Wind and extremely long bridges – a challenge for computer aided design

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Summary

With ever increasing span lengths of bridges thoroughly investigation of wind related phenomena has become more and more a topic of interest. Many effects were observed on already existing bridges and suitable theoretical models were developed. As bridges with such span lengths are becoming more and more a standard situation in bridge design and engineering, also the need for according design tools increases. In this article the necessary steps to expand existing models and to incorporate them into the design process is outlined. Practical examples for different aspects are discussed.

Keywords: Wind, CFD, vortex shedding, flutter, buffeting.

1. Introduction

Modern construction techniques and materials as well as increasing experience and expertise in bridge design allow for still increasing span lengths of large bridges. The needs of well established infrastructure and the trend to make our ways as short as possible demand to construct bridges where it would not have been possible a few decades ago, for example across wide spans of open water. In principle this improvement process is based on the development of new methods and/or materials, followed by a phase in which this new development is pushed to the limit of applicability.

Most of the bridges of such enormous span length are also subject to strong wind forces due to their exposed placement. Because of their slenderness and related dynamic behaviour it is no longer sufficient to treat wind gusts and other fluctuations by equivalent static wind forces. Instead different investigation methods developed for such extreme situations must be applied to examine the interaction of oncoming wind and bridge. Many of these methods were inspired and motivated by observations made on existing bridges or by comparison to similar effects in the aeronautic industry.

An important topic in wind analysis is the data management and information interchange. Because of the complexity of the task, many engineers working on different fields must work closely together. This starts with the measurement and evaluation of meteorological data to describe the local wind situation and ends with final wind check calculations and according decisions. In this sense, not only the analysis methods, but also the data storage and interchange model plays an important role to accomplish an efficient design process. To this end, a software implementation which serves as data container as well as analysis tool was developed. In this paper, the used concepts and methods are presented, and different aspects of the calculation are discussed with practical examples.

2. Numerical modelling of interaction of bridges with wind

The first step to the numerical modelling is a careful investigation of the airflow around the concerned bridge cross sections. This is done by applying a CFD module based on the Discrete

Vortex Method (DVM). By applied post processing statistical evaluation, a set of steady state aerodynamic coefficients and flutter derivatives can be obtained.

The subsequent wind buffeting analysis separates static and dynamic wind force contributions. The static part can be applied as distributed constant load. The dynamic wind load can be split into aerodynamic damping and stiffness and contributions due to fluctuating wind. The structural response is calculated by transforming the equations into modal space and frequency domain. By providing suitable wind profile data the excitation power spectrum can be calculated and the structure peak response can be estimated by statistical methods.

By solving simplified versions of the buffeting and flutter equations, wind checks can be obtained for galloping, torsional divergence, torsional flutter and classical flutter phenomena. According critical wind velocities can be estimated.

3. Application example

Calculations were performed for the Hardanger bridge in Norway. Some results of the wind buffeting analysis are shown in Fig. 1.

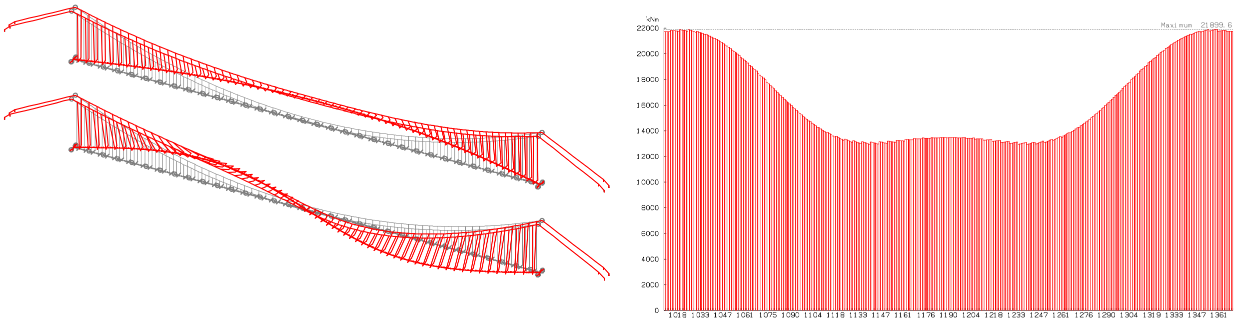

Fig. 1. First two eigenmodes (left) and internal longitudinal twisting moment due to dynamic wind (right) of Hardanger bridge.

