Monitoring of Wind Effects on the Cable-stayed Bridge across the Ziegelgraben

Dr.-Ing. Karl Kleinhanß *
Dr.-Ing. Björn Schmidt-Hurtienne **

* DEGES, Deutsche Einheit Fernstraßenplanungs- und –baugesellschaft, Berlin, Germany, kleinhanss@deges.de
** EHS beratende Ingenieure für Bauwesen Dr.-Ing. Schmidt-Hurtienne - Dr.-Ing. Osteroth GmbH, Lohfelden, Germany, bsh@ehs-ingenieure.de

Abstract:
The second Strelasund link between the island of Rügen and the city of Stralsund crosses the Ziegelgraben with a cable-stayed bridge 42 m above sea level. To prevent frequent traffic restrictions due to strong winds, wind shielding walls have been installed after investigating their effectiveness by theoretical simulations and wind tunnel tests.

In order to calibrate the traffic regulations under in situ conditions, an extensive monitoring system has been installed collecting data of the wind profile above the bridge deck and on top of the 128 m high pylon. Wind speed and direction are evaluated to determine the tolerable maximum wind speed for light vehicles.

The 32 cables of the Ziegelgrabenbrücke consist of parallel strands, a first application for German interstate roads. The aerodynamic properties of the cables, partially equipped with hydraulic dampers, are controlled by acceleration gauges to measure wind- and traffic- induced vibrations.

This report points out how precautionary measures during the construction and early service phase - adding functional elements and supervising them by a carefully designed monitoring program – will contribute to serviceability and structural integrity of a cable-stayed bridge exposed to strong winds.

Keywords: Wind monitoring, wind-induced cable vibrations, cable damping, fatigue strength

1 Introduction

The 4100 m long Strelasund link including the Rügen Bridge provides an efficient connection between the island of Rügen, the city of Stralsund situated on the shore of the Baltic Sea and the "Ostseeautobahn" A20 preventing traffic jams quite common along the 70 year old Rügendamm.

Figure 1: Cable-stayed Ziegelgraben bridge

The link crosses the eastern part of Stralsund, the Ziegelgraben, the island of Dänholm and the Strelasund and consists of six individual bridges, - a beam slab girder from prestressed concrete, a steel composite bridge, a cable-stayed steel bridge and three prestressed box girder bridges.

The cable-stayed bridge crosses the Ziegelgraben with a minimum clearance for shipping vessels of 42 m and a main span of 198 m. Its aerodynamically shaped bridge deck is supported by 32 cables suspended from a 128 m high, framed steel pylon (figure 1).

The bridge incorporates a number of innovations and first applications in Germany among other things stay cables from parallel strands, self compacting concrete, wind-shielding walls, high containment guardrails and the adaptation of superposition rules in the design codes [3].

To evaluate the occurrence of wind-induced cable vibrations and to verify the effectiveness of the wind-shielding walls a monitoring program has been installed on the cable-stayed bridge during the construction phase and the early service phase.

2 Innovation Management

2.1 Stay Cables

The tender documents asked for locked coil ropes (∅ 120 mm) from St 1570 traditionally used in Germany and meticulously described in the corresponding codes. In addition, the technical requirements for the application of parallel strand cables were given. After the award of the contract, this alternative was proposed by the contractor and accepted by the owner. The chosen system consists of 34 strands - with the option of adding 3 more strands - ∅ 0,6" (15,7 mm) of St 1570/1770, which are anchored by wedges only and may, hence, be individually replaced, figure 2 [4].

