particular, the support structure can be decomposed in the following parts:
  a. foundation: the part which transfers the loads acting on the structure to the seabed;
  b. substructure: the part which extends itself upwards from the seabed and connects the foundation to the tower;
  c. tower: the part which connects the sub-structure to the rotor-nacelle assembly;
- *Microscopic* (Micro level), related to smaller geometric dimensions with specialized structural role: these are the components or elements.

The scheme of Fig. 1 can be then appreciated when related to Fig. 2, where the main parts of an offshore wind turbine structure are exposed. In this figure, it is shown that several substructure types could be adopted: the choice is related principally to water depth (h), soil characteristics and economical reasons. According to DNV-OS-J101 (2004), the following rough classification can be used: monopile, gravity and suction buckets ($h < 25$ m); tripod, jacket and lattice tower (20 m $< h <$ 40 ÷ 50 m); low-roll floaters and tension leg platform ($h > 50$ m). In the present study, attention has been focused on monopile, tripod and jacket types which are schematically depicted in Fig. 2.

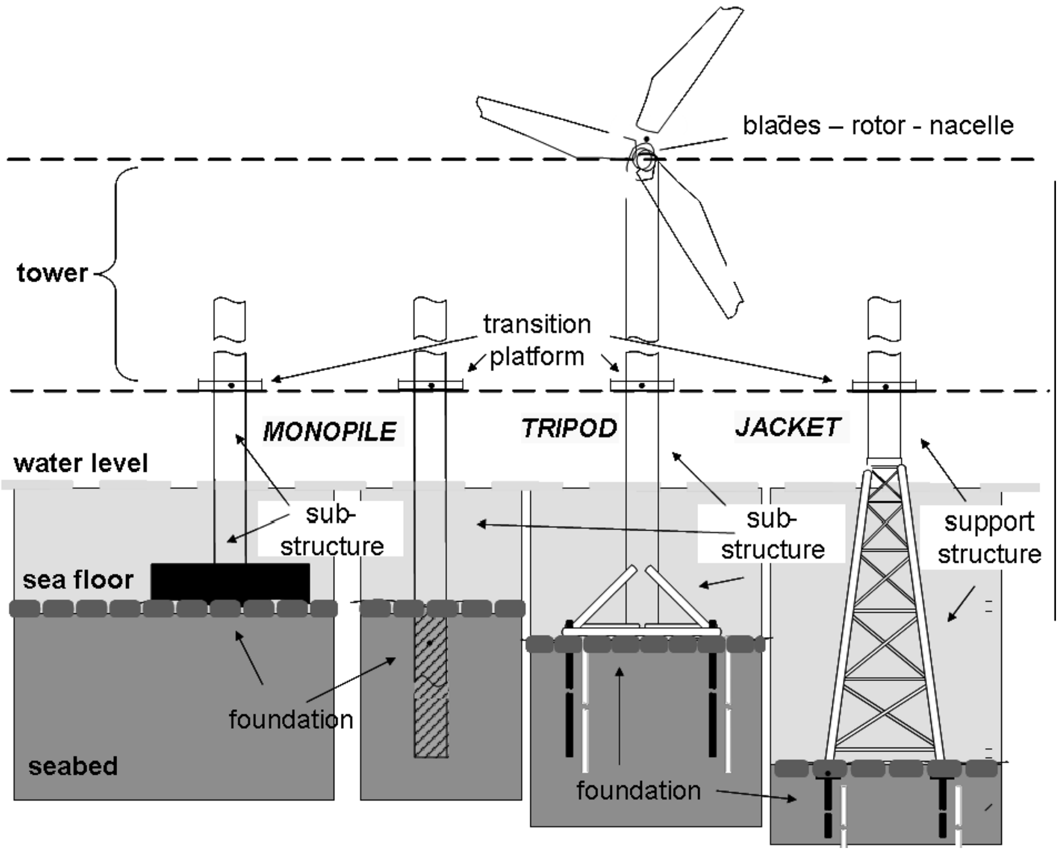


Fig. 2 Main parts of an offshore wind turbine structure with reference to three typologies (monopile, tripod, jacket) (a) foundation, (b) sub-structure, (c) tower, (d) blades-rotor-nacelle

The previous introduced structural decomposition has several meanings. One of the most important is related to the different safety and reliability characterization of the recognized parts: usually, foundations have, due to their fundamental role, safety requirements larger than the upper parts which can be more fallible. Roughly speaking, it means that one accepts to lose, for some reason, first, the rotor and the tower, then, the substructure and at last the foundations. There are also constructability differentiations, being the towers usually standardized parts, and also, obviously, differences on constructive tolerances. Of course, the most mechanical intensive parts, the blades-rotor-nacelle, have manufacturing standards more oriented toward industrial engineering than the remaining parts.

2.2 Actions decomposition

The second step of the structural system decomposition is connected to the actions. These can be decomposed as shown in Fig. 3, from which one can appreciate the high variety of acting loads. These actions can be connected with the environmental conditions as properly decomposed in Fig. 4.

It is important to underline that, since the environmental conditions have, in general, a stochastic

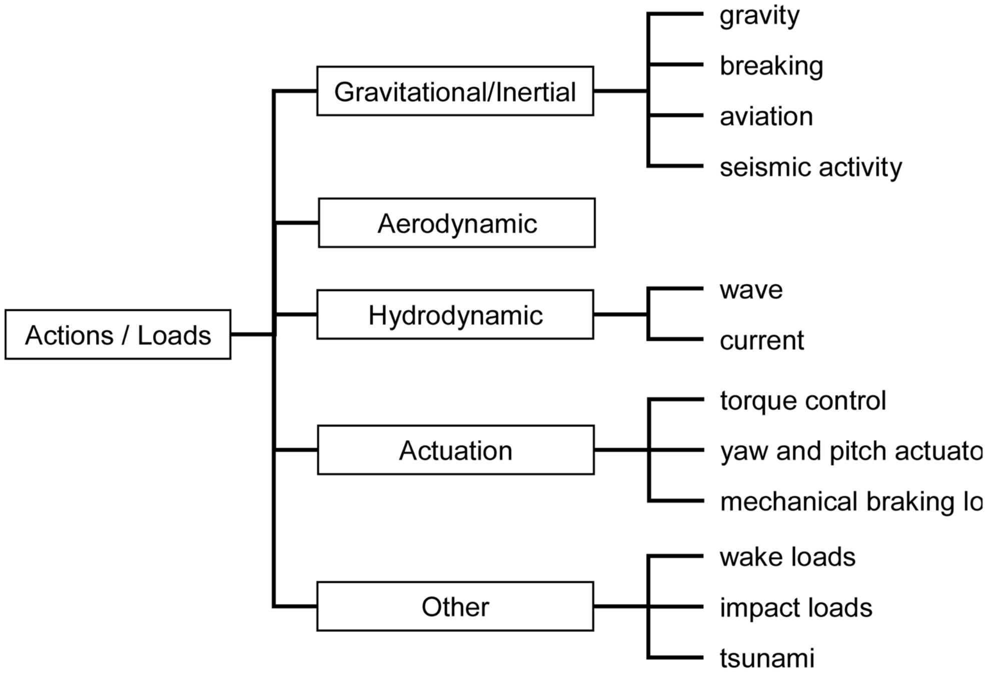


Fig. 3 Environmental actions/loads decomposition

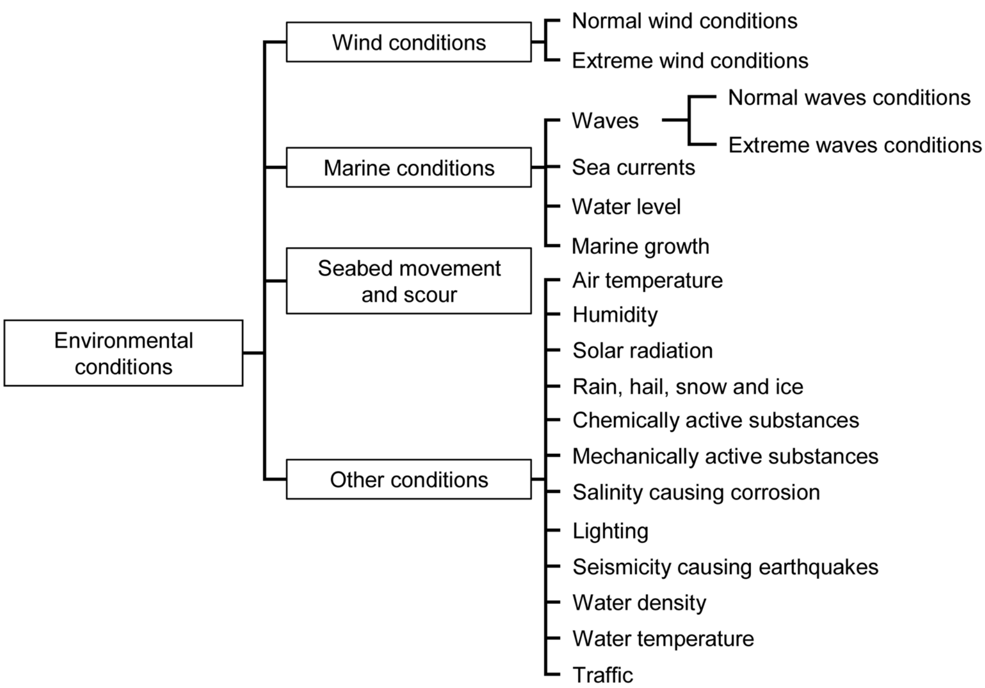


Fig. 4 Environmental conditions classification (partially adapted from IEC 61400-3)

nature, the magnitude of the actions involved can usually be characterized by a certain return period T_R: low values of T_R are associated with the so called "normal condition" while high values of T_R are associated with "extreme conditions". It is obvious that one of the most delicate design decisions is the choice of this value.