4. Summary and conclusions

In this paper the necessary steps for a complete computer aided wind design of long span bridges were discussed. The theory of the analysis process can be split into three parts: first the air flow around the structures cross sections are characterized by means of CFD methods. The structural response is then estimated with a wind buffeting analysis based on a quasi-steady theory. Additionally, wind design checks can be applied to estimate critical wind velocities.

For the design process it is important to provide the possibility to perform the calculations for different cross section variations as well as wind scenarios. This has been taken into account by providing two storage containers for aerodynamic coefficients (“aero class”) and wind profiles. This information can be arbitrarily combined to check every possible situation.

The developed computer program was applied to perform different wind calculations for the planned Hardanger suspension bridge. A comparison of the deck twisting moment for static and dynamic wind revealed the importance of such wind buffeting calculations.

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With ever increasing span lengths of bridges thoroughly investigation of wind related phenomena has become more and more a topic of interest. Many effects were observed on already existing bridges and suitable theoretical models were developed. As bridges with such span lengths are becoming more and more a standard situation in bridge design and engineering, also the need for according design tools increases. In this article the necessary steps to expand existing models and to incorporate them into the design process is outlined. Practical examples for different aspects are discussed.

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1. Introduction

Modern construction techniques and materials as well as increasing experience and expertise in bridge design allow for still increasing span lengths of large bridges. The needs of well established infrastructure and the trend to make our ways as short as possible demand to construct bridges where it would not have been possible a few decades ago, for example across wide spans of open water. In principle this improvement process is based on the development of new methods and/or materials, followed by a phase in which this new development is pushed to the limit of applicability. An example of such an evolution is shown e.g. in Fig. 1 for suspension bridges.

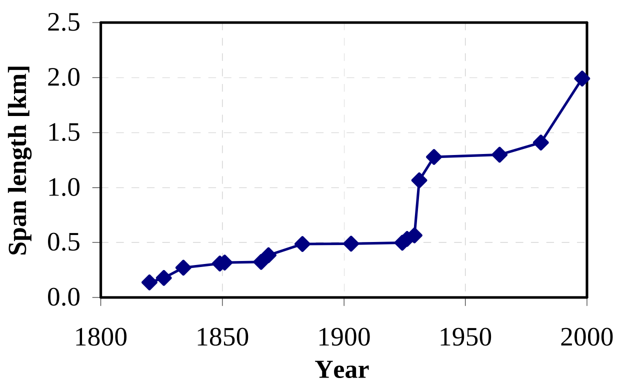


Fig. 1. *Evolution of span length of suspension bridges [1].*

Most of the bridges of such enormous span length are also subject to strong wind forces due to their exposed placement. Because of their slenderness and related dynamic behaviour it is no longer sufficient to treat wind gusts and other fluctuations by equivalent static wind forces. Instead different investigation methods developed for such extreme situations must be applied to examine the interaction of oncoming wind and bridge. Many of these methods were inspired and motivated by observations made on existing bridges or by comparison to similar effects in the aeronautic industry.

An important topic in wind analysis is the data management and information interchange. Because of the complexity of the task, many engineers working on different fields must work closely together. This starts with the measurement and evaluation of meteorological data to describe the local wind situation and ends with final wind check calculations and according decisions. In this sense, not only the analysis methods, but also the data storage and interchange model plays an important role to accomplish an efficient design process. To this end, a software implementation which serves as data container as well as analysis tool was developed. In this paper, the used concepts and methods are presented, and different aspects of the calculation are discussed with practical examples.

2. Numerical modelling of interaction of bridges with wind

The most complete description of the interaction would be a fully consistent calculation by coupling the equations of motion of the bridge with the Navier-Stokes equations which govern the time evolution of the surrounding air flow. The boundary condition which establishes the coupling is the no slip condition at the bridge surface. Additionally, specific inflow conditions which possess the correct statistical properties according to the local weather conditions by a suitable set of parameters must be prescribed. This very involved calculation is not possible for memory and performance reasons on standard computers.