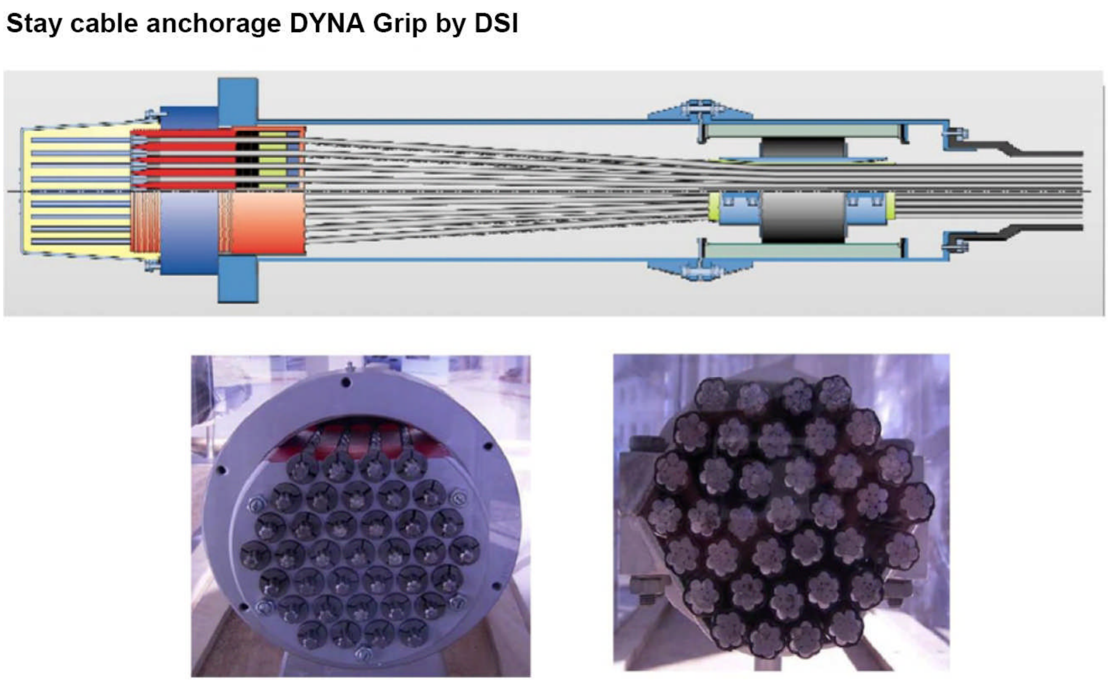

Figure 2: Parallel strand cables with anchorage

The corrosion protection consists of
- galvanizing of the wires
- a PE sheath with a wax filling
- a HDPE-pipe ∅ 180 x 10 with an extruded Scruton helix.

2.2 Wind Shielding

One of the main points of interest was road safety, to guarantee that traffic on the bridge can run efficiently, safely and without interference. To fulfill these requirements, the wind speeds being considerably higher at greater altitude and the impact of crosswinds were taken into account. Numerical simulations and wind tunnel tests proved that a glass wind-shielding with a height of 1.5 meters integrated into the railing would provide sufficient shelter even from strong wind. Therefore, it can be assumed that cars as well as loaded trucks will be able to pass the bridge even at times of strong winds. The verification of the effectiveness of the wind shielding was one of the objectives of a comprehensive measuring program described in sections 3 to 5.

2.2.1 Wind design

Considering that wind has no great impact on most bridges, this point is regulated on a rather general basis in the Appendix N of the DIN report 101.

The wind loads indicated in table N.1, for example, are given independent of wind zone and cross-section design. They only depend on the height of the bridge as well as the width to depth relation.
The wind load which – for the first time in German regulation – has to be calculated along the longitude of the superstructure is also regulated in rather general terms (i.e. independent of the surface roughness e.g. due to transversal girders) to be 25 % and 50 %.
In order to optimize the design parameters, a more precise series of verifications had to be given for the Ziegelgraben bridge, since it is located right on the shore of the Baltic Sea and has a height above sea level of approximately 50 meters.

2.2.2 Crosswind

To shield traffic, especially caravans and unloaded trucks, from strong winds it was decided to install wind shielding screens on the upstand at the edge of the bridge, even though the detailed design had almost been completed.
Via numerical fluid dynamics simulation and wind tunnel tests the impact of these wind shielding screens on the overturning moment of a truck – 4 meters in height – and on the load exercised on the super structure and the pylons was investigated. Numerical analyses clearly show the shielding effect of the wind shielding screens (figure 10)
The overturning moment affecting the truck is reduced by approximately 50 % while the wind speed considered to be dangerous is raised by 40 %.

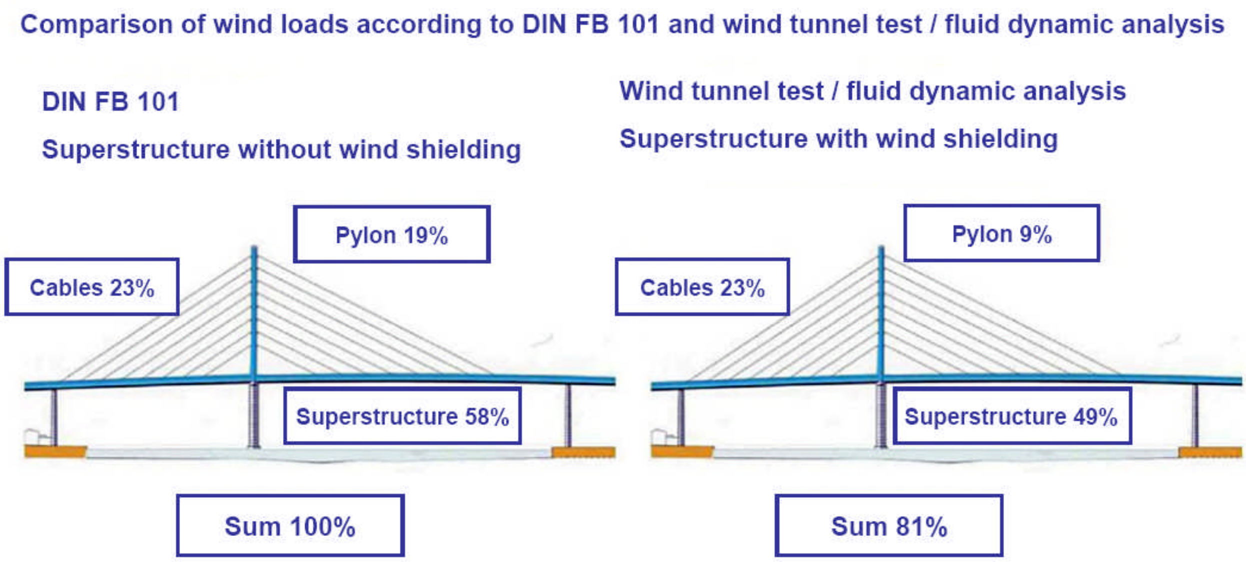

Figure 3: Reduction of wind loads due to wind tunnel test and numerical simulation

2.2.3 Wind on superstructure and pylon

A comparison between wind loads according to the DIN report and the wind tunnel test is shown in figure 3 [3].
The precise calculation of coefficients resulted in loads that - in spite of the wind shielding screen - were 20 % lower than calculations made according to DIN FB 101.
This leads to interesting conclusions for the retrofitting of other bridges. Numerical simulations and wind tunnel tests of aerodynamically shaped super- and substructures yield lower loads. The design of protection barriers must not necessarily lead to higher cost for the structure.