2.3 Performances decomposition

The performance requirements express qualitatively the performances that the structure must provide. These can be identified and decomposed as follows:

- assurance of the serviceability and operability of the turbine, as well as of the structure in general; as a consequence, the structural characteristics (stiffness, inertia, etc.) have to be equally distributed and balanced along the structure;
- assurance of a proper level of reliability for the entire life-span of the turbine; as a consequence, a check of the degradation due to fatigue and corrosion phenomena is required;
- safety assurance with reference to collapse in probable extreme conditions; this is applicable also to the transient phases in which the structure, or parts of it, may reside (e.g., transportation and assembly) which have to be verified as well;
- assurance of sufficient robustness of the structural system, that is to assure the proportionality between possible damage and resistance capacity, independently from the triggering cause providing, at the same time, an eventual endurance of the structure in hypothetical extreme conditions.

For the structural system identified, some criteria, which express quantitatively the performances, can be identified and, eventually, associated with appropriate Limit States subdivided in Serviceability Limit State - SLS, Ultimate Limit State - ULS and Accidental Limit State - ALS:

- Dynamic characterization of the turbine as dictated by the functionality requirements (SLS):
  - natural vibration frequencies of the whole turbine (comprehensive of the rotor-nacelle assembly), the support structure and the foundations;
  - compatibility of the intrinsic vibration characteristics of the structural system with those of the acting forces and loads;
  - compatibility assessment of the movement and the accelerations of the support system with the functionality of the turbine;
- Structural behavior regarding the serviceability (SLS):
  - limitation of deformations;
  - connections decompression;
- Preservation of the structural integrity in time (SLS):
  - durability with regards to the corrosion phenomenon;
  - structural behavior related to fatigue;
- Structural behavior for near collapse conditions (ULS):
  - assessment of the solicitations, both individual and as a complex, for the whole structural system, its parts, its elements and connections;
  - assessment of the resistance for global and local instability phenomena;
  - assessment of the global resistance of the structural system;
- Structural behavior in presence of accidental scenarios (ALS):
  - decrease in the load bearing capacity proportionally to the damage;
  - survival of the structural system in presence of extreme and/or unforeseen situations, such as

the possibility of a ship impacting the structural system (support system or blades), with consequences accounted for in risk scenarios.

3. Design environment and actions analytical models

The hierarchy that would be followed, in modeling the environmental loads acting on the structure, implies, first of all, the modeling of the generic environmental field (e.g., the wind velocity and hydrodynamic fields) and, successively, the modeling of the environment-structure interactions from which the environmental actions arise (e.g., the aerodynamic or aeroelastic phenomena for what wind actions and hydrodynamics for waves are concerned).

It is known that an environmental action, if it is observed during a short time period, is composed by two parts: a mean (or slowly variable) part and a stochastic part. In the case of the aerodynamic and hydrodynamic actions, the first component is generated by the mean wind velocity and by the sea current, while the stochastic component is generated by the turbulence wind velocity and by the not rotational (exception made for breaking waves) waves.

The definition "mean" must be specified in the sense that it is consistent only if one considers a

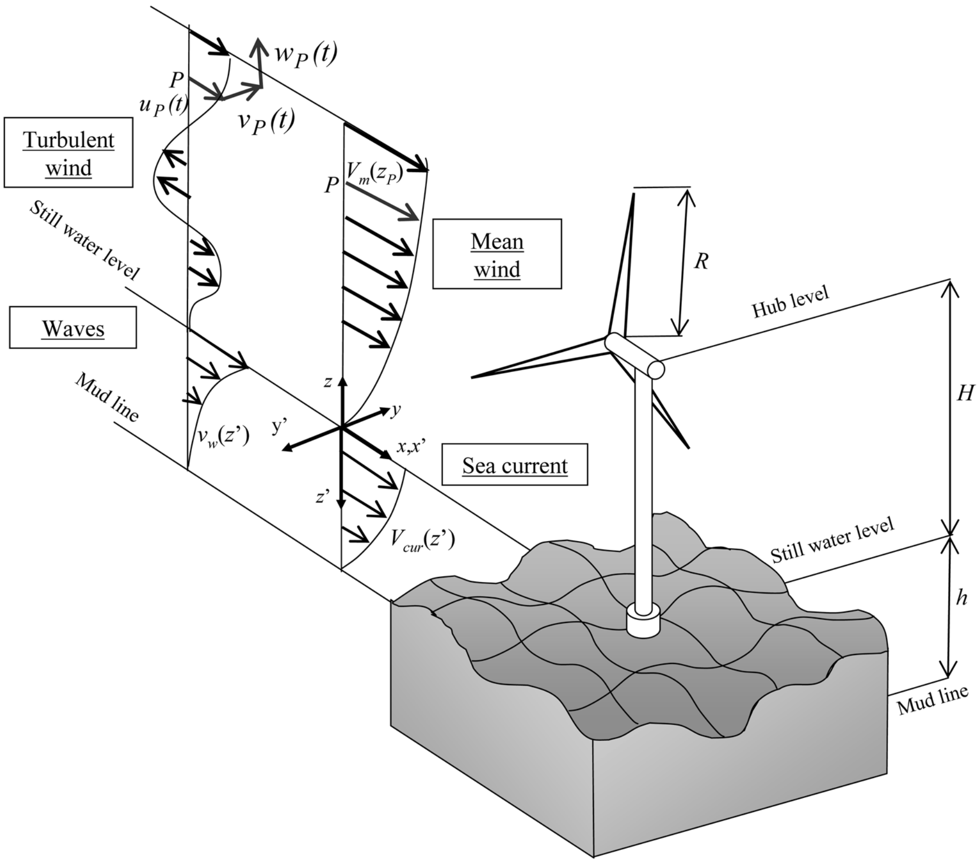


Fig. 5 Problem statement and main wind and wave actions configuration

specified "short time period" (usually less than 1 hour); on the contrary, the so called "mean component" varies in a stochastic manner during long time periods. For this reason, in what follows, the mean components will be considered as constant only for short periods of analyses.

The generic environmental configuration is shown in Fig. 5, where the macro geometric parameters defining the problem are also represented. These are: the water mean depth (h), the hub height above the mean water level (H) and the blades length (or rotor radius) (R).

A correct prediction of the structural response under extreme and normal load conditions requires the definition of their probability distribution and statistical parameters; these are site specific and have to be estimated by carrying out statistical analyses of the measurements database. In particular, two kinds of investigations are usually carried out: short term and long term statistics for performing fatigue and ultimate limit state analysis respectively.

To define the extreme load cases one needs to estimate the probability distribution for: (i) the extreme 10-min average wind velocity at the reference height; (ii) the significant wave height estimated in a 3-hour reference period along with the associated range of wave peak spectral periods.

When no information is available for defining the long term joint probability distribution of extreme wind and waves, the extreme 10-min mean wind speed, with 50-year recurrence period occurring during the extreme sea state with 50-year recurrence period (IEC 64100-3), shall be assumed and, possibly, reduced to suitable values.

3.1 Wind field model

Concerning the wind modeling for aerodynamic actions computation, a Cartesian three-dimensional coordinates system (x, y, z), with origin at water level and the z-axis oriented upward is adopted, as shown in Fig. 5. Focusing on a short time period analysis, the three components of the wind velocity field $V_x(j)$, $V_y(j)$, $V_z(j)$ at each spatial point j (the variation with time is omitted for simplicity) can be expressed as the sum of a mean (time-invariant) value V_m and turbulent components $u(j)$, $v(j)$, $w(j)$ with mean value equal to zero. Assuming that the mean value of the velocity is non zero only in the x direction, the three components are given by

$$V_x(j) = V_m(j) + u(j); \quad V_y(j) = v(j); \quad V_z(j) = w(j) \tag{1}$$

The mean velocity $V_m(j)$ can be determined by a database of values recorded at or near the site and evaluated as the record average over a proper time interval (e.g., 10 minutes).

