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Prediction of long-term extreme load effects due to wave and wind actions for cable-supported bridges with floating pylons

Yuwang Xu[a,*], Ole Øiseth[a], Torgeir Moan[b], Arvid Naess[c]

[a] Department of Structural Engineering, Norwegian University of Science and Technology, Richard Birkelands vei 1, Trondheim, Norway
[b] Centre for Autonomous Marine Operations and Systems, Centre for Ships and Ocean Structures, Norwegian University of Science and Technology, O. Nielsens v 10, Trondheim, Norway
[c] Department of Mathematical Sciences, Norwegian University of Science and Technology, Alfred Getz vei 1, Trondheim, Norway

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ABSTRACT

The characteristic values of the extreme environmental load effects should correspond to a specified annual probability of exceedance. These load effects can be calculated using short-term or long-term methods. The full long-term method is considered the most accurate approach, but it requires tremendous computational effort for complicated structures, especially when nonlinearities must be considered. In a case study of the dynamic behavior of a three-span suspension bridge with two floating pylons, these nonlinearities are found to have a significant effect on the extreme values of some of the load effects. It is thus recommended to determine these responses in the time domain. However, time-domain simulations can be very time consuming even by using simplified approaches such as the environmental contour method (ECM) and the inverse first-order reliability method (IFORM). Therefore, this paper introduces a computationally efficient approach utilizing the ECM and the IFORM to determine long-term extreme values based on responses from combined frequency- and time-domain simulations.

1. Introduction

During the design of offshore structures, it is necessary to estimate the characteristic values of extreme load effects corresponding to specified annual exceedance probabilities. These load effects are calculated using short-term or long-term methods. Short-term approaches are used to analyze load effects during storms with N-year return periods with specified durations, e.g., three hours for offshore structures subject to waves and normally one hour for structures experiencing combined wind and wave actions; meanwhile, long-term approaches consider all storms that occur in the long-term period.

In principle, the full long-term methods (FLM) represents the most accurate approaches for determining the characteristic values of extreme load effects on a structure for ultimate limit state (ULS) and accidental limit state (ALS) design checks. In Norwegian rules and regulations [1], the ULS and ALS values normally correspond to annual exceedance probabilities of 10^{-2} and 10^{-4}, respectively, for offshore structures. The FLM essentially integrates short-term response statistics (i.e., distributions of all peaks, distributions of extreme values or mean upcrossing rate) over all short-term environmental conditions [2]. It incorporates both the long-term variability of environmental conditions represented by a joint probability distribution of environmental parameters and the variability of short-term extreme values characterized by the conditional distribution of short-term responses with regard to the environmental conditions.

However, the FLM clearly does not represent the most economical approach from a computational perspective because they must account for contributions from all possible short-term states [3]. Determination of the annual probability of exceedance given a response is analogous to determining probability of failure (if failure is defined as exceeding a given response). Hence, structural reliability methods (e.g. FORM) [4–6] can be used to determine the distribution of long term extreme response values. If the annual probability of exceedance or return period is given, the inverse method (e.g. IFORM) [7–9] needs to be used.

The environmental contour method, which is a simplification of IFORM, decouples the uncertainty in the environmental conditions and the short-term extreme values and the latter is disregarded [10–13]. Fundamentally, the ECM calculates the contour line corresponding to a selected return period. It is further assumed that the most important combination of environmental parameters along the contour line can be used to approximate the long-term extreme value. Neglecting the short-term variability in the extreme values can give non-conservative results. Thus, a higher percentile than the expected maximum is used as the

* Corresponding author.
E-mail address: yuwang.xu@ntnu.no (Y. Xu).

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short-term characteristic value rather than selecting the median extreme response [10]. Another alternative is to introduce a correction factor that is typically between 1.1 and 1.3 to make the prediction conservative [13–15]. The ECM has been frequently applied in ocean engineering endeavors to search for the appropriate short-term design case. This method makes it possible to estimate the long-term extreme response without conducting a full long-term analysis, which is especially beneficial for complex structures.

To an extent, a simplified FLM can guarantee both accurate and computationally efficient results because not all of the conditions contribute to the long-term extreme value distribution [13]. It is therefore necessary to assess whether the environmental conditions yield significant contributions; if not, they could be disregarded. By determining an appropriate range for the environmental parameters, e.g., wind velocities, wave heights and peak periods, significant reduction of computational times can be achieved.

Most of the research so far is focusing on wave induced load effects for offshore structures, while there exist some studies on combined wind and wave load. This paper addresses a very complex structural response problem, i.e., a three-span suspension bridge with two floating pylons subjected to combined wind and wave loading. Three environmental parameters are considered, namely, the mean wind velocity, the significant wave height and the peak wave period. Due to their computational efficiency, frequency domain methods are normally the first choice for obtaining the structural response required for long-term extreme value analyses. The accuracy of the simplified FLM, ECM and IFORM is validated through a comparison with the results applying the FLM. The results show that the simplified methods provide adequate results and can thus be used for predicting the wind- and wave-induced extreme load effects in this new bridge concept.

The time domain simulations presented demonstrate that nonlinearities constitute a difference of approximately 20% in the extreme values of the bending moment due to vertical deformation at the most important position along the girder. This means that frequency domain approaches may underestimate the long-term extreme response. However, time domain simulations can be very time consuming, even by applying the ECM or IFORM. Therefore, a computationally efficient approach is proposed to predict the long-term extreme response values based on the combined frequency- and time-domain simulation results and the use of IFORM and ECM. The idea of using IFORM arises from the observation in the case study that the search for the design point converges quickly and most of the iterations are located in a small area near the design point. Thus the domain of environmental parameters can be divided into a frequency-domain region and time-domain region. Time-domain simulations are utilized only as the iteration is performed in the time-domain region. The time-domain region constitutes only a small percentage, which is the key to avoid tremendous computational time.

