

Field Observation of Global and Local Dynamics of a Cable-Stayed Bridge



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Abstract This work examines dynamic characteristics of a cable-stayed bridge, utilizing acceleration data from the bridge deck as well as the cables. A weather station installed on the bridge provides additional data for the interpretation of the observed wind and traffic induced vibrations. The measurement data are acquired on the Stavanger City Bridge. The bridge layout, combining steel and concrete girder segments supported by a tower and stay cables, makes it an interesting case for system identification analysis especially as all three parts of the bridge structured are monitored to some extent. Especially as cable vibration sensors are rarely included in long-term wind and structural health monitoring of the cable-stayed bridges. The data analysis investigates the characteristics of the suspended steel box and the supporting concrete girder, synchronization in response between the different structural components and the interaction between deck vibrations and cable vibrations. The paper explores the performance of system identification techniques for estimation of damping, which is challenging for damping levels as low as 0.05%, such as in the case of the stay cables. It is found that there is a strong interaction between the cables and the deck structure. Detailed identification of the cable properties showed a clear dependency between the total cable damping and the wind velocity, revealing the contribution of aerodynamic damping.

Keywords Cable-stayed bridge · Vibrations · Acceleration · System identification · Damping

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1 Introduction

Large vibration amplitudes of the stay cables of the Stavanger City Bridge, in Norway, associated with the combined effects of wind and rain have been reported on different occasions [1]. Previous work [2, 3] has also established that the structural damping of the cable vibrations is very low, with damping ratios of the order of 10^{-4} . It is of interest to explore further the simultaneous bridge girder and stay cables dynamic response, with special attention to methods suitable for identification of low damping levels of stay cables. In the following, the key information about the studied cable-stayed bridge and the monitoring system is presented, followed by the analysis of the selected bridge girder and cable vibration data.

1.1 The Study Case

The central part of the Stavanger City bridge is a cable-stayed bridge with a suspended main span of 185 m supported by a 75 m-high A-shaped tower (see Fig. 1). The main span is a steel box girder of 15.5 m width and 2.4 m depth, whereas the rest of bridge consists of pre-stressed concrete side-spans supported by concrete columns [1].

Three sets of stay-cables at each side connect the pylon with the bridge deck. The length of the stays ranges from 61 to 141 m with constant diameter of 79 mm. The stay cables are of the locked-coil ropes type.

Two of the stay cable-sets on each side are comprised of four individual cables having a center-to-center distance of 320 mm and 480 mm to 580 mm with rigid connections between individual cables at two or three locations along the cable length, as depicted in Fig. 1b. The third and shortest cable stays are in a pair, with rigid connections placed in two locations along the stay span. Passive rubber dampers

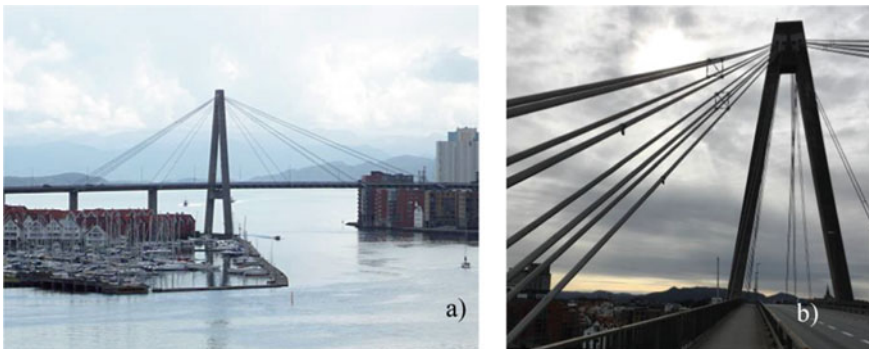


Fig. 1 The Stavanger City Bridge. **a** A side view from west. **b** A view towards south, showing the cable setup, the accelerometers on one side and the anemometer pole

at the ends of the stay cables are installed to reduce vibration effects on the anchor heads.

The following sections provide an overview of the bridge instrumentation, the acceleration data collected, and analysis of a relevant vibration event.

