Experience from the Global Analysis of Stonecutter's Bridge and Sutong Bridge

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ABSTRACT:

The design of long span cable-stayed bridges yields many challenges for the design team and the used design tools. These challenges, related to topics like optimizing the stressing sequence of the cables, simulation of erection procedure and dynamic problems such as wind-induced vibrations, and their solution in the design process of 2 major bridges are addressed in this contribution.

INTRODUCTION

The Stonecutter's Bridge in Hong Kong has been the first cable stayed bridge designed to span more than 1000 m (1018 m, currently under the construction) and the Sutong Bridge in China (1088 m, closed in summer 2007) is the first already built stay cable bridge breaking this limit.

Whereas many international companies were in charge with the design and construction of the Stonecutter's Bridge in Hongkong, the Chinese government strongly intended to handle the Sutong project as an all-China project with mainly Chinese companies involved. However, foreign companies have been engaged as sub-contractors for special tasks, in the design process as well as in the construction phase. E.g., COWI Consultants and CHODAI Co. Ltd have independently performed the review of the design documents.

The RM software package has been used for the global analysis or these bridges, by consultants Ove-Arup (Hong Kong) for the Stonecutter's Bridge and HPDI (Beijing) for the Sutong Bridge. TDV at Graz, Austria was selected to install its bridge design software RM in the charged design institutes and to assist and support the design teams in close cooperation throughout the whole design process.

The reason for choosing TDV was the proven versatility of the software product together with its experienced and solution oriented development and consulting staff, giving confidence that all problems, even any not yet known arising ones, can be solved successfully with joint endeavor.

These problems required sophisticated analyses taking into account large displacements, dynamic behavior with respect to wind impact as well as with respect to seismic events and ship impacts on the pylons, and the full construction history with optimization of the cable tensioning procedure.

Similar modeling and analysis problems had to be solved for both bridges and this paper presents some of experiences gained during the design, analysis and construction sequence. Engineering problems will be outlined, corresponding engineering solutions will be presented and theoretical basis needed for technical understanding will be briefly described.

All solutions gained with both reference projects have been further implemented into the general bridge design software package RM for being used in practical bridge design by other consultants. These include special features in optimizing of the stressing sequence of the cables, construction stage simulation and wind-induced vibrations (in present case wind buffeting).

THE BRIDGES

STONECUTTER'S BRIDGE - 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 stabilize the slender lightweight main span deck.

Prefabricated cable stays are arranged in a laterally-inclined fan arrangement to maximize the transverse and torsion stability of the main span. They include the world's longest bridge stay cable to date.

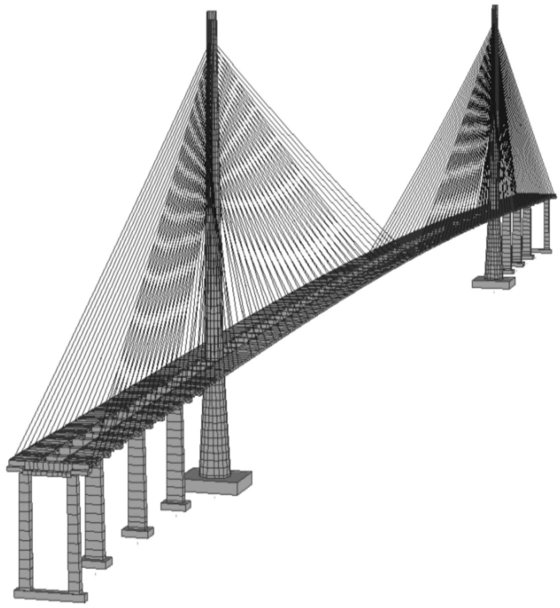

Figure 1 - Structural RM2006 model of Stonecutters Bridge

SUTONG BRIDGE - Taking full consideration of 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 2. 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.

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 the longitudinal direction with closed steel troughs. Transverse plate diaphragms are provided with a typical distance of 4.0 m and with smaller distances down to 2.27m locally around the two pylons. The standard cross-section of the girder is presented in Figure 3.

The inverted Y-shaped pylons are about 300 m in height and are made of concrete grade C50 according to the Chinese standard JTJ01—89. The stay-cables are anchored inside the steel boxes fixed to the concrete by shear studs at the pylon top. The maximum segment of a cable anchorage steel box weighs about 36 tons. The tie-beam between the pylon legs is fully post-tensioned for outward thrust from the pylon legs under service loads and seismic load.