The variation of the mean velocity V_m with the height z over a horizontal surface of homogeneous roughness can be described, as usual, by the exponential law

$$V_m(z) = V_{hub}\left(\frac{z}{z_{hub}}\right)^{\alpha} \tag{2}$$

In this expression, V_{hub} is the reference wind velocity at the rotor altitude z_{hub}, with $\alpha = 0.14$ for extreme wind conditions. The 10-minute wind speed V_{hub} is defined as a function of the return period T_R; it is the (1-1/T_R) quantile in the distribution of the annual maximum 10-minute mean wind speed, i.e., the 10-minute mean wind speed whose probability be exceeded in 1 year is 1/T_R. It is given by (DNV-OS-J101 2004)

$$V_{hub,T_R(z)} = F^{-1}_{V_{hub},\max,1\,year}\left(1-\frac{1}{T_R}\right) \tag{3}$$

where T_R > 1 year and $F_{V_{hub},\max,1\,year}(\bullet)$ is the cumulative distribution function of the annual maximum value of the 10-minute mean wind speed.

The turbulent components of the wind velocity have been modeled as zero-mean Gaussian ergodic independent processes. Moreover, in probabilistic calculations, only the random spatial variation with the height z has been taken into account by considering the wind acting on N vertically aligned points. By neglecting the vertical component w of the wind velocity, the turbulent components u and v are completely characterized by the power spectral density matrices $[S]_i$ ($i = u, v$).

The diagonal terms (auto-spectra) $S_{i_j i_j}(n)$ of $[S]_i$ (j = 1, 2, …, N) have been expressed in terms of normalized one-side power spectral density (Solari and Piccardo 2001) as

$$\frac{nS_{u_j u_j}(n)}{\sigma_u^2} = \frac{6.868\cdot n_u}{[1+10.302 n_u^2(z_j)]^{5/3}}$$
$$\frac{nS_{v_j v_j}(n)}{\sigma_v^2} = \frac{9.434\cdot n_v}{[1+14.151 n_v^2(z_j)]^{5/3}} \tag{4}$$

where n is the current frequency (in Hz), z_j is the height (in m) of point j, σ_u^2 and σ_v^2 are the variances of the velocity fluctuations, given by the relationships (Solari and Piccardo 2001)

$$\sigma_i^2 = [6\text{-}1.1\,\mathrm{arctang}(\log(z_0)+1.75)]u_*^2$$
$$\frac{\sigma_v}{\sigma_u} = 0.7 \tag{5}$$

Where z_0 is the roughness length, u_* is the friction or shear velocity (in m/s), given by: $(0.006)^{1/2}$ $V_m(z = 10)$, $n_i(z_j)$ is a non-dimensional height dependent frequency given by

$$n_i(z) = \frac{nL_i(z_j)}{V_m(z_j)} \tag{6}$$

The integral scale $L_i(z_j)$ of the turbulent component can be derived for $i = u, v$, according to the procedure given in ESDU (2001).

The out of diagonal terms (cross-spectra) $S_{i_j i_k}(n)$ of $[S]_i$ (j, k = 1, 2, …, N) are given by

$$S_{i_j i_k}(n) = \sqrt{S_{i_j i_j}(n) S_{i_k i_k}(n)}\cdot\exp(-f_{jk}(n)) \tag{7}$$

where for vertically aligned points (Di Paola 1998)

$$f_{jk}(n) = \frac{|n|\sqrt{C_z^2(z_j-z_k)^2}}{2\pi(V_m(z_j)+V_m(z_k))} \tag{8}$$

and C_z represents the *decay coefficient* that is inversely proportional to the spatial correlation of the process.

Using the proposed model, it is possible to generate samples of the wind action exerted on each point j of the structure.

3.2 Hydrodynamic field model

The hydrodynamic actions are due to sea currents and waves.

The sea currents caused by the tidal wave propagation in shallow water can be characterized by a practically horizontal velocity field whose intensity slowly decreases with the depth. Adopting a Cartesian coordinate system (x', y', z') with origin at the still water level and the z-axis oriented downward (Fig. 5), the variation in current velocity with the depth is given by (DNV-OS-J101 2004)

$$V_{cur}(z') = V_{tide}(z') + V_{wind}(z')$$
$$V_{tide}(z') = V_{tide0} \cdot \left(\frac{h-z'}{h}\right)^{1/7}$$
$$V_{wind}(z') = V_{wind0} \cdot \left(\frac{h_0-z'}{h_0}\right) \tag{9}$$

where $V_{tide}(z')$ and $V_{wind}(z')$ are the velocities generated by the tide and the wind; z' is the depth under the mean still water level; V_{tide0} and V_{wind0} are the tidal current and the wind-generated current at still water level; h is the water depth from still water level (taken as positive); h_0 is a reference depth (typically taken equal to 20 m).

In absence of site-specific measurements, the wind-generated current velocity may be taken equal to (DNV-OS-J101 2004)

$$V_{wind0} = 0.01 \cdot V_{1hour}(z = 10\text{ m}) \tag{10}$$

where V_{1hour} is the 1-hour mean wind speed.

Waves act on the submerged structural elements and on the transition zone above the still water level; therefore, the wave actions are due to the motion of the fluid particles and to the breaking waves, which may occur in shallow water conditions.

In general, the wave height is a time-dependent stochastic variable, described by:

- the significant wave height H_S, defined as four times the standard deviation of the sea elevation process; it is the measure of the intensity of the wave climate as well as of the variability in the arbitrary wave heights;
- the spectral peak period T_P, related to the mean zero-crossing period of the sea elevation process.

For extreme event analysis, the significant wave height is defined as a function of the return period T_R (DNV-OS-J101 2004)

$$H_{S,T_R}(z) = F^{-1}_{H_S,\max,1year}\left(1-\frac{1}{T_R}\right) \tag{11}$$

where $F_{H_S,\max,1year}$ represents the maximum annual significant wave height, which can be deduced by means of a Weibull distribution.

For particular performance investigations, like the fatigue analysis of the structure subjected to the wave action, it is necessary to define an appropriate spectral density of the sea surface elevation.

The characteristic spectral density of the specific sea-state $S(f)$ is defined by means of the parameters H_S and T_P and an appropriate mathematical model. Usually the Jonswap spectrum is adopted for a developing sea, given by

$$S(f) = \frac{\alpha g^2}{(2\pi)^4} f^{-5} \exp\left[-\frac{5}{4}\left(\frac{f}{f_P}\right)^{-4}\right] \gamma^{\exp\left[-0.5\left(\frac{f-f_P}{\sigma f_P}\right)^2\right]} \tag{12}$$

where $f = 2\pi/T$ is the frequency; $f_P = 2\pi/T_P$ is the peak frequency; α and g are constants; σ and γ are parameters depending on H_S and T_P.

In general, the sea state is characterized by a distribution of the energy spectral density dependent on the direction of the wave components: this can be obtained by multiplying the one-dimensional spectrum $S(f)$ by a function that takes into account the directional spreading and is symmetrical with respect to the principal direction of the wave propagation.

Finally, the designer has to identify the analytical or numerical wave theories and their range of validity, which may represent the kinematics of waves:

- linear wave theory (Airy theory) for small-amplitude deep water waves; wave profile is represented by a sine function;
- Stokes wave theories for high waves;
- stream function theory, based on numerical methods accurately representing the wave kinematics over a broad range of water depths;
- Boussinesq higher-order theory for shallow water waves;
- solitary wave theory for waves in very shallow water.

In the numerical calculations, for the sake of simplicity, the kinematics of waves has been described by the linear wave theory applied to small-amplitude deep water waves, and the wave profile has been represented by a sinusoidal function.

3.3 Actions model

In general, the actions components could be calculated separately for all structural elements adopting a common frame of reference and then superimposed by a vectorial sum in a phase-correct manner.

The aerodynamic force can be decomposed, as usual, in a drag (parallel to the mean wind velocity) and a lift (orthogonal to the mean wind velocity) component, while moments have been neglected in the present paper. These can be computed for each structural component, for the specific wind velocity field and for each structural configuration (for example, extreme wind and parked turbine configurations), by using well known expressions (Petrini *et al*. 2007, Bontempi *et al*. 2008b). Equivalent static load can be derived by using peak factors based on the probabilistic characteristics of the wind velocity modeled as stochastic process (Van Binh *et al.* 2008). Another important aspect, concerning the wind action, regards the presence of the so called aerodynamic damping in addition to the material one. This arises when, depending on the actual tower top movement and blade deflection, the relative wind speed vector is reduced or enlarged, with a consequent change in the wind-blades angle of attack. This effect is relevant only in the load cases characterized by the operational activity of the turbine rotor.

Concerning the hydrodynamic loads on a structural slender cylindrical member (D/L < 0.2, with: D member diameter normal to the fluid flow, L wave length), both wave and (stationary) current generate the following two kind of forces:

- a force per unit length acting in the direction perpendicular to the axis of the member and is related to the orthogonal (with respect to the member) components of the water particle velocity (wave v_w plus current V_{cur} induced) and acceleration (wave only); it can be estimated by means of Morison equation (Brebbia and Walker 1979)

$$dF(z',t) = \left(c_i \frac{\rho_{wat}\pi D^2}{4}\dot{v}_w(z',t) + \frac{1}{2}c_d\rho_{wat}D(v_w(z',t)+V_{cur}(z',t))\left|v_w(z',t)+V_{cur}(z',t)\right|\right)dz' \quad (13)$$

where ρ_{wat} is the water density, c_i and c_d are the inertia (including added mass for a moving member) and drag coefficient respectively, which are related to structural geometry, flow condition and surface roughness: the dot indicates the time derivate. Periodic functions are adopted for both wave velocities and accelerations (Brebbia and Walker 1979).

