LONG SPAN BRIDGES
COMPUTER AIDED WIND DESIGN

JANJIC, Dorian
Managing Director
TDV GmbH
Graz, Austria
office@tdv.at

Dorian Janjic, born 1960, civil engineering degree from the Faculty of Civil Engineering, Sarajevo. 15 years of experience in technical research, software development.

Summary

The challenges for long-span bridges such as stay cable or suspended bridges with high pylons and slender steel or concrete decks, are mainly related to optimizing the stressing sequence of the cables, to the geometrically non-linear behaviour of the structure, and to dynamic problems such as wind-induced vibrations.

Referring to geometric non-linearity, we realise, that using the linear elastic theory is mostly sufficient for small and medium size bridges. The accuracy of the linear results is usually adequate, since non-linearity effects are small and can be accounted for by safety factors or even be ignored. Some non-linearity effects can – if required – also be locally taken into account in post-processing (design check) procedures. However, when analysing long-span bridges, especially cable suspended bridges, non-linearity effects often reach magnitudes, which cannot be neglected in the structural analysis.

This paper presents the findings of a research project investigating the wind impact. Long-span bridges require sophisticated wind buffeting analyses with considering both, the aero-elastic behavior of the structure and the wind loading correlation.

Further, this article deals with the application of Computational Fluid Dynamics (CFD) Methods for static and dynamic wind-effect analysis. The main attention will be paid to the consideration of suspension bridges with long spans, because they are very susceptible for wind induced oscillations. We will go into Discrete Vortex Method (DVM) for determination of the aerodynamic features more into detail.

The theoretical background is outlined and the implementation into the bridge design software is briefly described. A practical application example illustrates the described principles.

Keywords
Geometric non-linearity, p-delta, cable sagging, aero-dynamics, wind buffeting, aero-elastic damping, Stonecutters bridge, Shenzen Western corridor, Sutong Bridge

1 INTRODUCTION

As a matter of fact, a very long span length results in a more flexible structural system, which makes the complicated static as well as dynamic behaviour. Of course, excessive deflections and vibrations due to strong wind may occur.

Upgraded construction methods and novel materials allow increasingly bridge construction with extraordinary long spans. Contemporaneously the height to width proportion of the main girder cross section can be reduced more and more – at least considering only static requirements. However, the construction becomes more susceptible for wind induced oscillations and instabilities. Therefore increased attention must be paid to the bridge-wind interaction during the planning process in order to ensure stability at local design wind velocities.

Wind tunnel measurements have been proved to be a very credible method for examination of these interactions during the planning process. However two problems arise which should not be disregarded. On the one hand these measurements cause high expenses and on the other hand it comes to time delay in the planning process because they are usually not performed in-house of design companies.

For this reasons *Computational Wind Engineering* (CWE), which is performed as preparation or simultaneously to wind tunnel measurements, is becoming more and more a method of choice. The computer aided modelling provides generally the opportunity to calculate the influence of cross-section modifications on the wind-bridge interaction fast and straightforward. By using adequate numerical and analytical models both steady state and dynamic wind loading can be calculated. In the present case a *Discrete Vortex Method* (DVM) serves to calculate the airflow around considered cross sections. Design checks and calculation of critical velocities are performed for classical flutter, one-dimensional torsional flutter, across wind oscillation and torsional divergence.

As method of choice DVM is implemented for resolving the Navier-Stokes equations, dealing with the aerodynamic coefficients which are essential for the design checks. Finally, practical examples are discussed.

During the design process the engineer is looking for the best solution for given criteria by changing specific system parameters. Engineering experience helps to reduce the time required, but there will still be a need for many iteration steps until the design criteria are met. Computer programs nowadays should provide the best possible support for this design process.

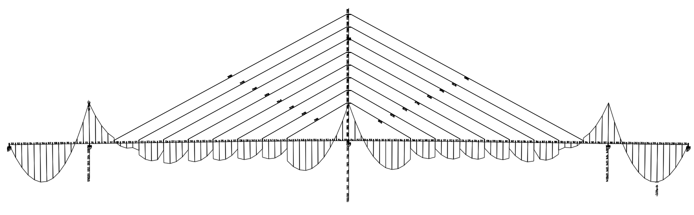


Figure 1: Target bending moment at the final stage should be achieved with construction sequence.

The construction sequence combined with long-term effects has an influence on the target engineering design. Within structural analyses it is necessary to account for long-term effects in the calculation and to minimize undesired influences.

The determination of the structural eigenmodes and eigenvalues under the current total loading in the displaced structural shape is an important step in the bridge design process. Because of the flexible structural system all computations are based on the tangential stiffness of the structure at a given point in time – the structure under permanent loading and mean wind – allowing for including all prior non-linear effects. The usage of the structural “tangent stiffness” results in the best possible linearization for the subsequent dynamic analysis steps in the frequency domain.

2 Discrete Vortex Method

The Navier-Stokes equations for incompressible fluids of constant density ρ and temperature provide a basis for the numerical treatment of the air flow. By taking the curl, the so-called *vorticity transport equation* (VTE) is obtained, which reads for two-dimensional cross sections:

$$\frac{\partial \omega}{\partial t} + (u \cdot \nabla)\omega = \nu \nabla^2 u \quad (1)$$

Here, the vorticity ω is the curl of the velocity u and ν denotes the kinematic viscosity of air. The vortex field ω is approximated by a large number of vortex particles of given circulation Γ_i and size σ located at x_i:

$$\omega(x) = \sum_i \delta_\sigma (x - x_i)\Gamma_i . \qquad (2)$$

The perimeter of the cross section is divided into a given number of straight boundary elements along which either vanishing tangential velocity (*no-slip*) or vanishing normal velocity (*no-penetration*) is demanded. To resolve the boundary conditions, the vorticity field at the perimeter is modelled by a surface circulation which varies linearly along the surface panels. The solution is made unique by imposing conservation of total circulation.