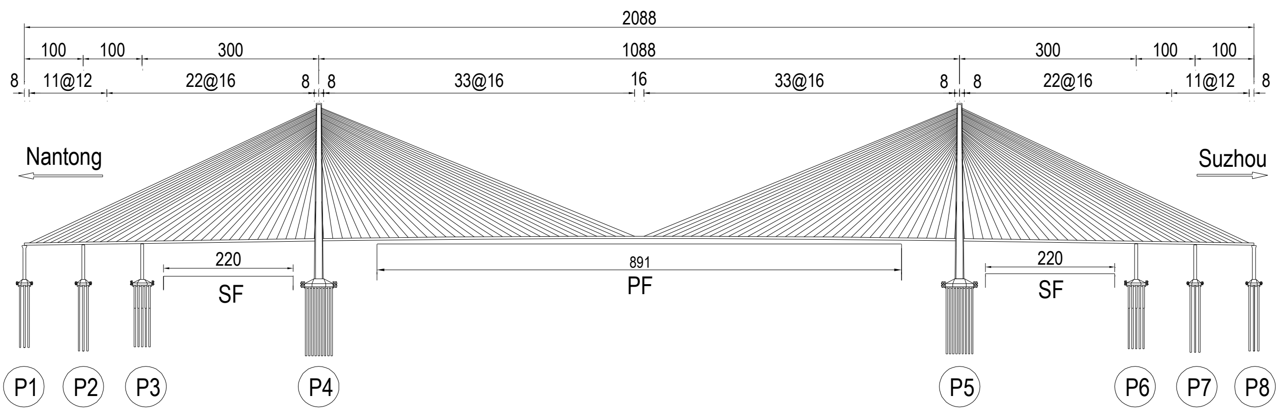

Figure 2 – Sutong Bridge – span arrangement [m]

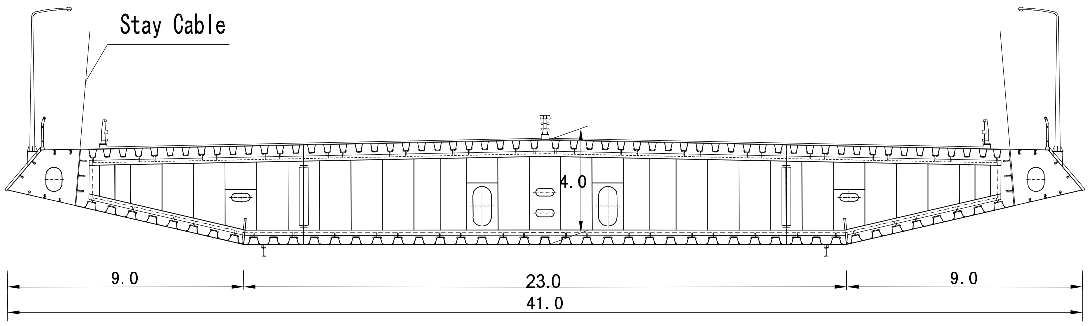

Figure 3 – Sutong Bridge - Cross-section of the Girder [m]

The stay cables are arranged in double inclined cable planes with standard spacing of 16 m in the central span and 12 m near the ends of the back spans along the girder. To reduce the effect of wind loads, the cable stay systems are made of the parallel wire strand consisting of 7 mm wires, each with a cross sectional area of 38.48 mm^2. Cable sizes range from a minimum of PES7-139 for the main span stays near the pylons to a maximum of PES7-313 for the longest backstay. The longest cable is about 577 m with a weight of 59 tons.

The problem of cable vibration due to wind and possibly in combination with rain or parametric excitation has been studied during the design process. Different measures to minimize stay cable vibrations have been investigated including two kinds of cable surface treatments to prevent formation of rainwater flows on the cables and internal or additional external damping devices. The final measures will be chosen after detailed testing.

OPTIMIZATION OF STAY CABLE STRESSING

TARGET DESIGN STATE – In the design process, the engineer is looking for the best solution for given criteria by changing specific system parameters.

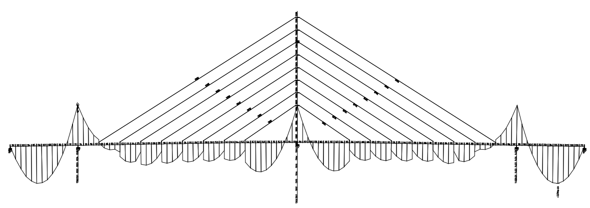

Figure 4 - Target bending moment to be achieved with construction sequence

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.