1.2 Instrumentation

The cable monitoring system is comprised of four wireless battery-powered tri-axial accelerometers (G-Link200-8G from Microstrain), which are installed on stay cables referred to as C1 and C2 (on each side of the deck) about 4 m above deck level, see Fig. 2a, b. The sampling frequency of the sensors is 64 Hz. The accelerations components of each sensor are normal to the cable axis along the in-plane (vertical) and out-of-plane (lateral) direction.

Two tri-axial accelerometers (CUSP-3D from Canterbury Seismic Instruments) are located inside the bridge deck on each side of the pylon. One is mounted on the east wall of the concrete deck north of the tower, 20.8 m from the anchor beam of the stay cables C1 and C2 whereas the other one is in the steel box girder 35 m south of the tower. The sampling frequency of the sensors is set to 50 Hz. The deck accelerometers north and south of the tower are denoted DN and DS, respectively, see Fig. 2.

A weather transmitter (WXT530 from Vaisala) is mounted on a 3.5 m-high pole between the anchors of cable C1-C2 and C3 on the east side of the bridge deck (Fig. 2). The instrument measures the horizontal wind components, the absolute temperature, relative humidity, pressure, and rain intensity with a sampling frequency up to 4 Hz.

Data from the accelerometers (DN & DS) inside the bridge deck and the weather station are gathered and synchronized using a single data acquisition unit whereas the wireless accelerometers installed on the cables transmit data to a dedicated logging unit. Synchronization between all signals is then carried out in the pre-processing phase, based on individual time stamps.

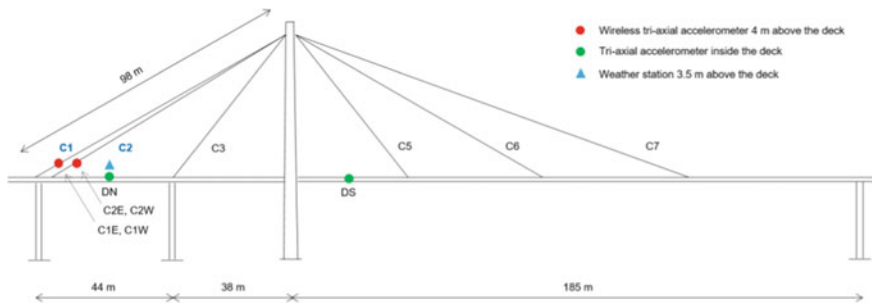


Fig. 2 Instrumentation of the Stavanger City Bridge

2 Data Analysis

The following data analyses are based on vibration data gathered over 24 h on 28.08.2019. The focus is on the dynamic properties of the bridge, including the natural frequencies, and damping of the cables.

Figure 3 shows the mean wind velocity and direction relative to the bridge axis over the 24-h period (UTC) on August 28, 2019. The velocity was in the range of 3 m/s to 9 m/s, and the wind direction varied from Southeast during the first 12 h toward South-southwest in the afternoon. It should be noted that the bridge axis lies about -18° from North, towards Northwest. It rained in the afternoon around 15:30 and again around 16:30, but as the mean wind velocity was below 5 m/s and the wind direction was along the bridge axis at the time, large wind-rain induced vibrations did not occur.

Figure 4 (left) shows the vibration time-series recorded by the tri-axial accelerometer in the steel deck at location (DS). As can be seen the acceleration amplitude generally follows the magnitude of the mean wind velocity. Traffic-induced vibrations of short duration are also noticeable as regular spikes on top of the wind-induced response. It is also clear that the vertical vibration is about three times the transversal vibration and close to 7 times higher than the axial vibrations in the deck. The accelerations in the concrete deck on the other side of the tower (DN), follow the same amplitude pattern, but the magnitude is about half of the acceleration levels at (DS) in the steel deck.

Figure 4 (right) shows the cable accelerations recorded during the 24-h period at cable location C1E. As can be seen the sway and heave accelerations are a magnitude larger than the axial vibrations. It is also clear that the sway and heave acceleration levels follow the wind velocity variations closely.

