Coupled wind and wave load analyses of multi-span suspension bridge supported by floating foundations

Mitja Papinutti

Wind OnDemand, Graz, Austria

Ketil Aas-Jakobsen, Lars Halvor Kaasa

Aas-Jakobsen, Oslo, Norway

Arne Bruer, Mathias Huuse Marley

TDA-COWI, Oslo, Norway

Johannes Veie, Simen Hellgren Holtberget

Norwegian Public Road Administration, Oslo, Norway

Contact: mitja.papinutti@gmail.com

Abstract

The TLP suspension bridge concept is a novel design proposed for crossing the wide and deep fjords along the E39 highway on the west coast of Norway. The bridge concept consists of a multi-span suspension bridge supported by one or more tension leg platforms. An accurate and representative load formulation for wind and wave excitation is essential for design of these structures. This article presents time domain analysis of a TLP suspension bridge subject to wind and wave loading. Analyses are performed for both coupled and separate wind and wave loading, in order to investigate possible coupling effects between the two environmental loads. A fully coupled nonlinear Newmark time integration scheme is used, where structural geometric nonlinearities, frequency-dependent hydrodynamic radiation properties and wind interaction are included. New developed tools are suitable for parallel calculation of the step time integration. Presented study shows a search strategy on coupled effect for different structural components.

Keywords: Multi-span suspension bridge; time domain analysis; coupled wind and wave analyses; wind interaction; hydrodynamics; parallel calculation.

1 Introduction

The Ferry free E39 project is an ongoing project lead by the Norwegian Public Roads Administration (NPRA), whose aim is to create a fixed link between Kristiansand and Trondheim, along the west coast of Norway. Several fjords need to be crossed along the route, and the fjord topography introduces some new challenges to bridge engineering.

One such fjord is the Bjørnafjord, just south of Bergen. The fjord is about 5 kilometres wide, and is one half kilometre deep. Several concepts have been evaluated for this crossing, including a multi-span suspension bridge on floating foundations. The concept is currently being developed as a cooperation between the NPRA and a group of consultants companies; Aas-Jakobsen, COWI, Johs. Holt, Moss Maritime, Wind OnDemand, Aker Solutions, NGI and Plan Arkitekter.

A multi-span suspension bridge itself is a challenging project, and when combined the Bjørnafjord topography, the challenge becomes even greater. However, Norway has over 60 years of offshore oil industry development, where several types of floating platforms have been developed. Among these platforms concepts are the Spar platform, Semi Sub platform, and the Tension Legg Platform (TLP). As a suspension bridge foundation, the TLP has proven the most promising due to the limited pitch and roll motion.

The concept for the Bjørnafjord crossing consists of a three-span bridge with two rock founded towers on each side of the fjord, and two floating pylons in the fjord. The floating pylons are founded on TLP-platforms, at a depth of 550 and 450 meters. These TLP-floaters introduce some new aspects to the design of a multi-span suspension bridge, and introduce additional load scenarios needed to be considered. Normally the design loads would depend on wind, traffic and road traffic accidents, but in the case of Bjørnafjorden we also need to cope for wave, current and ship impact loading. To take these loads into account a coupled wind and wave analysis procedure has been developed [1].

Time domain tools are used as the most accurate analysis tools available, providing important information on nonlinear and coupled effects [2]. Project requirement is analysis of multiple load scenarios, where a good overview of the loads and load combination necessary. Dynamic project environment is requiring a good data management strategy and automatized time domain calculation. Long calculation time require a use of multicore machines and severs. For this project a Visual Basic (VBA) excel user interface was developed and used on the presented study case of coupled analysis.