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.

OPTIMIZATION PROCEDURE - Special optimization procedures are necessary for the standard bridge design process. The *AddCon* Method (Additional Constraint Method) is a novel solution for optimization problems in structural engineering (Bokan et al., 2006).

If structural response is not linear, the optimization 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. Again, these effects can be treated with the same method de-scribed above. With a mild non-linearity grade, we cover almost all problems.

Optimization 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 optimization of tensioning of temporary stays etc) are another great help in the design process. The algorithm implemented in RM 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 traveler etc.), related to the individual erection procedure, are also calculated systematic. 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 optimized in additional constraint module. Further details are given in (Janjic et al., 2003).

CONSTRUCTION STAGES, ERECTION CONTROL, PRE-CAMBER

CONSTRUCTION STAGE ANALYSIS – The forward analysis with the *AddCon* method implemented in RM was employed for all erection stages to achieve the optimum final dead load situation as required by the designer. All temporary supports, tie-downs, and movements of derricks for construction, temporary loading, and permanent loading were included in the model at various stages.

Continuous change of structural systems is a major reason for getting non-linearity in structural analysis. To understand the physical reason, a very simple example of using a temporary support in the construction schedule is shown. Bending moments on the main girder before closure (with temporary primary support active) and after closure on the main girder due to temporary support removing are presented in Figure 5.

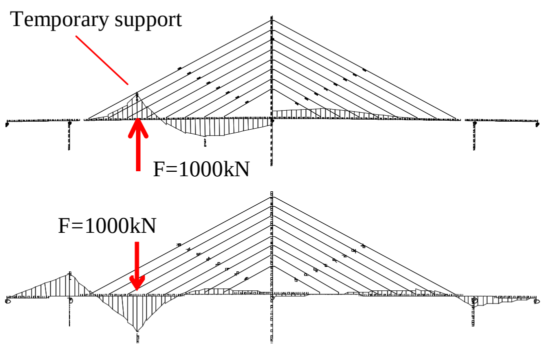

Figure 5 – Bending moments before and after change of structural system

PRE-CAMBER – RM computes the pre-camber line of all construction stages automatically. The third-order effect of pre-camber shapes apart from the design elevation of the deck is also considered. The results of the construction stage analyses of both bridges showed that the global stiffness is very small before closure. In the case of the Sutong Bridge for instance, the initial tensioning of the longest stay cable in mid-span yields a vertical deflection of 1.3 m at the end of the cantilever. Even after closure, the superimposed dead load (including paving, baluster, etc.) will still yield a vertical deflection of 1.8 m in the centre of mid-span, i.e. geometric nonlinearity has obviously remarkable effects, especially for the erection geometry of the deck.

A special erection control facility has therefore been implemented in the program allowing the design team for predicting and monitoring the bridge erection process. The program also gives information if force action on site is necessary when assembling the new segments. This allows determining any necessary equipment and possible construction problem already in the early design stage (Janjic et al., 2006).

LARGE DISPLACEMENTS - The designers have given much attention to geometric nonlinearities all the way from the preliminary design phases to the detailed design. Special studies on nonlinear effects were carried out, giving some notable remarks on the influence of geometrical nonlinearities:

- Compared with the linear analysis, effects of geometrical non-linearity may induce an offset of 10 to 20 % of the maximum or minimum stress of the girder and the pylons, and the critical location can be shifted.
- Each of the stay-cables was divided into specially developed "catenary" elements in order to consider cable sagging more properly than by approximating this effect with a fictitious effective Young's modulus. Comparisons showed that for cable-stayed bridges beyond 1000 m this is essential to achieve the required accuracy.
- In the construction phase, big deviations from the design shape have to be adopted as pre-camber values, in order to get the required design shape under permanent load at the end of the construction sequence without accepting forbidden internal constraint forces.

TIME EFFECTS - In practical engineering, time effects (creep, shrinkage and relaxation) must be considered in the analysis. The occurrence of time dependent plastic strain is a material property of concrete. Although the girders of the two bridges are steel box girders, creep and shrinkage of the concrete pylons play a certain not negligible role due to considerably influencing the stressing state of the stay cables.