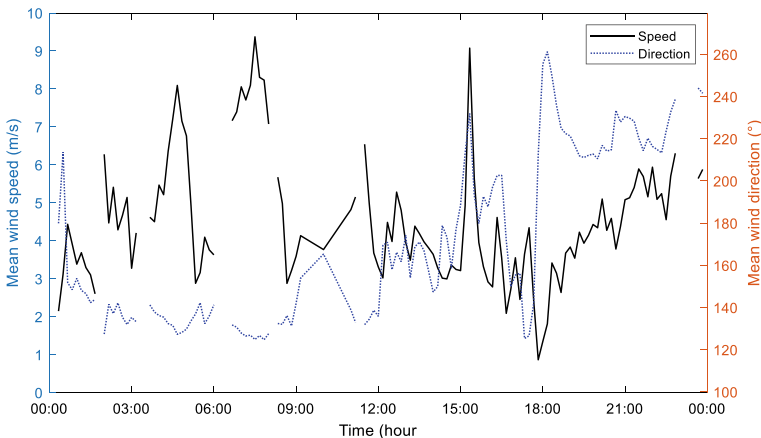


Fig. 3. 10-min mean wind velocity and wind direction relative to the bridge axis on 28.02.2019

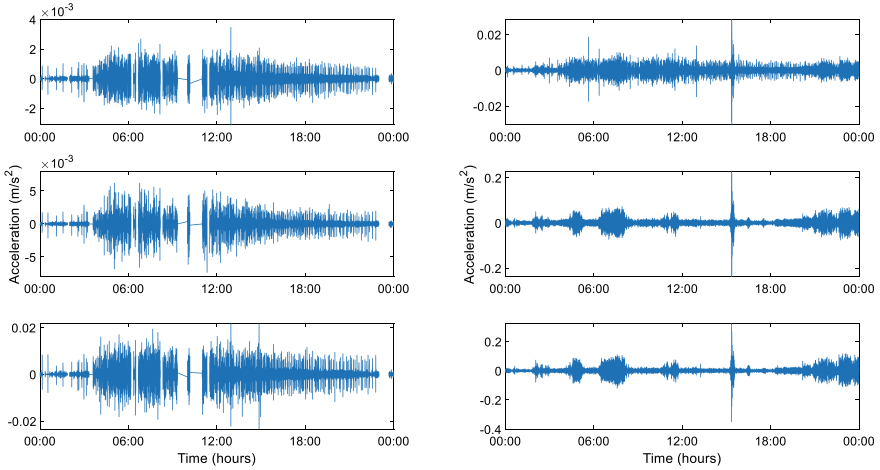


Fig. 4 Accelerations recorded on 28.8.2019 in the steel deck at DS (left) and on cable C1E (right). Axial component (top), sway component (middle) and heave component (bottom)

2.1 Spectral Analysis

The recorded signals were investigated in terms of their spectral properties, as illustrated in Fig. 5. The variance normalized spectra of three acceleration components in the steel and the concrete deck are shown, along with the associated co-coherence between the corresponding vibration components. The axial vibrations in both parts of the deck show a similar spectral pattern and have a relatively high coherence over the whole frequency range. Negative co-coherence is observed for the across-deck (y) accelerations below 5 Hz, whereas the co-coherence is largely positive at the lower frequencies for the along-deck (x) and the vertical (z) accelerations. This is partially linked to the influence from the cable vibrations, especially for the vertical acceleration. It is particularly the y-acceleration that clearly demonstrates some deck-modes that cannot be directly linked to the cable vibrations.

2.2 System Identification Using the SSI-COV Method

The modal properties of the cables and bridge deck were derived from each sensor system, i.e. the three channels, using an automated covariance-driven stochastic subspace identification algorithm (SSI-COV) toolbox from [4], which was inspired by [5].