The wind shielding screen was designed with c_f = 1,7 resulting in a wind load of 3,67 kN/m² (with a dynamic pressure of 2,16 kN/m² or a velocity of 59 m/s at a height of 50 meters), more than the values indicated by DIN FB 101.

It could be proved by a Finite Element calculation taking into account nonlinear third order theory that for a continuously supported laminated safety glass a thickness of 2 x 6 mm is sufficient.

2.3. Adaptation of Design Rules

DIN FB 101 indicates different ways to carry out superposition of wind and temperature: Appendix C.2.1.1 states that for road bridges wind and temperature influences should not be taken into account at the same time; Appendix O.1 (2), though, states that wind and temperature should be entirely superposed.
This coincidence of extreme temperatures and hurricane force winds is not to be expected at the location of the bridge. Therefore, a combinational coefficient $\Psi_0 = 0{,}6$ was introduced.
In pylon axis 170 the complete dilatation could thus be reduced from 933 to 787 mm, which made it possible to simplify expansion joints.

3 Monitoring Program

Due to the special conditions and the innovations introduced during construction of this cable-stayed bridge [2], a comprehensive supervision and monitoring program was installed. It was used during the test stage to support the assumptions made during the planning and construction process and to create a database for future operation.

The measuring program includes measurements to analyze load bearing capacity and stiffness characteristics of the structure. Further measurements are performed to determine climatic conditions that could influence traffic.

When constructing bridges of this size, measurements are necessary to verify mathematical assumptions, since there are certain factors like excitation mechanisms for cable vibrations that can only be forecast to a certain extent.

The assumptions made by the DIN FB 101 concerning the superposition of wind in longitudinal direction with extreme temperature strain and the diminution of wind load assumptions based on the wind tunnel tests should also be verified through measurements of the dilatation on the expansion joints.

Measurements have been continuously in place since approximately 10 months prior to the opening of the bridge to public traffic in October 2007.

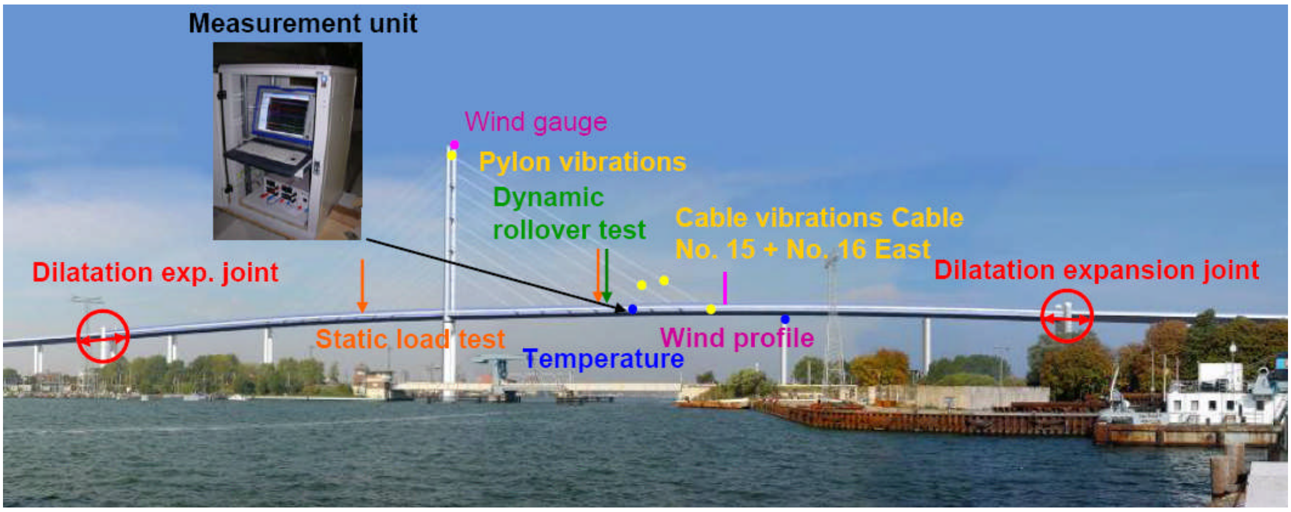

Figure 4: Monitoring program

Figure 4 gives an overview of the measurement program.

Static load tests were performed with three 30 t vehicles showing excellent agreement with the static analysis. Vibration characteristics of the superstructure were determined via dynamic rollover tests across sleepers.
The dynamic rollover tests demonstrated the insensitivity of the superstructure to traffic induced vibrations.

Continuous measurements of cable vibrations in the longest cables facing the sea closely monitor the amplitudes and determine whether additional measures have to be taken to attenuate cable vibration.

