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Modelling the stochastic dynamic behaviour of a pontoon bridge: A case study

Knut Andreas Kvåle, Ragnar Sigbjörnsson, Ole Øiseth *

Department of Structural Engineering, Norwegian University of Science and Technology, Trondheim, Norway

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ABSTRACT

Herein, a study on the hydrodynamic modelling of pontoon bridges is presented, with the Bergsøysund Bridge as a representative example. The model relies on the finite element method and linearized potential theory. The primary emphasis is placed on the stochastic response analysis within the framework of the power spectral density method. The quadratic eigenvalue problem is solved using a state-space representation and an iterative algorithm. The contribution of the fluid–structure interaction to the overall modal damping is investigated. Response effects due to changes in the sea state are studied. A frequency-independent approximation of the hydrodynamic coefficients is presented and discussed.

1. Introduction

Although the history of floating bridges may be traced back as far as 2000 BC [1], only in recent decades have floating bridges begun to be developed to a sufficient degree of sophistication such that they can be applied as critical components of modern infrastructure. Compared with land-based bridges, including cable-stayed bridges, only limited information on floating bridges is currently available, particularly regarding construction records, environmental conditions, durability, operations and performance. This is clear from the fact that only approximately twenty long floating bridges currently exist throughout the world. The major trends in the development of floating bridges and other very large floating structures (commonly abbreviated VLFSs) have been presented by Wang et al. [2] and Wang and Wang [3].

The state-of-the-art design philosophy for floating bridges in 1997 was outlined by Moe [4]. It was remarked that standard engineering practices were not directly applicable to floating bridges. A verified design code for floating bridge design would drastically reduce the effort required during the planning stage and would thus increase the potential economic advantages of floating bridges over many alternative bridge concepts. From a broader perspective, a unifying, efficient, and reliable method for simulating the behaviour of floating bridges is the primary goal.

The Norwegian Public Roads Administration (NPRA) is currently investigating possible technological solutions for a ferry-free Coastal Highway Route E39 along the western coast of Norway. This route stretches 1100 km between the cities of Kristiansand and Trondheim and requires multiple crossings of deep and wide fjords. The ferry-free crossings of these deep fjords represent considerable engineering challenges that are difficult or impossible to solve using existing bridge technology; pontoon-type floating bridges have been proposed as feasible options.

Of all existing floating bridges, only a few rely on discretely distributed pontoons, whereas the remainder are based on continuous pontoon girders. The majority of these bridges are also provided with additional stiffness through side-mooring. Only two long-span end-supported floating bridges exist in the world: the Bergsøysund Bridge and the Nordhordland Bridge, both relying on discretely distributed pontoons and both located in Norway. In connection with the planning of these structures, interest in the stochastic dynamic behaviour of floating bridges flourished in certain research communities, who combined the knowledge from the highly developed Norwegian offshore industry with knowledge gained during the construction of the floating bridges found in the State of Washington, USA, and in British Columbia, Canada. Much of this pioneering work can be credited to the research groups of Holand and Hartz (see, e.g., [5–11]) and Borgman [12]. The methodology was further developed, elaborated and exemplified by Sigbjörnsson [13] and by Langen and Sigbjörnsson [14].

Since the remarkable efforts contributed to the methodology in the 70s and early 80s, few case studies have been performed on real floating bridges. The effects of the flexibility of the superstructure of a pontoon bridge were studied by Kumamoto and Maruyama [15], who emphasized the relevance of such a study in regard to the design of the Yumeshima–Maishima (Yumemai) Bridge in Osaka, Japan, around the year 2000. This unique bridge is described in [16] and is the successor to the previous massive

* Corresponding author.

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research project concerning VLFSs in Japan: the Mega-Float. Seif and Inoue [17] performed a conceptual case study of the Bergsøysund Bridge, in which the response of the bridge was simulated in the time domain for various wave directions and spreading indices for a specified crest length.

Morris et al. [18] performed a frequency-domain analysis of the planned William R. Bennett Floating Bridge in British Columbia. Among other relevant contributions of more recent vintage are [19,20].

Floating bridges play a modest role in modern infrastructure, partly because of the limited knowledge of the uncertainties that arise with increased spans. The longest existing floating bridges are moored to the seabed and rely on continuous pontoon solutions. However, individual pontoons are beneficial in many cases, and for deep straits such as fjords, it is not practically feasible to incorporate anchoring. From this kind of design follows a greater importance of the correlation of the wave action field.

An intermediate study concerning the stochastic modelling of the dynamic behaviour of the Bergsøysund Bridge was performed by Kvåle et al. [21]. The cited paper presents a similar study of the Bergsøysund Bridge; however, the current paper is far more elaborate and extensive, with respect to both the model and the interpretation of the analyses. The current paper presents a two-part combined model of the Bergsøysund Bridge, in which the fluid–structure interaction is considered using linear potential theory and the superstructure is represented by a finite element (FE) model consisting of beams and shells. The presented model serves as a basis for evaluating and discussing the damping contribution from the fluid–structure interaction. The effects of changes in the sea state, as represented by the crest length and the significant wave height, are studied in terms of both the wave excitation and the global response of the bridge. Because of the discretely distributed pontoons used in the bridge design, the wave excitation acts at only a few well-separated points. Thus, the correlation of the wave action on the bridge is an important issue and a vital aspect of this paper. With time-domain analyses in mind, the memory effect in the contribution from the fluid–structure interaction is avoided by applying two different frequency-independent approximations, and the resulting errors are discussed.

