

# 1 Measured buffeting response of a long-span suspension bridge compared with 2 numerical predictions based on design wind spectra

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9 turbulence characteristics

## 10 Abstract

11 Wind-induced vibrations of the Hardanger Bridge deck were studied with reference to turbulence  
12 characteristics at the bridge site to evaluate the performance of the state-of-the-art methods for buffeting  
13 response analysis. Long-term monitoring data from an extensive monitoring system were used to obtain  
14 the bridge vibrations and wind characteristics. The acceleration response of the bridge was calculated  
15 in the frequency domain using multimode buffeting theory. Design regulations were used directly and  
16 also modified using measurement data to deduce the wind turbulence spectra. The aerodynamic  
17 properties of the bridge section obtained from previous wind tunnel tests were used in the analyses. The  
18 predicted root mean square (RMS) acceleration response was compared to the measured response. The  
19 analysis using the design methodology gave underestimations of the measured responses. The use of  
20 average values of wind statistics obtained from the monitoring data only slightly improved the results.  
21 When the variability of the wind field was reflected into the design method by using the probability  
22 distributions of the wind field parameters, more satisfactory design curves were obtained.

## 23 1. Introduction

24 Along the western coast of Norway, Coastal Highway E39 connects Trondheim to Kristiansand. This  
25 highway is of vital importance to the Norwegian economy, as the majority of the country's exports are  
26 transported along this coastline. However, due to the unique topography of the Norwegian fjords, eight

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27 ferries currently operate on this highway, resulting in increased travel time. Recently, the Norwegian  
28 Public Roads Administration (NPRA) has initiated an effort to replace the ferry connections with road  
29 transportation. The straits to be crossed vary in width from 1.5 km to 4 km (Nordfjorden, 1.5 km;  
30 Halsafjorden, 2 km; Sulafjorden, 3.8 km; and Sognefjorden, 3.7), and the seabed in these areas is  
31 generally very deep (600 – 1500 meters). For most of the crossings, suspension bridges are considered  
32 to be the primary option, which would require designing and constructing suspension bridges with  
33 unmatched scale in a complex, wind-prone terrain. Experience suggests that wind effects on slender  
34 structures such as these can be critical and even destructive; therefore, accurately predicting the wind-  
35 induced dynamic response is essential for reliable design (Larsen and Larose 2015; Miyata 2003).

36 In wind resistant design of long-span suspension bridges, predicting the buffeting response is one of the  
37 most important steps, particularly for the serviceability and fatigue limit states (Simiu and Scanlan  
38 1996; Xu 2013). First formulated by Davenport (1962), the stochastic theory for buffeting response  
39 analysis was later improved by Scanlan and Tomko (1971) the introduction of aerodynamic derivatives  
40 for describing the self-excited forces. Recently, a multimode approach has been adopted by many  
41 researchers. This analysis can be conducted either in frequency domain (Chen et al. 2001; Jain et al.  
42 1996; Øiseth et al. 2010; Scanlan 1978; Zhu and Xu 2005) or time domain (Caracoglia and Jones 2003;  
43 Chen et al. 2000; Chen and Kareem 2001; Costa et al. 2007; Øiseth et al. 2012).

44 In recent years, structural health monitoring (SHM) systems have been installed on many long-span  
45 cable-supported bridges around the world (Brownjohn et al. 1994; Caetano et al. 2015; Cross et al.  
46 2013; Hui et al. 2009; Macdonald 2003; Miyata et al. 2002; Wang et al. 2011, 2013) to ensure safety  
47 and to monitor the structural behavior of these large and complex structures. The environmental and  
48 structural data obtained from these measurements have been used extensively by researchers in several  
49 applications, e.g., to establish wind characteristics, to study and predict the dynamic response or to  
50 calibrate finite element (FE) models. However, the number of studies where the measured response was  
51 compared to analytical predictions are rather limited. Bietry et al. (1995) compared the acceleration  
52 response of a suspension bridge with analytical predictions and reported that using quasi-steady theory  
53 and aerodynamic admittance functions set to unity led to conservative predictions. Macdonald (2003)  
54 used quasi-steady theory and Davenport's original formulation to calculate the wind-induced response

55 of the Clifton Bridge. Although the design predictions were satisfactory, the study showed that using  
56 the site measurements of the wind field parameters in the prediction instead of the design values would  
57 underestimate the torsional and vertical responses by 40%. Xu and Zhu (2005) studied the dynamic  
58 response of the Tsing Ma Bridge in China using the framework proposed by Zhu and Xu (2005), which  
59 accounts for skew-winds. Researchers included the bridge towers and cables in their analysis, and their  
60 predictions showed reasonable agreement with the field data for the studied 1-hour recording. Wang et  
61 al. (2011) used the same framework to conduct time domain analyses for the Runyang Suspension  
62 Bridge. The measured and predicted responses were reasonably similar for the single event considered.  
63 Wang et al. (2013) compared the buffeting analysis results for the Sutong Bridge that were obtained  
64 using the measured and design spectra. Although the agreement between the two responses were  
65 excellent, no comparisons were made with the measured response. More recently, Cheynet et al. (2016)  
66 studied the buffeting response of the Lysefjord Suspension Bridge in Norway. They compared the  
67 measured displacement response with their frequency-domain predictions for one day of continuous  
68 measurements. The measured turbulence spectra were used to calculate the response and the results  
69 indicated underestimations of the vertical and lateral responses, presumably as a result of the complex  
70 topography. The valuable research efforts listed above provide confidence in the existing methods for  
71 dynamic response prediction of cable-supported bridges. However, in most of the studies, the response  
72 was predicted considering a single event or using limited data. More recordings of strong winds are  
73 needed to consider the actual variability in the response. Objective evaluations of the design  
74 methodology are not possible if the variability in the measured response is not presented. The sources  
75 of discrepancies also remain to be investigated to develop more accurate methods for both predictions  
76 and design.

77 In this study, long-term monitoring data of wind velocity and acceleration from a slender long-span  
78 suspension bridge are used to study its dynamic response. The bridge is located in complex terrain in  
79 the Norwegian fjords and is subjected to strong winds that are mainly perpendicular to the bridge  
80 longitudinal axis. The buffeting response of the bridge is calculated using state-of-the-art methods. The  
81 wind turbulence spectra are deduced using design guidelines and also the measured wind characteristics.

82 The measured acceleration response is compared with the analytical results in the form of buffeting  
83 curves.

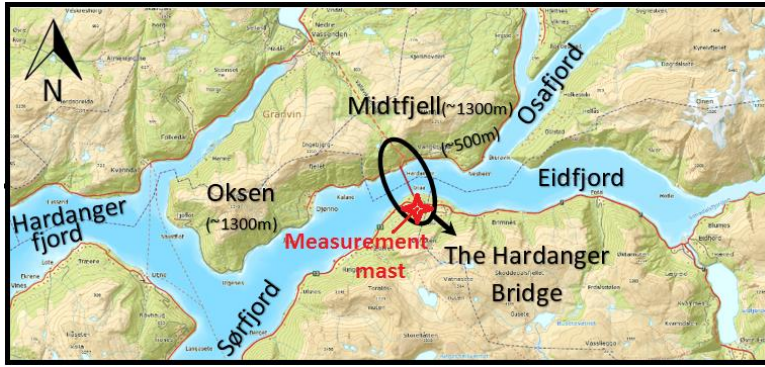
## 84 2. Hardanger Bridge

85 The Hardanger Bridge (Fig. 1) crosses the Hardangerfjord in Norway and serves as an important link  
86 on the highway between the major cities of Oslo and Bergen. Since it opened to the public in 2013 after  
87 a four-year construction period, it has stood as the longest suspension bridge in Norway, with a main  
88 span of 1310 meters. It is located in a mountainous region (Fig. 2) that is subjected to strong wind  
89 storms. Hardanger Bridge supports only two traffic lanes in each direction with an additional lane for  
90 bicycles and pedestrians; thus, with its long main span, this bridge constitutes an exceptionally slender  
91 and lightweight structure. The bridge girder is a streamlined steel box girder that is 3.2 meters high and  
92 18.3 meters wide. In the design stage, the shape of the girder was governed by wind effects (flutter and  
93 vortex shedding); a cross-sectional drawing of the girder is shown in Fig. 3. The bridge was constructed  
94 by individually lifting 60-meter long sections that were fastened to the hangers. Transverse bulkheads  
95 were added every four meters along the deck and guide vanes were installed underneath the girder to  
96 mitigate vortex-induced vibrations. The girder is supported by 130 hangers and 2 main cables, which  
97 are located at either side of the girder. The longest hanger is 128 meters long, and the shortest one is 2  
98 meters long; the hanger diameter is 70 mm. Each of the bridge cables consists of 19 strands that are  
99 made of 528 galvanized 5.3 mm wires; the cables were assembled by pulling each wire into place using  
100 a spinning wheel travelling along the cable path. The two bridge towers are supported by concrete  
101 foundations on rock and they reach up to 200 meters. Each tower consists of two massive concrete  
102 columns joined by three cross-beams, and the design of the concrete pylons was governed by wind  
103 effects in the ultimate limit state.



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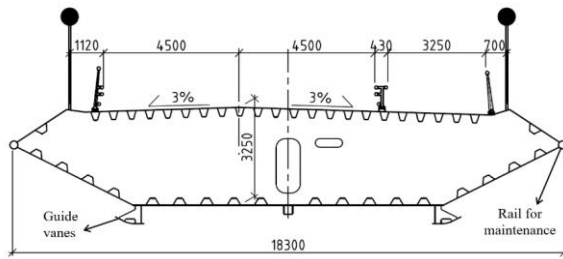
105 **Fig. 1.** Hardanger Bridge (image by the authors)



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**Fig. 2.** Local topography around Hardanger Bridge



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**Fig. 3.** Scale drawing of the Hardanger Bridge cross-section

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### 3. Design basis for Hardanger Bridge

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The Norwegian handbook for bridge design (N400, Statens-Vegvesen 2009) provides regulations for

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the design of bridge structures, including suspension bridges. This document states that field

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measurements must be performed if the bridge span is longer than 300 meters and the wind load is

114

significant. According to N400, the along-wind ( $I_u$ ) and vertical ( $I_w$ ) turbulence intensities are given as

115

$$I_u(z) = \frac{c_{tt}}{\ln(z/z_0)}, \quad I_w = 0.5I_u \quad (1)$$

116

where  $c_{tt}$  is the turbulence factor,  $z$  is the height above the ground and  $z_0$  is the roughness length, which

117

can be taken as 0.01 meters, in accordance with the document. The turbulence factor was given as 1.2

118

for the Hardanger Bridge by the NPRA, which accounts for topographical influences. The above

119

formula gives a turbulence intensity of 11.5% for the along-wind turbulence and 5.7% for the vertical

120

turbulence. To calculate the one-point auto-spectra of the along-wind ( $S_u$ ) and vertical ( $S_w$ ) turbulence

121

components, N400 recommends the following expression, which is in the form of the Kaimal spectra

122

(Kaimal et al. 1972):

123 
$$\frac{fS_i}{\sigma_i^2} = \frac{a_i \hat{n}_i}{(1 + 1.5a_i \hat{n}_i)^{5/3}}, \quad \hat{n}_i = \frac{f \cdot {}^xL_i(z)}{U(z)}, \quad i = u, w \quad (2)$$

124 where  $f$  is the frequency in Hz,  $U$  is the mean wind speed,  $\sigma_{u,w}$  are the standard deviations of the  $u$  and  
 125  $w$  turbulence components,  ${}^xL_u(z)$  and  ${}^xL_w(z)$  denote the longitudinal and vertical length scales,  
 126 respectively, which are given as functions of height above the ground  $z$ . For the Hardanger Bridge deck,  
 127  ${}^xL_u(z)$  and  ${}^xL_w(z)$  correspond to 171 meters and 14 meters, respectively, at a height of approximately  
 128 60 meters above sea level, the spectral parameters are given as  $a_u = 6.8$  and  $a_w = 9.4$ . The document  
 129 adopts Davenport's (1961) expression for the normalized cross-spectrum, which can be written as

130 
$$C(f, \Delta x) = \exp\left(-K \frac{f \cdot \Delta x}{U}\right) \quad (3)$$

131 where  $K$  is the decay coefficient and  $\Delta x$  is the span-wise separation. The recommended values for the  
 132 decay coefficients are given as  $K_u = 10$  and  $K_w = 6.5$ .