Horizontal as well as vertical acceleration gauges record the vibrations at a distance of 0.1 L from the anchors of the cables. Via additional acceleration gauges on the pylon and on the base of the cable it is analyzed whether parameter excitation can be identified as a cause for cable vibration.

The impact of the wind on cable vibration is constantly monitored by recording wind direction and speed.
The air speed indicators are installed at the base of the pylon (2 m above sea level), on the superstructure (ca. 44 m above sea level) and on the pylon head. This way the impact of the topographical roughness on wind speed is measured.
Additionally the wind profile is continuously measured on the bridge deck before and after installation of the wind shielding.
On the one hand, these measurements serve to verify theoretical analysis of the wind shielding, on the other hand they provide data concerning wind power under operating conditions and thus supply necessary data for decisions about when to close the bridge for traffic in case of strong winds.
Deformation through temperature and wind in longitudinal direction of the bridge are measured in both bridge joints with wire displacement sensors to verify the choice of the combinational coefficient.

Outside air temperature is measured via a shielded air temperature sensor on axis 210. The steel temperature is measured in the box girder on the upper and lower flange via surface temperature sensors in the center of the main segment. In addition, the inside air temperature within the box girder is recorded. These temperature measurements are not only supposed to show correlation with dilatation within joints but also provide data on temperature differences between upper and lower part of the box girder before and after installation of the road surface.

All measuring data is centrally recorded by a measuring unit in the box girder (figure 4).
Remote control is used to access results and control system configurations for sample rate, filter and trigger values.

4 Cable Vibrations

Stress and vibration under wind action result from a multitude of excitation mechanisms that have not yet been fully described. The parallel strand cables used here are less susceptible to excitation mechanisms compared to other high-strength tension members, since

- the supports work as dampers
- the Scruton helix on the sheathing attenuates wind- and rain-wind induced vibrations as no oscillating water rivulet is generated.

According to theoretical dynamic analysis the excitation of cable vibrations is most likely to be caused by periodic displacements of the anchorages at the pylon.

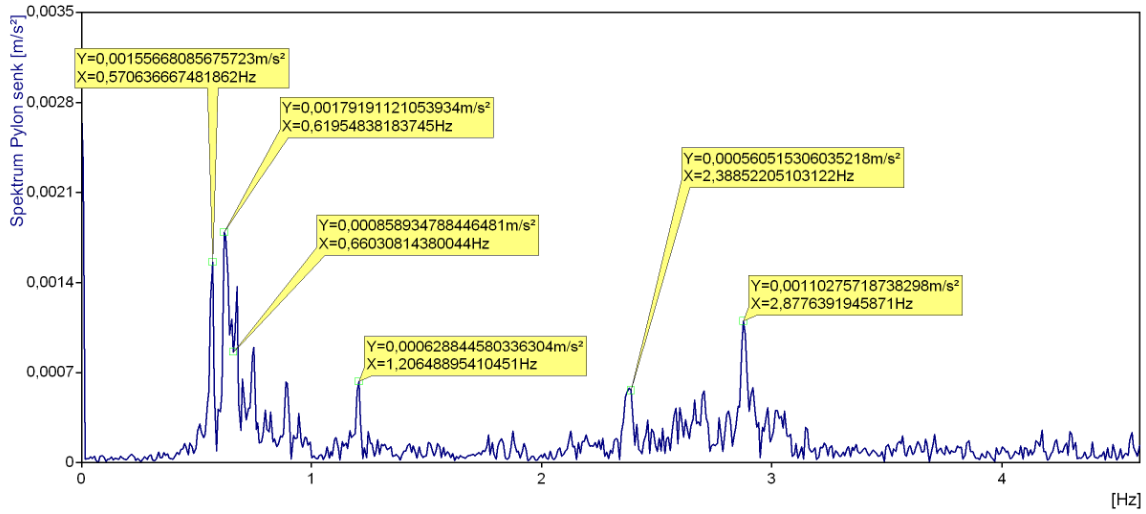

Figure. 5: FFT of transversal vibrations of pylon

Figure 5 shows the FFT of the transversal pylon vibrations as recorded by the monitoring program. The lowest frequency of transverse pylon vibrations with 0,62 Hz is close to the base frequencies of cables 15 and 16 with 0,66 Hz and 0,57 Hz. The critical wind speed for vortex shedding on the drop-shaped cross-section of the pylon is about 30 km/h. Moderate steady wind in longitudinal direction is most likely to stimulate cable vibrations on the longest cables.

The criterion for the maximum allowable amplitude of cable vibrations was chosen according to the psychologically motivated limit recommended in the fib bulletin 30 [1]. The amplitude of first mode vibrations is limited to f = L/1700. For the longest cable this corresponds to an amplitude of 10 cm.

Considering fatigue this psychologically based criterion causes

with $\Delta\alpha = 2\,x\,\dfrac{\pi \cdot f}{L} = 3{,}70x\,10^{-3}\,rad$

a resulting bending stress of $\Delta\sigma_B$ = 9 N/mm² at the anchor head and 18 N/mm² at the support amounting only to 17 % of the allowable fatigue strength.

Due to the fatigue load model (n = 2 x 106) of DIN Fb 101 the calculated stress amplitudes are only 35 % of the acceptable value (figure 6).