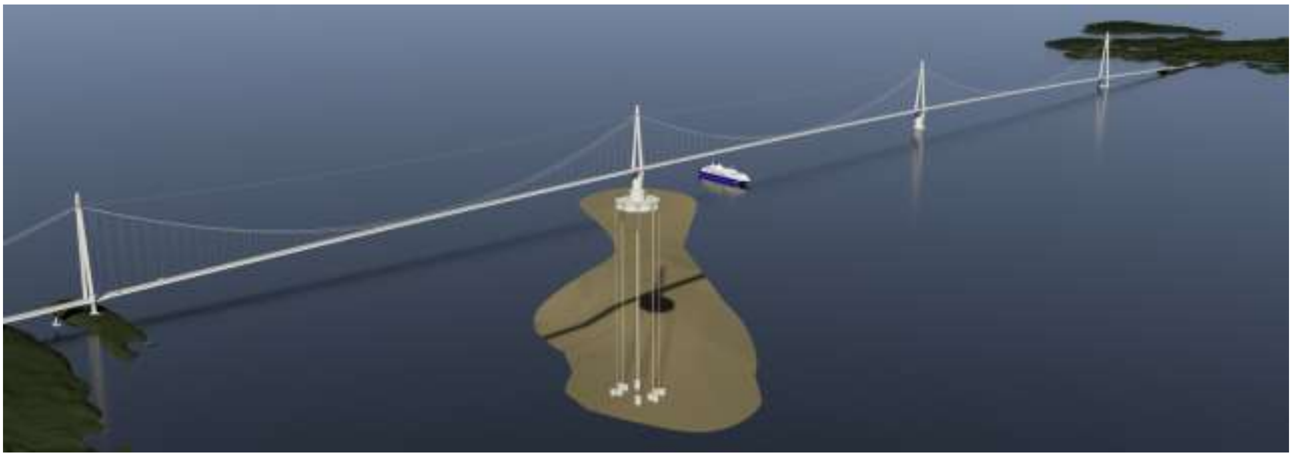

Figure 1. Rendering example of the multi-span suspension bridge on floating foundations crossing the Bjørnafjord.

2 Project analysis setup

2.1 Naming system

For the Bjørnafjorden project the TLP bridge is subject to multiple environmental loads. In first step, these loads are analysed separately, to yield extreme response for the bridge. The extreme combinations are then combined in environmental events, used in fully coupled time domain analysis. In one event, different loads are introduced with different properties. For better project overview a special naming system was developed for naming of the loads. This provides a good overview and immediate information on what kind of the loading was applied, when looking at the results.

Each load has assigned letter, describe the environmental load type, such as: A is wave load, B is swell load, C is wind load and D is sea current load. After the letter, six-digit number follows to designate basic load properties. First digit (1) stands for describing agreed return period of the load. Next three digits (2,3,4) are determining the horizontal angle of load attack. The last two digits (5,6) are reserved for event number describing different load properties. For example, a wind environmental load can be defined as:

C409001

This name represents a wind load with 4 = 100 years return period, 090 = 90-degree angle of attack, with the wind turbulent properties defined in the 01 wind profile number defined in the analysis software. Uniquely defined loads are then combined in events, for example:

E001: A231501 B209001 C418001 D000000

In this event (E001), loads are combined with wave and swell loads from east, together with the turbulent wind load from the south. There are no sea currents included in above presented event.

This uniquely defined combinations are practical for organizing and reviewing of multiple calculated events. A special VBA Excel program has been developed, tailored to this naming system and used for easy overview and parallel execution of time integrations of multiple realizations of a large number of load combinations.

2.2 Automated VBA tool

A VBA Excel program has been developed for analysis of multiple environmental scenarios [3] in RM Bridge Advanced [4]. This provides a good overview of analysed load events, and makes it simple to set up, execute and post-process large number of coupled analyses. The user interface is based on Excel sheet input. In the first sheet, “Screening_Overview”, all relevant loads are defined with their parameters. All parameters are named with uniquely defined loads, described above. The “Analyses_Run” sheet is combining load into events, based in the previous defined names. This combination can be marked and calculated in the background. The engineer selects wanted combinations, provide input parameters as duration and number of realizations of each event, and an automatized procedure is starting all calculations and performing post processing of the results. The calculation work is distributed among the available number of CPUs by always running as many analyses in parallel as there are CPUs, and immediately starting new analyses when other are finished. This approach is very suitable for servers, with big CPU power capacity.