- a non-stationary (lift) force per unit length acting in the direction perpendicular both to the axis of the slender member and to the water current. This component is induced by a vortex shedding past the cylinder and inverts direction at the frequency f_l of eddies separation which is related to flow field and structural geometry through Strouhal number $St = Df_l/V_{cur}$. It should be kept far from the structure's natural frequency to avoid resonances.

In the case of static analysis, equivalent static forces are applied considering the maximum values of the fluctuating actions components and, eventually, applying certain load amplification factors.

4. Numerical modeling

As previously stated, in order to reduce the model uncertainties, a differentiation of the modeling level is adopted. The level of a generic model of the structure is here identified by means of two parameters: the maximum detail level and the scale of the model. If the finite element method is adopted, at each model level it is possible to associate a certain typology of finite element which is mainly used to build the model. To explain this point, one can say that, in general, four model levels are defined for the structure:

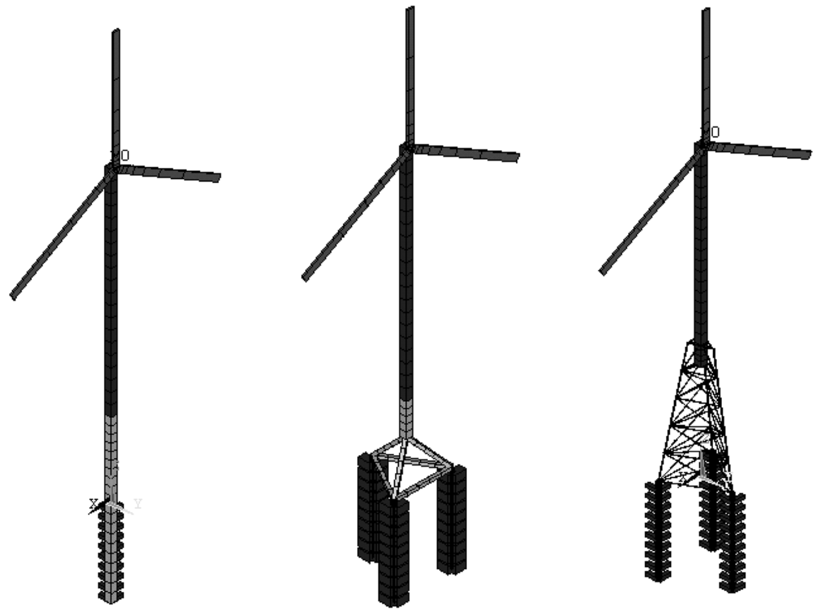
1. *systemic-level* (S): the model scale comprises the whole wind farm and can be adopted for evaluating the robustness of the overall plant. Highly idealized model components are used in block diagram simulators;
2. *macro level or global modeling* (G): in these models the scale is reduced to the single turbine, neglecting the connections among different structural parts; the component shapes are modeled in an approximate way, only the geometric ratios among the components being correctly reproduced; beam finite elements are mainly adopted;
3. *meso-level or extended modeling* (E): these models are characterized by the same scale of the previous level but with a higher degree of detail: the actual shape of the structural components is accounted for and the influence of geometrical parameters on the local structural behavior is evaluated. Shell elements are adopted for investigating the internal state of stress and strain (e.g., for fatigue life and buckling analysis) in the structure extrapolated from previous models;
4. *micro-level or detail modeling* (D): this kind of models is characterized by the highest degree of detail and is used for simulating the structural behavior of specific individual components, including connecting parts, for which a complex internal state of stress has been previously identified e.g. due to the presence of concentrated loads. Shell or even solid finite elements are used.

The different structural model levels features are summarized in Table 1; an analogous distinction can be made for the specification of the external loads.

According to what stated above, at the initial stage of investigation, structural analyses have been

Table 1 Definition of the model levels

Model level	Scale	Maximum detail level	Main adopted Finite Elements
Systemic level	wind farm	approximate shape of the structural components	BLOCK elements
Macro level	single turbine	approximate shape of the structural components, correct geometry	BEAM elements
Meso level	single turbine	detailed shape of the structural components	SHELL, SOLID elements
micro level	individual components	detailed shape of the connecting parts	SHELL, SOLID elements



(a) (b) (c)

Fig. 6 Macro level finite element models: (a) monopile, (b) tripod and (c) jacket support types, springs for preliminary soil-foundation-structure interaction are shown

carried out with macro-level and meso-level models for the three offshore wind turbine types previously described. Some of the developed structural models (macro level models only) are shown in Fig. 6.

The effect of the foundation soil should be simulated with a full non-linear model in order to account for possible plastic effects and load time-history induced variation of the mechanical properties (Zaaijer 2006). At this level of investigation an idealized soil has been simulated through:

- linear springs - such technique has been adopted for macro-level models. Springs are applied at the pile surface and act in the two coordinate horizontal directions. The corresponding mechanical parameters have been set up on the basis of soil characteristic available to simulate its lateral resistance at the pile interface;

Table 2 Model and analysis type selection

Structural component	Performance		Model level	Analysis type
Tower	Stress Level (ULS)	→	Macro Meso	Static extreme
	Global Buckling (ULS)	→	Macro Meso	Static incremental
	Local Buckling (ULS)	→	Meso Micro	Static incremental
	Fatigue (FLS)	→	Macro (poor) Meso Micro	Dynamic

- solid elements - used for meso-level models; these three dimensional elements simulate the linear mechanical behavior of the soil. The extension of the foundation included in the model has been selected in order to minimize boundary effects.

Both kinds of models have been used for evaluating the modal response of the structural system.

The decomposition of both structural system and performances and the differentiation of the model levels can be used to guide and optimize the numerical analysis efforts in this design phase: focusing on a certain structural component and selecting a specific performance that has to be investigated for that specific component, the choice of both the model level and the type of analysis to be adopted can be done so as to obtain the best efficiency of the analysis (deriving from the balance between the level of outputs detail and the computational efforts).

For example, focusing the attention on the tower (having a steel tubular section), the maximum stresses for Ultimate Limit States calculation can be preliminary obtained by adopting a macro level model and by carrying out a static extreme analysis (characterized by extreme values of the environmental loads). If one wants to assess the local buckling phenomena, at least a meso-level structural model and a static incremental analysis are needed. These considerations are resumed in Table 2.

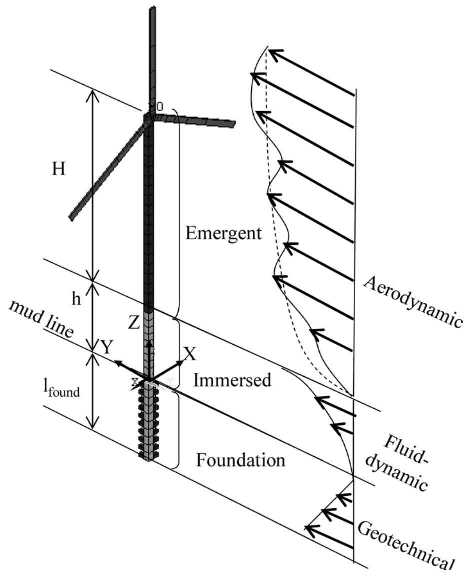
5. Application

The numerical analyses have been conducted for the three support typologies previously introduced for OWT, designed for a wind farm project in the Mediterranean Sea. The main structural characteristics of the models are listed in Table 3.

A typical situation in the initial design step is the choice among various design configurations. This decision has to be taken essentially on the basis of the results deriving from basic numerical analyses (e.g., static or modal analysis). In a successive design phase, more advanced (prevalently dynamic) analyses have to be carried out in order to optimize the performances of the previously selected configuration.

Following this philosophy, first of all, the performances of the three support typologies are compared by carrying out modal and extreme static analyses. Successively, the structural response of the selected support typology (jacket) is investigated by means of dynamic analyses carried out adopting both time and frequency domain techniques.

Table 3 Main structural characteristics

	Monopile [m]	Tripod [m]	Jacket [m]
	$H = 100$ h = 35 $l_{found} = 40$ $D = 5$ $t_w = 0.05$ $D_{found} = 6$	$H = 100$ $h = 35$ $l_{found} = 40$ $D = 5$ $t_w = 0.05$ $D_{tripod} = 2{,}5$ $t_{w\ tripod} = 0{,}05$ $D_{found} = 2.5$	$H = 100$ $h = 35$ $l_{found} = 40$ $D = 5$ $t_w = 0.05$ jacket members: $D_{vert} = 1.3$ $t_{w\ vert} = 0.026$ $D_{hor} = 0.6$ $t_{w\ hor} = 0.016$ $D_{diag} = 0.5$ $t_{w\ diag} = 0.016$

D = tubular tower diameter;
D_{found} = foundation piles diameter (concrete);
D_{tripod} = tubular tripod arm diameter;
$D_{vert,\ hor,\ diag}$ = diameter of the jacket vertical, horizontal or diagonal tubular members
t_w = thickness of the tower tubular member;
$t_{w\ tripod}$ = thickness of the tripod arm tubular member;
$t_{w\ vert,\ hor,\ diag}$ = thickness of the jacket vertical, horizontal or diagonal tubular members

5.1 Modal analysis

The preliminary task of the modal analysis is to assess the natural modes of vibration in order to avoid that a non-stationary load could cause the system resonance when excitation and natural frequencies are closer.