A summary of the results for the cable vibrations is shown in Fig. 6 along with a typical stabilization diagram in Fig. 8. It was found that most of the spectral peaks gave several frequency values, as can be seen in Fig. 7, which affected the evaluated

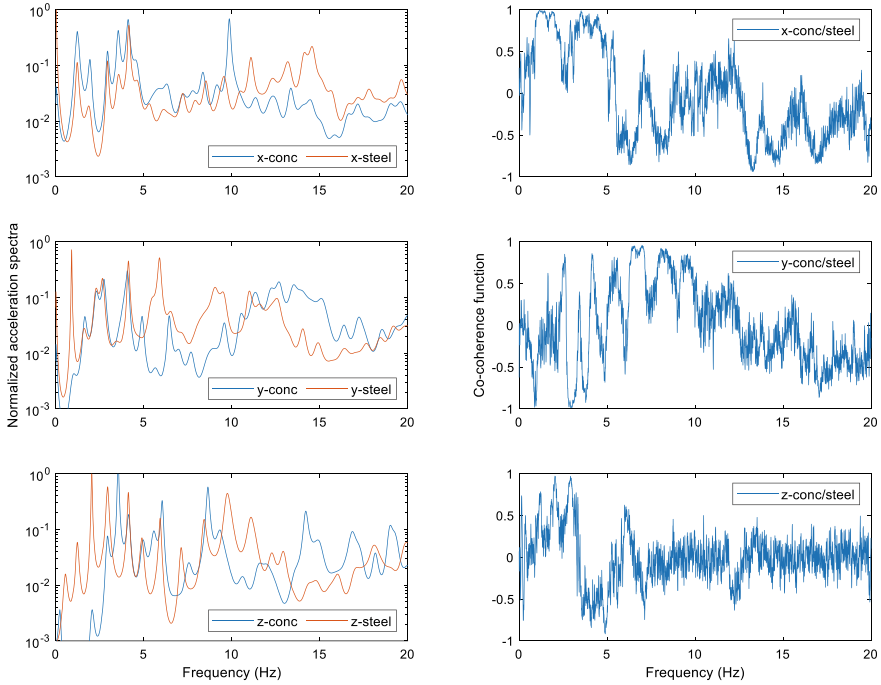


Fig. 5 Variance normalized spectra and coherence of accelerations in the concrete (DN) and steel (DS) parts of the bridge deck, based on 24-h of data from 28.08.2019

damping. The damping varies more when evaluated for all three components of motion than for a single component, see Sect. 2.3. However, it is seen that the damping is generally of the order of 10^{-3} and below 0.005, at least for the first three and last three of the eight cable modes reported. The variability in the damping estimation is found to be greatest for modes 4 and 5, which are likely more influenced by the deck structure than the other modes.

Although not apparent from Fig. 6, it was found that sensor C2W, gives the lowest damping on average and a slightly higher natural frequencies than the other sensors, indicating that the C2W cable may have somewhat higher tension than the other cables. Similarly, C2W showed the highest damping on average.

System identification results from the deck vibrations are shown in Fig. 8, for frequencies up to 21 Hz. As can be seen, the eigen-frequencies fall in many instances close to the cable frequencies, shown by dashed lines. This indicates the close coupling between the deck and the cables, particularly for the lower cable modes.

The analysis shows that the bridge deck is partly excited by the cable vibrations. Therefore, the vibrations recorded in the deck are to some extent forced vibrations, rather than ambient vibrations. An output only system identification method is therefore somewhat inadequate, when analyzing the deck vibrations, as it is difficult to

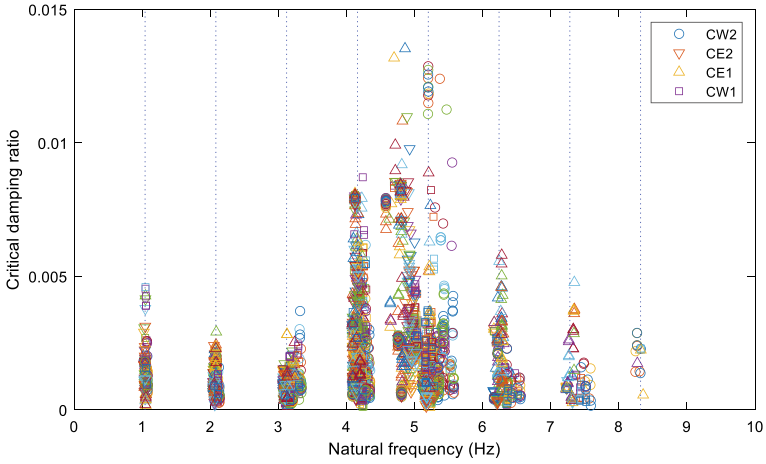


Fig. 6 Damping ratio as a function of natural frequency for the sway and heave motion from all cable sensors for each hour on 28.8.2019, estimated using the SSI-COV method. The vertical lines give a regular multiple of the first eigen frequency (1.04 Hz)

