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Influence of wind–waves energy transfer on the impulsive hydrodynamic loads acting on offshore wind turbines

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ABSTRACT

This paper draws some preliminary considerations about the direct wind effects on the kinematics and dynamics of steep extreme waves propagating near offshore wind turbines. Most of the hydrodynamic load models currently employed in designing offshore wind turbines take into account only indirectly the role of the wind. In fact, once the sea severity upon a certain wind speed is established, the sea state is fully determined by means of the significant wave height and the peak spectral period. In contrast, recent experimental results show that the local wind strongly influences the evolution of steep waves. A wind–waves energy transfer model is here implemented in a fully nonlinear potential flow model for wave-propagation. First, the laboratory experimental evidences about the influence of the wind on the propagation of steep waves are confirmed numerically by simulating the 2D propagation of a Gaussian wave packet. Second, real environmental conditions are reproduced in the near field of an offshore wind turbine using linear–nonlinear potential flow models within a novel coupling strategy. Different wind-wave relative speeds are simulated attempting to quantify the influence of the wind on the wave propagation. Although a reliable evaluation of this influence is still an open issue, the importance of some physical parameters, such as the duration of the energy transfer, is highlighted.

1. Introduction and background

Because of their intrinsic nature, during their lifetime, wind turbines may likely be exposed to harsh weather conditions. Extreme sea states can undermine the integrity of offshore structures so that much attention has to be paid in the loads assessment. When a strong wind blows for long fetches, wave steepening might originate breaking phenomena. In such a circumstance, classical design tools, based on the standard linear wave theory, fail their prediction. A breaking wave hitting a wind turbine structure induces an additional impulsive load that can be even three times larger than the non-impulsive contributions. For this reason the global hydrodynamic loads on the tower must account for the impulsive term originating from the wave impact phenomena in addition to the usual drag and inertial contributions (Morison et al., 1950).

A proper prediction of the impulsive loads depends both on the accuracy of the wave-propagation model used to estimate the kinematics and the height of the wave at the impact time instant, and on the reliability of the wave impact model. Recently, Marino (2010) and Marino et al. (2011) proposed an improved numerical framework for the design of the offshore wind turbines. Within a numerical weak coupling, linear and fully nonlinear potential flow models are used to predict the wave propagation in the hydrodynamic field around the structure. When a breaking wave near the offshore structure is expected, linear solution initializes a fully nonlinear potential flow model to get a reliable evaluation of the kinematics and dynamics of the breaking wave at the impact time instant. These data become the input of the analytical wave impact model proposed in Wienke and Oumeraci (2005). Although other modalities of impact can induce local impulsive loads (see Peregrine, 2003), in the present algorithm only impacts caused by plunging breaking waves are considered. Finally the novel strategy for the wave evolution against the structure is coupled with FAST (Jonkman and Buhl, 2005), an open source hydro–aeroelastic solver able to reproduce the system response in the time domain. The main advantage of the novel numerical strategy lies in the coupling algorithm developed, which permits keeping the computational cost comparable with the typical effort required by the models based on the linear wave theory.

However, Marino (2010) and Marino et al. (2011) do not consider any direct influence of the wind on the wave kinematics and dynamics. This assumption is valid as long as the fetch increases; therefore, it is not consistent with the case of wind

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turbines subjected to extreme environmental conditions as it is implicitly assumed that the sea storm is generated by the same wind the rotor is exposed to.

Kharif et al. (2008) has recently pointed out the role of the wind on the kinematics of extreme waves. Thus, in the light of these interesting results, the present paper attempts to draw some considerations about the effects of the wind–waves energy transfer mechanism on the wave field surrounding the offshore wind turbines.

The coupling strategy is shortly recalled in Section 2. Then Section 3 describes the fully nonlinear potential flow wave model, including the airflow-water free surface sheltering mechanism. Section 4 focuses on the validation of the numerical model through experimental results without wind effect. The effect of the wind is then considered through the propagation of a Gaussian wave packet for which the wave steepness at the focusing is known a priori. By generalizing the results of Section 4, 5 proposes an application of the global simulation scheme. Finally, summary and conclusions are discussed in Section 6.

2. Global coupling strategy

The coupling strategy, extensively discussed in Marino et al. (2011), is here shortly recollected to emphasize the main features implemented.

Once the site-specific environmental parameters, i.e. wind speed, significant wave height and peak period, are prescribed and once the wind and waves spectra are chosen, a linear random wave model is used to reproduce the evolution of the wave elevation $\eta(x=x_t,t)$ in the space–time domain $\mathcal{D}(t,p)=[x_{min},x_{max}]\times[0,T_{sim}]$, where T_{sim} is the overall simulation time, while $x_{\min}$ and $x_{\max}$ are the spatial lower and upper bounds of the domain,respectively. The zero-crossing analysis of $\eta(x=x_t,t)$ at the wind turbine location $x_t\in[x_{min},x_{max}]$ allows determining the plunging breaker occurrence in accordance with the criterion $(H/L)_{break.}=0.142\tanh(kd)$(Sarpkaya and Isaacson, 1981). Here k indicates the local wave number and d the water depth.