Instead the problem is divided into several smaller sub-problems by making simplifying, but well justified, assumptions. The main step is to break down the analysis into a preliminary calculation of the air flow around 2D fixed or moving cross sections and a subsequent structural analysis part. The reduction of 3D to 2D is in general applicable, because one dimension – the main span, or the height of the pylons – of a structural part is dominant with respect to the other two dimensions, cf. Fig. 2 for the Stonecutters bridge. Since the structural analysis is based on a modal approach, the air flow can be adequately described by characterizing the wind forces for fixed cross sections and certain type of movement.

Fig. 2. Model of Stonecutters Bridge.

2.1 Aerodynamic characterization of 2D cross sections

The numerical investigation of the airflow around the considered cross sections is based on a suitable discretization of the Navier-Stokes equations for viscous fluids. For the investigations presented in this paper a Discrete Vortex Method (DVM) was applied. The advantage of this class of methods is that they work grid free, which is very desirable when calculating moving cross sections. Details on the implementation and application to the Hardanger bridge as well as several bridges in Brazil are given in [2] and [3].

From an engineering point of view the most important feature of the implemented CFD method is that it is embedded within the structural analysis part. This allows for an easy transfer of cross sectional data as input and the calculation results as output. Next to geometric data only a few calculation parameters as well as the range of wind directions to be considered must be passed to the CFD module. The necessary calculations are performed in an automated batch mode with almost no necessary user interaction.

Basic output of the CFD calculations are time histories of drag and lift force and overturning moment. Even for smooth oncoming flow and fixed cross sections these time histories show oscillation patterns due to vortex shedding in the wake of the considered cross section as indicated in Fig. 3. Within the following post-processing procedure these time histories are reduced to a set of characteristic coefficients for the cross section. For static wind calculations it is sufficient to provide the mean wind forces and moment acting on the structure.

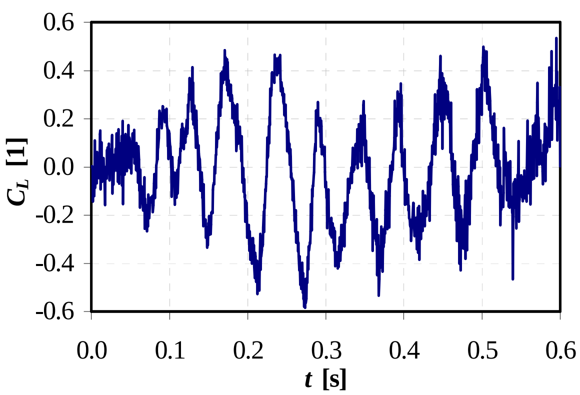


Fig. 3. Typical time history of lift coefficient.

They are given in non-dimensional form according to the steady state coefficients C_D, C_L and C_M

$$C_D = \frac{D}{\frac{1}{2}\rho U_\infty^2 H} \qquad C_L = \frac{L}{\frac{1}{2}\rho U_\infty^2 B} \qquad C_M = \frac{M}{\frac{1}{2}\rho U_\infty^2 B^2} \tag{1}$$

where normally the cross section height H is chosen as normalization length for drag and the width B for lift and moment. The air density is indicated by r and is assumed to be constant for all calculations, and the mean wind velocity is given by U_∞.

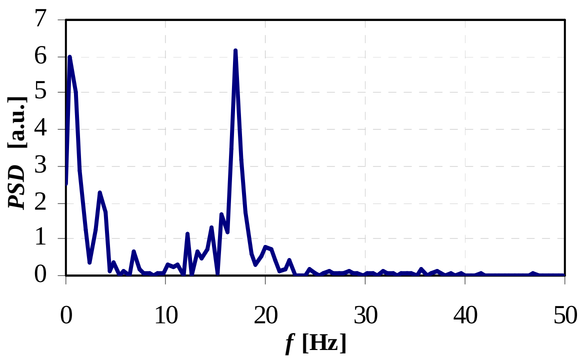


Fig. 4. Power spectrum of lift coefficient.