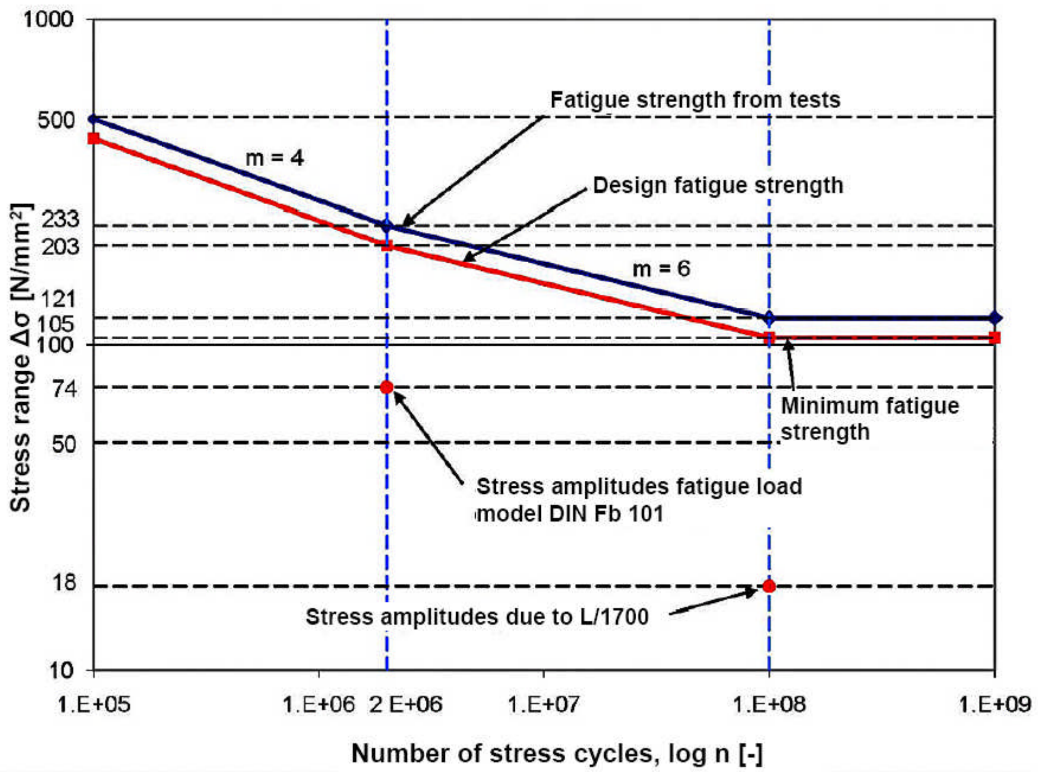

Figure 6: Fatigue strength of stay cable system

The limit for cable vibrations is designed with a large margin to the fatigue strength of the stay cable system. The L/1700 criterion was set as a threshold for taking measures to retrofit the cables with damping devices as provided for in the tender documents. Fatigue-relevant cable vibrations require amplitudes of more than 50 cm regarding the longest cables.

4.1 Construction phase

After completion of the steel structure cable vibrations were recorded on the two longest seaside cables No. 15 and 16.

The largest cable vibrations were recorded in February 2007 before installation of the mastic asphalt on the bridge deck (figure 7).

On February 28th the maximum horizontal amplitude at the acceleration gauge of cable No. 15 was recorded as 47 mm amounting to 153.5 mm at center of the cable with first mode vibrations excited. With concurring vertical vibrations of 40.8 mm the total amplitude amounted to 159 mm exceeding the criterion of L/1700 by more than 60 %. The diagram in figure 7 includes wind speed and direction measured 4 m above the bridge deck.

During the incident of cable vibrations the wind direction was southwest (215° to 230°) which corresponds to the longitudinal axis of the bridge (red line in the compass of figure 7). As predicted in the prognosis the wind speed was moderate and oscillated between 20 and 30 km/h, which is in the range of the critical wind speed for vortex shedding.