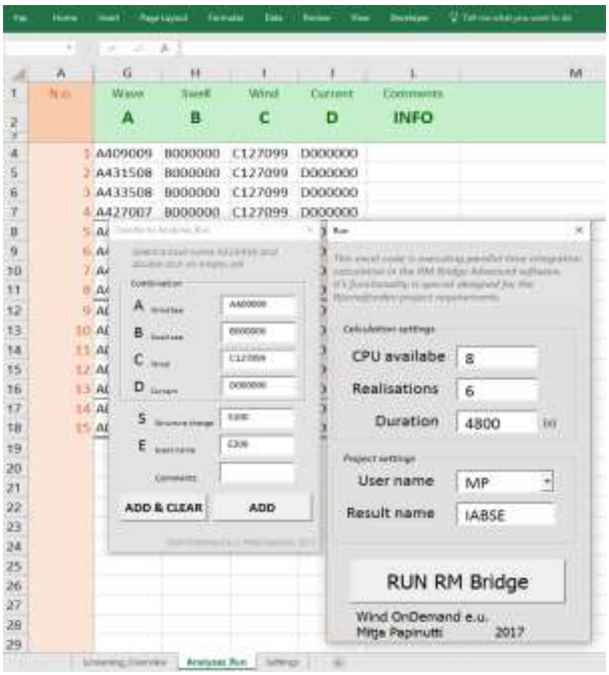

Figure 2: Analysis interface for analysis for overview and execution of time integrations.

2.3 Post processing of the results

The results are generated from multiple time integrations, containing extreme envelopes or time dependent curves. These results are then collected in the result folder, where some automatized post-processing is performed. Extreme values of all realizations are averaged for each event, and this values are further superposed to obtain the design values. Manual and more detailed post-processing of the response signal is required for extreme value estimation, fatigue analysis or studies of the coupling effects, as presented later in this paper.

The following scheme has been used for calculation of the results, depicted in Figure 3.

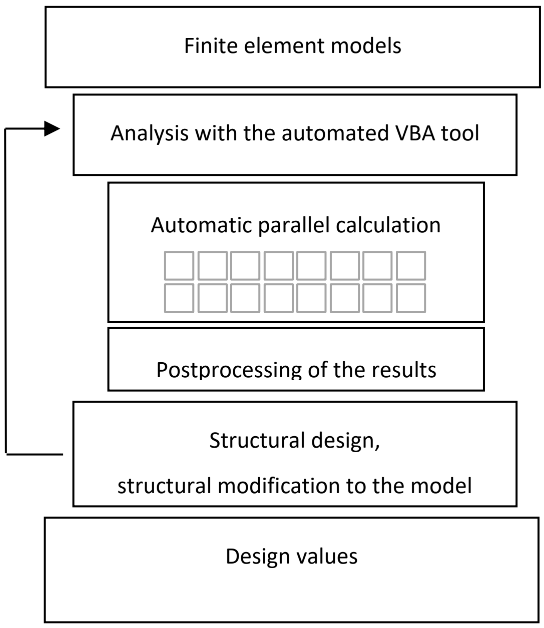

Figure 3. Project workflow and parallel calculation strategy for coupled time integration analysis.

Both manual and automatized post-processing are used for extraction of the design values. The provided tool is successfully handling the large generated data of the time domain simulation. This is used during the project, where CPU processor power needs to be shared between different machines, working groups and available licences.

3 Coupled response investigation

An example of ongoing investigation shows the use of time domain tools [1,2] together with VBA interface, developed by Wind OnDemand [3]. This study case is focused on preliminary study of coupling effects of different loads. The focus is for given finite element analysis model performing the investigation of coupling of the loads. This provides an important information about the possible effect that are required to be considered.

3.1 Structural model

A structural model of the bridge, depicted in Figure 4, is developed in RM Bridge. Line finite elements are used for the pylons and bridge deck, where cable finite elements are used to model the main cables, hangers and tethers. A high-tensioned top cable system is suspended between each span and anchored in the spread chamber of the anchor foundation. Top cable reduces the Pylon top displacement from unfavorable traffic position. The connection between the deck and the floating is laterally restrained and has free longitudinal bearings. The bridge deck ends have restrained lateral motion and have free longitudinal motion in SLS condition. Additional 15MN end stoppers are activated for exceeded SLS motion, however not used for this investigation.