133 During the design of the Hardanger Bridge, the turbulence spectra given by N400 were refined, using  
 134 field measurements from a 45-meter-high mast (Harstveit 2007) and wind tunnel tests on a terrain model  
 135 of the bridge site (Sætran and Malvik 1991); the location of the measurement mast is shown in Fig. 2.  
 136 The four-year (1988-1992) data from the mast were combined with long-term (1981-2006) data from a  
 137 nearby lighthouse to obtain the wind characteristics. Considering both the field and wind tunnel  
 138 measurements data, the N400 recommendations were calibrated by NPRA, to form the design basis for  
 139 the wind characteristics (Statens-Vegvesen 2006). The turbulence intensities for the along-wind and  
 140 vertical turbulence were reported as 13.7% and 7%, respectively. The expression given in Eqn. (2) was  
 141 used to calculate one-point spectra of turbulence, where the length scale values were adjusted to 360  
 142 meters for the along-wind component and 21 meters for the vertical component to match the  
 143 measurements. The following expression was used for the normalized cross-spectra:

144 
$$C_i(f, \Delta x) = \left(1 - \frac{1}{2} \kappa \Delta x\right) \cdot \exp(-\kappa \Delta x), \quad \kappa = b_i \sqrt{\left(\frac{2\pi f}{U}\right)^2 + \left(\frac{1}{c_i \cdot {}^xL_i}\right)^2}, \quad i = u, w \quad (4)$$

145 where the coefficients are given as  $b_u = 1$ ,  $c_u = 1.5$ ,  $b_w = 0.5$  and  $c_w = 1$ . This expression is based on the  
 146 original formulation by Krenk (1996), and the coefficients were introduced to provide better agreement  
 147 with the site measurements. This expression is superior to the classical exponential expression for two

148 reasons: it allows for values smaller than unity for large separations at zero frequency, and it allows for  
149 negative values. This equation overcomes the theoretical problem of Eqn. (3), which contradicts the  
150 zero mean definition of the turbulence component (Holmes 2007; Krenk 1996).

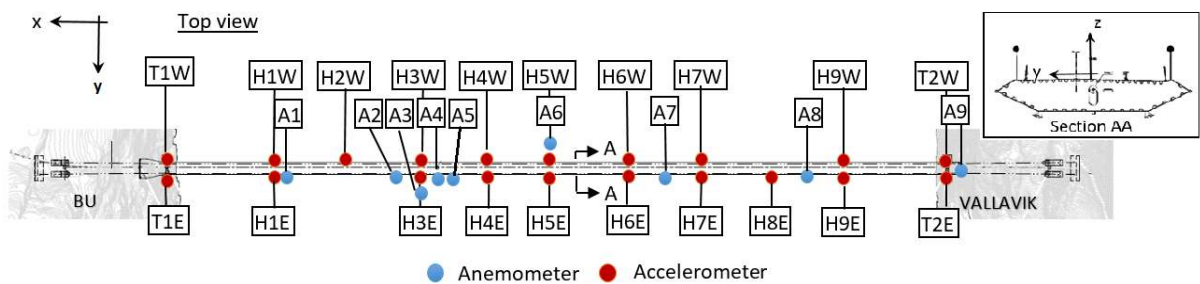
#### 151 4. Monitoring of Hardanger Bridge

##### 152 Measurement system

153 Immediately after Hardanger Bridge opened in 2013, it was instrumented with an extensive monitoring  
154 system with the aim of measuring wind velocities and accelerations at several locations along the bridge  
155 deck and at the tower tops. The monitoring system consists of 20 triaxial accelerometers and 9 3D  
156 anemometers; the sensor layout is given in Fig. 4. The monitoring system consists of 20 triaxial  
157 accelerometers and 9 anemometers; the sensor layout is given in Fig. 4. For wind measurements,  
158 WindMaster Pro 3-d ultrasonic anemometers from Gill Instruments were used. The wind sensors  
159 provide a measurement range of 0-65 m/s, 0.001 m/s resolution and up to 32 Hz data output rate. The  
160 accelerometers installed on the bridge are of CUSP-3D series strong motion accelerographs by  
161 Canterbury Seismic Instruments. These sensors are robust triaxial accelerometers and are capable of  
162 measuring accelerations in  $\pm 4g$  range with 200 Hz data output rate. Of the accelerometers, 16 are  
163 located inside the bridge girder, 14 of which are installed on both ends of the girder as pairs to capture  
164 the torsional motion of the girder, and the remaining 4 are fixed inside the two tower tops. Eight of the  
165 anemometers are distributed along the bridge span, and they are attached to the hangers 8 meters above  
166 the girder to avoid wind flow disturbances due to the deck and traffic. The last anemometer stands on  
167 top of the Vallavik (North) tower.

168 The accelerometer data are sampled at 200 Hz initially, whereas the anemometer data are sampled at  
169 32 Hz. A common sampling frequency of 20 Hz is used for both the accelerometer and anemometer  
170 data in this study, so both data were downsampled to 20 Hz prior to use. The wind data were transformed  
171 to a coordinate system oriented in the mean wind direction, and its mean and fluctuating parts  
172 (turbulence components) were decomposed using a 10-minute averaging interval for the wind  
173 characteristics study. The fluctuations in the along-wind direction are denoted as  $u(t)$ , where the  
174 fluctuations in the cross-wind and vertical directions are denoted as  $v(t)$  and  $w(t)$  turbulence

175 components, respectively. The vertical and lateral accelerations of the bridge girder were calculated by  
 176 averaging two signals from the accelerometers on each side of the girder; the torsional acceleration was  
 177 then calculated by dividing the difference between the two signals by the deck width. Since only the  
 178 wind-induced vibrations of the bridge are of interest, it is desired to exclude vibrations induced by other  
 179 sources from the data. Looking at the frequency content of the acceleration signals, it is seen that the  
 180 response is dominated by low frequency vibrations when the wind speed is above 8 m/s. Among the  
 181 recordings with lower wind speeds, the ones dominated by high frequency vibrations were removed  
 182 from the data, assuming the traffic-induced vibrations were profound. Moreover, the remaining  
 183 acceleration data were passed through a low-pass filter with 1 Hz cut-off frequency, since it is expected  
 184 that wind-induced response of the bridge is in the low frequency (0-1 Hz) range (Brownjohn et al. 1994;  
 185 Xu and Zhu 2005a). It should also be noted that the traffic density in Hardanger Bridge is usually very  
 186 low; therefore, large variations in response due to traffic is not likely.



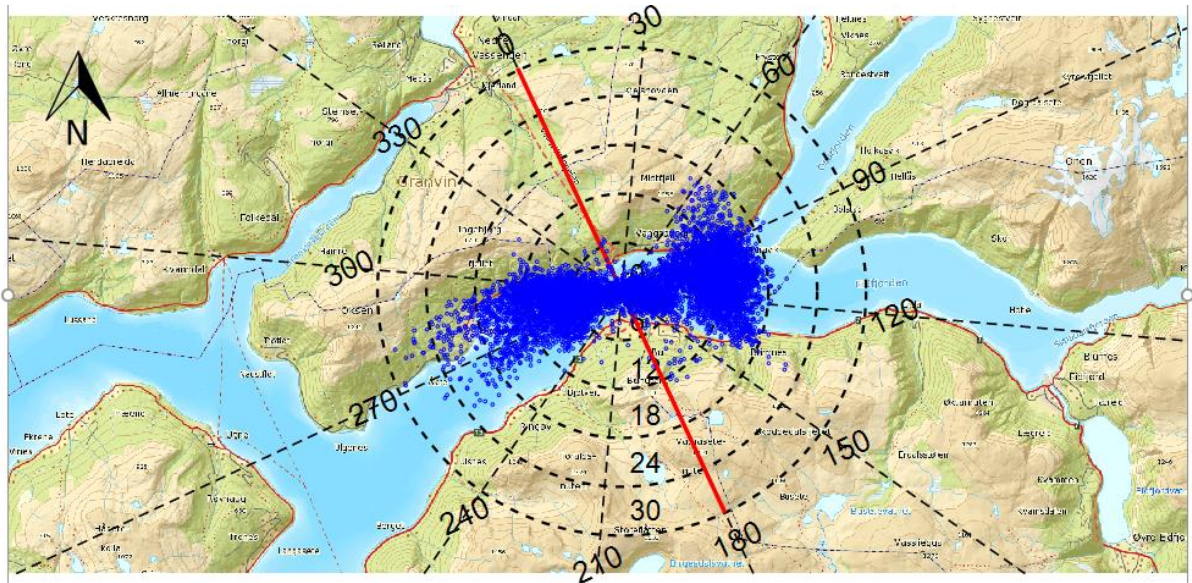
187  
 188 **Fig. 4.** Sensor layout

189 Mean wind speed and direction

190 The monitoring system has been recording data in a discontinuous manner since 2013. The system is  
 191 triggered if the wind velocity in the horizontal plane exceeds 15 m/s in any of the anemometers. After  
 192 the system is triggered, the acceleration and wind velocity are recorded for 30 minutes. Occasionally,  
 193 the system is also triggered manually to also include recordings with low wind speeds in the database.,  
 194 A total of 8530 10-minute recordings from the database are considered in this paper; these data were  
 195 recorded between December 2013 and September 2016 (a total of 35 months). A threshold wind speed  
 196 of 3 m/s is used, meaning that recordings with lower wind speeds were discarded and are not presented  
 197 in this paper. Fig. 5 shows a wind rose plot of the 10-minute mean wind speed on the local topography  
 198 map of the bridge site. In the figure, the 0° direction corresponds to the longitudinal axis of the bridge.