A numerical solution to the VTE is obtained by applying a fractional step method to the convection term (second term on left hand side) and diffusion term (right hand side). The convection term is treated either with forward Euler differencing or second order Runge-Kutta. The velocity is recovered from the vorticity with the Biot-Savart relation

$$u(x) = U_\infty - \frac{1}{2\pi} \int \frac{\omega(x_0) \times (x_0 - x)}{|x_0 - x|^2} dx_0 \qquad (3)$$

where the oncoming velocity is given by U_∞. Diffusion is treated by a random walk method.

The application of DVM in bridge engineering was expatiated e.g. by Morgenthal (2002) und Walther (1994).

3 AERODYNAMIC COEFFICIENTS

3.1 Steady State Coefficients

A dimensional analysis of the VTE (1) shows that there exists an unique solution for all flows which are characterized by the same Reynolds number

$$\mathrm{Re} = \frac{U_\infty \ell}{\nu} \qquad (4)$$

where, ℓ is a typical length scale for the considered cross-section. This fact is used advantageously for wind tunnel experiments where scale models of the cross section are examined. The forces measured for the considered models can be renormalized to obtain the forces on the bridge.

In order to calculate the static wind load, the dimensionless steady state coefficients C_D, C_L and C_M are considered. They are given as ratio of wind load and force due to dynamic pressure:

$$C_D = \frac{D}{\frac{1}{2}\rho U_\infty^2 \ell_D}, \quad C_L = \frac{L}{\frac{1}{2}\rho U_\infty^2 \ell_L}, \quad C_M = \frac{M}{\frac{1}{2}\rho U_\infty^2 A} \qquad (5)$$

The drag force D acts along the wind direction while L denotes the lateral lift force normal to the wind direction. The overturning moment is given by M. The mass density of air is described by ρ. Normally the normalization lengths ℓ_D and ℓ_L are equal to the height H and width B of the cross section. The moment coefficient is usually normalized with $A = B^2$.

The dominant frequency of vortex separation at the cross section is expressed by the dimensionless Strouhal number St. Normally this frequency is equal to the frequency at which alternating across wind forces act on the cross section. The Strouhal number is given by

$$\mathrm{St} = \frac{f \ell_{\mathrm{St}}}{U_\infty} \qquad (6)$$

where f denotes the frequency and ℓ_{St} is the normalization length which equals the height H of the cross section in most cases.

3.2 Flutter Derivatives

Classical flutter is a wind induced oscillation with two degrees of freedom. In this case the equations of motion which govern vertical deviation h and torsional deviation α must be considered as a coupled system. Theodorsen developed a complete analytic theory for the special terms that dominate in aircraft construction. For bridge deck cross sections Simiu (1996) proposes a linearization of the aerodynamic forces by means of the aeroelastic coefficients or flutter derivatives H_i^* and A_i^*, i = 1,...,4:

$$C_{L,ae} = KH_1^* \frac{\dot{h}}{U_\infty} + KH_2^* \frac{B\dot{\alpha}}{U_\infty} + K^2 H_3^* \alpha + K^2 H_4^* \frac{h}{B} \qquad (7)$$

$$C_{M,ae} = KA_1^* \frac{\dot{h}}{U_\infty} + KA_2^* \frac{B\dot{\alpha}}{U_\infty} + K^2 A_3^* \alpha + K^2 A_4^* \frac{h}{B} \qquad (8)$$

In the equations above the dynamic lift and moment must be considered as additional contribution next to steady state forces. Normalization is in analogy to (5) and the

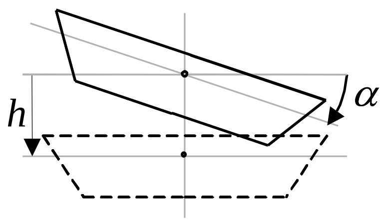


Figure 13: Vertical and torsional displacement in classical flutter theory.

oscillation frequency is expressed by the reduced frequency $K = \omega_{osc}B/U_\infty$. The flutter derivatives also depend on frequency and are commonly expressed as function of the reduced velocity $v_r = 2\pi/K$. The definition of the vertical and torsional displacement h and α is shown in figure 1. For reasons of consistency with the Theodorsen theory, vertical displacement and force are often given with positive sign downwards. In some cases a factor 2 is used additionally on the right hand side of the calculation.

To determine the flutter derivatives with CFD two separate calculations are performed. Once a forced vertical oscillation of the cross section is considered and once a torsional oscillation with the axes normal to the cross section plane. In both cases lift force and moment can be expected to oscillate with the same frequency as the cross section with a certain phase shift. With the obtained phase shifts and oscillation amplitudes the flutter derivatives can be obtained from the model equations (7) and (8), cf. Larsen (1998). For wind tunnel measurements an elastically suspended model is examined. For a proper scaling of the extracted dynamic coefficients 15 similarity parameters must be matched instead of the single Reynolds number in the steady state case. A short abstract of the procedure can be found for example in Lechner (2006).

4 Design Checks

4.1 Galloping

Galloping is a self-excited oscillation of the cross section in across wind direction due to alternating lateral forces. According to Simiu (1996), a necessary condition for galloping is the so-called Glauert-Den Hartog criterion

$$\left[C_L^{'}(0) + C_D(0)\right] < 0 \qquad (9)$$

which is valid as long as drag and lift coefficients are normalized with the same length scale $\ell_D = \ell_L$. In the more general case with different length scales the criterion must be modified accordingly to

$$\left[C_L^{'}(0) + \frac{\ell_D}{\ell_L} C_D(0)\right] < 0 \qquad (10)$$

4.2 Torsional Divergence

For slender cross sections as those of bridge girders, the overturning moment due to wind increases in general with increasing wind attack angle. If the torsional stiffness is weak it might be not possible to find a stable equilibrium solution for the torsional displacement. This phenomenon can be observed if the wind velocity is higher than the critical velocity

$$U_{c,div} = \sqrt{\frac{2k_\alpha}{\rho A C_m^{'}(0)}} \qquad (11)$$

Here k_α denotes the torsional stiffness and A the normalization area of the moment coefficient C_M. Normally one considers the lowest eigen-frequency f_α in practice so that the torsional stiffness can be expressed by the generalised moment of inertia I as $k_a = I\,\omega_\alpha^2$, where $\omega_\alpha = 2\pi f_\alpha$ is the circular frequency associated with the eigen-frequency.