When the criterion is not satisfied, hydrodynamic loads on the structure are estimated through the standard Morison's equation (Morison et al., 1950). In contrast, when breaking waves are expected, fully nonlinear potential flow model, suitably initialized with the linear solution, reproduces the waves propagation in the near field sub-domain Ω_i defined as follows:

$$\mathcal{D}(t,p)\supset\Omega_i(t,p)=[t_{b_i}-\delta t_b,t_{b_i}+\delta t_b]\times[x_t-\delta x_t,x_t+\delta x_t]$$

To avoid numerical instabilities due to a sudden transition from the linear to nonlinear solution a spatial ramp function is applied in a very short space range, shorter than 10 times the size of a boundary element. Because the free surface is discretized with approximately 40 boundary elements per mean wavelength, the spatial ramp influences the solution in a range smaller than 0.25 times the mean wavelength.

The kinematical features of the wave at the impact time instant, that is the impact velocity of the mass of water and the maximum wave elevation at the monopile, are the input data for the analytical impact model (Wienke and Oumeraci, 2005). In this way the impulsive loads are estimated and added to the drag and inertial contributions provided by Morison's equation.

In the following, the main features of the fully nonlinear potential flow and local impact models will be shortly described. On the contrary, concerning the algorithms that are not the innovative key points of the present work (i.e. the open source solver FAST (Jonkman and Buhl, 2005) and the linear potential flow theory (e.g. Dean and Dalrymple, 1988)) the reader can refer to the extensive literature available.

3. Fully nonlinear wave propagation problem: mathematical formulation

3.1. Governing equations

Under the assumptions of inviscid fluid and irrotational flow, potential flow model allows the description of fully nonlinear water wave propagation in a domain $\Omega(t)$. At a fixed time instant t, the velocity of a fluid particle $p\in\Omega(t)$ is $\overline{v}(p,t)=\nabla\phi(p,t)$, $\phi(p,t)$ being the velocity potential function. Continuity equation leads to Laplace's equation

$$\nabla^2\phi(p,t)=0 \quad \forall p\in\Omega(t) \tag{1}$$

In 2D conditions, the fluid domain $\Omega(t)$ is bounded by four boundaries: the inflow wall $\Gamma_i(t)$, the bottom $\Gamma_b(t)$, the outflow wall $\Gamma_o(t)$ and, finally, the free surface $\Gamma_f(t)$

On rigid boundaries, moving (i.e. wavemaker) or fixed (i.e. ends wall and bottom), Neumann boundary conditions

$$v^n(p,t)=\nabla\phi(p,t)\overline{n} \quad \forall p\in\Gamma(t) \tag{2}$$

are enforced. Here $\overline{n}$ is the outward unit normal vector and $v^n(p,t)$ is the normal velocity prescribed through the linear spectral approach.

On the free surface, i.e. a geometric surface deforming in space and time, dynamic and kinematic boundary conditions

$$\frac{D\phi(p,t)}{Dt}=-\frac{p_{fs}}{\rho_w}-g\eta+\frac{1}{2}\nabla\phi(p,t)\nabla\phi(p,t) \qquad \forall p\in\Gamma_f(t) \tag{3}$$

$$\frac{D\overline{r}(p,t)}{Dt}=\overline{v}(p,t)=\nabla\phi(p,t) \qquad \forall p\in\Gamma_f(t) \tag{4}$$

are imposed; p and t denote the water particle and the time, respectively, p_{fs}denotesthe pressure on the free surface, ρ_w the water density, η the fee surface elevation and g the gravity. Finally, $\overline{r}$ indicates the Lagrangian position vector of the particle on the free surface.

The initial-boundary-value problem defined in Eqs. (1)–(4) is reformulated within a boundary integral equation, and numerically solved with a Mixed Eulerian–Lagrangian formulation. To that purpose, a second order Taylor series expansion is used for the time integration (Peregrine and Dold, 1986) and a higher order Boundary Element Method (BEM) for the space discretization (Brebbia and Dominguez, 1998).

3.2. Jeffreys' sheltering mechanism for the wind wave energy transfer

Free surface schematization imposes a pressure p_{fs} constant (see Eq. (3)) and equal to the atmospheric pressure. Here, to take into account the effect of the wind forcing, a pressure patch is imposed on the free surface in accordance with the Jeffreys' formulation. In the original formulation, Jeffreys (1925) assumed flow separation on the lee side of the wave crest. This implies an asymmetry of the pressure and then a drag that causes growth and persistence of steep waves. A similar physical mechanism was recently observed experimentally in Kharif et al. (2008) and Chambarel et al. (2010). As a consequence, the pressure on the free surface is related to the surface slope and to the relative wind speed through the following equation:

$$p_{fs}=p_a+\rho_a s(U_{wind}-U_{wave})^2\frac{\partial\eta}{\partial x} \tag{5}$$

with p_a being the atmospheric pressure without wind, ρ_a the air density, $s=0.5$ the sheltering coefficient estimated in Kharif et al. (2008), U_{wind} the wind velocity and U_{wave} the wave velocity.

Touboul et al. (2006) proposed a threshold value $(\partial\eta/\partial x)_{min}$ for the free surface slope above which airflow separation occurs and