199 Mean wind speeds of up to 30 m/s were recorded, and the wind direction was almost perpendicular to  
 200 the bridge axis. The wind was in general blowing along the fjord due to the steep mountains on either  
 201 end of the bridge (Fig. 5); however, skew winds with deviations of up to 30° from the perpendicular  
 202 direction were also measured.



203  
 204 **Fig. 5.** Wind rose plot of mean wind speed averaged over 10 minutes

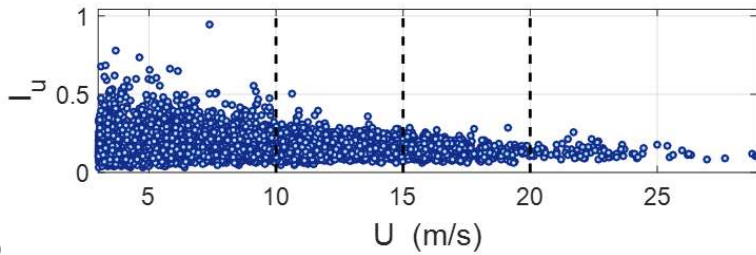
205 Turbulence intensity

206 The turbulence intensity provides direct information on the turbulent energy of the wind, and is  
 207 therefore critically important for describing the characteristics of atmospheric turbulence. The along-  
 208 wind (u) and vertical (w) turbulence intensities were calculated for the 10-minute recordings and were  
 209 plotted against the mean wind speed (Fig. 6). Fig. 7 shows the probability distribution plots of the along-  
 210 wind (u) and vertical (w) turbulence intensities. The cross-wind (v) component is assumed to have a  
 211 negligible influence on bridge dynamic response and is, therefore not presented. Since the turbulence  
 212 intensity is dependent on the mean wind speed, the data were divided into four segments with different  
 213 wind speeds. Lognormal distributions were fitted to the data and are shown in the same figures; the  
 214 plots indicate that lognormal distributions can represent the data fairly well. The probability distribution  
 215 function of the lognormal distribution is written as

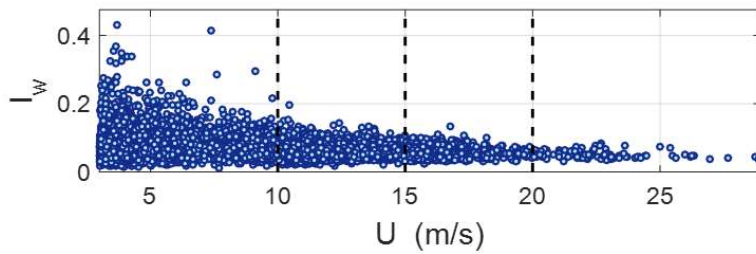
216

$$f(x | \mu, \sigma) = \frac{1}{x\sigma\sqrt{2\pi}} e^{\left\{ \frac{-(\ln x - \mu)^2}{2\sigma^2} \right\}}; x > 0 \quad (5)$$

217 where  $\mu$  and  $\sigma$  are the parameters of the distribution (the mean and standard deviation of the natural  
 218 logarithm of the random variable, respectively) and are given in the figures for the fitted distributions.  
 219 The mean values of the along-wind and vertical turbulence intensities were 16.5% and 7.1%,  
 220 respectively.

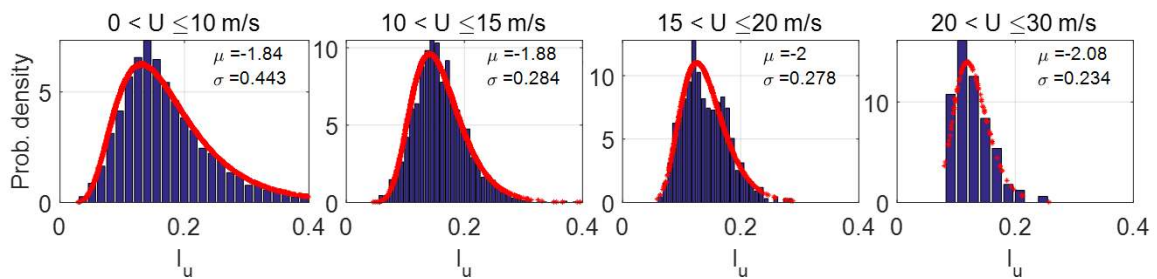


221 (a)

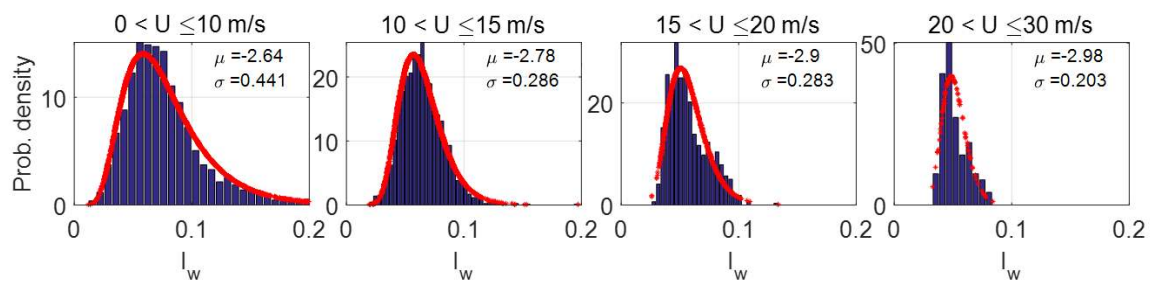


222 (b)

223 **Fig. 6.** Turbulence intensity vs. mean wind speed: (a) u component and (b) w component



224 (a)



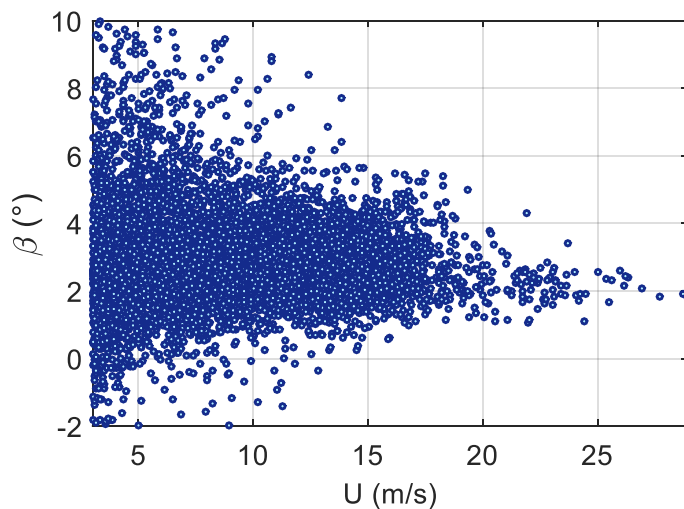
225 (b)

226 **Fig. 7.** Probability distributions of turbulence intensity: (a) along-wind and (b) vertical turbulence.

227 Angle-of-attack

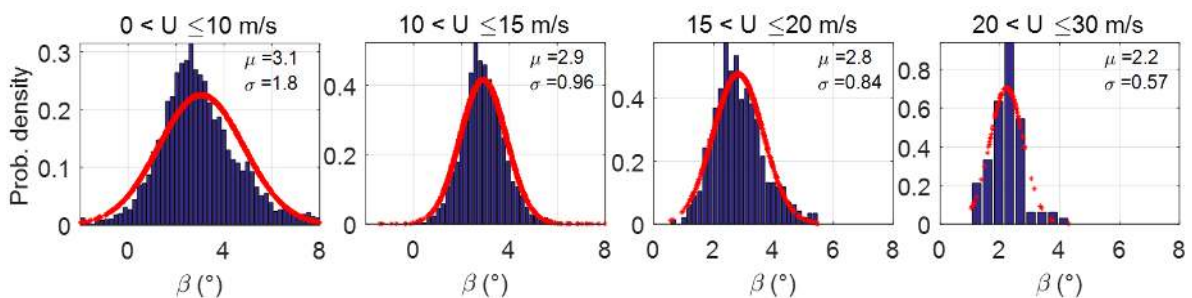
228 The angle-of-attack ( $\beta$ ) is defined here as the angle between the mean wind velocity vector and the  
 229 horizontal plane. The aerodynamic properties of the bridge section are affected by the inclination of the

230 mean wind; consequently, the structural response is influenced by the angle-of-attack. The angle-of-  
 231 attack was calculated using the anemometer data and plotted against the mean wind speed for all  
 232 recordings, as shown in Fig. 8. This figure indicates that the angle-of-attack exhibits significant  
 233 variability at low and moderate wind speeds. Large angles were obtained from the recordings with low  
 234 wind speeds and a nonstationary nature. In general, the mean wind velocity was inclined towards the  
 235 positive z direction (upwards), and the mean value of the angle-of-attack was  $3^\circ$  for whole velocity  
 236 range, which is rather substantial. The conditional probability distributions of  $\beta$  are given in Fig. 9 for  
 237 different velocity ranges along with the normal distribution fits to the data. The angle-of-attack can be  
 238 approximated as normally distributed, with a mean value of approximately  $2^\circ$ - $3^\circ$ , even for the strong  
 239 winds.



240

241 **Fig. 8.** Angle-of-attack vs. mean wind speed



242

243 **Fig. 9.** Probability distributions of the angle-of-attack ( $\beta$ )

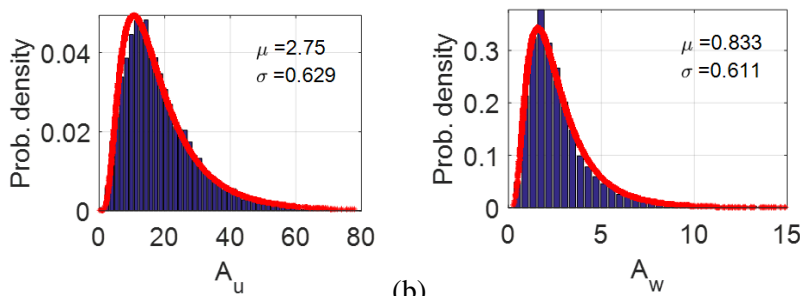
244 One-point spectra of turbulence

245 The one-point auto-spectra of the along-wind and vertical turbulence components are often used to  
246 describe the gust loading on suspension bridges in dynamic response calculations. However, the one-  
247 point cross-spectra of u and w components, are often neglected assuming that their effects on the  
248 dynamic response are insignificant. Therefore, the auto power spectral densities (PSDs) of the u and w  
249 turbulence components were estimated for the entire database. For each 10-minute signal, the spectra  
250 were estimated using Welch's averaged periodogram method (Welch 1967). The time series of the  
251 turbulence components were divided into 8 segments with 50% overlap; then, the PSDs were calculated  
252 using the Fast Fourier Transform (FFT) method and were averaged after applying a Hamming window  
253 to each segment. The Welch estimate of the PSD results in high variance; thus, the estimates were  
254 smoothed using a parametric least squares fit. The following parametric expression, in the form of the  
255 Kaimal spectra, was fitted to each estimate:

$$256 \quad \frac{S_{u,w} f}{\sigma_{u,w}^2} = \frac{A_{u,w} f_z}{(1 + 1.5 A_{u,w} f_z)^{5/3}}, \quad f_z = \frac{f z}{U} \quad (6)$$

257 where  $f$  is the frequency in Hz,  $U$  is the mean wind speed,  $z$  is the height above the ground,  $S_{u,w}$  are the  
258 auto-spectra and  $\sigma_{u,w}$  are the standard deviations of the u and w turbulence components. The remaining  
259 non-dimensional parameters  $A_{u,w}$  are determined by the least squares fit to the measurement data. The  
260 resulting probability distributions of the spectral parameters are presented in Fig. 10 along with the  
261 fitted lognormal distributions. The data is not divided into velocity intervals, since no strong dependence  
262 was observed with the mean wind speed. The corresponding mean values were 18.8 and 2.8 for  $A_u$  and  
263  $A_w$ , respectively. The parameters of the fitted distributions are indicated on the plots.