4.3 Classical Flutter

Classical flutter is a coupling of vertical and torsional oscillation. Consequently, one searches for harmonic solutions of the system

$$\begin{aligned} m\left[\ddot{h} + a\ddot{\alpha} + 2\zeta_h\omega_h\dot{h} + \omega_h^2\right] &= L_h \\ I\left[\frac{a}{r_g^2}\ddot{h} + \ddot{\alpha} + 2\zeta_\alpha\omega_\alpha\dot{\alpha} + \omega_\alpha^2\right] &= M_\alpha \end{aligned} \qquad (12)$$

The vertical and torsional displacement, respectively, are given by h and α and the corresponding circular frequency and critical damping is denoted by ω and ζ. If the centre of mass and shear centre do not coincided, also the eccentricity a and radius of gyrations r_g must be given. The aerodynamic forces L_h and M_a can be obtained from the flutter derivatives according to equations (7) and (8).

By assuming harmonic oscillations as solution of (12) a system of equations for the amplitudes h_0 and α_0 is obtained. The system is only solvable if the determinant of the coefficient matrix vanishes. For the assumed type of solution the determinant is a complex valued polynomial of fourth order in the unknown flutter frequency ω. Since the frequency must be real valued for harmonic oscillation the polynomial can be decomposed into two real valued polynomials. Simultaneous solutions to these polynomials can only be found for certain critical velocities $U_{c,2dof}$. A detailed derivation for the evaluation of the critical velocity is described in Walter (1994) and Simiu (1996).

4.4 Torsional Flutter

If the flutter derivative A_2^* exposes a pronounced change of sign a pure torsional oscillation becomes possible as pointed out by Simiu (1996). Since A_2^* can also be interpreted as additional damping a vanishing total damping can occur if A_2^* is positive. In this case a necessary condition for torsional flutter can be derived from the second equation of motion (12):

$$A_2^* = \frac{4\zeta_\alpha I}{\rho A \ell_F^2} \qquad (13)$$

From the critical value of A_2^* a critical for the reduced velocity can be deduced, and with the given eigen-frequency, a critical velocity can be calculated.

4.5 Wind-induced vibration of the whole bridge

It is natural that wind load effects are getting bigger on the design of a cable-stayed bridge with its longer span lengths. The wind vibrations are conventionally classified into buffeting (gust response), vortex excitations, galloping and torsional flutter of the whole bridge.

Wind induced vibrations are to be verified also for various stages during the erection as well as, for such structural elements as pylon and stay cables, after completion.

The buffeting of a structure always takes place in the direct effects of turbulent natural wind. In the case of a long span bridge exposed to wind, there will be random vibrations of lateral bending by drag, vertical bending by lift and torsion around the bridge axis by pitching moment.

Referring to wind impact, long-span bridges require sophisticated wind buffeting analyses with considering both, the aero-elastic behavior of the structure and the wind loading correlation.

There are numerous assumptions that should be considered in order to deduce mathematical models for the fluid-structure interaction problem. For the analysis of bridges they are well established and the most important ones for the present aims can be listed as follows:

a) For the superposition principle of aerodynamic forces to hold, vibration amplitudes of a bridge deck are assumed to be small (lower than ±3 Deg in torsion, say).
b) The aeroelastic loads and the associated flutter derivatives are assumed to be functions of the mean reduced frequency and static twisting angle of the deck. The spanwise correlation of aeroelastic loads is assumed to be perfect.
c) The aerodynamic strip hypothesis is valid, i.e. the aerodynamic forces acting on a deck section (strip) are not influenced by the flow conditions at the strip vicinity.
d) The spatial correlation of fluid velocity fluctuations and the buffeting load they induce are considered to be identical.
e) The dynamical system is representable by means of the linear equations of motion around the equilibrium position. The equilibrium position is dependent upon the mean wind velocity.
f) Winds considered are assumed to be strong, the mean values of order 10 m/s or higher, for the referred turbulence models to be valid.
g) The buffeting excitation is assumed to be a stationary ergodic random process, i.e. the conditions of rapid change at mean wind velocity (rising and settling phases of storms) are not considered.
h) The horizontal across-wind component of turbulence spectrum $S(n)$ is assumed to have unimportant effects on the structural response and is neglected for the computational efficiency.
i) Lateral wind velocity components perpendicular to bridge spans are assumed to produce dominant wind actions to bridge decks.

Generally, none of the assumptions mentioned above are found to be restrictive in the sense, that they should be relaxed in analysis of typical long-span bridges, with expectations:

j) In the analysis of a bridge construction stage, the strip hypothesis may yield error, if the effects of the finite aspect ratio of the girder are not accounted for.
k) Winds parallel to bridge span can cause notable buffeting during the cantilever construction of cable-stayed bridges.
l) Conservative models for coherence decay characteristics can be assumed to account for a possible increase of load correlation vs. fluctuation velocity correlations.

The assumptions are not intended to be fully accepted in all cases, but in the present development it is believed that their implications are small or negligible in comparison to uncertainties in the structural and aerodynamic data.

The presented solution for structural wind buffeting calculation is performed in the modal space and in the frequency domain. It includes aerodynamic damping and stiffness effects due to structural movement caused by the wind flow. All computations are based on the tangential stiffness of the structure at a given point in time – the structure under permanent loading and mean wind – allowing to include all prior non-linear effects.