The submerged parts of the floaters are modelled as rigid bodies connected to the seabed by massless cable elements representing the tendons. The hydrodynamic properties are included in hydrodynamic points, defined at each pylon in one node at the sea level.

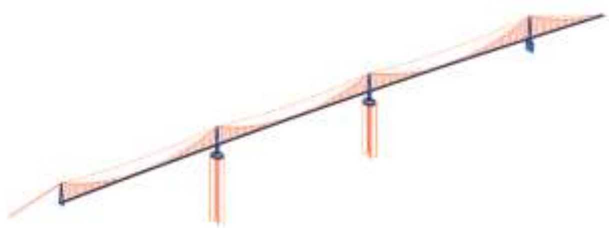

Figure 4. Overview of RM Bridge Analysis model with the 4-legged steel floater concept.

3.2 Environmental loads

3.2.1 Wind loads

Wind buffeting load application is done with assumption of Quasi-Steady State (QSS) theory, where a fully developed wind flow around the deck is producing an aerodynamic force [5]. Non-frequency dependent load model is function of the wind angle of attack β. Total wind load vector R is combined from mean wind V_m, time dependent turbulences v,u and structural movements u_z, u_y. Full load vector is applied to the structure and is presented in Figure 5:

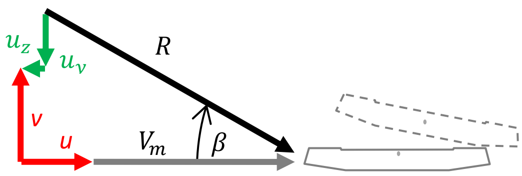

Figure 5. Wind buffeting load vector.

The aerodynamic cross section properties are described with the drag coefficients C_D, lift coefficients C_L and moment coefficients C_M. The fluctuating wind velocities are described with Power Spectrum Densities (PSD) of wind fluctuations and the cross spectra coherence. The Kaimal PSD with parameters of length scales Lu=245 m, Lv=28 m, Lw=85 m and exponential factor epsilon=0.3 are describing the turbulence over the height of the bridge. The turbulence intensity is defined as TI_u=0.14, TI_v=0.07 and TI_w=0.105 with constant values over the bridge height. The exponential wind profile is approximated with a constant one, to simulate the homogeneous wind field properties. Investigation shows that a good approximation can be found for the matching deck level wind speed V_m=37 m/s. Mean wind speed is calculated for 100 year return period and shown in Figure 6.

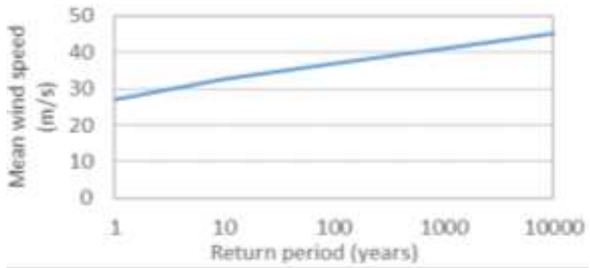

Figure 6. Lateral mean wind speed at deck level.

3.2.2 Wave loads

Wave excitation is modelled by 6 DOF load time histories per pylon, developed by external software [6]. Two separate wave time series A433508 and B431513 were calculated. The A433508 is local wind-generated waves with a significant wave height (H_s) of 2.5m and peak period T_p of 6.2s with a mean wave direction of 25deg from the longitudinal bridge direction. The swell waves B431513 are defined parameters with H_s = 0.4m and Tp=14s with a mean direction of 45deg to the bridge longitudinal axis.

Both sea states are representative of the 100-year condition in Bjørnafjorden. For this investigation, both swell load and wave load were simulated. Input spectra and their time domain realisations, of local wind sea A433508, are shown in Figure 7.