264



265 (a)

(b)

266 **Fig. 10.** Probability distributions of the spectral parameters: (a) u component and (b) w component

267 .

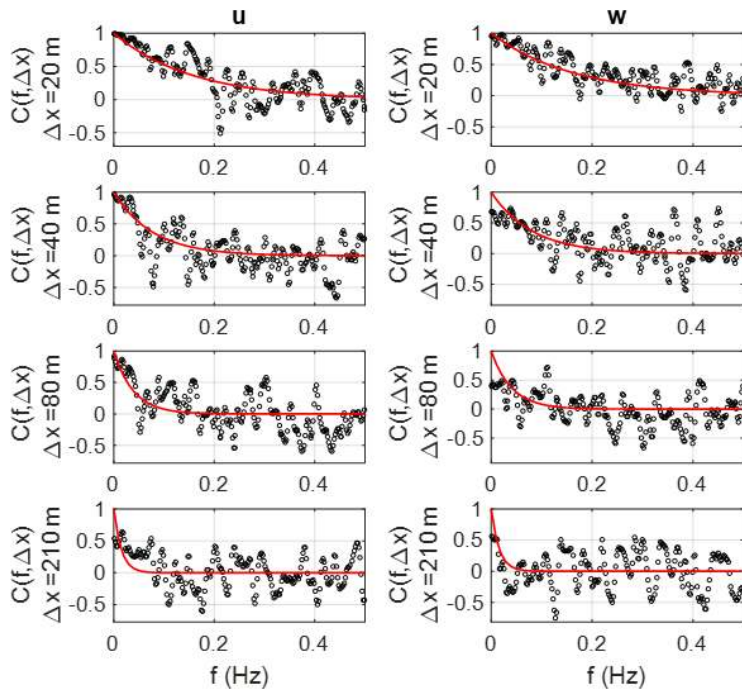
268 Normalized cross-spectra

269 A complete description of the atmospheric turbulence and consequently the gust loading on any line-  
270 like structure requires spatial correlation information of the turbulence components in addition to the  
271 one-point statistics. This is commonly achieved using normalized cross-spectra. The normalized cross-  
272 spectrum can be interpreted as a frequency-dependent correlation coefficient and is defined as (Dyrbye  
273 and Hansen 1997)

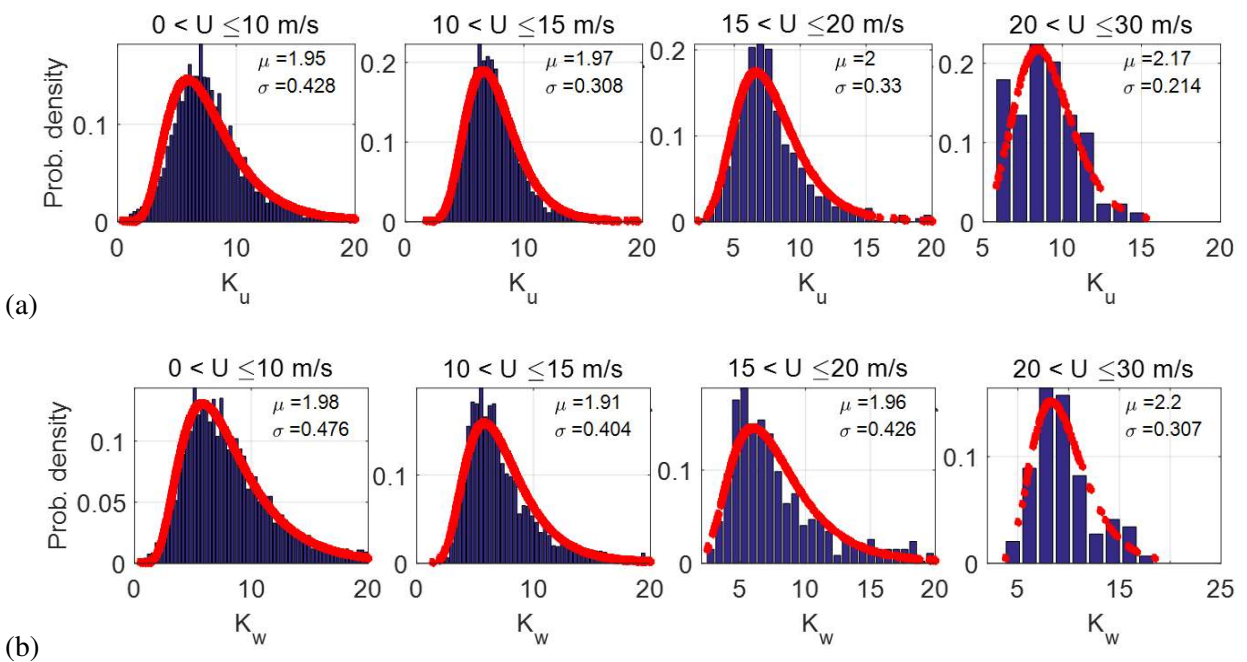
$$274 \quad C_{nm}(f, \Delta x) = \frac{S_{nm}(f)}{\sqrt{S_n(f)S_m(f)}} \quad (7)$$

275 where,  $S_{nm}$   $n \in \{u, w\}$ ,  $m \in \{u, w\}$  denotes the cross-spectral density while  $S_n$   $n \in \{u, w\}$  and  $S_m$   $m \in \{u, w\}$   
276 denote the auto-spectral densities at two points separated by a distance  $\Delta x$ . The complex part of the  
277 cross-spectral density contains the phase information, which is regarded as small for separations  
278 transverse to the wind flow and is often neglected in practice. The normalized cross-spectra for the 10-  
279 minute recordings were calculated using the Welch spectral density estimates and neglecting the  
280 complex part of the cross-spectra. The normalized cross-spectra are usually represented by the simple  
281 exponential expression proposed by Davenport (1961), the expression of which is given in Eqn. (3).  
282 The decay coefficients  $K_{u,w}$  can be estimated using a least squares approximation of the data. Despite  
283 its weaknesses at low frequencies and large separation distances (Simiu and Scanlan 1996), Davenport's  
284 expression approximates the actual normalized cross-spectra reasonably well and is frequently used in  
285 practice. The expression was fitted to the measurement data in least squares sense and the decay  
286 coefficients were obtained for all recordings. Four sensor pairs were used to estimate the root  
287 coherences from the database; the corresponding separation distances were 20, 40, 80 and 210 meters  
288 for the four sensor pairs. The curve fitting of the root coherence function is shown in Fig. 11 for a 10-  
289 minute recording as an example. The resulting probability distributions of the decay coefficients are  
290 presented in Fig. 12 for the u and w components along with the corresponding lognormal distributions  
291 that were fitted to the data. The mean values of the decay coefficients were 7.63 for the along-wind  
292 component and 7.78 for the vertical component. This expression causes difficulties when the correlation

293 is not apparent; thus, any  $K$  value above 20 is considered non-coherent and is not included in the  
 294 probability distribution.



295  
 296 **Fig. 11.** Estimation of the decay coefficients using least squares fitting for a sample 10-minute  
 297 recording ( $K_u = 6.55$ ,  $K_w = 6.31$ ). The histograms represent the measured data and the curves were fitted  
 298 using Eqn. (3).



300 (b)  
 301  
 302 **Fig. 12.** Probability distributions of the decay coefficients: (a) u component and (b) w component

303

304 5. Buffeting analysis in frequency domain

305 Buffeting analysis method

306 The buffeting response of Hardanger Bridge was calculated in frequency domain using the classical  
 307 multimode theory (Chen et al. 2001; Jain et al. 1996). The analytical procedure is briefly introduced  
 308 here; more detailed formulations can be found elsewhere (Katsuchi et al. 1998; Øiseth et al. 2010). The  
 309 analysis is based on the solution of the fully coupled system of equations of motion, where the bridge  
 310 displacements are represented in generalized coordinates of the mode shapes in still-air. The multimode  
 311 system of equations of motion including the aeroelastic effects is written in frequency domain as

$$312 \quad \mathbf{M}_0^c \mathbf{G}_{\ddot{\mathbf{x}}}( \omega ) + (\mathbf{C}_0^c - \mathbf{C}_{ae}^c(U, \omega)) \mathbf{G}_{\dot{\mathbf{x}}}( \omega ) + (\mathbf{K}_0^c - \mathbf{K}_{ae}^c(U, \omega)) \mathbf{G}_{\eta}( \omega ) = \mathbf{G}_{\mathbf{Q}_{buff}}( \omega ) \quad (8)$$

313 where  $\omega$  is the circular frequency,  $\mathbf{M}_0^c$ ,  $\mathbf{K}_0^c$  and  $\mathbf{C}_0^c$  are the generalized mass, stiffness and damping  
 314 matrices in still-air, respectively,  $\mathbf{C}_{ae}^c$  and  $\mathbf{K}_{ae}^c$  are the generalized aeroelastic damping and stiffness  
 315 matrices respectively,  $\mathbf{G}_{\ddot{\mathbf{x}}}$ ,  $\mathbf{G}_{\dot{\mathbf{x}}}$  and  $\mathbf{G}_{\eta}$  denote the Fourier transforms of the acceleration, velocity and  
 316 displacement responses in generalized coordinates, respectively, and  $\mathbf{G}_{\mathbf{Q}_{buff}}$  denotes the generalized  
 317 buffeting force. According to random vibration theory (Wirsching et al. 2006), the PSD matrices of the  
 318 generalized displacement response ( $\mathbf{S}_R^c(\omega)$ ) and the buffeting force ( $\mathbf{S}_{\mathbf{Q}_{buff}}(\omega)$ ) are related as follows:

$$319 \quad \mathbf{S}_R^c(\omega) = \left[ \mathbf{E}_{\eta}^{-1}(\omega) \mathbf{S}_{\mathbf{Q}_{buff}}(\omega) \mathbf{E}_{\eta}^{-1*}(\omega) \right] \quad (9)$$

$$\mathbf{E}_{\eta}(U, \omega) = \left[ -\mathbf{M}_0^c \omega^2 + (\mathbf{C}_0^c - \mathbf{C}_{ae}^c(U, \omega)) i\omega + (\mathbf{K}_0^c - \mathbf{K}_{ae}^c(U, \omega)) \right]$$