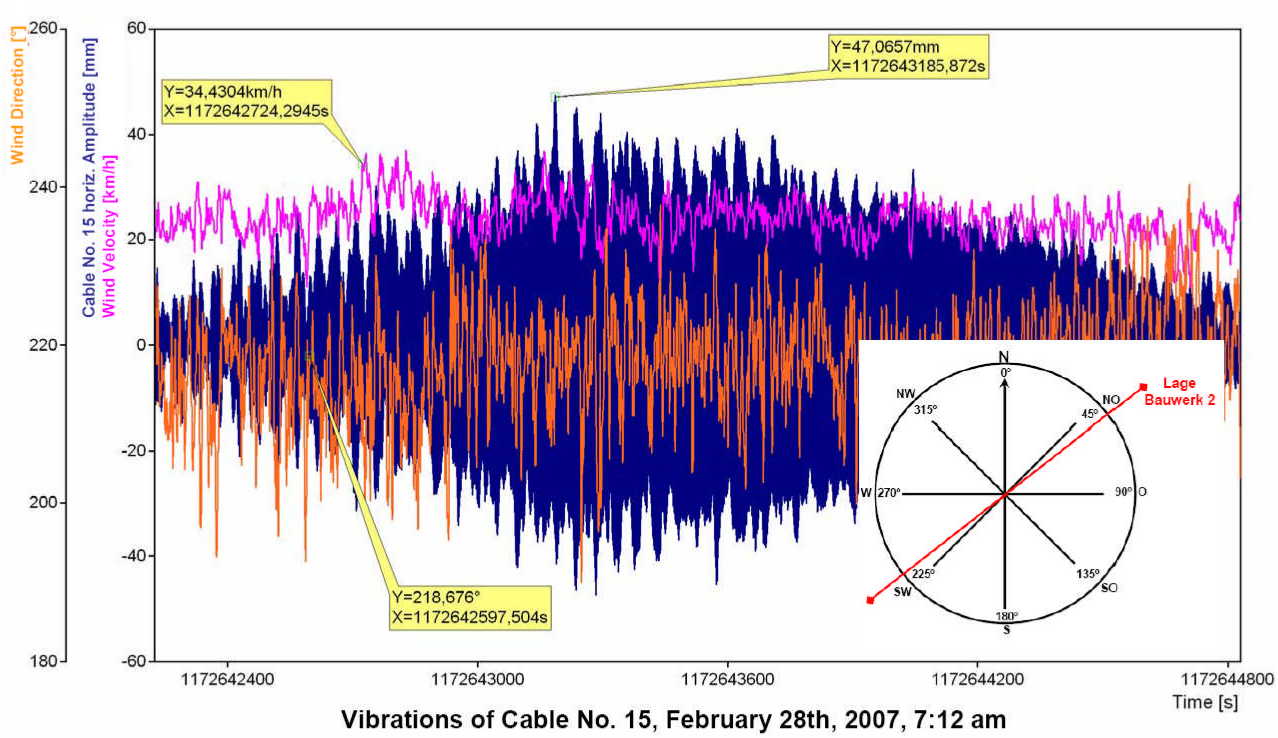

Figure 7: Cable vibrations of cable No. 15 before installation of damper

4.2 Damping

Due to infrequent cable vibrations exceeding the maximum allowable amplitude of L/1700 on cables 15 and 16 the longest cable pairs 1, 2, 15 and 16 were subsequently equipped with dampers.

The hydraulic dampers were designed for a damping constant of 20400 Ns/m and mounted on a cantilevered steel structure that was attached to the bridge deck on anticipatorily designed protective sheets. Figure 8 proves that an elegant architecturally unobtrusive damping device can be integrated harmonically.

The effectiveness of the dampers was checked by manual excitation tests that were recorded by the measurement unit.

Figure 8: Cable anchorage with cantilevered hydraulic damper

4.3 Service phase

After installing the dampers the monitoring of cable vibrations on cables No. 15 and 16 did not record any significant vibrations in vertical direction (in plane of pylon and cable). Horizontally the maximum amplitudes are substantially reduced by the dampers. Horizontal vibrations are extremely sporadic and coincide with pylon vibrations (figure 9).

In Figure 9 horizontal amplitudes of pylon vibrations in transverse direction of 7,5 mm (dark blue curve) lead to first mode cable vibrations of about 60 mm on cable 16 (red curve). These vibrations occur only during longitudinal inflow with moderate wind speeds of 25 to 30 km/h. All cable vibrations recorded after installation of the dampers stay well below the allowable criterion of L/1700.

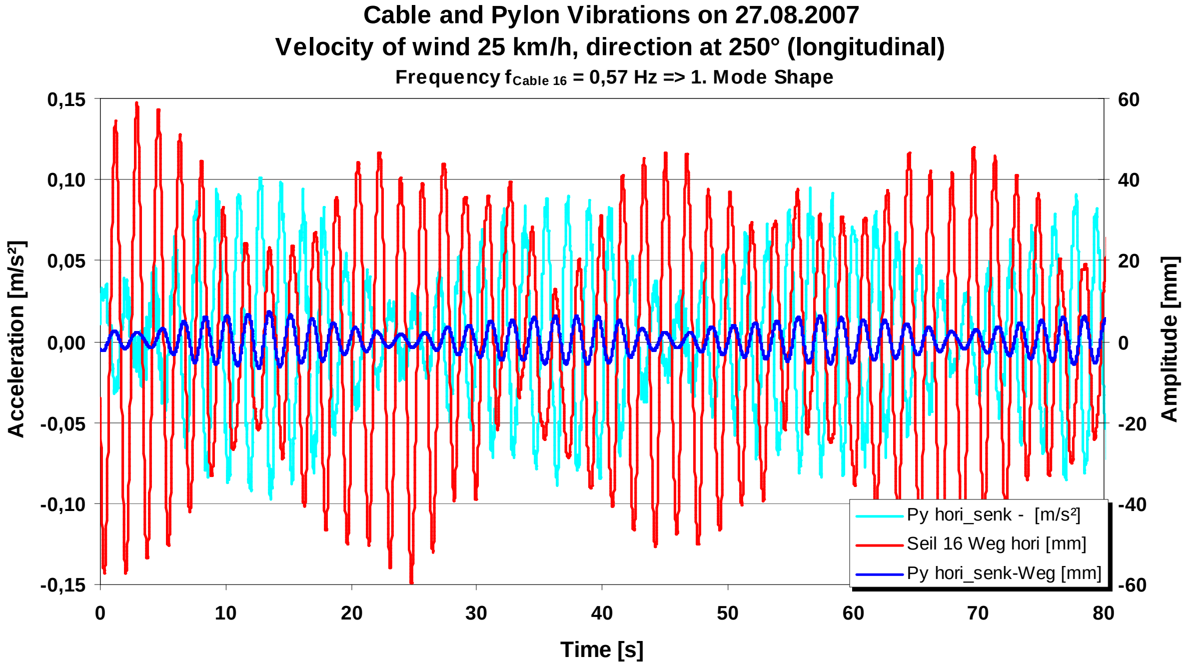

Figure 9: Horizontal cable and pylon vibrations

During gale-force winds no excitation of cable vibrations has been recorded. Wind-buffeting does not lead to instabilities.