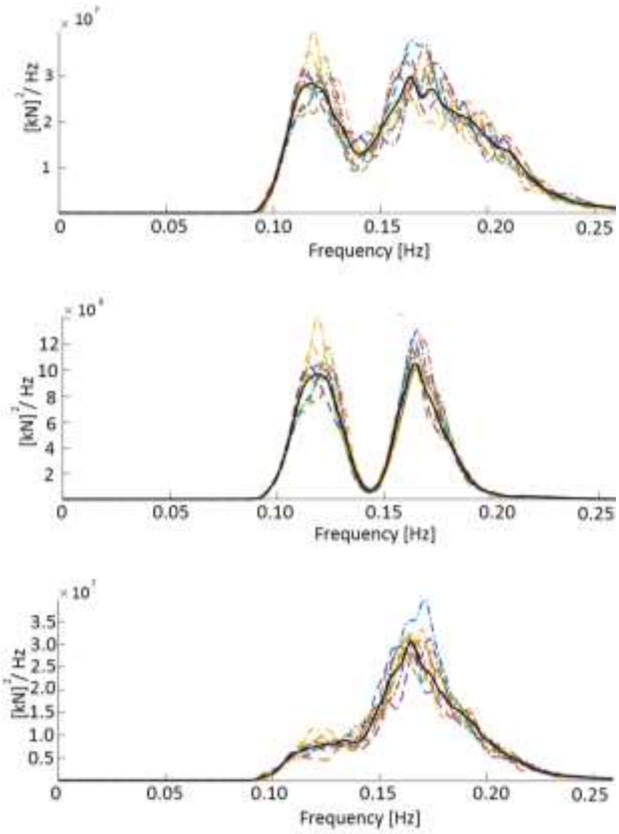

Figure 7. Generated load spectra for wind sea wave load on each hull. Top: Fx (along bridge), middle: Fy (vertical), bottom: Fz (across bridge).

4 Bridge response

4.1 General

To analyse coupling effects on the bridge response, analyses of wind sea response, swell response and wind response have been conducted both separately and combined in one analysis. This has revealed one significant coupling effect, which is the effect of wind induced aerodynamic damping on wave response. Besides that, no coupling effects significant for bridge design have been identified in the presented analyses.

4.2 Applied load events

Only results for a limited set of environmental loads are presented herein. Four load events have been included: wind sea waves only, swell waves only, lateral wind only and all of them applied together in a coupled analysis. Current loads have not been included. This is summarized in Table 1.

Table 1: Applied load events

Event	Wind Sea Waves	Swell Waves	Wind	Current
E001	A433508	B000000	C127099	D000000
E002	A000000	B431513	C127099	D000000
E003	A000000	B000000	C427021	D000000
E005	A433508	B431513	C427021	D000000

4.3 Effect of wind on wave response

Bridge response due to wave loading (wave response) is more high frequent and narrow banded than wind loading. Also, swell wave loading has harmonic characteristics by nature, and may produce highly resonant response if the wave period matches a natural period of the bridge. Both makes the swell response sensitive to damping.

Vertical wave response of bridge deck is dependent of aerodynamic damping introduced by lateral wind, see Figure 8. The vertical movement of the bridge girder changes the effective angle of attack of the wind, which increases the lift force. This lift effectively dampens the movement of the girder.

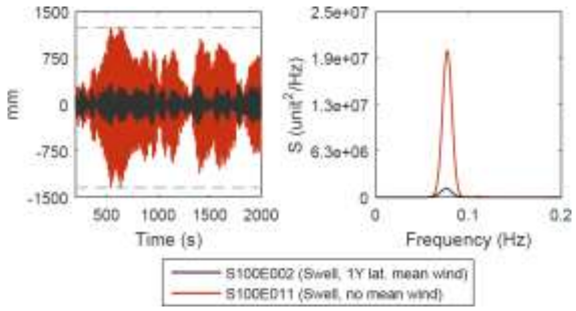

Figure 8: Effect of mean wind on vertical displacement at middle of middle main span.