320

321 The frequency-dependent modal aeroelastic stiffness and damping matrices can be obtained by

$$322 \quad \mathbf{K}_{ae}^c(U, \omega) = \int_L (\mathbf{\Phi}^T \mathbf{K}_{ae}(U, \omega) \mathbf{\Phi}) dx \quad (10)$$

$$\mathbf{C}_{ae}^c(U, \omega) = \int_L (\mathbf{\Phi}^T \mathbf{C}_{ae}(U, \omega) \mathbf{\Phi}) dx$$

323 where

$$324 \quad \mathbf{K}_{ae} = \frac{\rho B^2}{2} \omega^2 \begin{bmatrix} P_4^* & P_6^* & BP_3^* \\ H_6^* & H_4^* & BH_3^* \\ BA_6^* & BA_4^* & B^2 A_3^* \end{bmatrix}, \quad \mathbf{C}_{ae} = \frac{\rho B^2}{2} \omega \begin{bmatrix} P_1^* & P_5^* & BP_2^* \\ H_5^* & H_1^* & BH_2^* \\ BA_5^* & BA_1^* & B^2 A_2^* \end{bmatrix} \quad (11)$$

325 In the above expressions,  $\boldsymbol{\varphi}_i = [\phi_y, \phi_z, \phi_\theta]^T$  is the mode shape vector,  $\boldsymbol{\Phi}$  denotes the matrix of the mode  
326 shapes,  $P_{1,2,\dots,6}^*$ ,  $H_{1,2,\dots,6}^*$ ,  $A_{1,2,\dots,6}^*$  denote the dimensionless aerodynamic derivatives, B is the width of the  
327 girder and  $\rho$  is the air density. Having established the system matrices, the buffeting action needs to be  
328 defined. The elements of the spectral matrix of the buffeting force can be written in generalized  
329 coordinates as

$$330 \quad \mathbf{S}_{Q_{buff}}(\omega) = \int_L \int_L \boldsymbol{\Phi}^T(x_1) \mathbf{B}_q(\omega) \mathbf{S}_V(\Delta x, \omega) \mathbf{B}_q^T(\omega) \boldsymbol{\Phi}(x_2) dx_1 dx_2 \quad (12)$$

$$\mathbf{S}_V(\Delta x, \omega) = \begin{bmatrix} S_{uu}(\Delta x, \omega) & S_{uw}(\Delta x, \omega) \\ S_{uw}(\Delta x, \omega) & S_{ww}(\Delta x, \omega) \end{bmatrix}$$

331 where  $\mathbf{S}_V(\Delta x, \omega)$  is the cross-spectral density matrix containing the auto and cross-spectral densities of  
332 the turbulence components at the two points  $x_1$  and  $x_2$ , which are separated by a distance of  $\Delta x$ . The  
333 matrix  $\mathbf{B}_q(\omega)$  includes the steady-state force coefficients:

$$334 \quad \mathbf{B}_q(\omega) = \frac{\rho UB}{2} \begin{bmatrix} 2(D/B)\bar{C}_D & ((D/B)C'_D - \bar{C}_L) \\ 2\bar{C}_L & (C'_L + (D/B)\bar{C}_D) \\ 2B\bar{C}_M & BC'_M \end{bmatrix} \quad (13)$$

335 Here,  $\bar{C}_D$ ,  $\bar{C}_L$  and  $\bar{C}_M$  are the mean values of the steady-state force coefficients associated with the  
336 drag, lift and moment, respectively,  $C'_D$ ,  $C'_L$  and  $C'_M$  are the corresponding derivatives, and D denotes  
337 the girder height. The steady-state force coefficients were obtained from the wind tunnel tests by  
338 Hansen et al. (2006) and are given in Table 1. The aerodynamic admittance functions are taken as unity  
339 throughout the study due to lack experimental data, which is expected to yield conservative results  
340 (Bietry et al. 1995; Larose and Mann 1998; Macdonald 2003). Finally, the root mean square (RMS)  
341 acceleration response can be obtained from the displacement spectra using

$$342 \quad \sigma_R = \sqrt{\int_0^\infty S_{\ddot{x}}(\omega) d\omega}, \quad S_{\ddot{x}}(\omega) = \omega^4 \left[ \boldsymbol{\Phi}(x) \mathbf{S}_R^G(\omega) \boldsymbol{\Phi}^T(x) \right] \quad (14)$$

343 where  $S_{\ddot{x}}(\omega)$  and  $S_R(\omega)$  are the acceleration and displacement spectra in global coordinates,  
344 respectively.

345 **Table 1.** Steady-state force coefficients for the Hardanger Bridge section (Hansen et al. 2006)

$\bar{C}_D$	$C'_D$	$\bar{C}_L$	$C'_L$	$\bar{C}_M$	$C'_M$
0.7	0	-0.25	2.4	0.01	0.74



346 4.2 Modal analysis

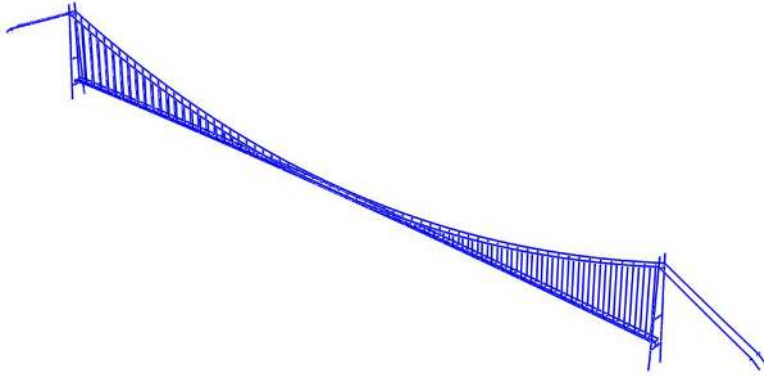
347 An eigenvalue analysis was performed to obtain the still-air vibration frequencies and mode shapes of  
 348 Hardanger Bridge, to be used in the buffeting calculations. The FE model of the bridge, which was  
 349 originally constructed by the NPRA in ABAQUS (Dassault Systèmes Simulia et al. 2013), was used in  
 350 the analysis. The FE model is shown in Fig. 13. The first 100 natural frequencies and the corresponding  
 351 mode shapes were obtained by solving the classical eigenvalue problem, after the application of dead  
 352 loads, accounting for the geometric stiffness utilized by the cables. The first 20 modes and the  
 353 corresponding natural frequencies and periods are listed in Table 2 along with the dominant nature of  
 354 each mode; some of the mode shapes are illustrated in Fig. 14.

355

356 **Table 2.** First 20 vibration modes of Hardanger Bridge

Mode	Frequency (Hz)	Period (seconds)	Description of the dominant motion
1	0.05	20.00	Symmetric lateral vibration of the deck
2	0.098	10.20	Asymmetric lateral vibration of the deck
3	0.11	9.09	Asymmetric vertical vibration of the deck
4	0.14	7.14	Symmetric vertical vibration of the deck
5	0.169	5.92	Symmetric lateral vibration of the deck
6	0.197	5.08	Symmetric vertical vibration of the deck
7	0.21	4.76	Asymmetric vertical vibration of the deck
8	0.225	4.44	Symmetric lateral vibration of the cables
9	0.232	4.31	Asymmetric lateral vibration of the cables
10	0.233	4.29	Asymmetric lateral vibration of the deck and the cables
11	0.244	4.10	Symmetric lateral vibration of the deck and the cables
12	0.272	3.68	Symmetric vertical vibration of the deck
13	0.293	3.41	Asymmetric lateral vibration of the deck
14	0.33	3.03	Asymmetric vertical vibration of the deck
15	0.36	2.78	Symmetric torsional vibration of the deck
16	0.373	2.68	Symmetric lateral vibration of the cables
17	0.392	2.55	Symm. lateral vibration of the deck accompanied by torsion
18	0.394	2.54	Symmetric vertical vibration of the deck
19	0.406	2.46	Asymmetric lateral vibration of the deck and the cables
20	0.407	2.46	Asymmetric lateral vibration of the cables

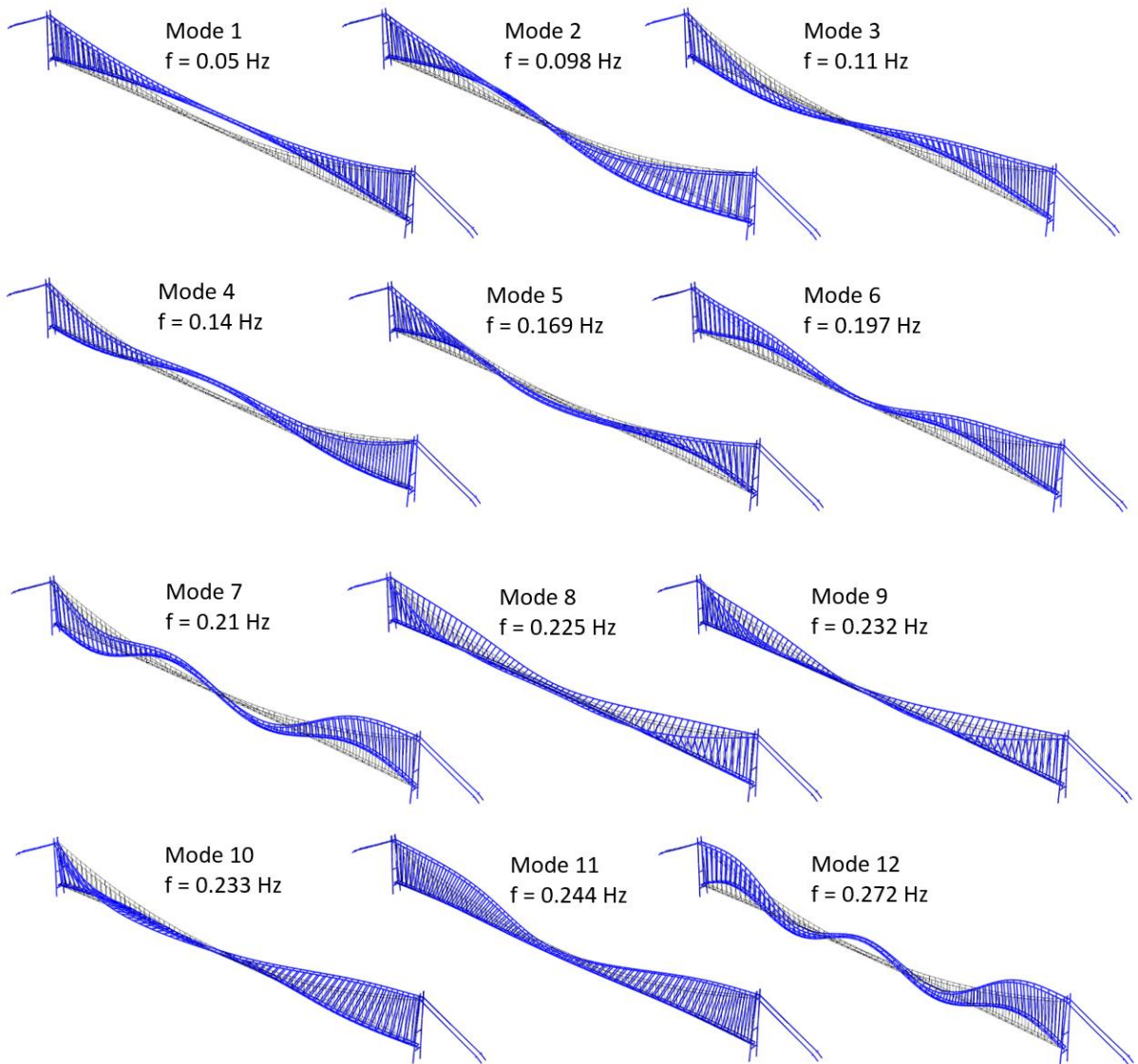
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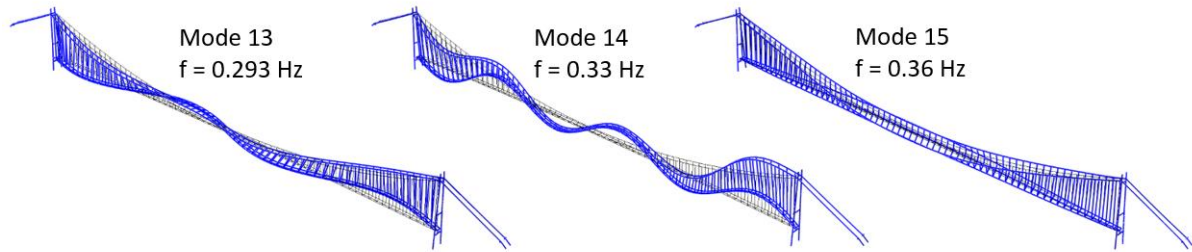
359 **Fig. 13.** FE model of Hardanger Bridge