The effect of the loading history on the plastic strains in concrete is very important. Considerable strain changes (creep recovery) can continue for a very long time after removal of all loads. Modern national design codes take effects of the loading history into account, resulting in much more accurate deformation prediction than with previous approaches. Concrete creep models are generally defined by separated creep factors for each stress increment in the loading history. Rather complicated calculation of creep factors for each stress increment is impossible for hand calculation but presents no difficulty for computer implementation (Janjic et al. 2004).

WIND DESIGN

Several different investigations had to be performed with respect to the heavy wind impact. First of all, appropriate girder cross-sections had to be developed, which satisfied operational demands and bearing capacity requirements as well as the requirements with respect to wind loading. For both projects, the wind tunnel tests were performed at the Tongji university. They led to streamlined closed steel box girders with wind fairings.

For the Sutong bridge, the total width is 41.0 m accommodating dual 8 traffic lanes. The cross-section height is 4.0 m as previously shown in Figure 3. The cross-section of the Stonecutter's Bridge consists of two more or less rigidly connected steel box girders as presented in Figure 6.

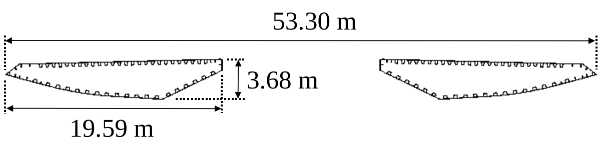

Figure 6 - Twin girder system of Stonecutters Bridge

CFD CALCULATIONS – Sophisticated CFD analyses for the decks were performed in addition to the wind tunnel tests. They gave a much deeper insight in the flow pattern around the deck and the resulting forces, as the respective mathematical model allows for performing a variety of influence studies, which can hardly be done with physical models due to economic reasons. In addition, these investigations can be performed on the full scale cross section model with the correct Reynolds number, whereas admissible Reynolds numbers in wind tunnels are of order 10^5 while in reality Re > 10^7 (Stampler et al. 2007).

APPLICATION TO STONECUTTERS BRIDGE – The application of CFD analysis is presented by considering the twin girder deck of the Stonecutter's Bridge in Hong Kong. A detailed presentation of one girder is illustrated in Figure 7, the twin girder system is outlined in Figure 6.

A CFD calculation was performed for the deck, where rigid connections between the twin girders are assumed. The total width of 53.3 m was chosen as normalization length for all coefficients. Good agreement was found with results obtained with DVMFLOW (Larsen and Walther, 1998).

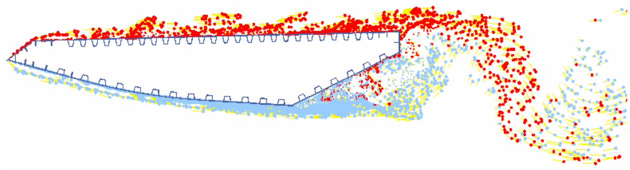

Figure 7 - Typical flow pattern of Vortex particles around cross section

From the CFD calculations, time averaged forces and moments can be easily and directly obtained and further processed in the structural analysis process. In non-dimensional form they are related to the dynamic pressure ½ ρU_∞^2 to yield the steady state coefficients for fixed cross sections:

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

Secondly, the problem of cable vibration due to wind and possibly in combination with rain or periodic excitation had to be studied. Different measures to minimize stay cable vibrations have been investigated including two kinds of cable surface treatments to prevent the formation of rainwater flows on the cables and internal or additional external damping devices.

Further dynamic wind phenomena had to be checked (Domaingo et al., 2008) in addition to the buffeting analysis, which is the primary design criterion and in detail described below. These phenomena include for instance galloping instability, which is a simplified case of the buffeting analysis.

Torsional divergence is another phenomenon, occurring if the twist of the cross section induced by the aerodynamic torsional moment increases the effective wind attack angle. If the torsional stiffness of the structure is not high enough to counterbalance this increase of moment, divergence will be observed.

The classical flutter phenomenon refers to a flow driven coupled two-degree-of-freedom oscillation of the cross section. To solve the flutter problem, a coupled system of equations of motion is constituted. The critical velocity is the point of transition from damped oscillation to sustained oscillation. (Simiu and Scanlan 1996).

The vortex shedding phenomenon is accompanied by large oscillating lifting 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 for the different natural frequencies. Design checks demand that further investigations are performed for a given natural mode if $U_{c,,x} < f_s\, U$ where f_s = additional safety factor (e.g. f_s = 1.25 in Eurocode).

