ASPECTS OF WIND BUFFETING RESPONSE AND NON-LINEAR STRUCTURAL ANALYSIS FOR CABLE STAYED BRIDGES

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Summary

As the bridge engineering community sets sails to using longer and longer spans, more and more sophisticated analysis models have to be used in the design process. For large prestressed concrete bridges built using the incremental launching or free cantilevering methods, this applies mainly to accurately modelling the erection process in time, with considering the different construction stages, the time dependent behaviour (creep and shrinkage), and the required pre-cambering to achieve the design shape after the construction process has been finished.

However, the challenges for ultra long-span bridges such as stay cable or suspended bridges with high pylons and slender steel or concrete decks, are mainly related to optimising the stressing sequence of the cables, to the geometrically non-linear behaviour of the structure, and to dynamic problems such as wind-induced vibrations. Addressing these last two topics more in detail is the aim of this contribution.

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 theoretical background of geometrically non-linear structural bridge analysis, and finding suitable ways of implementing non-linear procedures into a general bridge design software package for being used in a practical bridge design environment. 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.

1.1 Target design state

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.

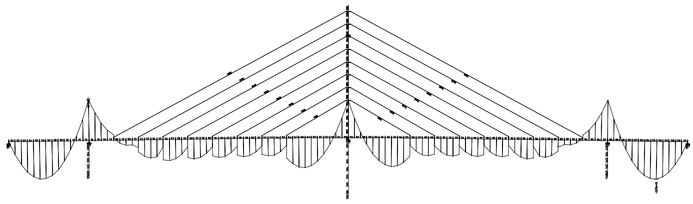


Figure 1: Target bending moment at the final stage should be achieved with construction sequence.

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 minimise undesired influences.

1.2 Natural frequency and structural design

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.

1.3 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.

1.4 Buffeting (gust response)

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.

2. NUMERICAL SOLUTION

2.1 Optimisation procedure

Special optimisation procedures are necessary for the standard bridge design process. The AddCon Method (The Additional Constraint Method) is a novel solution for optimisation problems in structural engineering [1].

If structural response is not linear, the optimisation problem is non-linear from the very beginning. In practical cases this non-linearity is not too far away from a linear solution. Design experience shows that non-linear effects are usually within 20 % of the linear solution. This is the same order of magnitude as the non-linearity due to time effects [8]. Again, these effects can be treated with the same method de-scribed above. With a mild non-linearity grade we cover almost all problems.

Optimisation procedures (e.g. for evaluating the required stay cable stressing sequence in order to achieve a given maximum stress state in the superstructure or for optimisation of tensioning of temporary stays etc) are another great help in the design process. The algorithm implemented in **RM2006** models in detail every construction stage. The tensioning of each single cable is considered at first as a unit load case taking into account the current structural system and then influencing all previously applied unit load cases.

All other loadings (e.g.: self weight of the new segment, moving the traveller etc.), related to the individual erection procedure, are also calculated step by step. All displacements and internal forces are accumulated and divided into one „constant“ (self weight etc.) and several „variable“ components. Each „variable“ component is related to one tensioning unit loading case and optimised in additional constraint module. Further details are given in [5].

2.2 Wind buffeting response

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.

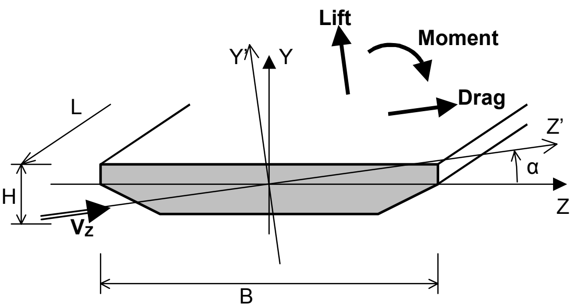
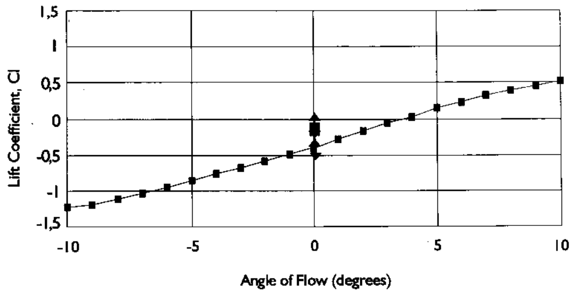
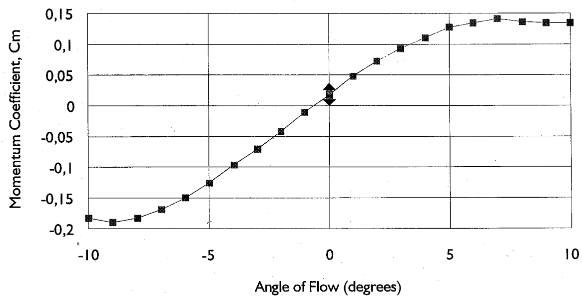