360



361

362



363

364 **Fig. 14.** Mode shapes of Hardanger Bridge for the first 15 vibration modes

365 Wind turbulence spectra

366 The wind turbulence spectral matrix ( $S_v$  in Eqn. (12)), must be established to calculate the buffeting  
 367 response. Four different spectral matrices are described here to be used in the response analyses. The  
 368 first two spectral matrices were calculated according to N400 guidelines and the design basis for  
 369 Hardanger Bridge, the expressions for which are given in Section 2. Then, as a third case, the probability  
 370 distributions of the turbulence parameters were used to calculate the turbulence spectra. The spectral  
 371 parameters  $A_{u,w}$  and the decay coefficients  $K_{u,w}$  were taken as the 50th percentile values from the  
 372 fitted lognormal distributions considering the whole wind speed range. Due to their strong dependence  
 373 on the wind speed (Fig. 6), the turbulence intensities were taken as the 50th percentile values from the  
 374 conditional probability distributions for different wind speed segments (Fig. 7). For the fourth case,  
 375 instead of the 50th percentile values, the 5th percentile values for the decay coefficients and the 95th  
 376 percentile values for the turbulence intensities were used to calculate the spectral density matrix; the  
 377 spectral parameter was the same as in the third case. For both the third and the fourth cases, the auto-  
 378 spectra and the normalized cross-spectra were calculated using Eqns. (3) and (6). The cross-spectra of  
 379 the  $u$  and  $w$  components were neglected in all cases. Throughout the rest of the paper, the four cases  
 380 listed above are referred to as the ‘N400’, ‘design’, ‘modified design’ and ‘conservative’ spectra,  
 381 respectively.

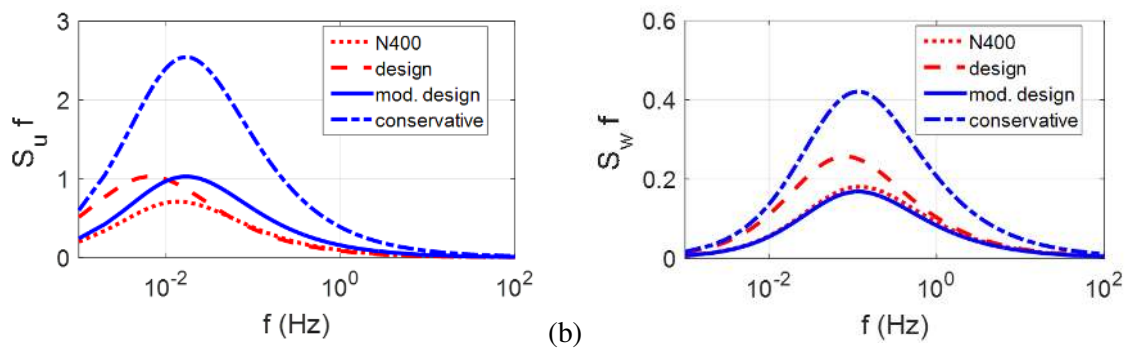
382 For comparison, the corresponding parameters given in Eqns. (3) and (6) were calculated for the N400  
 383 guidelines and for the design basis; the resulting parameters corresponding to each spectra are presented  
 384 in Table 3. The auto-spectral densities and the normalized cross-spectra for the  $u$  and  $w$  components are

385 presented in Fig. 15 and Fig. 16 using the parameters given in Table 3 and a mean wind speed of 16  
 386 m/s.

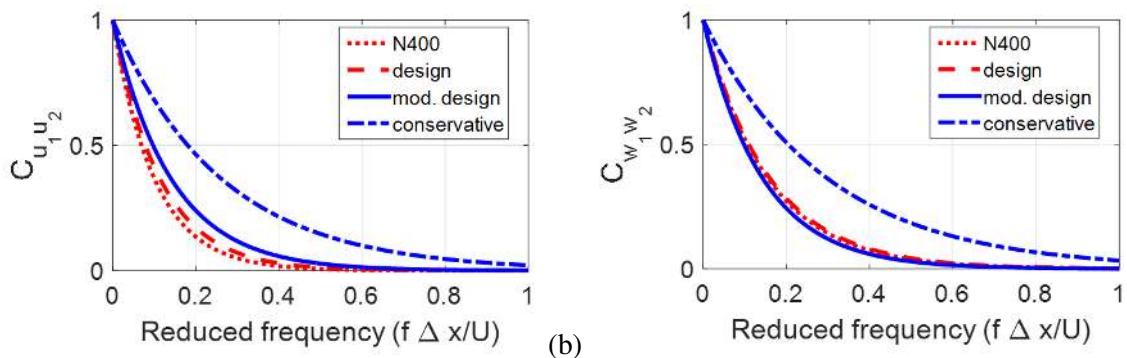
387 **Table 3.** Parameters for the spectral density and normalized cross-spectra of turbulence from Eqns. (6)  
 388 & (3)

	$A_u$	$A_w$	$K_u$	$K_w$	$I_u$	$I_w$
<b>N400</b>	19.4	2.2	10	6.5	0.113	0.057
<b>Design</b>	40.8	3.3	8.8	6.3	0.136	0.068
<b>Modified design</b>						
$0 < U \leq 10$	15.7	2.3	7.14	7.06	0.159	0.071
$10 < U \leq 15$	15.7	2.3	7.14	7.06	0.153	0.062
$15 < U \leq 20$	15.7	2.3	7.14	7.06	0.136	0.055
$20 < U \leq 30$	15.7	2.3	7.14	7.06	0.125	0.051
<b>Conservative</b>						
$0 < U \leq 10$	15.7	2.3	3.84	3.39	0.33	0.15
$10 < U \leq 15$	15.7	2.3	3.84	3.39	0.244	0.099
$15 < U \leq 20$	15.7	2.3	3.84	3.39	0.214	0.087
$20 < U \leq 30$	15.7	2.3	3.84	3.39	0.184	0.071

389



390 (a) (b)  
 391 **Fig. 15.** One-point spectra of turbulence ( $U = 16$  m/s): (a)  $u$  component and (b)  $w$  component

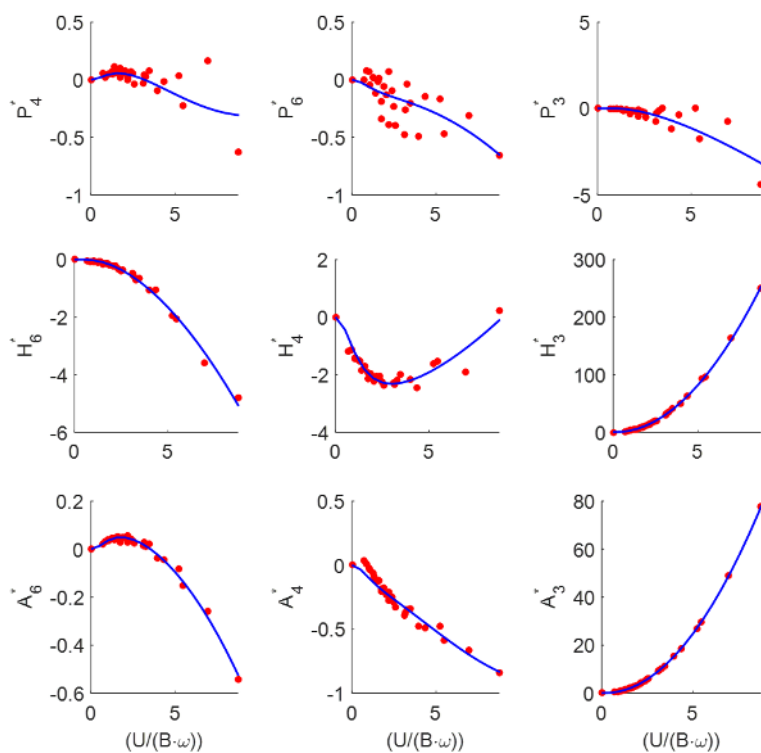


392 (a) (b)  
 393 **Fig. 16.** Normalized cross-spectra of turbulence ( $U = 16$  m/s): (a)  $u$  component and (b)  $w$  component

394 Self-excited forces

395 Motion-dependent self-excited forces were included in the analysis using the aeroelastic stiffness and  
396 damping matrices given in Eqns. (10) and (11). These matrices consist of frequency dependent  
397 aerodynamic derivatives, which are distinct properties of the bridge section that are obtained  
398 experimentally. The aerodynamic derivatives were obtained by Siedziako et al. (2017) from recent  
399 forced vibration wind tunnel tests on a section model of Hardanger Bridge. The experimental results  
400 are shown in Fig. 17 and Fig. 18 along with rational function approximations using a nonlinear least  
401 squares fit to obtain the 18 aerodynamic derivatives as continuous functions of reduced velocity. Here,  
402 it should be noted that the aerodynamic derivatives are sensitive to the curve fit where there is no  
403 experimental data points. This is the case for the torsional motion, where the interested reduced velocity  
404 range (0-0.7) is quite low. Therefore, the curve fit of the derivative  $A_2$  is forced to stay negative in that  
405 range to avoid negative damping in the buffeting analysis.