4.4 Search for other coupling effects

To search for other coupling effects besides the aerodynamic damping introduced by vertical girder motion, a mean wind with 1 year return period has been applied to the separate wave analyses (coupled analyses include wind loading with 100 year return period). Coupling effects can now be found by comparing the sum of wave and wind response with coupled analysis response. Plots of bending moments at bottom of floating pylons, tether forces, top cable forces and bridge girder movement are depicted with colours in Figure 9.

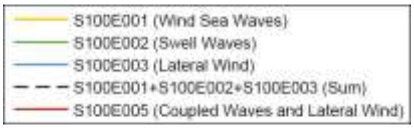

Figure 9: Legend of response plots

4.4.1 Bending moments at bottom of south floating pylon

Bending moments stems mainly from wind loading, however Figure 10 shows that swell also feeds energy into modes with natural periods near the swell period (14 s, 0.71 Hz). Wind sea waves trigger a mode at 1.2 Hz for moments about lateral axis. No coupling is observed Figure 10 and Figure 11.

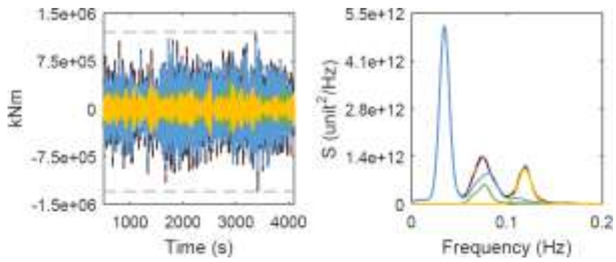

Figure 10: Bending moment about bridge lateral axis at bottom of south floating pylon

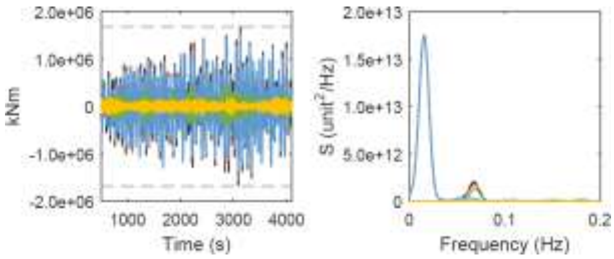

Figure 11: Bending moment about bridge longitudinal axis at bottom of south floating pylon

4.4.2 Tether forces

Tethers forces are also dominated by wind induced response. The most significant modes from moment response in Figure 10 and Figure 11, are directly correlated to the tether normal forces identified in Figure 12. An additional two minor wind excited frequencies are observed for frequencies 0.28Hz and 0.39 Hz.

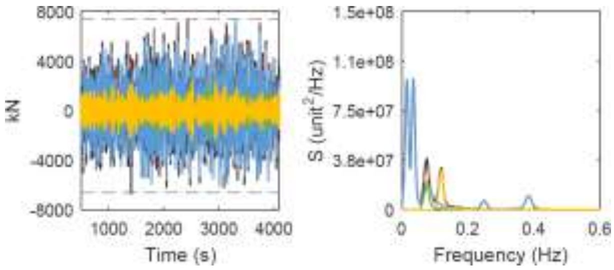

Figure 12: Dynamic part of axial force in one tether connected to south floating pylon

4.4.3 Bridge girder motion at middle of middle span

Girder motion at middle of the bridge is completely dominated by wind induced response, depicted in Figure 13, Figure 14 and Figure 15. Wind feed energy into a broad banded frequency range, while wave loading is more narrow banded. Hence wind drives the low frequency/large amplitude modes that produces large girder displacements. The principal modes are the same that were identified when looking at pylon bending moments above.

Swell loads contribute to a mode where the two floating pylons are given a pitch motion so that pylon tops are moving towards and apart from each other. However, this parametric excitation of the bridge girder is limited due to the aerodynamic damping.