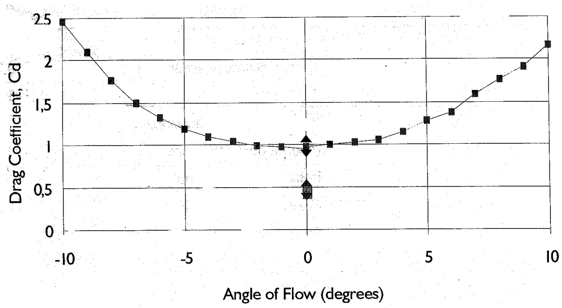
Figure 2: Wind forces acting on the bridge section.

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.

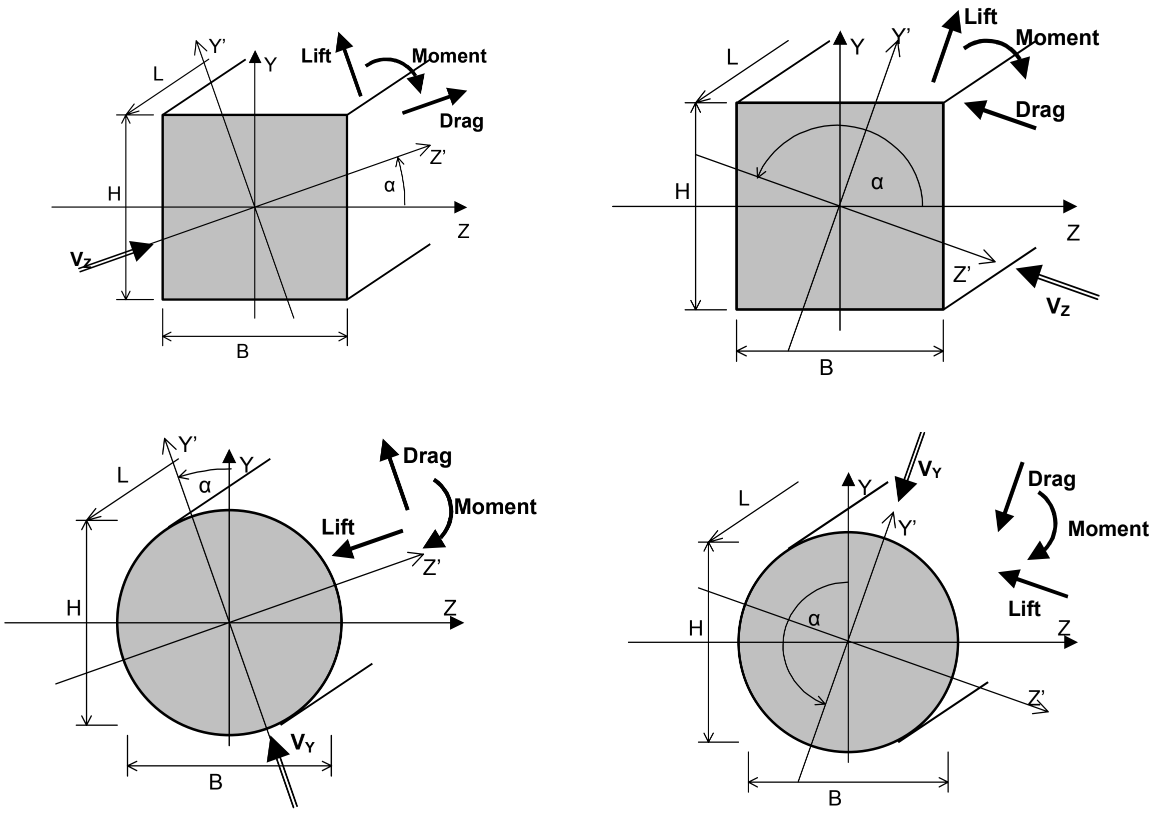
These *aerodynamic coefficiants* leed into curves for Drag, Momentum and Lift.