5 Wind measurements

In Figure 10 the location of the wind gauges is superimposed on the results of the numerical simulation of the wind velocity field showing the intensity of the wind flux around the cross-section of the superstructure with wind-shielding as a percentage of the inflow velocity.

The stands are set up close to the wind-shielding not to disturb the traffic. A substantial reduction of wind speeds should be recorded on the windward side only by the lowest wind gauge, whereas leeward all gauges should show wind speeds below the inflow velocity.

In Figure 11 a characteristic wind measurement is shown with longitudinal inflow as well as transversal inflow. The latter is critical for the evaluation of the effectiveness of the wind shielding protecting light vehicles from crosswind.

During longitudinal inflow all wind gauges record similar wind speeds increasing marginally with height. However when the incoming wind is transversely directed towards the bridge, the wind speed at the lowest windward sensor is for example reduced to 12 km/h, whereas the second sensor 3,4 m above the pavement records wind speeds of 48 km/h.

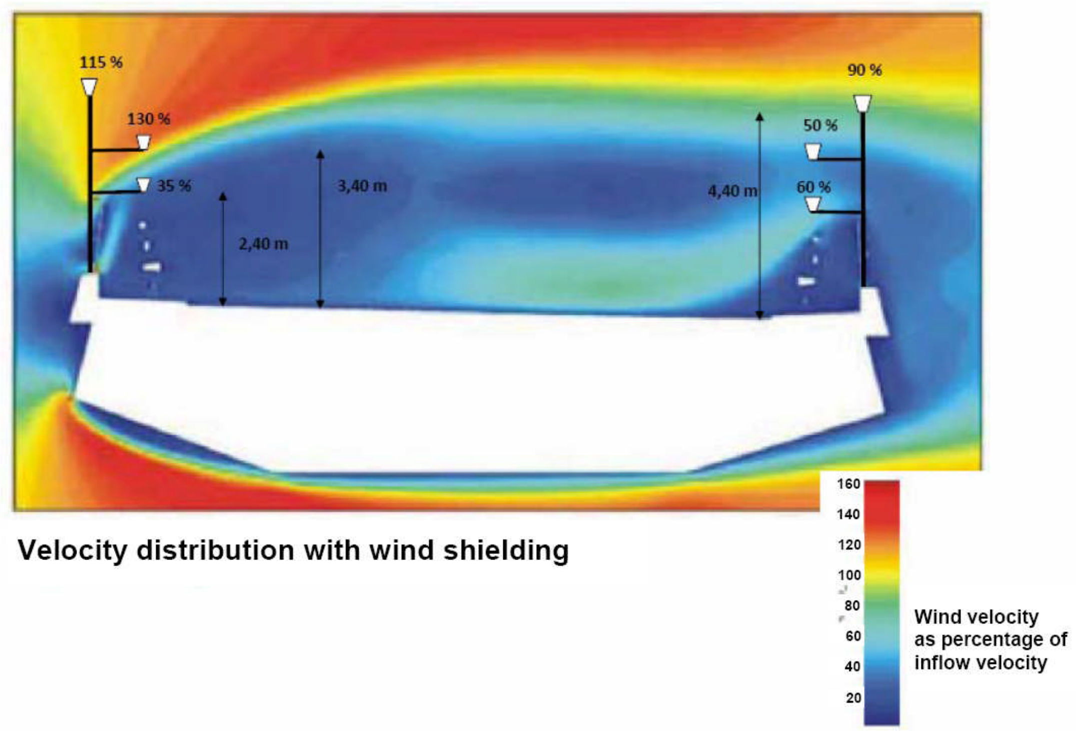

Figure 10: Numerical simulation of velocity field and location of wind gauges

This corresponds to a wind speed ratio of

$$v_{2,4m}/v_{3,4m} = 12 / 48 = 0,25,$$

the numerical simulation yielded by way of comparison:

$$v_{2,4m}/v_{3,4m} = 35\ \% / 130\ \% = 0,27.$$

The measurements prove the shelter effect due to the wind shielding and confirm the results from the wind tunnel tests and numerical simulations.

Regarding the influence of strong longitudinal winds on the dilatation at the expansion joints the measurements have given no hint at a longitudinal movement of the bridge due to exposure of the pylon, cable and bridge deck areas to wind action. The reduction of the combinational coefficient was confirmed by the measurements. The dilatation at the expansion joints results exclusively from temperature, creep and shrinkage. The necessity to superimpose longitudinal wind action with other loads should be reviewed in the European standards.