By considering the power spectrum of the time history, cf. Fig. 4, often pronounced peaks at non-zero frequencies are observed. The most dominant peak indicates the vortex shedding frequency. It is expressed in a non-dimensional manner by the Strouhal number

$$St = \frac{fH}{U_\infty} \tag{2}$$

For dynamic calculations the forces must be evaluated also for moving cross sections. In the case of a modal approach to the structural response calculations, it turns out that it is sufficient to consider harmonic oscillations of the cross section. Following [4], the time dependent forces can be obtained by a linear combination of the cross section displacement and velocity according to

$$\begin{aligned}
D &= \tfrac{1}{2}\rho U_\infty^2 H\left(KP_1^* \frac{\dot{p}}{U} + KP_2^* \frac{B\dot{\alpha}}{U} + K^2 P_3^* \alpha + K^2 P_4^* \frac{p}{B}\right)\\
L &= \tfrac{1}{2}\rho U_\infty^2 B\left(KH_1^* \frac{\dot{h}}{U} + KH_2^* \frac{B\dot{\alpha}}{U} + K^2 H_3^* \alpha + K^2 H_4^* \frac{h}{B}\right)\\
M &= \tfrac{1}{2}\rho U_\infty^2 B^2\left(KA_1^* \frac{\dot{h}}{U} + KA_2^* \frac{B\dot{\alpha}}{U} + K^2 A_3^* \alpha + K^2 A_4^* \frac{h}{B}\right)
\end{aligned} \tag{3}$$

where p, h and α indicate horizontal, vertical and torsional displacement, respectively. The reduced circular frequency of oscillation is given by $K = B\omega/U_\infty$. The linear coefficients P_i^*, H_i^* and A_i^* in Eq. (3) are the so-called flutter derivates.

2.2 Wind buffeting forces and analysis

The term wind buffeting refers to the forces acting on the structure due to turbulent velocity fluctuations. In most of the applications it is justified to assume that the velocity fluctuations and resulting structural displacement velocities are small compared to the mean wind velocity. In this case the total forces can be derived from a quasi-steady theory based on the steady state coefficients. This approach is presented for example in [5] and [6]. The overall wind forces can be written as

$$\begin{pmatrix} D \\ L \\ M \end{pmatrix} = \frac{1}{2}\rho U_\infty^2 \begin{pmatrix} HC_D \\ BC_L \\ B^2 C_M \end{pmatrix}_{\mathrm{st}} + \rho U_\infty \begin{pmatrix} HC_D u + \frac{1}{2}\left(HC_D' - BC_L\right) w \\ BC_L u + \frac{1}{2}\left(BC_L' + HC_D\right) w \\ B^2 C_M u + \frac{1}{2} B^2 C_M' w \end{pmatrix}_{\mathrm{dyn}} - \rho U_\infty \begin{pmatrix} HC_D \dot{r}_z + \frac{1}{2}\left(HC_D' - BC_L\right)\dot{r}_y \\ BC_L \dot{r}_z + \frac{1}{2}\left(BC_L' + HC_D\right)\dot{r}_y \\ B^2 C_M \dot{r}_z + \frac{1}{2} B^2 C_M' \dot{r}_y \end{pmatrix}_{\mathrm{damp}} + \rho U_\infty \begin{pmatrix} \frac{1}{2} HC_D' \vartheta_x \\ \frac{1}{2} BC_L' \vartheta_x \\ \frac{1}{2} B^2 C_M' \vartheta_x \end{pmatrix}_{\mathrm{stiff}} \tag{4}$$

In this equation, the coefficients C_i are evaluated for the mean angle between wind and cross section, and C_i' indicates the slope at this angle. The first term takes into account the mean wind contributions, the second one the additional forces due to wind fluctuations. The third term is connected to the cross section velocity and can thus be considered together with structural damping; it is therefore also called “aerodynamic damping”. Analogously, the last term is called “aerodynamic stiffness, because it depends on the cross section deflection.