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: $q_y = 0.5C_D t\rho V^2$

Lift: $q_z = 0.5C_L B\rho V^2$

Moment: $M = 0.5C_M B^2\rho V^2$

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.

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

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

3. 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.

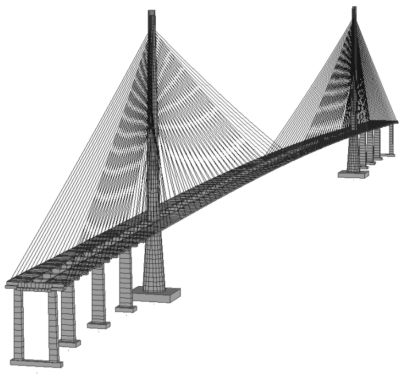


Figure 3: Structural RM2006 model.

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.

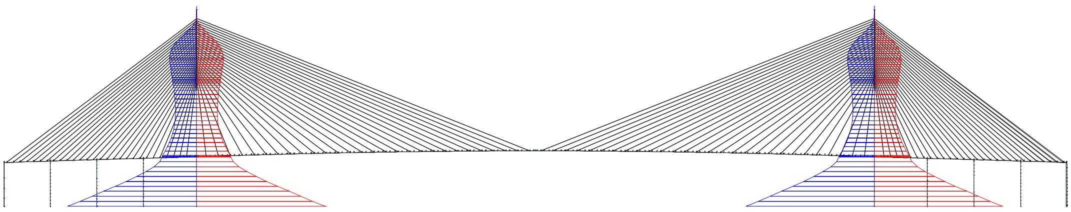


Figure 4: Tower bending moment.

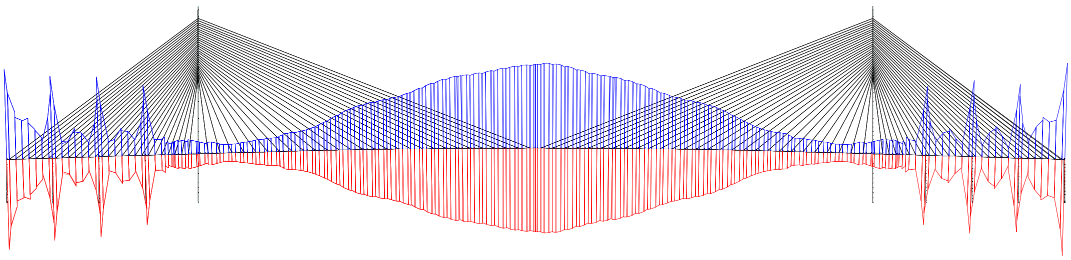


Figure 5: Deck bending moment.

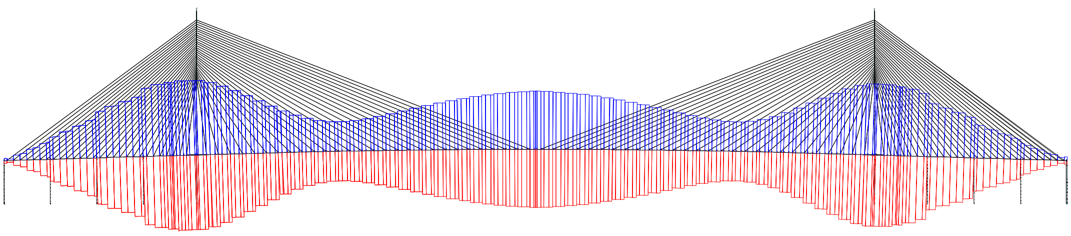


Figure 6: Deck normal force.

3.1 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.

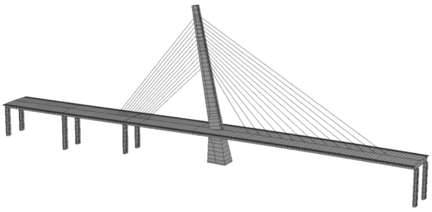


Figure 7: Structural RM2006 model.

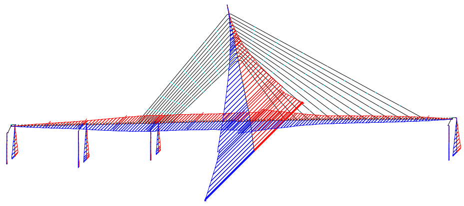


Figure 8: Lateral bending moment.