Geometrical parameters of the three support structures have thus been selected with the aim of maintaining the corresponding natural frequency far from the one of the non-stationary external forcing loads (wind and wave).

Finite element modal analysis has provided the deformed shapes given in Fig. 7: here, have been displayed only odd modes since modes i and $i + 1$ (with $i = 1, 3 \ldots$) have the same frequency but vibration occurs in orthogonal planes, according to the axial symmetry of the tower (the eccentricity of the mass of the blades is neglected).

In particular, in Fig. 7(d), the two x-parallel dashed lines correspond, when referred to a constant rotor velocity, to the mean rotor frequency ($1P$) and to the frequency of a single passing pale ($3P$) which is three times the former one, for a three bladed turbine. These frequencies determine the sampling period of the wind turbulent eddies and, as a consequence, the characteristics of the induced non-stationary actions.

As a consequence, they acquire importance when performing a dynamic analysis and are generally compared to the first natural frequency f_{nat} to classify the structural behavior:

- if f_{nat} falls below $1P$, the structure is said soft-soft. For this type of structures, the wave load could be dominant. Because of the structural low frequency, the fatigue effects could be relevant;

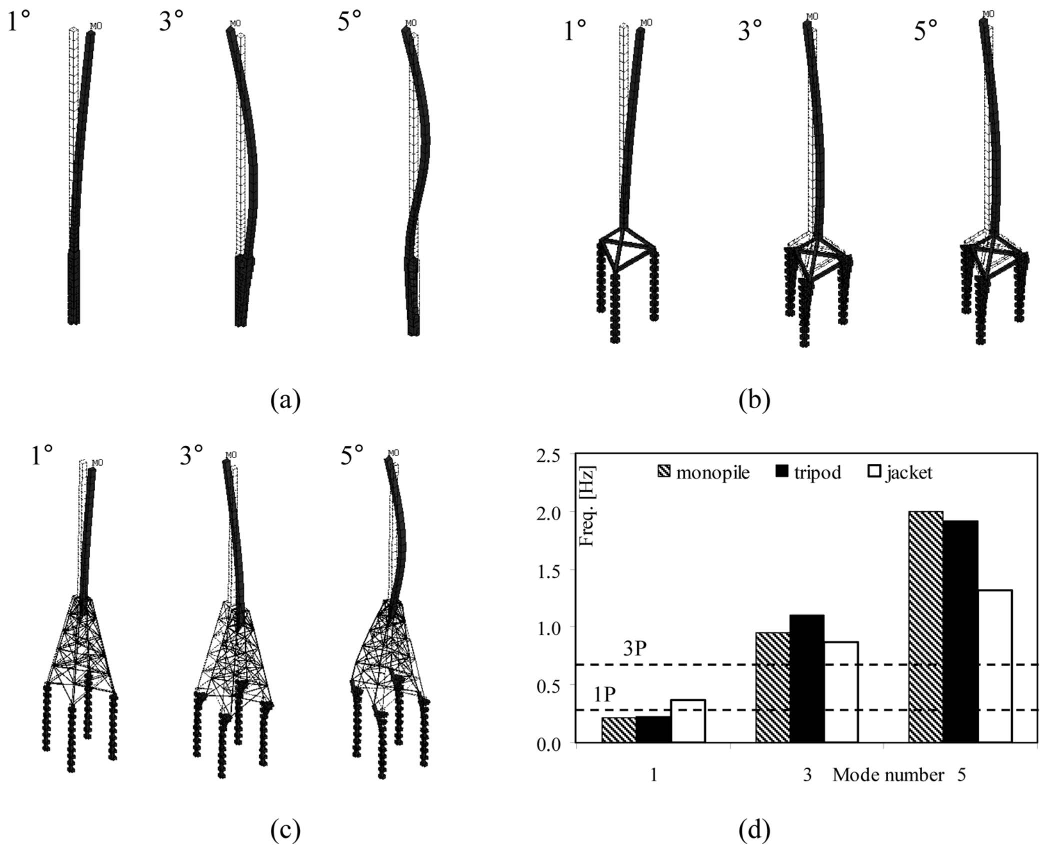


Fig. 7 Modal analysis (macro-level models). (a) Natural mode shapes for monopile, (b) tripod and (c) jacket support structures, (d) comparison of natural frequencies

- when f_{nat} is between $1P$ and $3P$ the structure is called soft-stiff. For this type of structures the wind action frequencies could be more dangerous than the waves' ones. The fatigue effects could still be relevant;
- if f_{nat} is greater than $3P$ the structure is called stiff-stiff; for this type of structures the fatigue effects are in general not relevant.

From the results plotted in Fig. 8(d) one can see that the structural system falls in the soft-stiff range only if the jacket support type is adopted.

For the first couple of modes, the dynamic behavior of the jacket is stiffer than that of other types, but the trend is inverted after the third mode.

5.2 Static extreme analysis

Static analysis under extreme environmental loads can be useful in order to obtain preliminary structural members dimensioning in the first design stage.

Here, steady loads have been assumed for the principal environmental actions with no functional loads (parked condition).

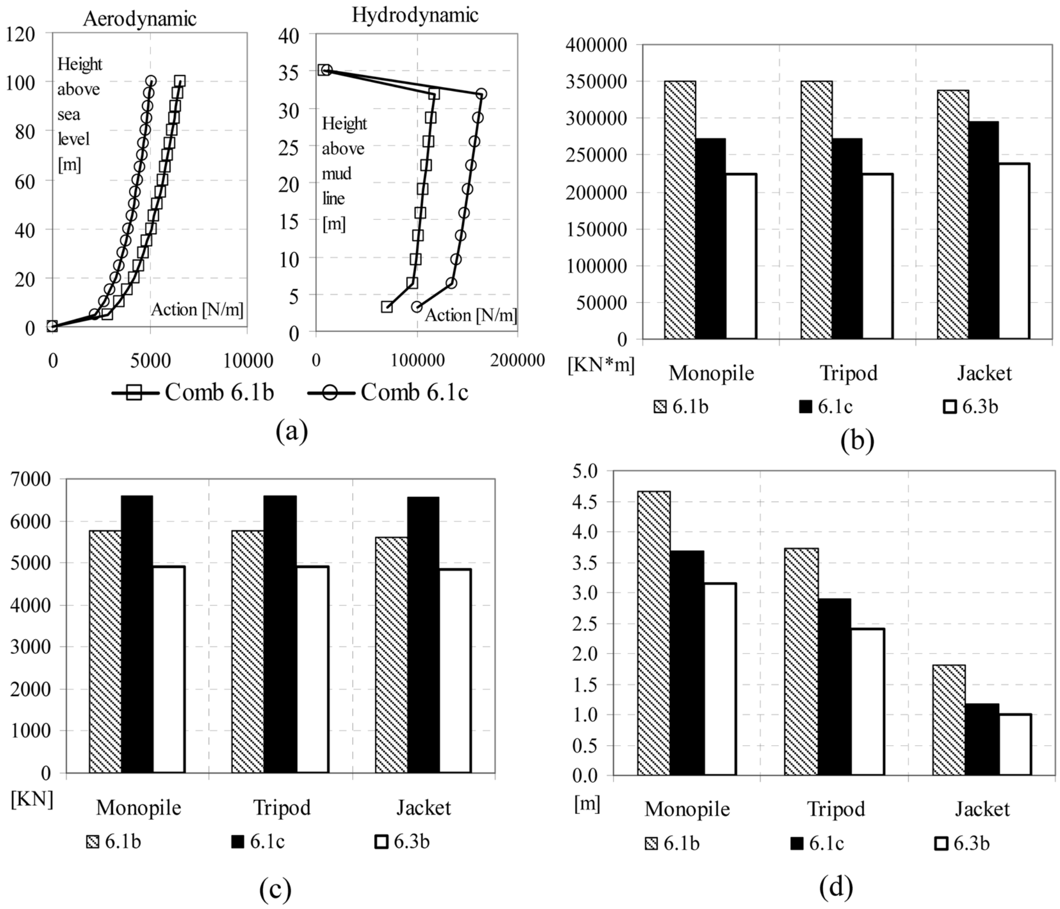


Fig. 8 (a)Actions (monopile), (b) overturning moment, (c) total shear reaction at the mud line and (d) hub displacements for the three different load cases of Table 4

Table 4 Load cases

Design Situation	Combina-tion name	Wind Condition	Marine Condition	Load Factors γ_F		
				Environmental	Gravitational	Inertial
Parked (standstill or idling)	6.1b	$V_{hub}=V_{e100}$	$H=H_{red100}$	1.35	1.1	1.25
	6.1c	$V_{hub}=V_{red100}$	$H=H_{max100}$	1.35	1.1	1.25
	6.3b	$V_{hub}=V_{e1}$	$H=H_{red100}$	1.35	1.1	1.25

The numerical analysis for the selected support structure types has been carried out considering the three load cases summarized in Table 4, where V_{hub} represents the wind velocity at the hub height; V_{eN} (with $N = 1$ or 100) represents the extreme wind velocity whit a return period T_R equal to N years and it is derived from the reference wind velocity associated with the same return period V_{refN} multiplied by a certain peak factor; V_{redN} represents the reduced wind velocity whit a return period T_R equal to N years and it is derived from the V_{refN} by applying a reduction factor. In the same table H_{maxN} and H_{redN} represent respectively the design wave maximum height and the design wave reduced height associated whit a return period T_R equal to N years.