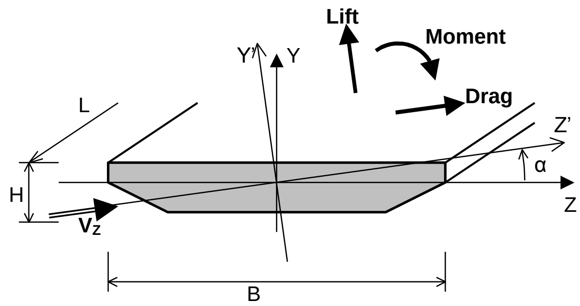
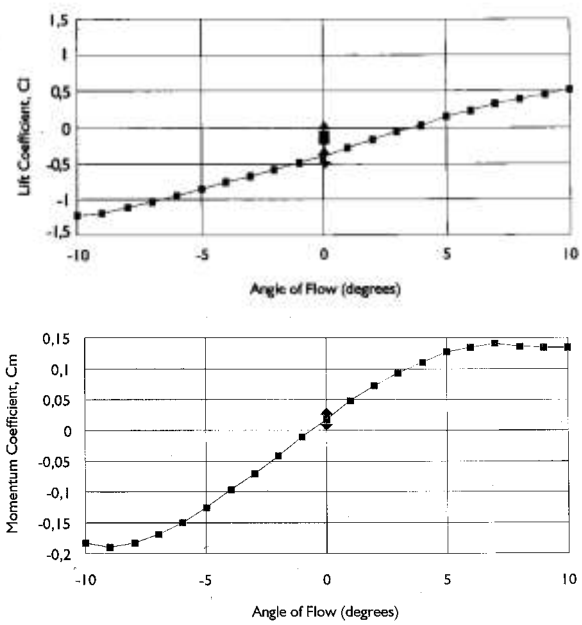
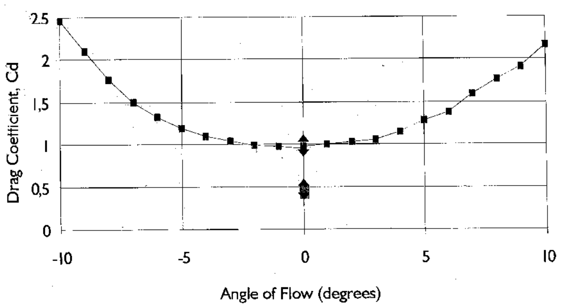


Figure 2: Wind forces acting on the bridge section.

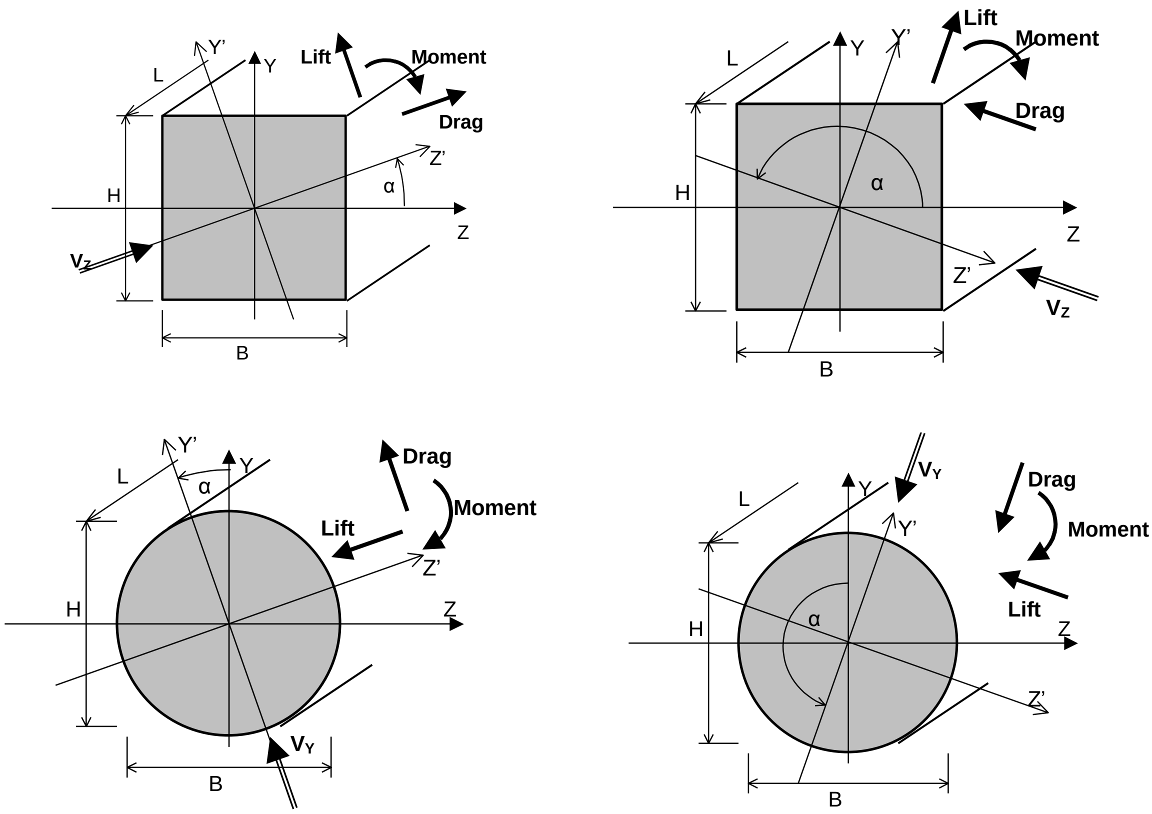
This calculation is commonly performed in the modal space, presuming linear behaviour based on the tangential stiffness matrix attained in a previous fully non-linear analysis for the dead loads. It includes aerodynamic damping and stiffness effects due to the structural movement caused by the wind flow. The analysis is based on the wind profile and on the aero-elastic parameters of the cross-sections (drag, lift, moment coefficients and derivatives). The wind profile is characterised by the mean wind velocity and the fluctuation (turbulence) velocity, both being a function of the height above terrain level. The stochastic is accounted for by using power spectra in the frequency domain.