Last not least, full dynamic wind buffeting analyses of the whole structure had to be performed for both, the bridge under traffic and the empty bridge. These analyses were based on the aerodynamic coefficients and derivatives derived from wind tunnel tests, and included the required non-linear damper elements provided for cable stabilization as well as for connecting the girder with the pylon.

WIND BUFFETING RESPONSE – A detailed statistical characterization of the local wind properties is necessary to describe the buffeting response of a bridge adequately. To this end, wind measurements are carried out to derive models for mean wind, turbulence intensity, power spectral density and wind coherence. This information is then used together with the aerodynamic coefficients in the framework of a modal analysis to estimate corresponding peak response of the bridge.

There are numerous assumptions that must 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 can be listed as follows:

a) For applying the superposition principle, the aerodynamic vibration amplitudes of a deck are assumed small (less than ±3° in torsion, say).
b) The aeroelastic loads and the associated flutter derivatives are assumed functions of the mean reduced frequency and static twisting angle of the deck. The span-wise correlation of loads is assumed 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 identical.
e) The dynamic system is described by linear equations of motion around the equilibrium position. The equilibrium position is dependent on the mean wind velocity.
f) Considered winds are assumed strong, with mean values of 10 m/s or higher, for the turbulence models being valid.
g) The buffeting excitation is assumed a stationary random process, i.e. the conditions of rapid change of mean wind velocity 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 as dominant wind actions to bridge decks.

The assumptions are not intended to be fully accepted in all cases, but for the investigations on the 2 presented bridges it was believed that their implications are small or negligible in comparison with other 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 (Janjic et al., 2004)

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 characterized by the mean wind velocity and the fluctuation (turbulence) velocity, both being a function of the height above terrain level. The stochastic component is accounted for by using power spectra in the frequency domain. Especially complicated wind conditions have been expected for the Stonecutter's Bridge. A 50 m high mast was therefore constructed in order to obtain accurate wind profile information such as wind speed, direction and turbulence intensities.

Figures 8 to 10 below show the resulting envelopes of dynamic internal force components from the buffeting analysis of the Stonecutter's Bridge.