A steady wind field has been assumed along with stationary and regular wave actions; both actions have been assumed to act in the same direction.

The design wind exerts a force distribution which is dependent on the undisturbed flow pattern: the resultant action on the rotor blades has been concentrated at the hub height while the drag forces acting on the support are distributed along the tower and the exposed piece of the substructure (jacket type only). The submerged part of the support structure is subject to combined drag and inertia forces induced by undisturbed wave and current induced flow field.

In Fig. 8(a), the calculated vertical profiles of the aerodynamic and hydrodynamic actions induced per unit length on the tower and the substructure respectively are shown for the monopile case.

The analyses carried out through macro-level models allow the evaluation of both the reactions at the mud line (shear and overturning moment) and the induced displacement at the hub height.

Results obtained with macro level models are summarized in Figs. 8(b),(c),(d): the maximum shear stress at the mud line is reached for load case 6.1c, which is the one characterized by the maximum wave height and reduced wind speed (see Table 4). On the other hand, the combination giving the maximum bending moment at the mud line corresponds to the extreme wind and reduced wave height (combination 6.1b).

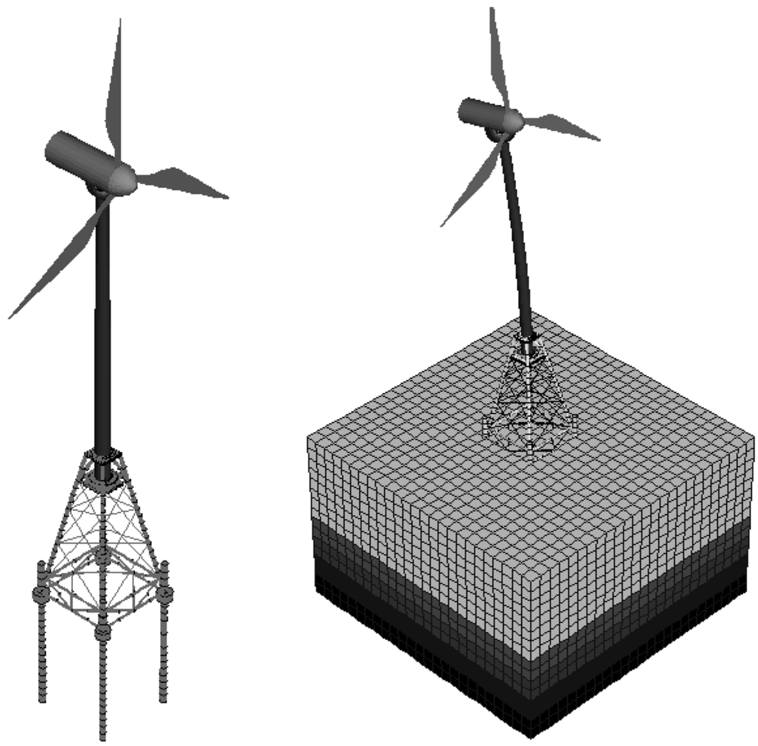
It follows that waves and currents deeply influence the resultant shear force, while the wind appears to be more critical for the overturning moment being distributed at a higher distance from the bottom.

Moreover, from the same figure, one can see that the three structural types experience approximately the same resultant shear and moment under each load combination. Concerning the horizontal displacement at the hub height, one can see an increasing stiffness of the support structure going from the monopile to the jacket type under each load combination. Maximum displacement occurs always for load case 6.1b, giving place to the higher overturning moment; for the jacket type it is almost one third of the monopile type.

In Table 5 the applied loads and the numerical results obtained for the more severe combination (6.1b) are reported. The maximum stress in the tower has been computed by combining compression (or tension) and bending stresses.

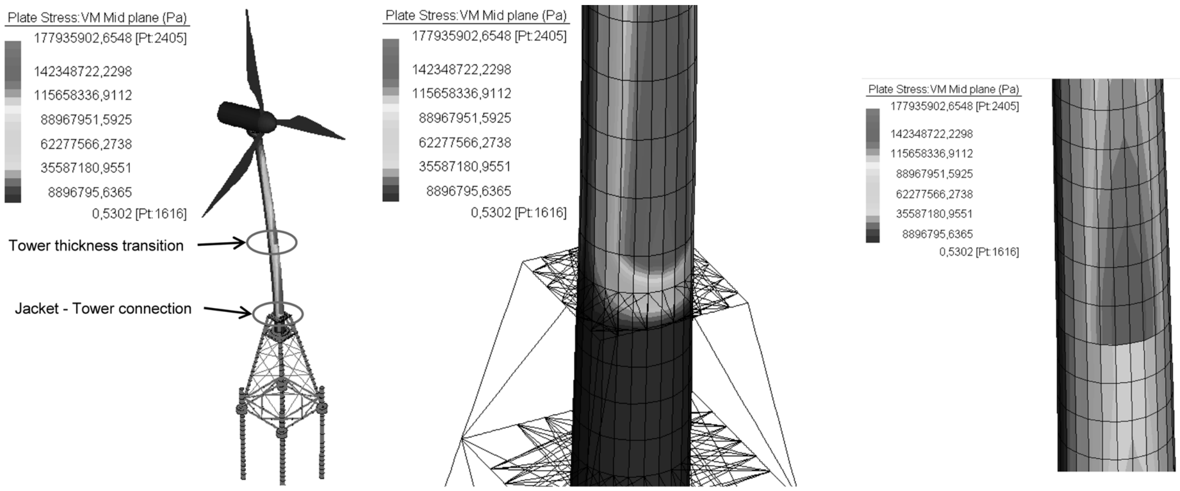
Table 5 Applied loads and the numerical results (loads combination 6.1b)

		Monopile	Tripod	Jacket
Actions	Wind on rotor (kN)	1663	1663	1663
	Wind on tower (kN)	740	740	428
	Wave and current (kN)	3372	3372	3500
	Overturning moment (kN*m)	350456	350456	337087
Reactions at mud line	Shear reaction at mud line (kN)	5775	5775	5591
	Vertical reaction at mud line (kN)	10714	10356.3 (max in pile=15018)	13768 (max in pile =9929)
Structural checks	Maximum stress in the tower (N/mm^2)	286	230	151
	Nacelle displacement (m)	4.66	3.72	1.82



(a) (b)

Fig. 9 Meso-level structural model of jacket type (a) and deformed shape under static aerodynamic and (b) hydrodynamic loads



(a) (b) (c)

Fig. 10 Elastic internal state of stress at critical zones (b) jacket-tower connection and (c) tower thickness transition

From the previous results it can be deduced that, for the present design context, the jacket support type is the best choice for what concerns the structural response (above all for the maximum stress in the tower and for the nacelle displacement) under extreme loads.

A meso-level model has been constructed for this type of support, including the ground model (five substrates with different mechanical characteristics) by using brick finite elements (Fig. 9(b)). The connection between the tower (shell elements) and the jacket has been modeled by using rigid beams elements (Fig. 10(b)). This detail level allows the designer to investigate the internal state of stress for critical parts (Fig. 10).

The meso-model is subjected to the load case referred to as 6.1b in Table 3 (most severe); the model gives a nacelle displacement equal to 2 m and a maximum stress in the tower equal to 178 MPa (jacket-tower connection). The results of the meso-level model are in good agreement with the macro-level one. The small differences are probably due to the variation in the tower diameter (ranging from 5.0 meters at the base to 3.4 meters at the top) along the vertical direction and to the thickness variation of the tubular member at a fixed transition section (see Fig. 10(a)), these two features are not detailed in macro-level models, where mean values are assumed along the tower for both tower diameter and thickness.

5.3 Buckling analysis

Another important aspect concerns the structural stability characteristics. A static incremental analysis has been conducted in order to assess the buckling load; in this case, the hydrodynamic actions have been schematically depicted using a single force acting on the jacket at the mean water level (see Fig. 11).

The analysis gives a multiple of 1.17 for the extreme load case referred to as 6.1b in Table 3. It is important to outline that the first buckling mode shows a local instability of the tower tubular section (Fig. 11) which cannot be seen in the macro models.