These *aerodynamic coefficiants* leed into curves for Drag, Momentum and Lift.





Note: *In case of a deck girder a range of this angle α between –10° and +10° is of interest. For pylons and piers this range might be from 0° to 360°, but symmetric conditions are often helpful.*



The aerodynamic forces per unit length are:

Drag: $q_y = 0.5C_Dt\rho V^2$

Lift: $q_z = 0.5C_LB\rho V^2$

Moment: $M = 0.5C_MB^2\rho V^2$

There C_D, C_L, C_M are shape factors for drag, lift and overturning moment.

The relation between "Autocorrelation function" and "Power Spectral Density" is a basic equation of the solution in the modal domain.

$$f(\Omega) = 2\int_{-\infty}^{\infty} \psi(\tau) e^{-i\Omega\tau} d\tau \tag{1}$$

$$\psi(\tau) = \frac{1}{2}\cdot\frac{1}{2\pi}\int_{-\infty}^{\infty} f(\Omega) e^{i\Omega\tau} d\Omega = \frac{1}{2\pi}\int_{0}^{\infty} f(\Omega) e^{i\Omega\tau} d\Omega$$

5 GREAT BELT BRIDGE

The cross section of the Great Belt (Storebœlt) bridge, a suspension bridge with a span of 1624 m in Denmark, is a well documented example in the CFD literature (e.g. Larsen, 1994 and Morgenthal, 2000). A schema of the cross section with width *B* = 31 m and height *H* = 4.34 is shown in figure 2. Since Reynolds numbers are only accessible up to 10^5 in the wind channel the CFD calculations were performed for the same Reynolds number for comparison. The structural data for wind design checks are given in table 1 (cf. Walther, 1994).

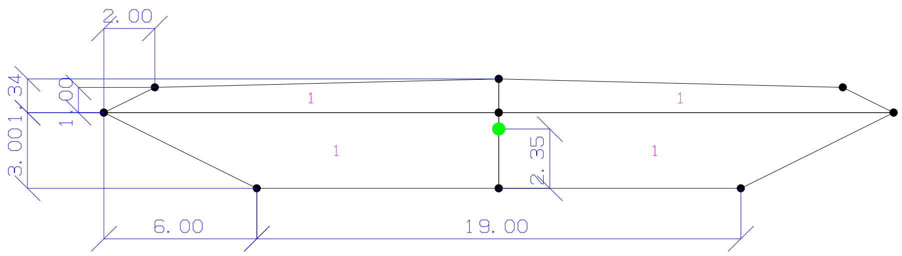


Figure 14: Schema of the Great Belt bridge main girder cross section. The green marks the shear centre.

ρ	m	I	f_h	f_α
[kg/m³]	[kg/m]	[kg m]	[Hz]	[Hz]
1.2	$17.8 \cdot 10^6$	$2.173 \cdot 10^8$	0.099	0.186

Table 1: Structural data for the wind design checks of the Storebœlt bridge.

The results of the steady state coefficient calculation are illustrated in figure 3. A comparison with Walther (1994) and Morgenthal (2002) shows a good agreement of the coefficients. The Strouhal number for horizontal wind direction is St = 0.12. This is in good accordance with the values found in literature which are between 0.08 and 0.25 (cf. Morgenthal, 2000). By inspecting the dependence of the lift coefficient on the wind direction, a positive slope is found for angles between –10° and +8°. Therefore, no galloping is expected for these angles. For the remaining region up to +10° the criterion (10) predicts no susceptibility for galloping.

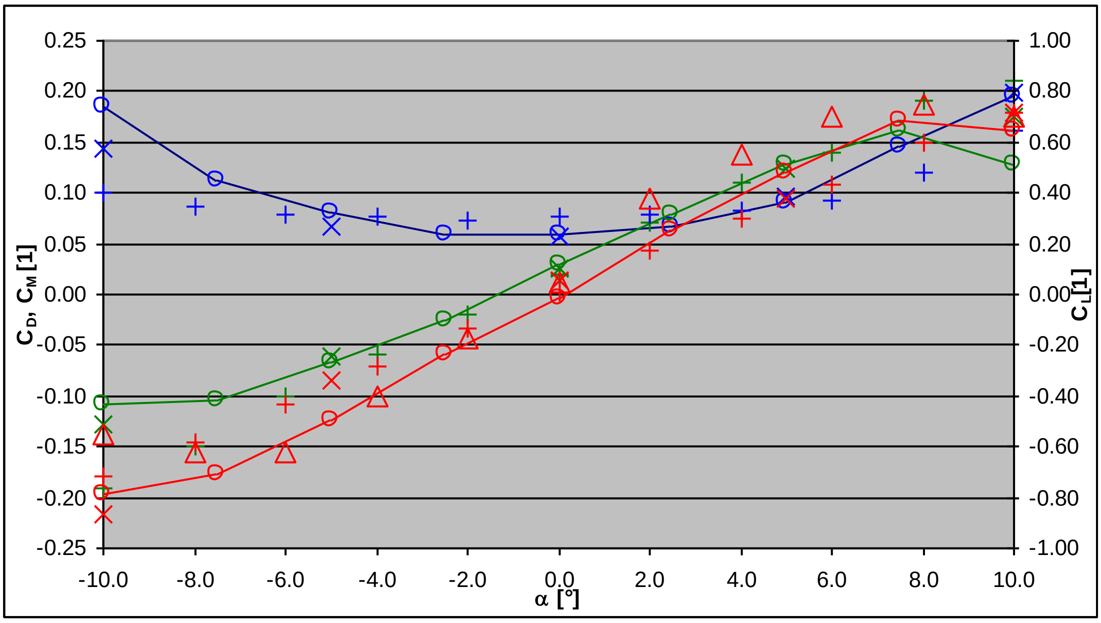


Figure 15: Steady state coefficients C_D (blue), C_L (red) and C_M (green) of Storebœlt bridge deck in dependence of wind direction. For comparison with the present results (o), results obtained by Walther (1994, x), Morgenthal (2002, Δ) and Reinhold (1992, +) are given.