This situation is indicated in Fig. 5. The overall effective wind velocity and incident angle results from a vectorial addition of the single velocity components. For evaluating the forces according to Eq. (4), only a linearization is applied.

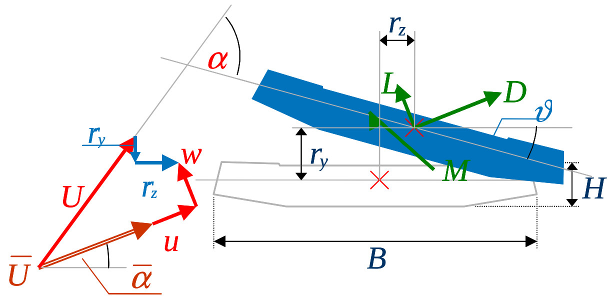


Fig. 5. Effective wind velocity and angle.

To handle the resulting equations of motion of the bridge structure numerically, the time-dependent deflections of the bridge are represented by a linear combination of suitable basis functions with time-dependent coefficients in the framework of a modal approach. To this end, the eigenfrequencies and –modes of the structure with applied permanent loading are calculated together with the linearized modal stiffness matrix. By consecutive weighting of the equations of motion with the basis functions, a set of differential equations for the linear coefficients is obtained. By performing a Fourier transformation from time to frequency domain, one end up with a linear set of equations for the Fourier transforms of the linear coefficients. This procedure is presented in detail for example in [7] and [6].

The terms in Eq. (4) which model damping and stiffness are easily included into the frequency domain representation. By combining the structural and aerodynamic contributions, the so-called mechanical admittance function is obtained. The problem is that a power spectrum for the forces due to velocity fluctuations must be established. To this end, a statistical description of the possible wind events must be provided. The basic parameters are the mean wind velocity, turbulence intensity, wind power spectrum and coherence. For practical reasons, the available data is commonly fitted to different standard models. By providing the necessary model parameters, the so-called joint acceptance function [7] can be evaluated which accounts for the span-wise statistical averaging of the different wind events.

By combining the mechanical admittance and joint acceptance function, the power spectrum of the linear coefficients and hence the standard deviation of deflection of the corresponding can be obtained. If it is assumed that the wind events can be characterized by Poisson statistics, the peak response is given by the product of standard deviation and a suitable peak factor. Finally, a statistical superposition of the single modes is used to estimate the overall peak response.

2.3 Wind design checks

Wind design checks are usually based on simplified versions of the full buffeting equations. If aerodynamic damping and stiffness are expressed by the flutter coefficients, one speaks of the flutter equations, and simplified solutions can yield insight into the dynamic behaviour of the considered structure. Within the developed program presented in this paper, the following checks were implemented.

2.3.1 Galloping

If only the lowest eigenmode with mainly vertical displacement is considered, a sustained oscillation of the deck is possible. A necessary condition for this so-called galloping phenomenon is that the total damping becomes negative. This is only possible if the Glauert-Den Hartog criterion holds: $C_L' + H/B\, C_D < 0$ [8].

2.3.2 Torsional divergence

This is a static effect related to the twist of the deck due to the mean wind overturning moment. If the moment increases with increasing twisting angle, there will be a critical point, at which the reactive moment of the deck is not sufficient to counterbalance the wind induced moment. This happens at a critical wind velocity for divergence, which is given by

$$U_{c,\mathrm{div}} = \sqrt{\frac{2I\omega^2}{\rho B^2 C'_M}} \tag{5}$$

where I is the moment of inertia of the considered cross section and ω the circular frequency of the lowest torsional eigenmode.

2.3.3 Torsional flutter

As for torsional divergence, a pure torsional deflection of the deck is considered in this case. However, the analysis is based on a dynamic approach via the flutter derivatives. By considering the lowest mainly torsional eigenmode it can be observed that the aerodynamic damping depends on the flutter coefficient A_2^*. By again considering vanishing total damping as the critical point, a critical value for this coefficient can be determined depending on the structural damping ratio to critical ζ:

$$A^*_{2,crit} = \frac{4\zeta I}{\rho B^4} \tag{6}$$

By looking up the reduced velocity at which this critical value is reached, the critical wind velocity can be deduced. As can be seen, it is a necessary condition for torsional flutter that A_2^* becomes positive. This behaviour is usually not found for plate-like girders, but can be observed for some open-truss girders, e.g. the Golden Gate Bridge.