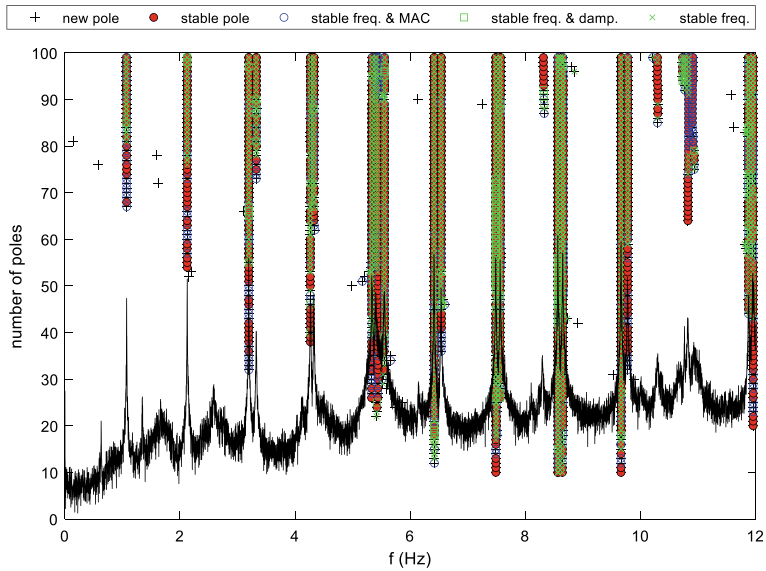


Fig. 7 An example of a stabilization diagram for the SSI-COV estimation process for cable C1E. The PSD estimate of the vertical response is superposed to the identified poles

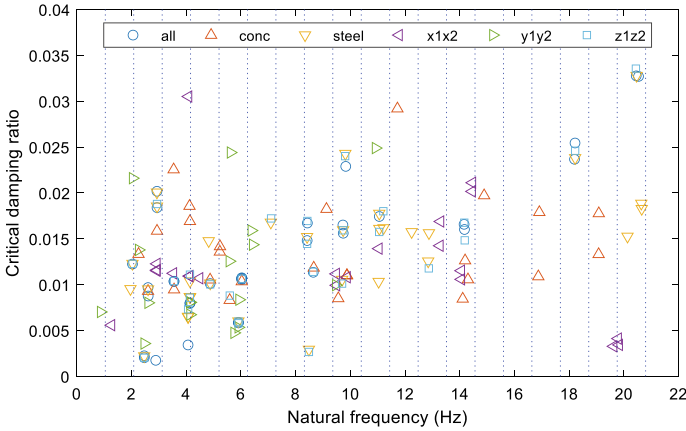


Fig. 8 Damping ratio as a function of natural frequency for different combinations of the components of deck vibrations from both sensors (DN & DS) over the 24-h period on 28.08.2019, estimated using the SSI-COV method

separate the deck modes and the cable modes since the steel deck is supported by the cables, which are then anchored in the concrete side span.

2.3 Cable Dynamics in Relation to Weather Conditions

The following describes a more detailed analysis of cable dynamic characteristics in relation to influencing parameters such as temperature and wind conditions, for the three lowest modes of the C1E cable. Distinct spectral peaks for the three sway and heave modes are utilized to isolate the resonant responses, in a 0.1 Hz wide frequency band centered at a considered peak frequency.