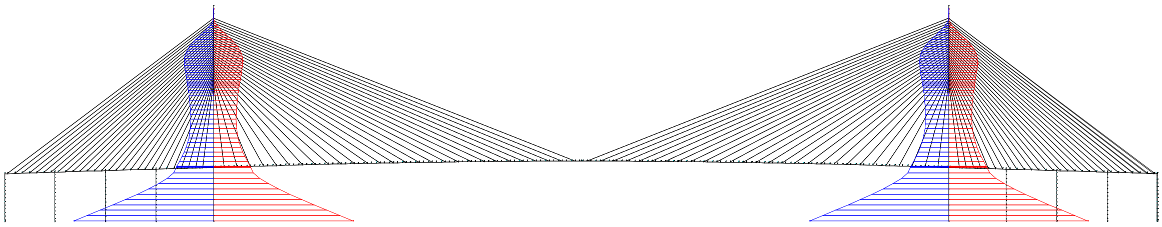

Figure 8 – Stonecutter's Bridge - amplitudes of bending moment in the pylons

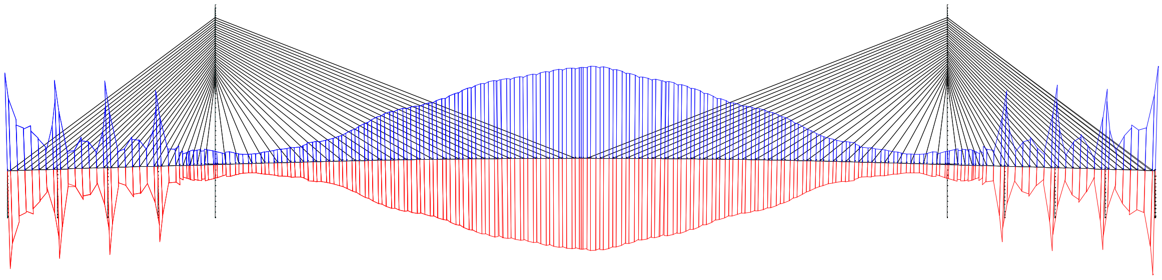

Figure 9 – Stonecutter's Bridge - amplitudes of bending moment in the deck

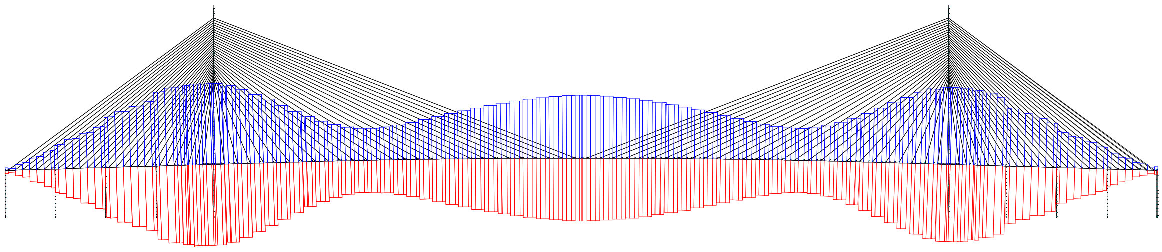

Figure 10 – Stonecutter's Bridge - amplitudes of normal force in the deck

SPECIAL DETAILS

CONNECTION BETWEEN GIRDER AND PYLONS – For the Sutong Bridge, the selected permanent connection between the girder and the pylons is accomplished by nonlinear dampers as used for the Great Belt East Bridge in Denmark.

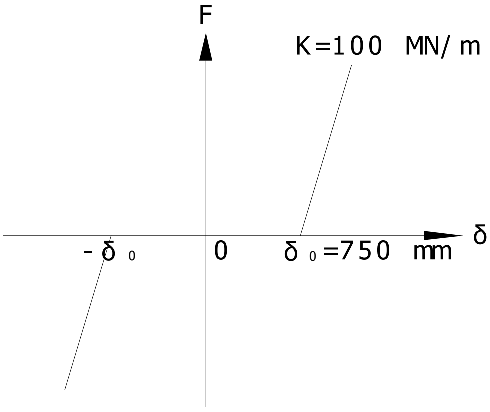

Figure 11 – Static force-displacement relationship of the dampers

These dampers do not confine the displacement of the steel girder induced by temperature, moderate wind, and vehicle traffic, but transfer the loads from the girder induced by gust, earthquake and other forces from specific load combinations to an alternative pylon.

Four dampers are placed at each pylon. A relative movement of 750 mm between girder and pylon is allowed before the displacement restriction equipment is activated. Each of the four dampers at one pylon has a linear stiffness of 100 MN/m in the event that a relative movement beyond 750 mm occurs. The static force - displacement relationship for each damper unit is shown in Figure 11.

The dynamic characteristics of the dampers is given as $F = C \cdot V^{\alpha}$, where V is the relative velocity between pylon and girder, α is a constant equal to 0.4 and C is a constant equal to 3750 kN/(m/s)$^{0.4}$.

This damper connection between girder and pylon is essential for the safety of the pylon under extreme wind and seismic loads. Based on detailed parametrical studies and some further considerations for installation tolerances and safety margins a maximum damper force of 10 MN was one of the design requirements. The dynamic calculations with RM showed, that these requirements can be achieved with the above specified damper characteristics.

SUMMARY

The numerical investigations of the Stonecutter's Bridge and of the Sutong Bridge have been carried out with the computer package RM2006 by the local design teams (Ove-Arup, HPDI) with strong support of and close collaboration with the program development team of TDV at Graz, Austria.

The extended demands of these challenging projects and the experiences gained in the cooperation phase gave considerable and valuable input for the further development of the software package. Many special functions for solving complex problems were implemented as direct consequence of the project work.

One of the special emphases has been on the solution of optimization problems with respect to the stay cable tensioning strategy. Another major problem was related to erection control issues with correct pre-cambering rules taking into account the stage-wise construction and time-dependency effects.

The third important field of problems was related to the susceptibility of these slender structures to wind-induced vibrations. The used wind related functions match nearly all needs for the design of long-span bridges. Arbitrary complicated wind profiles with varying wind speed and turbulence intensity were 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. The presented algorithm predicts wind-buffeting response within structural non-linear analysis.

The proposed analysis methods handle satisfactorily static and dynamic bridge behavior up to the time infinity; they are generally suitable for the investigation of cable stayed bridges. Cable sagging, p-delta effects, large displacements or even contact problems can be combined with long term effects within consistent analysis.

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