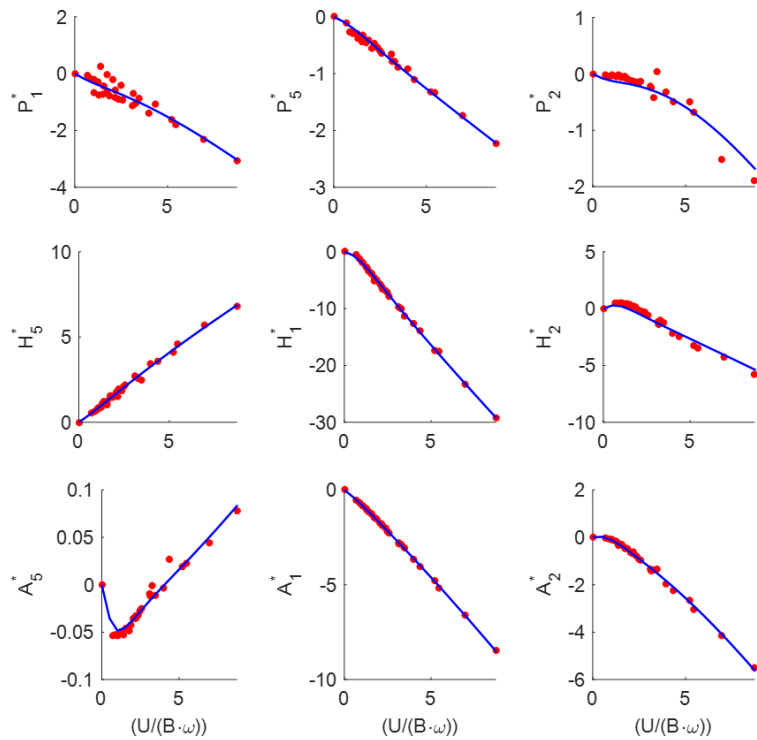
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407

408 **Fig. 17.** Aerodynamic derivatives associated with stiffness. (The dots represent experimental data and

409 the continuous curves represent the curve fit)



410

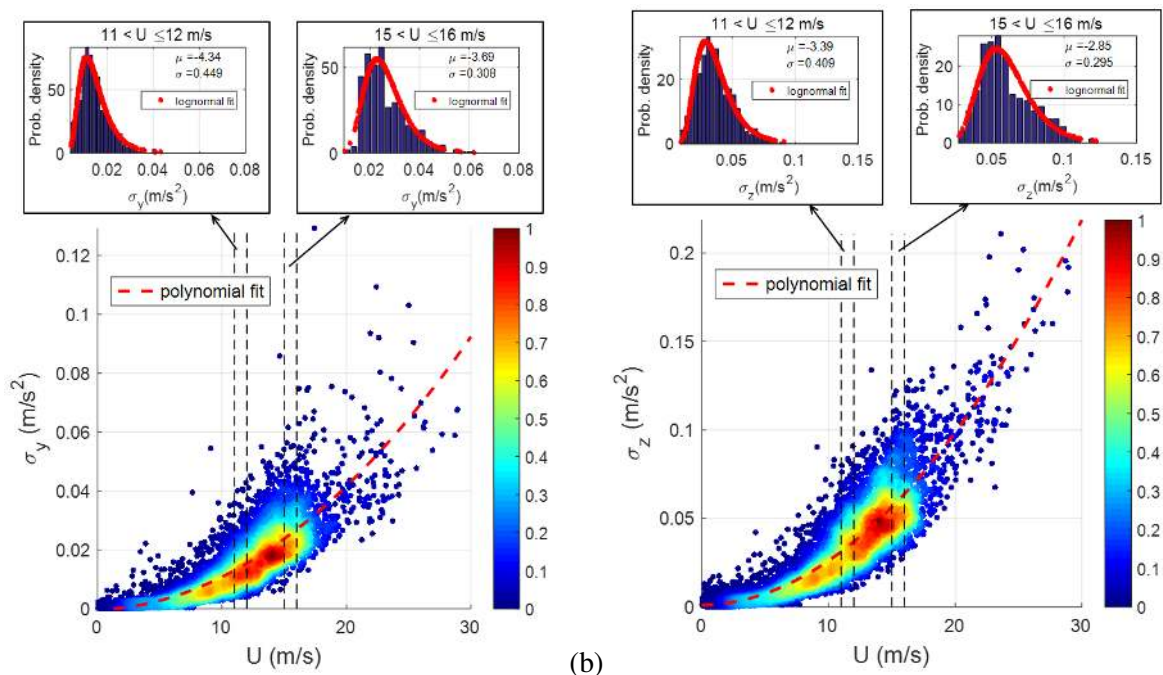
411 **Fig. 18.** Aerodynamic derivatives associated with damping. (The dots represent experimental data and  
 412 the continuous curves represent the curve fit)

413

414 6. Acceleration response at the midspan

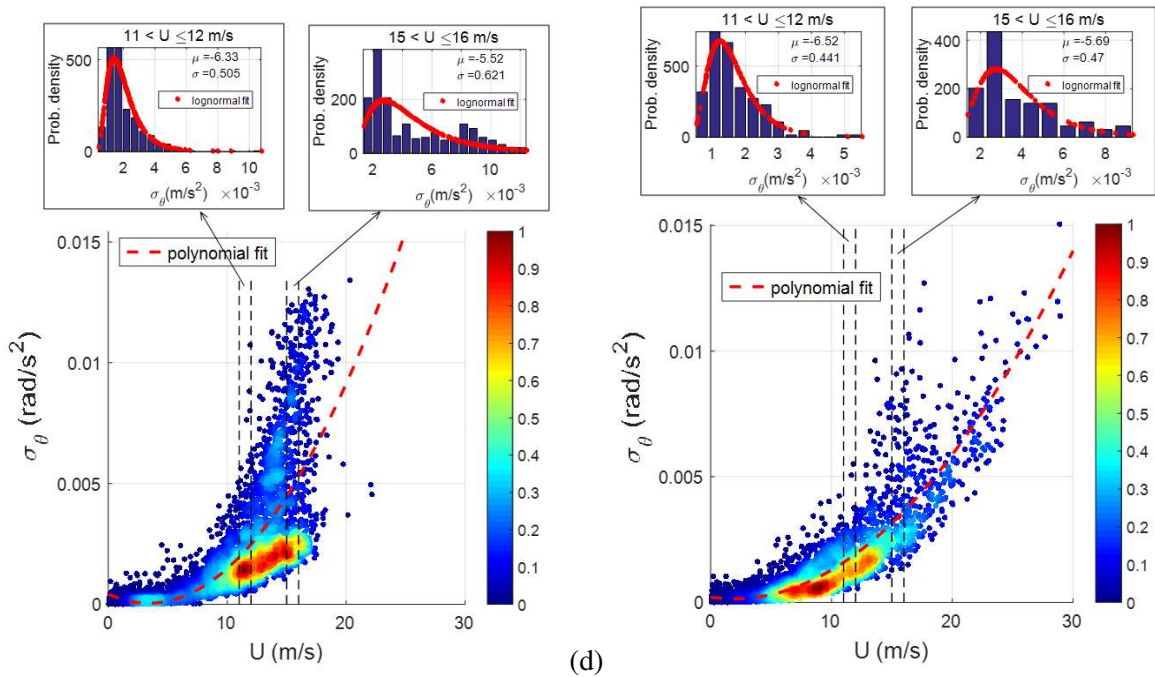
415 The standard deviations of the zero-mean turbulence components were obtained directly from the 10-  
 416 minute long time series of the lateral, vertical and torsional accelerations. The resulting root-mean  
 417 square (RMS) responses at the midspan are presented in Fig. 19 against the mean wind speed. Here,  
 418 due to the large number of data points and remarkable variability in the data, it is deemed important to  
 419 elaborate on how the scatter is distributed. For this purpose, the data points in the figures are color coded  
 420 to highlight the relative density of the data. The relative density corresponding to each data point was  
 421 calculated by dividing the plotting area into rectangular regions using a fine orthogonal grid. Then the  
 422 relative density corresponding to each data point was multiplied with the square of the mean wind speed  
 423 ( $U^2$ ) to give more weight to the data with higher wind speeds. This helps visualizing the distribution of  
 424 scatter in response for a given wind velocity. Furthermore, histograms of response components are also  
 425 given in figures for narrow velocity intervals. It is seen that the response data is in general log-normally  
 426 distributed, resembling the wind field statistics. The torsional response was plotted for the easterly and

427 the westerly winds separately due to an apparent distinction observed in two responses. The distinction  
 428 mainly arises from the differences in the upwind terrain of the two wind directions. The vertical  
 429 turbulence intensity of the easterly winds are in general much higher than the westerly winds especially  
 430 for the winds approaching from 60-100° direction range, due to the disturbance of the wind flow by the  
 431 mountains. It is observed that the distribution of scatter deviates from the lognormal distribution around  
 432 wind speeds of 15 m/s and this is more profound in the case of easterly winds. A least-squares  
 433 polynomial fit to the data is also shown in the figures to highlight the mean of the scatter.  
 434 The scatter observed in the plots are mainly due to terrain effects; however, many other factors  
 435 contribute to the variability. Nonstationary features in turbulent fluctuations, which is commonly  
 436 observed in such complex terrain can alter the wind characteristics (Chen et al. 2007; Tao et al. 2016;  
 437 Wang et al. 2016) and result into variations in the dynamic response. The effect of such features are  
 438 studied analytically by Chen (2015) and Hu et al. (2013, 2017). Although it is seen that non-stationarity  
 439 of the wind imposes variations on the response, such variations are small compared to the variability of  
 440 the wind characteristics and the response of the Hardanger Bridge. For reasonably high wind speeds ( $U$   
 441  $> 8$  m/s), rapid changes in mean wind speed or direction are not observed when an averaging interval  
 442 of 10 minutes is used. Therefore, the traditional stationary wind model was adopted in the present study,  
 443 mostly due to its common use in practice.