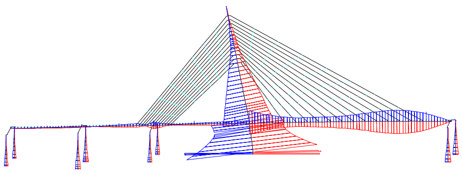


Figure 9: Vertical bending moment.

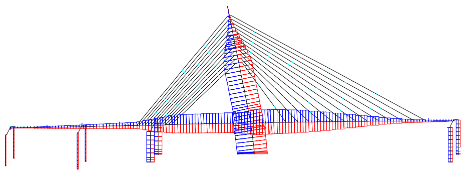


Figure 10: Normal force.

4. SUTONG BRIDGE

Sutong Bridge lies between Nantong City and Changshu in the east of the Jiangsu province and forms a link across the Changjiang River.

The total length covered by the project is 32.4 km. It consists of three parts: approach and link roads on both the north and south banks, and the main central crossing, which has a total length of 8.2 km. The main cable stayed bridge has a double cable plane, and a double-pylon arrangement, with a steel box girder deck. Its spans range from 100 m to the main span of 1088 m, which will be a world record when it is finished, overtaking the Stonecutters Bridge in Hong Kong.

The bridge over the other channel is 548 m long, and consists of a T-type steel girder bridge. The contract on the north bank includes approach roads of a total length of 15.1 km with two interchanges, a toll gate and a service zone; on the south bank the total length is 9.1 km, with one interchange.

Taking full consideration of various geotechnical conditions at the bridge site, technical feasibility, and especially construction ability, the selected design for the Primary Fairway Bridge is a double-plane twin-pylon cable-stayed bridge with a continuous span arrangement of (100+100+300+1088+300+100+100) m, as shown in Figure 11. Two auxiliary piers and one transitional pier are erected in each side span. The main span of the bridge is 1088 m, which is the longest main cable-stayed bridge span at present.

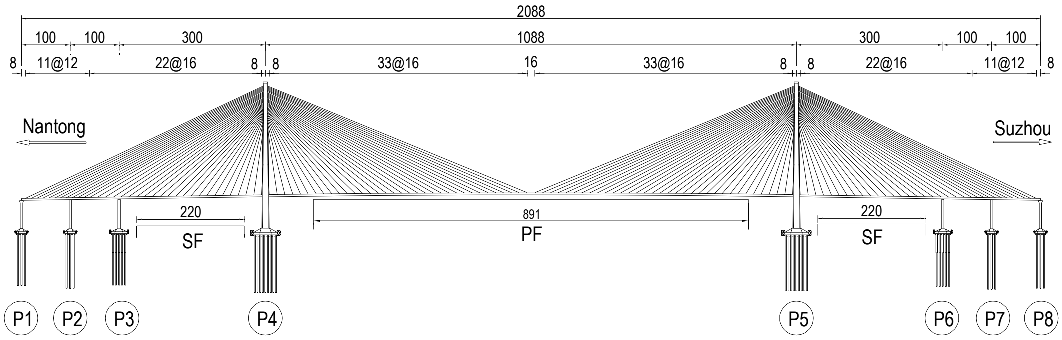


Fig. 11: Span Arrangement (unit: m).

The bridge girder is a streamlined closed flat steel box girder. The total width including wind fairing is 41.0 m accommodating dual 8 traffic lanes. The cross-section height is 4.0 m. The steel box is generally stiffened in longitudinal direction with closed steel troughs. The standard cross-section of the girder is illustrated in Figure 12.

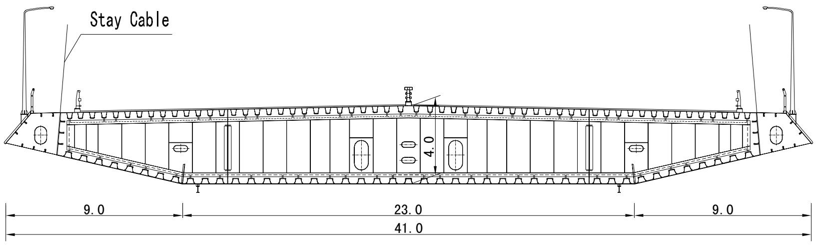


Fig. 12: Cross Section of the Girder (unit: m).

5. CONCLUSION

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.

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