2. Dynamic response of a cable-supported bridge with floating pylons

Fig. 1 shows a three-span suspension bridge with two floating pylons traversing Bjørnafjorden in Norway. The main cables are supported by two fixed pylons at each end of the bridge and two floating pylons in the middle. Similar to a tension leg platform, the bottom part of each floating pylon is moored by four groups of tethers that provide large stiffness coefficients for the heave, pitch and roll. The water depths at the left and right floating pylons are 550 m and 450 m, respectively. The dynamic behavior of the bridge can be simulated through both time- and frequency-domain approaches [16,17].

Fig. 1. Three-span suspension bridge with two floating pylons. Illustrated by Arne Jørgen Myhre, Statens vegvesen.

2.1. Multi-mode frequency-domain approach

Since multi-mode approaches can consider aerodynamic coupling effects among the modes, they demonstrate better performance in predicting the buffeting responses of bridges relative to conventional mode-by-mode approaches [18,19]. A cable-supported bridge with floating pylons experiences both wind and wave action, and the associated equation of motion can be written in the frequency domain as follows:

$$(\widetilde{\mathbf{M}}_s + \widetilde{\mathbf{M}}_h(\omega))\mathbf{G}_{\ddot{\eta}}(\omega) + (\widetilde{\mathbf{C}}_s + \widetilde{\mathbf{C}}_h(\omega) - \widetilde{\mathbf{C}}_{ae}(V,\omega))\mathbf{G}_{\dot{\eta}}(\omega) + (\widetilde{\mathbf{K}}_s + \widetilde{\mathbf{K}}_h - \widetilde{\mathbf{K}}_{ae}(V,\omega))\mathbf{G}_{\eta}(\omega) = \mathbf{G}_{\mathbf{Q}_{Buff}}(\omega) + \mathbf{G}_{\mathbf{Q}_{wave}}(\omega) \tag{1}$$

Here, $\mathbf{G}_\eta$ is the Fourier transform of the displacement response; $\widetilde{\mathbf{M}}_s$, $\widetilde{\mathbf{C}}_s$ and $\widetilde{\mathbf{K}}_s$ are the generalized mass, damping and stiffness matrices, respectively; $\widetilde{\mathbf{C}}_{ae}$ and $\widetilde{\mathbf{K}}_{ae}$ denote the generalized aerodynamic damping and stiffness matrices; $\widetilde{\mathbf{M}}_h$ and $\widetilde{\mathbf{C}}_h$ are the generalized hydrodynamic mass and damping matrices; $\widetilde{\mathbf{K}}_h$ is the hydrostatic restoring fore; $\mathbf{G}_{\mathbf{Q}_{Buff}}$ is the Fourier transform of the wind force on the girder; and $\mathbf{G}_{\mathbf{Q}_{wave}}$ is the Fourier transform of the first-order wave force on the pylons. The second-order wave forces are not considered in the paper since they are of minor importance for section forces in the cable-supported bridge with floating pylons [17].

The frequency domain approach mainly includes two steps: (1) the modal analysis of the structure, and (2) the modeling of the aerodynamic and hydrodynamic actions using generalized coordinates.

2.1.1. Structural modal analysis

The modal analysis is performed following a static analysis, wherein time-invariant mean wind forces are imposed upon the bridge. In addition, the added mass when the frequency goes to infinity and the hydrostatic restoring stiffness are added into the structural mass and stiffness matrices, respectively, since these effects will substantially alter the natural modes and frequencies. Consequently, fewer modes are required, and some computational time can be saved.

The deformation along the girder, pylons and pontoons for each natural mode must be applied for the calculations of the generalized wind and wave actions.

$$\begin{aligned}
&\Phi(\mathbf{x}) = [\boldsymbol{\varphi}_1 \cdots \boldsymbol{\varphi}_i \cdots \boldsymbol{\varphi}_{N_{\mathrm{mod}}}];\\
&\boldsymbol{\varphi}_i = [\varphi_y \quad \varphi_z \quad \varphi_{\theta x}]^T, \quad \textit{for} \quad \textit{girder}\\
&\boldsymbol{\varphi}_i = [\varphi_x \quad \varphi_y \quad \varphi_z \quad \varphi_{\theta x} \quad \varphi_{\theta y} \quad \varphi_{\theta z}]^T, \textit{for} \quad \textit{pylons}
\end{aligned} \tag{2}$$

where φ_n, $n \in \{x, y, z, \theta_x, \theta_y, \theta_z\}$ represents three translations and three rotations of the girder and pylons for each mode. The positive directions of the displacements along the girder, pylons, and pontoons are shown in Figs. 2 and 4. Not all the displacements are necessary for the girder since only the drag force, lift force and torsional moment along the girder are considered in this case study.

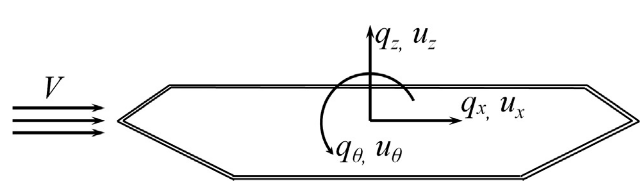

Fig. 2. Aerodynamic forces acting on a bridge deck cross-section.