5.4 Dynamic analyses

Modal analysis, quasi-static analysis and buckling analysis are cornerstones in the exploration of

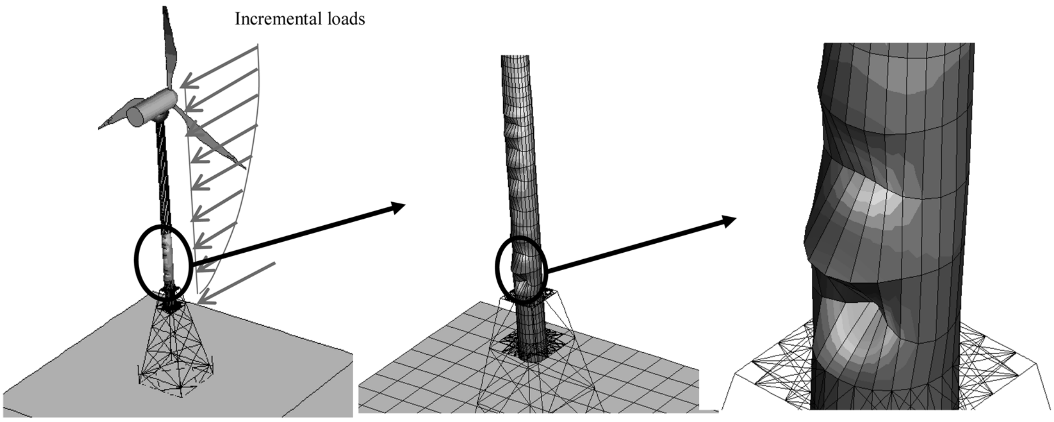


Fig. 11 Results of buckling analysis

the structural response of the proposed OWT supports. Nevertheless, dynamic analyses have to be carried out in the central design phases in order to consider some specific aspects like fatigue analysis, aeroelastic stability analysis, dynamic interactions, etc. As known, the structural dynamic analysis can be carried out both in time (Petrini *et al.* 2007) and in frequency domain (Solari 1997). The first one is widely adopted in the case of structures having a marked non-linear behavior and it requires the definition of actions time histories. If no experimental or full scale measurements are available, the actions time histories are usually generated by proper numerical algorithms (see Carassale and Solari 2006): the computational efforts may become remarkable in the case of multivariate processes and they are certainly more important if compared with the ones spent in the frequency domain approach.

On the other side, the frequency domain approach is characterized by less computational efforts but also by a lower level of detail for the results; for example, the peak response (r^p) is computed from the response variance and from the medium response by using the so called peak factor (Davenport 1998) as follows

$$r^p(h) = r_m + g_r \cdot \sigma_r(h) \tag{14}$$

where r_m is the medium response; σ_r is the response standard deviation and g_r is the peak factor. Along wind response of structures has a probability distribution which is closely Gaussian. For this case, Davenport derived the following expression for the peak factor

$$g_r = \sqrt{2\log_e(\eta \cdot T_{wind})} + \frac{0.577}{\sqrt{2\log_e(\eta \cdot T_{wind})}} \tag{15}$$

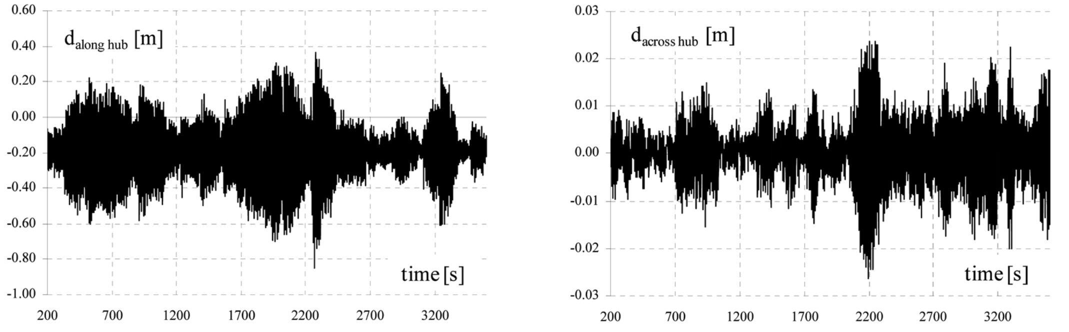
where η is the "cycling rate" of effective frequency for the response: this is often conservatively taken as the natural frequency, n_1. T_{wind} is the time interval over which the maximum value is required.

Here, both approaches have been implemented in order to investigate the dynamical response of the jacket-supported structure and to compare the results derived from the two techniques. For what concerns the dynamic actions modeling, here, the attention has been focused on the wind forces only. The hydrodynamic actions have been modeled again as static forces having the same value of the previous sections.

In order to compare the dynamic response computed by the two methods, the analyses have been conducted with reference to various mean wind velocity magnitudes: in the case of the time domain analysis, five distinct samples of the wind time histories have been generated and applied for each mean wind velocity magnitude at hub height (V_{hub}). The average value of the time domain result samples obtained for a certain wind velocity magnitude has been compared with the analogous result of the frequency domain approach. The time histories of the wind velocities have been numerically generated by adopting the so-called WAWS method, using the proper orthogonal decomposition technique for the power spectral matrix decomposition (Carassale and Solari 2006); both the along wind and the across wind actions have been applied and they have been considered uncorrelated.

The analyses have been conducted with reference to the parked (standstill or idling) configuration; in the model adopted for dynamic analyses, the foundation has not been modeled and then the jacket has been supposed fixed to the soil.

In Fig. 12 both the along and the across wind structural displacement time histories at hub height



(a) (b)

Fig. 12 Structural displacement time histories at hub height under a incident wind mean velocity of 30 m/s: (a) along-wind, (b) across-wind

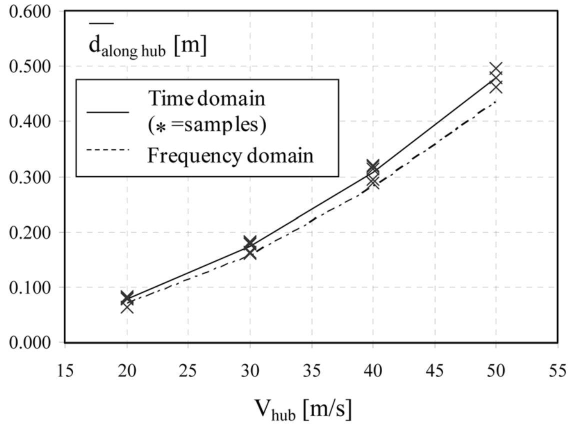


Fig. 13 Comparison between time and frequency domain analysis: mean along-wind displacements at hub height

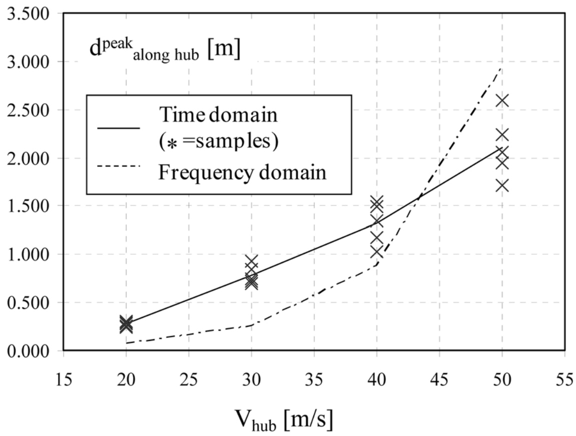


Fig. 14 Comparison between time and frequency domain analysis: peak along-wind displacements at hub height

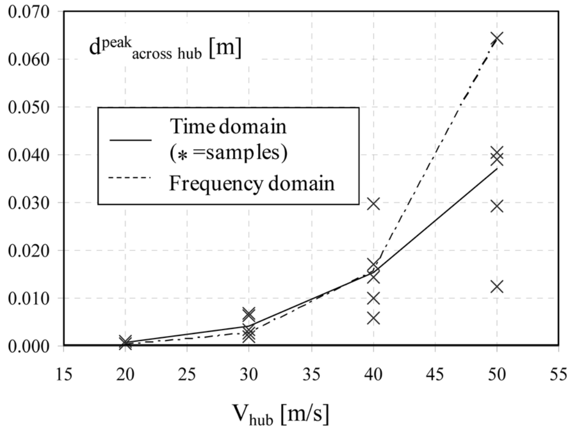


Fig. 15 Comparison between time and frequency domain analysis: peak across-wind displacements at hub height

are shown for an incident mean wind velocity at hub height equal to 30 m/s.

The results obtained by a total of 20 time domain simulations and the results from the frequency domain analyses are compared in Fig. 13 (mean along-wind displacement at hub height), in Fig. 14 (peak along-wind displacement at hub height) and in Fig. 15 (peak across-wind displacement at hub height), showing a fairly good agreement. Of course, this is a validating starting for more realistic time domain analysis, where nonlinear effects can be introduced.

6. Conclusions

In this paper, the systemic approach has been proposed as a conceptual approach for the design of offshore wind turbines: this vision can be effective in organizing the design problem, in collecting information and data and organizing models and structural analysis. The structural system decomposition has then been performed, with a focus on the structural analysis and on the performances definition.