To calculate the critical velocity for torsional divergence, the derivative of the moment coefficient is approximated by

$$C_M^{'}(0) \approx \frac{C_M(2.5°) - C_M(-2.5°)}{5°} \mathrel{\widehat{=}} 1.2 \frac{1}{\text{rad}} \qquad (13)$$

Inserting this value into equation (11) the critical velocity is $U_{c,div}$ = 65.5 m/s.

The flutter derivatives were calculated according to the procedure presented in section 3.2. The results are shown in figure 4 together with comparison results from another DVM calculation (Larsen, 1994). Please note that the present results were multiplied with a factor 0.5 for comparison reasons. Again a good compliance is reached.

The critical classical flutter velocity for zero critical damping $\zeta_h = \zeta_\alpha = 0$ calculates to $U_{c,2dof}$ = 41.8 m/s according to the method outlined in section 4.3. Reinhold (1992) obtained a value $U_{c,2dof}$ = 37.6 m/s with wind tunnel measurements. If the critical damping is raised to 0.5% the critical velocity is slightly increased to $U_{c,2dof}$ = 43.0 m/s.

By inspecting the dependence of the coefficient A_2^* on the reduced velocity it can be seen that there is no susceptibility to torsional flutter since the effective damping remains positive for all wind velocities.

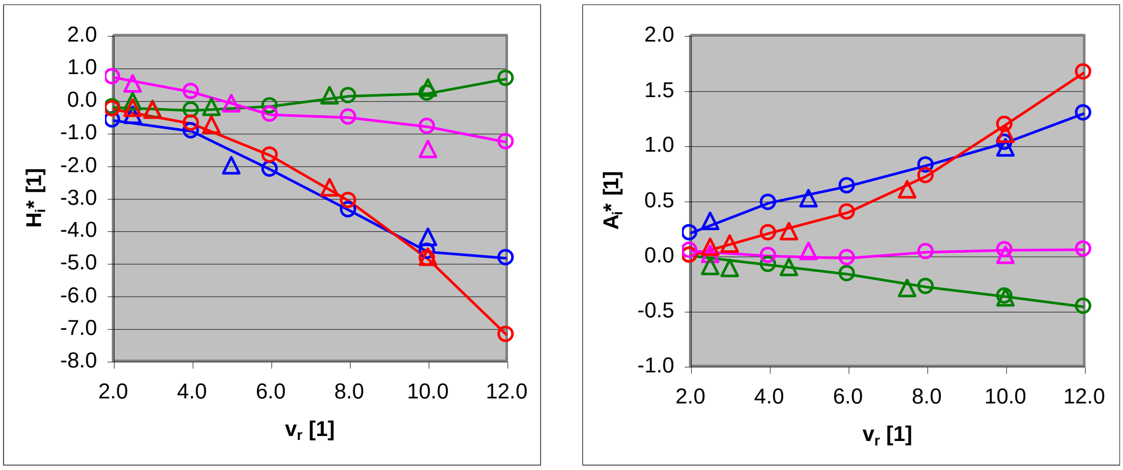


Figure 16: Flutter derivatives H_1*, A_1* (blue), H_2*, A_2* (green), H_3*, A_3* (red) and H_4*, A_4* (magenta) in dependence of the reduced velocity v_r. For comparison with the used DVM (o) results obtained by Walther (Δ) are given.

The time history of the lift coefficient is demonstrated in figure 5 for different situations: once for a fixed cross section and once for a cross section which performs a forced vertical oscillation with $v_r = 4$. After a certain settling time the time history for the fixed cross section can be modelled by a harmonic oscillation where the frequency is determined by the Strouhal number. The same frequency can be identified in the time history of the moving cross section but it is superimposed with an oscillation with the driving frequency. The amplitude and phase shift is used to calculate the flutter derivatives H_1* and H_4* according to (7).

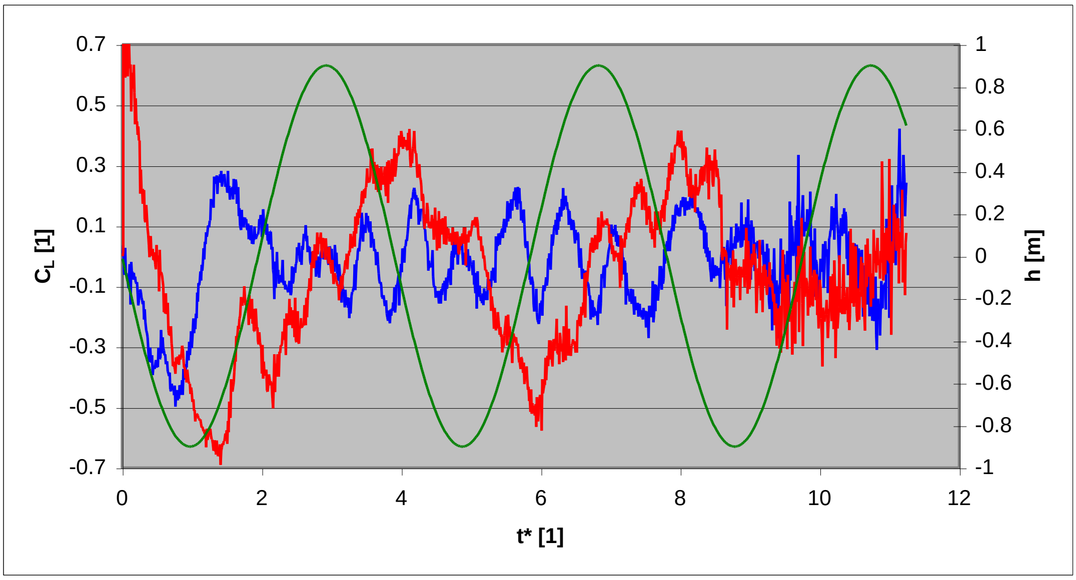
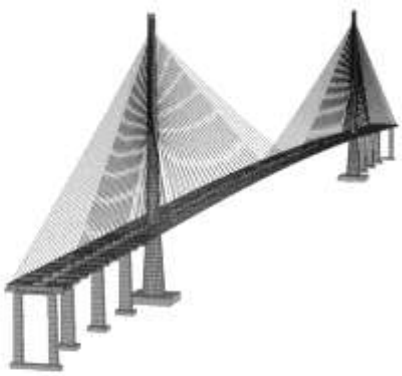