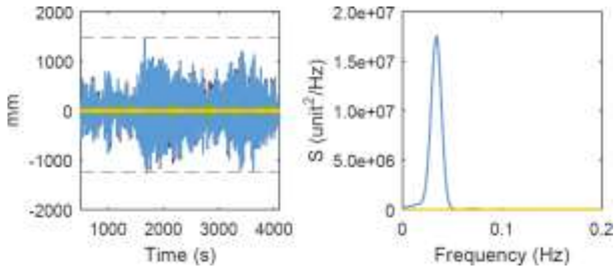

Figure 13: Longitudinal displacement of bridge girder at middle of middle span

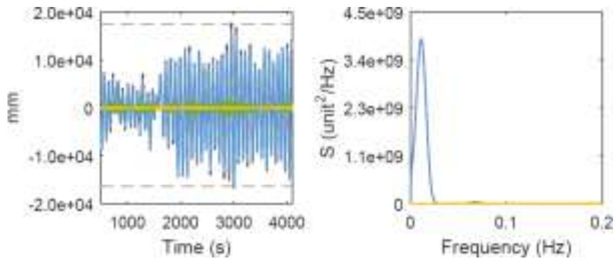

Figure 14: Lateral displacement of bridge girder at middle of middle span

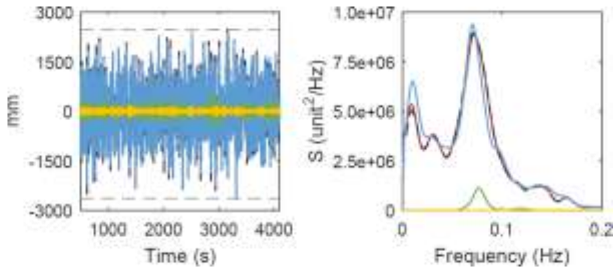

Figure 15: Vertical displacement at middle of middle span

4.4.4 Top cable response

The vertical motion of the top cable is driven by both wind induced response of the natural modes of the cable and by parametric excitation due to change of relative distance between pylon tops, as described for the bridge girder vertical motion above. The Figure 16 and Figure 17 are showing that the frequency content of relative motion between pylon tops is present in the vertical displacement of the top cable together with the first natural mode of the cable (at 0.13 Hz).

Minor coupling effects seems to be present in the power spectrum for relative distance between the pylon tops. This is most likely due to slightly lower applied mean wind in separate wave analyses (1 year return period) and in coupled analysis (100 year return period).

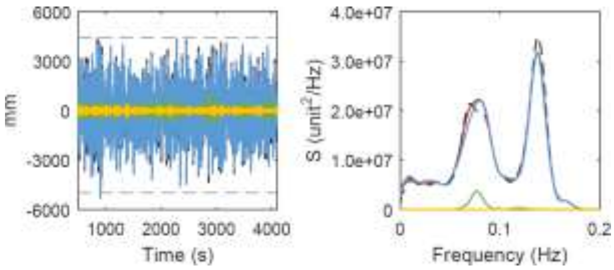

Figure 16: Vertical displacement of top cable at middle of middle span

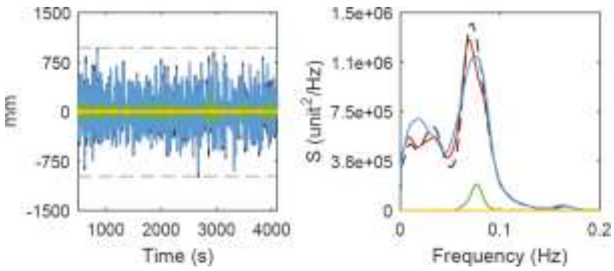

Figure 17: Relative distance between pylon tops of the two floating pylons

5 Conclusions

Present article shows the use of time domain tool to analyse of the coupling responses. Presented investigations are important milestone to understand the complicated structural response. This study provides us with information used for design of reliable and are long-lasting structures.

Only one significant coupling effect has been observed for the applied set of environmental loads, which is the effect of aerodynamic damping on wave response. Wave response induce vertical motion of the bridge girder, effectively altering the wind angle of attack on the girder, which in turn produce a significant change of lift force acting on the girder. This lift force resists the vertical motion and can hence be interpreted as a damping force. Since the damping is introduced by vertical girder movement, this coupling effect can be observed on all response that comprise modes involving this type of motion.

6 References

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