6 Conclusion

During the design and building of the second Strelasund link several innovations and first-time applications were implemented. For verification purposes a comprehensive monitoring program was established. Regarding vibrations of the stay cables consisting of parallel strands the measurement of accelerations on the longest cables led to the installation of hydraulic dampers on 8 cables thereby limiting vibrations below the strict criterion of L/1700 for vibration amplitudes. The limiting factor for cable vibrations is the psychologically tolerable amplitude not the fatigue strength of the cables at the anchorages.

The effectiveness of the wind shielding was first analysed by wind tunnel tests and fluid dynamic analysis and then proven by wind measurements. Additional wind measurements on the adjoining bridge without wind shielding and with a lower elevation show that the cross-sectional design of upstands reduces the crosswind load on vehicles already to the level on the mainland, so that the traffic along the complete bridge link requires no special restrictions during hurricane-strength winds.

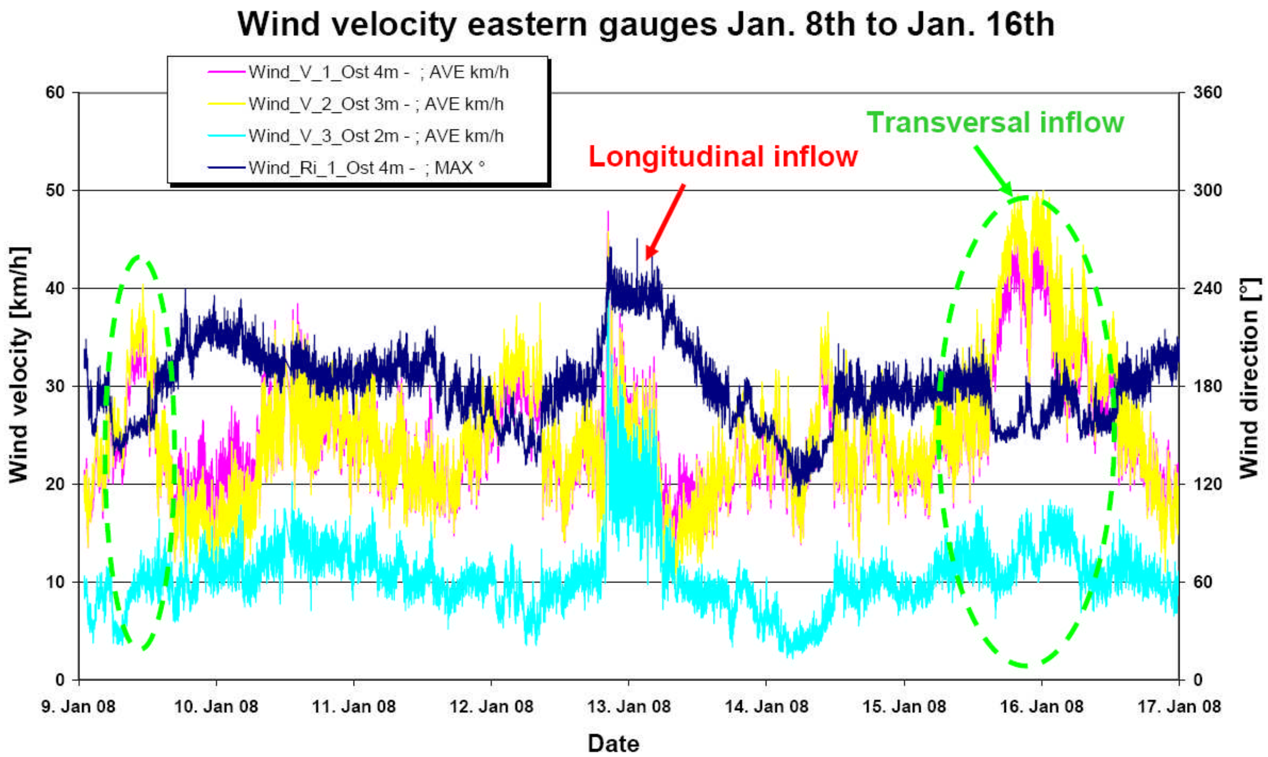

Figure 11: Wind speed and direction on the bridge deck (eastern gauges)

7 References

[1] fib, bulletin 30: Acceptance of stay cable systems using prestressing steels. Lausanne, January 2005.

[2] K. Kleinhanß, B. Schmidt-Hurtienne, M. Steinkühler: Die 2.Strelasundquerung mit der Rügenbrücke – Interaktive Entwicklungen in der Bauphase. VDI Bautechnik – Jahrbuch 2006/2007, pp. 224 – 240

[3] K. Kleinhanß, M. Romberg, R. Saul, B. Schmidt-Hurtienne: Die zweite Strelasundquerung mit der Schrägseilbrücke über den Ziegelgraben. Bauingenieur Vol. 82, 4/2007, pp 159 - 169.

[4] C. Gläser, M. Scheibe, K. Zilch: Die 2.Strelasundquerung – Erste deutsch Anwendung von Parallellitzenseilen". Bauingenieur Vol. 82, 4/2007

[5] K. Kleinhanß; R. Saul: The second Strelasund Crossing – a modern Cable-Stayed Bridge. Structural Engineering International 1/2007

[6] K. Kleinhanß: The "Ziegelgrabenbrücke", Main Structure of the second "Strelasundquerung", Proc. of IABSE, Weimar 2007