The eigen-frequencies and damping ratios are estimated considering one acceleration component at the time, using the eigen-value analysis (singular value decomposition) of the covariance block Hankel matrix [6, 7]. To ensure a robust analysis of the random signals in a limited frequency band, a convergence study of damping estimates was performed prior to an automatized analysis of all the signals. The range of identified eigen-frequencies and damping ratios are presented in Table 1. For the first mode, the mean heave frequency is 1% higher than the sway frequency, due to the cable sag effect [8]. The difference corresponds to a sag of about 0.25 m for the 98.3 m long cable, or a 2% increase in the effective tension force. For the second and the third mode, the average frequencies in the two directions differ by less than 0.1%.

In Fig. 9, the daily variation of all the six eigen-frequencies is displayed, together with the ambient temperature. A clear effect of temperature on the eigen-frequencies is demonstrated, with a temperature rise and fall giving a reduction and increase,

Table 1 Minimum, Mean and Maximum values of eigenfrequencies (f) and damping ratios (ξ) for the first three cable modes evaluated separately for heave and sway motion

	Mode 1				Mode 2				Mode 3			
	Heave		Sway		Heave		Sway		Heave		Sway	
	f Hz	ξ %	f Hz	ξ %	f Hz	ξ %	f Hz	ξ %	f Hz	ξ %	f Hz	ξ %
Min	1.032	0.027	1.022	0.066	2.041	0.033	2.043	0.038	3.068	0.031	3.071	0.042
Mean	1.038	0.090	1.028	0.139	2.055	0.082	2.056	0.101	3.088	0.078	3.091	0.098
Max	1.043	0.144	1.033	0.245	2.065	0.193	2.065	0.225	3.102	0.162	3.105	0.202

respectively, in the eigen-frequencies. This concurs with findings from another bridge [9].

In Figs. 10 and 11, the associated damping ratios for the three sway and heave modes during the 24 h are illustrated, together with the mean wind components in the two directions. A significant correlation between the magnitude of the relevant wind velocity components and damping can be observed, demonstrating the important contribution of aerodynamic damping to the estimated damping levels, which is largest for the lowest vibration mode. The results complement former field studies on wind-cable interaction [10] and [11]. In the absence of wind, pure structural damping is in the range 0.02% to 0.05%, in line with estimates from free vibration data analysis [3].

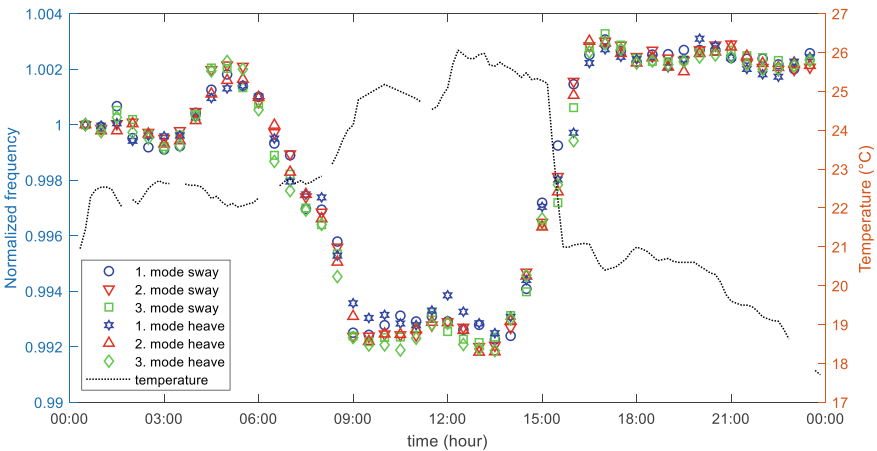


Fig. 9 Natural frequency for modes 1, 2 and 3 normalized by their initial value, as a function of time on 28.08.2019, along with 10 min mean values of atmospheric temperature