Figure 17: Time history of the lift coefficient C_L in dependence of the dimensionless time t* for fixed cross section (blue) and vertically oscillating cross section (red). The green line indicates the vertical displacement h of the cross section.

6 STONECUTTERS BRIDGE, HONG KONG



The Stonecutters Bridge is a cable-stayed bridge with a main span of 1018 m, side spans of 298 m, and two single towers of a height of 290 m. The bridge will straddle the Rambler Channel at the entrance to the busy Kwai Chung container port and its northern end will be located on reclaimed land that forms part of Container Terminal 8 at the eastern side of Stonecutters Island.

The deck of the main span is a twin girder steel deck, whilst the side spans are concrete; the side spans will be built in advance of the cable stay erection to counterbalance and stabilise the slender lightweight main span deck.

Figure 3: Structural RM2006 model.

Prefabricated cable stays are arranged in a laterally-inclined fan arrangement to maximise the transverse and torsional stability of the main span. They include the world‘s longest bridge stay to date. Prefabricated cable stays are arranged in a laterally inclined fan arrangement to maximise the transverse and torsional stability of the main span and include the world‘s longest bridge stay to date. Wind buffeting responses are shown in Figure 4 to Figure 6.

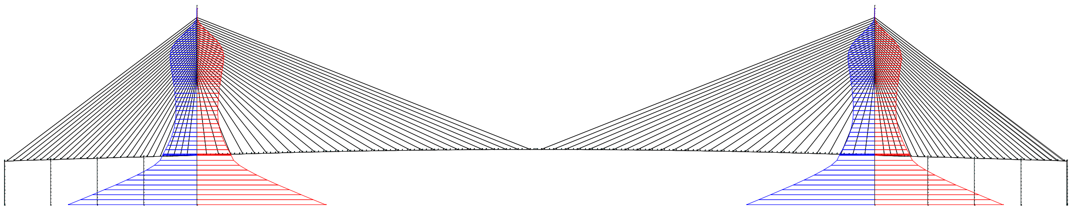


Figure 4: Tower bending moment.

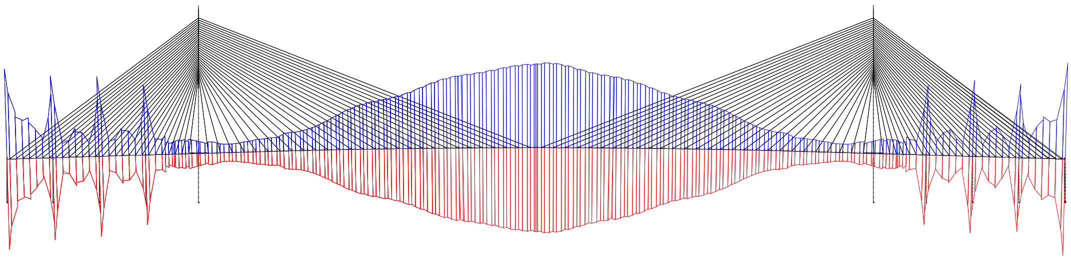


Figure 5: Deck bending moment.

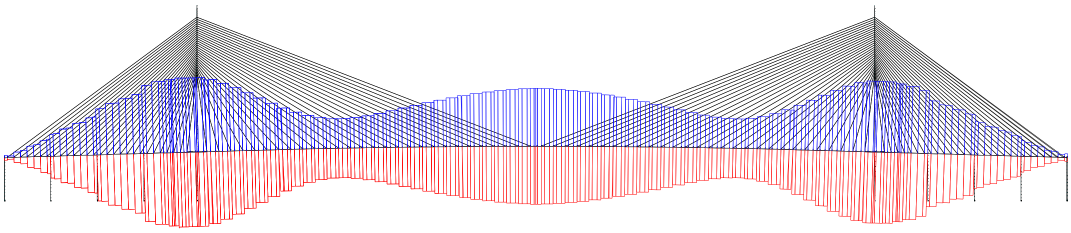


Figure 6: Deck normal force.

7 SHENZHEN WESTERN CORRIDOR

On the Hong-Kong side of the Shenzhen Western Corridor (Project Engineers: Ove-Arup, Hong-Kong) there is a cable stayed bridge with a main span of 210 m. The steel girder is supported by an inclined pylon of a height of 158 m.

The wind buffeting responses are shown in . Figure 8 to . Figure 10.

Figure 7: Structural RM2006 model.

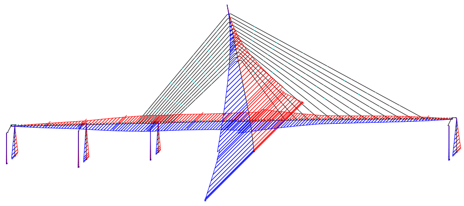


Figure 8: Lateral bending moment.