2. Outline of the theoretical model

A floating bridge is a complex structure, requiring theories from multiple scientific fields for the establishment of a complete numerical model. This section serves to outline the theoretical and mathematical framework needed for such a model. The frequency-domain equations of motion are established in Section 2.1. To solve these equations of motion with regard to the response, the power spectral density method is introduced in Section 2.2. The load acting on the structure is established through a random, Gaussian representation of the sea surface, which is established in Section 2.3 in the form of spectral densities. Furthermore, the load spectral densities are computed based on the sea surface spectral densities, as discussed in Section 2.4. To obtain a useful interpretation of the global system, a modal study is beneficial. Because of the self-exciting nature of a floating bridge, particular attention must be paid to the eigenvalue solution, as shown in Section 2.5.

2.1. Equations of motion

Within the framework of a finite element method (FEM) formulation, the equations of motion for a floating structure can be written as follows (see, e.g., Naess and Moan [22]):

$$[M_s]\{\ddot{u}(t)\} + [C_s]\{\dot{u}(t)\} + [K_s]\{u(t)\} = \{p_h(t)\} \tag{1}$$

where t is the time; $[M_s]$, $[C_s]$ and $[K_s]$ are the structural mass, damping and stiffness matrices, respectively; $\{u(t)\}$ is the displacement vector; and $\{p_h(t)\}$ is the total hydrodynamic action, including both the fluid–structure interaction and the wave action. The floating elements contribute via forces from the interaction between the water and the structure. The total hydrodynamic action can be formally expressed as follows:

$$\{p_h(t)\} = \int_{-\infty}^{\infty}[m_h(t-\tau)]\{\ddot{u}(t)\}d\tau + \int_{-\infty}^{\infty}[c_h(t-\tau)]\{\dot{u}(t)\}d\tau + [K_h]\{u(t)\} + \{p(t)\} \tag{2}$$

Here, $[m_h(t)]$ and $[c_h(t)]$ are the time-domain representations of the added hydrodynamic mass and the added hydrodynamic damping, respectively; and $\{p(t)\}$ represents the wave excitation forces. The first three terms on the right-hand side are models of the fluid–structure interaction forces. The time-domain representation of the added mass, $[m_h(t)]$, is related to the frequency-dependent hydrodynamic mass, $[M_h(\omega)]$, as follows:

$$[m_h(t)] = \frac{1}{2\pi}\int_{-\infty}^{\infty}[M_h(\omega)]e^{i\omega t}d\omega \tag{3}$$

Similarly, for the hydrodynamic damping, the following holds:

$$[c_h(t)] = \frac{1}{2\pi}\int_{-\infty}^{\infty}[C_h(\omega)]e^{i\omega t}d\omega \tag{4}$$

The restoring forces, however, are assumed to be independent of frequency. This implies that the frequency-domain and time-domain representations are identical. Here, the angular frequency is denoted by ω, and $i \equiv \sqrt{-1}$.

The wave excitation force, $\{p(t)\}$, is modelled herein as a homogeneous, stochastic, Gaussian process. The literature supports the validity of this assumption for the case of deep water and moderate wave heights (see, e.g., [23]). It follows that the response process inherits the properties of Gaussianity and homogeneity. It is assumed that the displacement and force processes can be expressed using generalized harmonic decomposition [24] as follows:

$$\{u(t)\} = \int_{-\infty}^{\infty} e^{i\omega t}\{dZ_u(\omega)\} \tag{5}$$

$$\{p(t)\} = \int_{-\infty}^{\infty} e^{i\omega t}\{dZ_p(\omega)\} \tag{6}$$

where $\{Z_u(\omega)\}$ and $\{Z_p(\omega)\}$ are the spectral processes corresponding to the response vector and the wave excitation force vector, respectively. The equations of motion can now be re-written in the frequency domain as follows:

$$\left(-\omega^2[M(\omega)] + i\omega[C(\omega)] + [K]\right)\{dZ_u(\omega)\} = \{dZ_p(\omega)\} \tag{7}$$

The fluid–structure interaction gives rise to inertia, damping and restoring forces. Hence, the system mass, damping and restoration (stiffness) are expressed as follows:

$$[M(\omega)] = [M_s] + [M_h(\omega)] \tag{8}$$

$$[C(\omega)] = [C_s] + [C_h(\omega)] \tag{9}$$

$$[K(\omega)] = [K_s] + [K_h] \tag{10}$$

By applying linearized potential theory, numerical values can be established for the wave excitation process, the hydrodynamic restoration matrix, the frequency-dependent added damping matrix and the frequency-dependent added mass matrix. This will be further discussed in Section 2.4. For further details regarding the establishment of the equations of motion, the reader is referred to [22].