2.3.4 Classical flutter

The term classical flutter refers to a coupled excitation of the lowest vertical and torsional eigenmode. The solution of the flutter equations involves the flutter coefficients H_i^* and A_i^* in this case. The searched critical velocity is again the point of transition from decaying to undamped or sustained oscillation.

2.3.5 Vortex shedding

The vortex shedding phenomenon is accompanied by large oscillating lift forces of the same frequency as the shedding. Related to this problem is the so called lock-in effect, which happens if the vortex shedding frequency lies close to a natural frequency of the structure. When the corresponding eigenmode is excited, the vortex shedding frequency will be pinned to the driving frequency and massive self interaction can be observed. To estimate the effect, vortex shedding velocities $U_{c,s}$ can be derived from Eq. (2) for the different natural frequencies.

2.4 Data information modelling

In order to perform large scale engineering calculations, the developed computer program must be capable to deal with structures consisting of several cross sections and different wind events, because all different possibilities must be checked for maximum security. To this end, two data containers were introduced to encapsulate the required information.

2.4.1 Aero classes

This container is designed too hold all cross section related aerodynamic data. This includes the steady state coefficients as well as flutter derivatives and Strouhal number. Since the normalization lengths may not correspond to width and height of the used cross section, additional factors may be entered. Moreover storage space is reserved for all structural constants which are needed for the different design checks.

2.4.2 Wind profiles

A wind profile contains the statistical description of an arbitrary stochastic wind event. The basic information is stored to describe the mean wind velocity profile, turbulence intensity, wind power spectra for the three velocity components and wind coherence data. For each type of wind information different theoretical models are available. Furthermore, the local construction site wind density can be specified for de-normalization of the aerodynamic coefficients.

3. Practical examples

The developed computer program is applied for the Hardanger bridge, a suspension bridge with a main span of 1310 m and a total length of 1380 m which crosses the Hardanger fjord in Norway. It will be number one in Norway and seven world wide. The opening is planned for 2011. The main deck will be 18.3 m wide, including two driving lanes and a separated cyclist and pedestrian path. The Pylons will be more than 180 m high and the maximum clearance for ship vessels will be around 50 m.

The very large ratio of main span to side spans (cf. Fig. 6) is due to the fact that the shore of the fjord drops very deeply, so that the pylons must be placed close to the water side. The construction will be done by lifting the individual girder segments and temporal connection to each other with hinges. Once all elements are lifted the final welding will be performed. During this time, the main girder will be even more susceptible to wind induced vibrations.

Fig. 6. Structural model of Hardanger bridge

The cable lengths for main cable and hangers were calculated by software by applying a set of constraints for sag and cable forces [9] for the bridge in service state. The final geometry of the main girder will not be straight but with a constant radius in elevation. This vertical radius is achieved by a constant bending moment induced between the pylons and the hangers closest to the pylons.

Basic considerations concerning the steady state coefficients of the main girder and wind shadow effects at the pylon legs were presented in [2]. Further CFD investigations were performed for the main girder with traffic. The two driving lanes are loaded with traffic according to the sketch presented in Fig. 7. Three different cases were considered: with both lanes loaded, only with the left lane and only with the right lane.

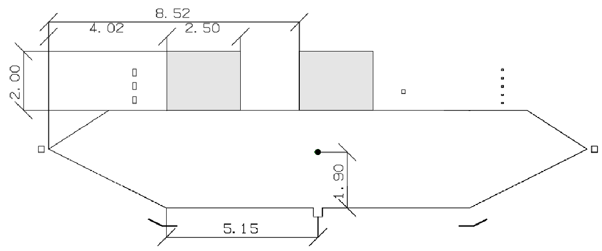


Fig. 7. Cross section for traffic calculation.