444 (a)

(b)



445 (c)

(d)

446 **Fig. 19.** RMS acceleration response at the midspan (color bar indicates the data density): (a) lateral  
 447 response, (b) vertical response, (c) torsional response for easterly winds, (d) torsional response for  
 448 westerly winds

## 449 7. Comparison of the acceleration response

450 The RMS acceleration response of Hardanger Bridge was calculated using the procedure described in  
 451 section 4.1. The first 100 mode shapes of the structure (0.05 – 1.6 Hz) are included in the analysis and  
 452 the RMS accelerations are obtained by numerically integrating the acceleration response spectra. In  
 453 general, the lateral response is dominated by a large peak at the first fundamental mode (0.05 Hz), where  
 454 significant contributions from higher modes can be observed in case of the vertical and torsional modes.  
 455 A damping ratio of 0.5% was assumed for the structural damping in the calculations and utilized for all  
 456 the participated modes. The damping ratio is selected based on the recommendation of N400, which  
 457 suggests the use of a damping ratio between 0.5 and 0.8% for steel structures. If a lower damping ratio  
 458 such as 3% is used in the analysis with the design method, 15%, 9% and 28% increase is observed under  
 459 30 m/s mean wind in the lateral, vertical and torsional RMS responses, respectively with the design  
 460 method. On the other hand, the use of a higher damping ratio of 1% resulted into 20%, 14% and 29%  
 461 decrease in the lateral, vertical and torsional RMS responses, respectively. It is observed that the  
 462 torsional response is the most sensitive to the structural damping because of the low aerodynamic



463 damping in torsional motion; however, such high variations in structural damping ratio are unrealistic.  
 464 The acceleration responses were evaluated for the four different turbulence spectra described in Section  
 465 5, and the resulting response prediction approaches are named after the corresponding spectra, i.e.,  
 466 ‘N400’, ‘design’, ‘modified design’ and ‘conservative’. For the modified design and conservative  
 467 approaches, the steady-state force coefficients were modified according to the mean angle-of-attack of  
 468 3°, which was obtained from the measurements in full-scale. The modified steady-state force  
 469 coefficients were obtained from the wind tunnel tests of Hansen et al. (2006), as listed in Table 4.

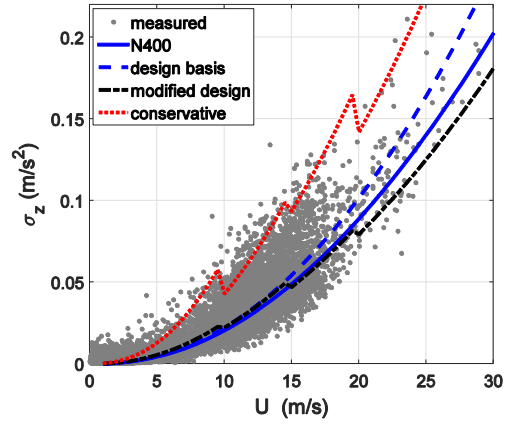
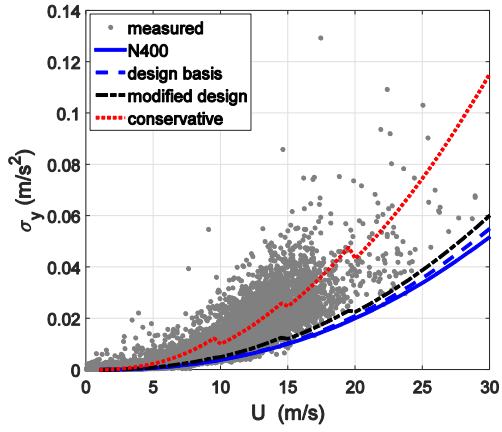
470 **Table 4.** Steady-state force coefficients for the Hardanger Bridge section for an angle-of-attack of 3°  
 471 (Hansen et al. 2006)

$\bar{C}_D$	$C'_D$	$\bar{C}_L$	$C'_L$	$\bar{C}_M$	$C'_M$
0.815	0	-0.112	2.5	0.036	0.86

472  
 473 RMS acceleration responses for the four analytical cases and the experimental data are shown in Fig.  
 474 20 and Fig. 21: Fig. 20 shows the acceleration response at the midspan (accelerometer pair H5E &  
 475 H5W), whereas Fig. 21 shows the acceleration response at approximately the quarter-span  
 476 (accelerometer pair H3E & H3W), which is 240 meters away (towards the south end) from the midspan.  
 477 The results yield very similar response levels at both points for all analytical cases and the field data,  
 478 so a common discussion is valid for both the midspan and quarter-span responses. For all response  
 479 components, the N400 method underestimated the measured response; this was somewhat improved  
 480 when the design values were used. The modified design approach resulted into similar curves as the  
 481 design approach. Compared to the design predictions, the vertical and torsional response predictions  
 482 were slightly higher for the low wind speeds and slightly lower for the high wind speeds, whereas the  
 483 lateral response predictions were similar. As expected, the conservative approach gave the highest  
 484 response predictions. Moreover, using the conditional probability distributions avoided severe  
 485 overestimation of the response at high wind speeds. The ‘conservative’ approach provided the most  
 486 satisfactory results as candidate curves for design of such structures. This observation indicates that the  
 487 wind-related variables are not independent from each other and depend strongly on the mean wind  
 488 speed. Consequently, better estimations can be obtained by considering the joint interactions of these  
 489 parameters.

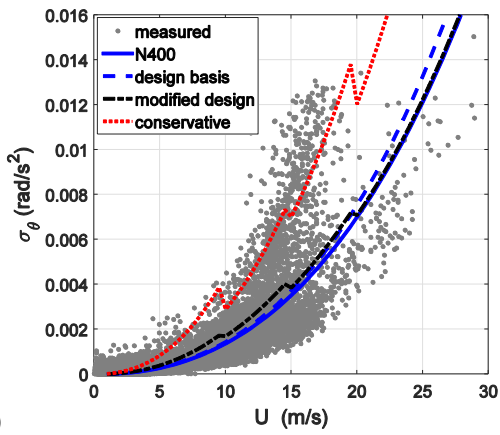
490 The analytical procedure gave systematically lower lateral response predictions compared to the  
491 measured response regardless of the adopted approach. The wind forces acting on the hangers and  
492 cables, which are expected to contribute to the lateral response, are neglected in the analyses. This was  
493 preferred due to the lack of information on the wind characteristics at the cable level; however, the  
494 degree of underestimation imposed is still of interest. In previous analytical studies, Xu et al. (2000)  
495 reported a 15% increase in lateral displacement response of Tsing Ma Bridge and Zhang (2007) reported  
496 a 20% increase for the Runyang Suspension Bridge. For the Hardanger Bridge, the drag force on one  
497 cable is around 32% of the force on the bridge deck. Considering the loading on both cables, if the  
498 analysis is repeated with a modified drag coefficient of 1.14 for the bridge deck (instead of 0.7),  
499 approximately 25% increase is obtained in the lateral response. Although this approach gives a  
500 maximum bound for the increase in response due to the cable forces, it is overly conservative since it  
501 assumes perfect correlation of wind forces at the cables and at the deck. Moreover, several researchers  
502 reported that the span-wise correlation of the wind buffeting forces is stronger than that of the wind  
503 turbulence (Jakobsen 1997; Larose and Mann 1998; Yan et al. 2016), which might partly explain the  
504 discrepancy.

505 To investigate the relative importance of the aerodynamic damping on different response components,  
506 the analysis with the design method is repeated neglecting the aerodynamic damping. The lateral and  
507 the vertical responses increased by 45% and 98% under a mean wind speed of 30 m/s, respectively. The  
508 change in torsional response was however very small; only a 0.5% increase was observed, indicating  
509 that the aerodynamic damping utilized by the use of aerodynamic derivatives was small for the torsional  
510 motion.



511 (a)  
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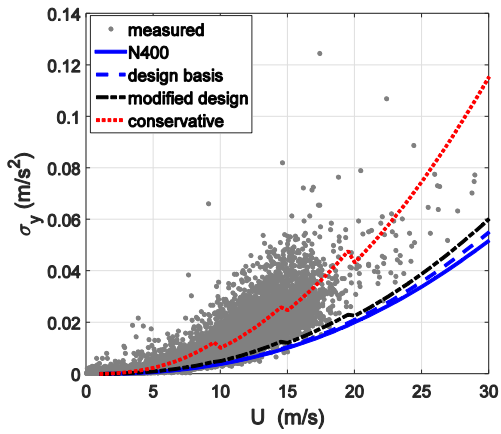
(b)



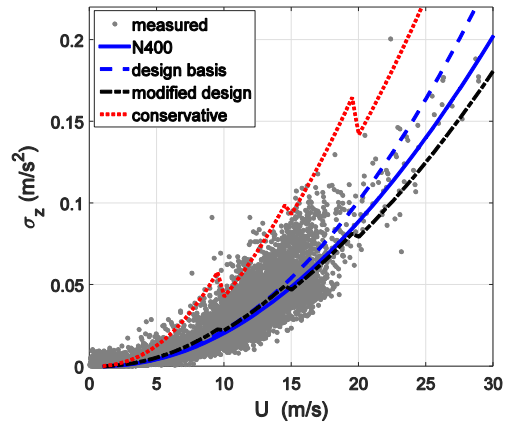
513 (c)

514 **Fig. 20.** Comparison of the RMS acceleration response at the midspan (a) lateral acceleration, (b)  
515 vertical acceleration and (c) torsional acceleration

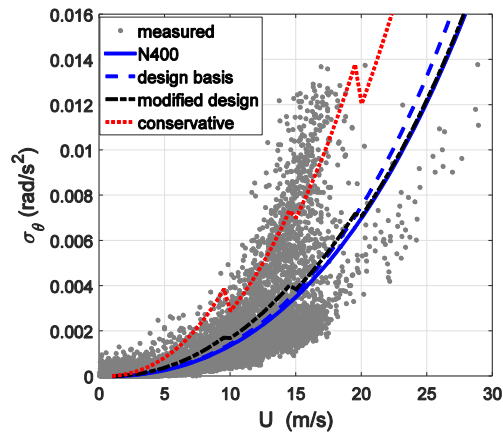
516



517 (a)



(b)



518 (c)

519 **Fig. 21.** Comparison of the RMS acceleration response at the quarter-span (at  $x = 240$  m) (a) lateral  
 520 acceleration, (b) vertical acceleration and (c) torsional acceleration

521 8. Conclusions

522 The long-term monitoring data of wind velocity and acceleration from Hardanger Bridge were used to  
 523 study the wind characteristics and to compare the acceleration response with analytical predictions. The  
 524 dynamic wind-induced response of the bridge was evaluated in frequency domain, using the design  
 525 spectrum and several modified design spectra. Comparing the results led to the following conclusions  
 526 for the specific case of Hardanger Bridge:

- 527 • The design curve underestimated the measured response. The design basis calibrated using field  
 528 measurements gave slightly better estimations compared to directly using the N400  
 529 recommendations.
- 530 • Using the 50th percentile values of the wind field parameters from the long-term monitoring  
 531 data improved the predictions; however, the resulting curves were still not considered  
 532 satisfactory design curves due to the scatter in the field data.
- 533 • The wind forces on the cables are thought to contribute to the dynamic response of the bridge,  
 534 especially for the lateral vibration response. More sophisticated analyses that include the cables  
 535 and hangers are necessary to account for such effects on the overall response prediction.
- 536 • A conservative approach that used 95th or 5th percentile values of the wind field parameters  
 537 from the monitoring data provided more desirable design curves. Overestimating the response

538 at high wind speeds was avoided by using conditional probability distributions for the  
539 turbulence intensities.

- 540 • The results imply that the wind field variability should be considered in the buffeting response  
541 analysis of such structures. Furthermore, the interactions among the wind field parameters  
542 should be considered, preferably using joint probability distributions or conditional probability  
543 distributions.

## 544 9. Acknowledgments

545 The research described in this paper was financially supported by the Norwegian Public Roads  
546 Administration.

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