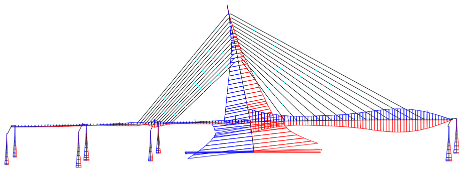


Figure 9: Vertical bending moment.

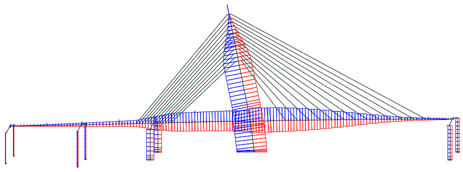


Figure 10: Normal force.

8 CONCLUSION

The presented results show good compliance with other publications both of theoretical and practical nature. The calculation time for the considered example was less than three hours with two Dual-Core processors at 2.67 GHz. With regard to the design time the computer aided calculation of wind effects can be regarded as a powerful tool in early design stages. Different cross sections and modifications of details can be examined fast and uncomplicated with good accuracy.

In the final design phase computer based calculations can be combined with wind tunnel measurements to guarantee optimal safety in the wind design process.

The numerical procedure outlined in this paper is implemented in the computer package RM2006. The presented algorithm predict wind buffeting response within structural non-linear analysis. Cable sagging, p-delta effects, large displacements or even contact problems can be combined with long term effects within consistent analysis. The proposed method for the numerical analysis can handle satisfactorily static and dynamic bridge behaviour up to the time infinity; it is generally suitable for the investigation of cable stayed bridges.

The wind related functions of RM2006 match nearly all needs for the design of long-span bridges. Arbitrary complicated wind profiles with varying wind speed and turbulence intensity are easily defined. Together with the cross-section related shape factor diagrams defining the dependency of the drag- lift- and moment coefficients on the attack angle of the wind impact, these wind profiles allow a comprehensive wind buffeting analysis taking into account the varying along-wind and lateral forces of gusty wind events.

References

[1] TDV GmbH., *"RM2006 – Technical Description"*, TDV-Austria, 2006.

[2] Zienkiewicz, O. C., Taylor, R. L., *"The Finite Element Method"*, Fifth Edition, McGraw-Hill, London, 2001.

[3] Stampler J., Bokan H., Janjic D., Heiden M., 2006. "Use of specialized software tools for modern Bridge Design", *International Conference on Bridges in Dubrovnik*, Croatia, 2006, to be printed.
[4] Janjic D, Pircher M., Pircher H., Bridge R.Q. "Towards a Holistic Approach to Bridge Design", *Proceedings: IABSE-Symposium 2002*, Melbourne, pp. 236-237.
[5] Janjic D, Bokan H., "Erection Control, TDV's unique tool solution for bridge design and construction", *Proceedings: IABSE-Symposium 2006*, Budapest, to be printed.
[6] Janjic D, Pircher M., Pircher H., "Optimisation of Cable Tensioning in Cable-Stayed Bridges", *Journal of Bridge Engineering*, ASCE, v8, n3, pp 131-137.
[7] Bokan H., Janjic D., Heiden M., "Optimisation of the Tensioning Schedule for Cable-Stayed Bridges using Dynamic Software", *Proceedings: IABSE-Symposium 2006*, Budapest, to be printed.
[8] Janjic D, Pircher H., "Consistent Numerical Model for Wind Buffeting Analysis of Long-Span Bridges", *Proceedings: IABSE-Symposium 2004*, Shanghai.
[9] Emil Simiu and Robert H.Scanlan, *"Wind effects on structures"*, John Wiley & Sons, New York, 1996.
[10] Miao J., Xiao R., Pei M., Zhang X., Janjic D, Pircher M., "Global Analysis of the SuTong Cable-Stayed Bridge", *Proceedings: IABSE-Symposium 2004*, New Delhi.
[11] Larsen, A., Walther, J.H., "Discrete vortex simulation of flow around five generic bridge deck sections", *J. Wind Eng. Ind. Aerodyn 1998., 77 & 78,* pp. 591 - 602.
[12] Lechner, B., „Aerodynamische Untersuchungen eines Modells der Hardangerbrücke im Windkanal des K+vif mit einer 5-Komponentenwaage", *Messbericht 2006, K+vif:Graz.*
[13] Morgenthal, G., "Comparison of Numerical Methods for Bridge-Deck Aerodynamics", *MPhil Thesis, University of Cambridge, Cambridge 2000.*
[14] Morgenthal, G., "Aerodynamic Analysis of Structures Using High-resolution Vortex Particle Methods", *PhD Thesis, University of Cambridge:* Cambridge 2002.
[15] Reinhold, T.A, Brinch, M., Damsgaard, A., "Wind tunnel tests for the Great Belt Link", *Proceedings of the First International Symposium on Aerodynamics of Large Bridges 1992*, pp. 255 - 267.
[16] Simiu, E., Scanlan, R.H., "Wind Effects on Structures", Third Edition, *John Wiley & Sons*: New York 1996.
[17] Walther, J.H., "Discrete Vortex Method for Two-Dimensional Flow past Bodies of Arbitrary Shape Undergoing Prescribed Rotary and Translational Motion", *PhD Thesis, Technical University of Denmark,* Lyngby 1994.
[18] Pircher, H., Janjic D., "FEMBRIDGE – Technical Project Description", *Internal report*, TDV-Austria, 2001.
[19] Pircher M., Janjic D., Pircher H., „Towards a Holistic Approach to Bridge Design", *Proceedings: IABSE-Symposium 2002*, Melbourne.
[20] Zienkiewicz, O. C., Taylor, R. L., "The Finite Element Method*"*, Fifth Edition, *McGraw-Hill Book Company*, London 2001.
[21] Bokan H., Janjic D., Heiden M., "Optimisation of the Tensioning Schedule for Cable-Stayed Bridges using Dynamic Software", *Proceedings: IABSE-Symposium* 2006, Budapest.