Since the application of traffic causes a symmetry break of the cross section layout, the CFD calculations must be performed for wind coming from the left (-10° to 10°) as well as from the right (170° to 190°). The computed results for the case with both traffic lanes are indicated in Fig. 8.

It can be observed that the slope of the lift coefficient for wind coming from the left is negative for negative angles. By evaluating the Glauert-Den Hartog criterion, slightly negative values are obtained, which indicates a tendency for galloping.

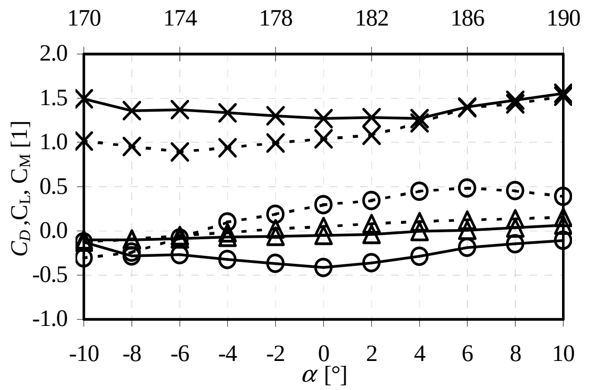
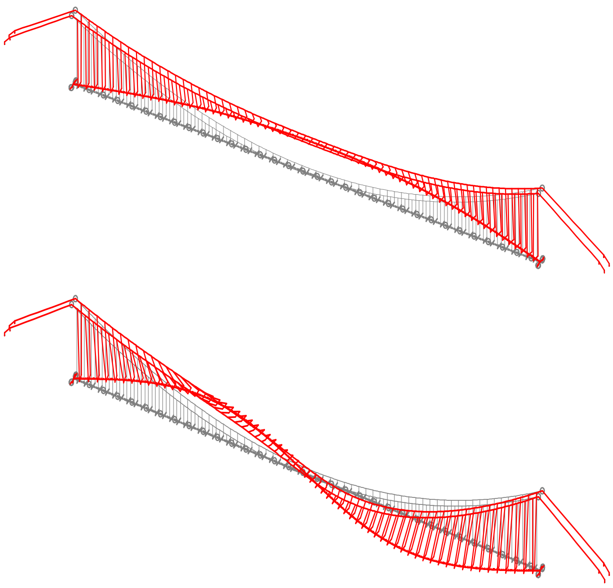


Fig. 8. Steady state coefficients C_D *(x),* C_L *(o) and* C_M *(Δ) for traffic on both lanes for wind from left (solid) and right (dashed).*



According to the considerations presented above, a set of eigenmodes under permanent loading was calculated for the bridge. The first two modes are shown in Fig. 9. Based on this set of basis functions, the buffeting analysis was performed for a wind profile where the mean wind is given by a logarithmic distribution and the power spectral density is of Kaimal type.

Some results of the buffeting analysis for later wind are presented in Fig. 10. By comparing with results for static wind only, it is observed that static and dynamic lateral forces are of same magnitude. The twisting moment is larger for the dynamic wind, because due to the fluctuating vertical wind component the effective wind incident angle varies more than due to static effects only. Thus the overall twisting of the deck is amplified and the internal moment is consequently higher

Fig. 9. First two eigenmodes of Hardanger bridge deck

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Fig. 10. Internal shear force(left) and longitudinal twisting moment (right) due to lateral wind.

4. Summary and conclusions

In this paper the necessary steps for a complete computer aided wind design of long span bridges were discussed. The theory of the analysis process can be split into three parts: first the air flow around the structures cross sections are characterized by means of CFD methods. The structural response is then estimated with a wind buffeting analysis based on a quasi-steady theory. Additionally, wind design checks can be applied to estimate critical wind velocities.

For the design process it is important to provide the possibility to perform the calculations for different cross section variations as well as wind scenarios. This has been taken into account by providing two storage containers for aerodynamic coefficients (“aero class”) and wind profiles. This information can be arbitrarily combined to check every possible situation.

The developed computer program was applied to perform different wind calculations for the planned Hardanger suspension bridge. A comparison of the deck twisting moment for static and dynamic wind revealed the importance of such wind buffeting calculations.

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