All these considerations have been applied to a design problem which compares the safety and the performances of three different types of support structures commonly adopted for water depth lower than 50 m: monopile, tripod and jacket support structures. Extreme loads with a recurrence period of 50 years have been applied. With this application, the first steps connected with the basis of design of offshore wind turbines, as a support to the decision making process, with specific reference to the structural safety and reliability for the entire lifespan have been enlightened.

A preliminary static analysis has been carried out simulating three different load combinations as prescribed by International Standards (Det Norske Veritas 2004): the relative influence of aerodynamic and hydrodynamic loads has been assessed on resultant shear force and overturning moment at the mud line and on horizontal displacement at hub height. This step is introductory to the selection of the jacket as the appropriate support structure type.

Moreover, the internal state of stress under the above mentioned steady extreme loads has been evaluated by means of two different levels of detail for the numerical models (macro and meso-level). The analyses have confirmed that macro-level model results can predict the basic aspects of the structural response, but the meso-level model provides an additional and more detailed picture of the structural behavior due to both greater capabilities of the adopted finite elements (shell and brick instead of beam) and higher geometrical resolution of the model.

An incremental analysis has been then carried out to assess the buckling load of the examined offshore wind turbine. The analysis has shown that there is a local buckling in the tower tubular section, with a multiplier equal to 1.17, for the most severe extreme loads.

Finally, the necessary exploration of the dynamic behavior has been considered, even if with preliminary modeling, both in the time domain and in the frequency domain.

In conclusion, it seems that the proposed approach is able to manage all the aspects related with the design of complex structures like offshore wind turbines, and can effectively guides the different activities.

Acknowledgements

The Authors thank Prof. Pier Giorgio Malerba, Prof. Giuliano Augusti, Prof. Marcello Ciampoli, Dr. Konstantinos Gkoumas and Dr. Sauro Manenti for their fundamental support to this study. The anonymous Reviewers are gratefully acknowledged.

References

API (1993), *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Load and Resistance Factor Design*, (suppl. 1997), RP 2A-LRFD, American Petroleum Institute.

ASCE (2010), *12th Biennial Conference EARTH & SPACE, Symposium 7: Advanced Topics in Offshore Wind Turbine Research*, Hawaii (USA), March 14-17, 2010, http://content.asce.org/conferences/earthspace2010/index.html.

Biehl, F. and Lehmann, E. (2006), "Collisions of ships and offshore wind turbines: calculation and risk evaluation", *Proceedings of the International Conference on Offshore Mech. & Arctic Engng (OMAE)*, Hamburg.

Bontempi, F. (2006), "Basis of design and expected performances for the messina strait bridge", *Proceeding of the International Conference on Bridge Engineering BRIDGE 2006 – Challenges in the 21st Century*, Hong Kong, November.

Bontempi, F., Gkoumas, K. and Arangio, S. (2008a), "Systemic approach for the maintenance of complex structural systems", *Struct. Infrastruct. E.*, **4**, 77-94.

Bontempi, F., Li, H., Petrini, F. and Manenti, S. (2008b), "Numerical modeling for the analysis and design of offshore wind turbines", *The 4th International Conference on Advances in Structural Engineering and Mechanics (ASEM'08)*, Jeju, May.

Bontempi, F., Li, H., Petrini, F. and Gkoumas, K. (2008c), "Basis of design of offshore wind turbines by system decomposition", *Proceedings of the International Conference the 4th International Conference on Advances in Structural Engineering and Mechanics (ASEM'08)*, Jeju, May.

Brebbia, C.A. and Walker, S. (1979), *Dynamic Analysis of Offshore Structures*, Nwenes-Butterworths.

Breton, S.P. and Moe, G. (2008), "Status, plans and technologies for Offshore Wind Turbines in Europe and North America", *Renew. Energ.*, **34**(3), 646-654.

BSH (2007), *Design of Offshore Wind Turbines*, Germany, 20 December, Bundesamt für Seeschifffahrt und Hydrographie.

Carassale, L. and Solari, G. (2006), "Monte Carlo simulation of wind velocity field on complex structures", *J. Wind Eng. Ind. Aerod.*, **94**(1), 323-339.

Davenport, A.G. (1998), "Probabilistic methods in wind engineering for long span bridges", *Proceedings of the International Symposium on Advances in Bridge Aerodynamics*, Copenhagen, Denmark, May.

DNV, Det Norske Veritas (2004), *DNV-OS-J101 Offshore Standard. Design of Offshore Wind Turbine Structures*, June 2004.

Erich, H. (2006), *Wind Turbines: Fundamentals, Technologies, Application, Economics*, 2nd Edition. Springer-Verlag Berlin Heidelberg.

ESDU (2001), "Report N. 86010: Characteristic of atmospheric turbulence near the ground. Part III: variations in space and time for strong winds (neutral atmosphere)", Available at www.esdu.com [assessed 1st March 2010].

Gerdes, G., Tiedemann, A. and Zeelenberg, S. (2006), "Case study: European offshore wind farms. A Survey for the analysis of the experiences and lessons learned by developers of offshore wind farms", Report, POWER, Deutsche Windguard, Deutsche Energie Agentur, University of Groningen, Groningen. Available at www.offshore-power.net [assessed 1st March 2010].

GL-OWT, Germanischer Lloyd Wind Energie GmbH (2005), *Guideline for the Certification of Offshore Wind Turbines*, Germanischer Lloyd, Hamburg.

GL, Germanischer Lloyd Wind Energie GmbH (2002), *Richtlinie zur Erstellung von technischen Risikoanalysen für Offshore-Windparks*, Germanischer Lloyd, Hamburg.

Henderson, A.R. and Patel, M.H. (2003), "On the modeling of a floating offshore wind turbines", *Wind Energy*, **6**, 53-86.

IEC (2005), *Wind Turbine Generator Systems – Part 3: Safety Requirements for Offshore Wind Turbines*, IEC

61400-3, TC88 WG3: Committee draft, December.
Jonkman, J.M. and Buhl, Jr., M.L. (2007), "Loads analysis of a floating offshore wind turbine using fully coupled simulation", *Presented at WindPower 2007 Conference & Exhibition*, Los Angeles, California, June.
Martìnez, E., Sanz, F., Pellegrini, S., Jiménez, E. and Blanco, J. (2009), "Life cycle assessment of a multi-megawatt wind turbine", *Renew. Energ.,* **34**(3), 667-673.
NASA (2007), *Systems Engineering Handbook*, online at: http://education.ksc.nasa.gov/esmdspacegrant/Documents/NASA%20SP-2007-6105%20Rev%201%20Final%2031Dec2007.pdf [assessed 1st March 2010].
Petrini, F., Giuliano, F. and Bontempi, F. (2007), "Comparison of time domain techniques for the evaluation of the response and the stability in long span suspension bridges", *Comput. Struct.*, **85**, 1032-1048.
Petrini, F., Manenti, S., Gkoumas, K. and Bontempi, F. (2010). "Structural design and analysis of offshore wind turbines from a system point of view", *Wind Eng.*, **34**(1), 85-108.
Simon, H.A. (1998), *The Sciences of the Artificial*, The MIT Press, Cambridge.
Snel, H. (2003), "Review of Aerodynamics for wind turbines", *Wind Energy*, **6**, 203-211.
Solari, G. and Piccardo, G. (2001), "Probabilistic 3-D turbulence modeling for gust buffeting of structures", *Probabilist. Eng. Mech.*, **16**, 73-86.
Solari, G. (1997), "Wind-excited response of structures with uncertain parameters", *Probabilist. Eng. Mech.*, **12**(2), 75-87.
Sumur, B.M. and Fredsøe, J. (2002), *The Mechanics of Scour in the Marine Environment*, World Scientific Publishing Co., Singapore.
Tempel, J. van der, Zaaijer, M.B. and Subroto, H. (2004), "The effects of scour on the design of offshore wind turbines", *Proceedings of the 3rd International Conference on Marine Renewable Energy Marec*, 27-35. London, IMarest.
Tempel, J. van der. (2006), "Design of support structures for offshore wind turbines", Ph.D. Thesis, Technische Universiteit Delft, Delft, Netherlands.
Van Binh, L., Ishihara, T., Van Phuc, P. and Fujino, Y. (2008), "A peak factor for non-Gaussian response analysis of wind turbine tower", *J. Wind Eng. Ind. Aerod.*, **96**, 2217-2227.
Veldkamp, D. (2007), "A probabilistic approach to wind turbine fatigue design", *Proceedings (CD-ROM) of the European Wind Energy Conference and Exhibition 2007 (EWEC 2007)*, Milan, Italy, May.
Weinzettel, J., Reenaas, M., Solli, C. and Hertwich, E.G. (2009), "Life cycle assessment of a floating offshore wind turbine", *Renew. Energ.*, **34**(3), 742-747.
Westgate, Z.J. and DeJong, J.T. (2005), *Geotechnical Considerations for Offshore Wind Turbines Report for MTC OTC Project*, 1available online at: www.masstech.org/renewableenergy/Owec_pdfs/GeotechOffshoreFoundations-MTCOWC.pdf [assessed 1st March 2010].
Zaaijer, M.B. (2006), "Foundation modelling to assess dynamic behaviour of offshore wind turbines", *Appl. Ocean Res.*, **28**, 45-57.