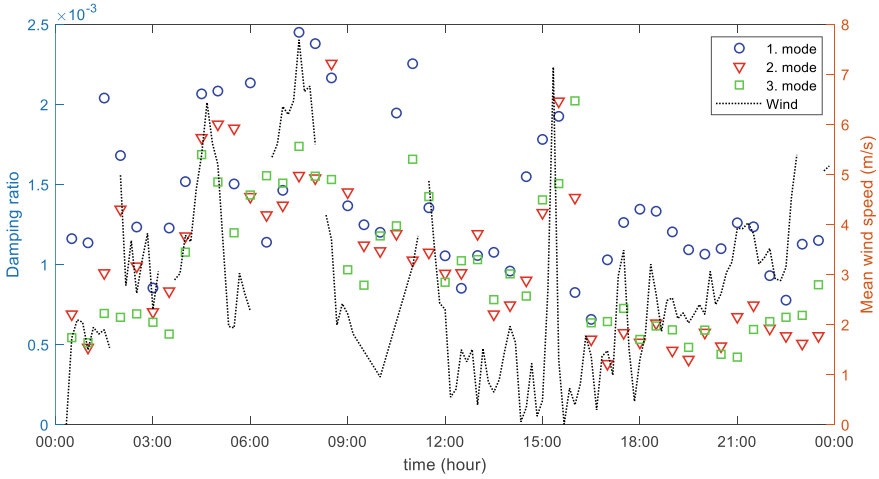


Fig. 10 Damping ratio for modes 1, 2 and 3, for sway motion evaluated based on 24 h acceleration data from 28.08.2019 plotted as a function of time, along with mean values over 10 min of the wind velocity acting horizontally across the cable

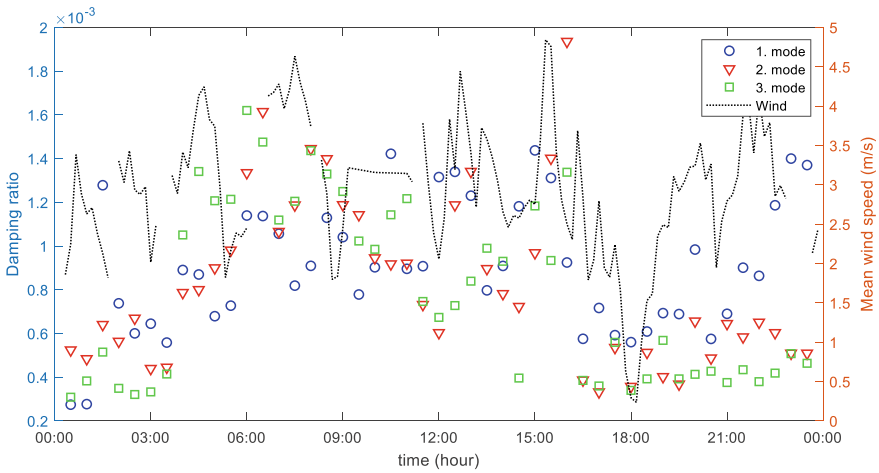


Fig. 11 Damping ratio for modes 1, 2 and 3 for heave motion, evaluated based on 24 h acceleration data from 28.08.2019, plotted as a function of time, along with mean values over 10 min of the wind velocity acting upward across the cable

3 Summary and Conclusions

The paper studies acceleration data from a cable stayed bridge, gathered over 24 h on 28.08.2019. Both deck and cable accelerations are studied. The bridge is a complex structure to analyze, as it combines a concrete deck and tower, that anchors and

supports the cable stayed steel deck providing the crossing over the entrance of Stavanger harbor.

The natural frequencies observed in the deck data are partly the same as the natural frequencies of the cable vibrations, indicating that the deck vibrations are to some extent forced by the cable actions, in addition to being induced by wind action and traffic.

Evaluation of the deck damping gave traditional values in the range of 1% to 2%. Whereas the cable damping was found to be more than an order of magnitude lower, except for modes with a frequency between 4 and 6 Hz, where there is likely a coupling between deck modes and cable modes.

The frequency and damping of the three lowest cable modes were studied in more detail for a single cable. It was found that the natural frequency of the cable was influenced by the ambient temperature changes over the 24 h, with higher ambient temperature resulting in lower natural frequencies. It was also found that the overall damping ratios of cable were strongly correlated with the variations in wind velocity during the 24-h, indicating the significance of aerodynamic damping.

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