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EXPERIMENTAL AND NUMERICAL VERIFICATION OF VORTEX-INDUCED VIBRATION OF HANGERS ON THE FOOTBRIDGE

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Abstract: *The paper deals with vortex-induced oscillations of footbridge hangers at a very low wind speed. The two-spans (2x50m) reinforced concrete structure with the central pylon (25m height) is designed as the harp system of hangers with circular cross-section. The oscillations were observed at the low wind speed and the high double amplitudes has been measured. The wind-excited deformation shape corresponded to the 6th, 7th and 8th natural modes of a hanger. The individual hanger as well as their group were analysed to estimate the dynamic response, possible aeroelastic vibration and special interference or coupling effects. It also includes proposal for the elimination of the extensive vibration. The analysis is complemented by numerical investigation using Computational Fluid Dynamics methods.*

Keywords: ***Footbridge, vortex-induced oscillations, Kármán vortex, fluid dynamics.***

1. Introduction

Long-span footbridges rank among engineering structures with a high social significance. Their architecture design must in general satisfy matters such as attractive look, reliability and weight. These requirements determine the bridge to be sensitive to the wind effects and the assessment against the wind becomes crucial. One of the several wind phenomena occurring on such a type of structures is termed the vortex shedding with consequent vortex induced vibration. This flow-induced excitation originating from the periodic separation of the vortices may arise even at a low wind velocity. According to the character and the intensity of an excitations, the adequate treatment to reduce or avoid the oscillations should be carried out also in cases when the vibration amplitudes could lead to the pedestrian discomfort or panic.

In this paper, the unacceptable vibrations of the bridge hangers were measured and investigated with respect to the pedestrian comfort. The examination includes a frequency analysis of the hangers accompanied by the analysis of the origin of the unstable behaviour. Moreover the numerical simulation of such an aero-elastic system, which provides addition information to discover the problem, was carried out.

2. Bridge description

The suspended three-section bridge with the almost straight deck, see Fig.1 was examined. The bearing structure consists of pre-fabricated and pre-stressed concrete sections. An A-shaped steel pylon supports the first two sections. The bridge deck core is composed from the steal beams of 800 mm height with the distance between their axes d=3.3 m. The bridge deck is 3.9 m wide. The pre-fabricated section material is made from C55/67 concrete.

The supports are V-shaped, made of steel tubes and are founded on the drilled piles. The supporting structure rest upon the roller bearings. The bearing structure is hanging on pairs of hangers

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formed by three cables; the cables are placed in a polyethylene tube. The hangers are strained up to 45 pct of their strength at maximum and 10 pct at minimum. Maximum amplitude of tension is 80 MPa at 1200 MPa. The hangers are semi harp-type, exchangeable and rectifiable.

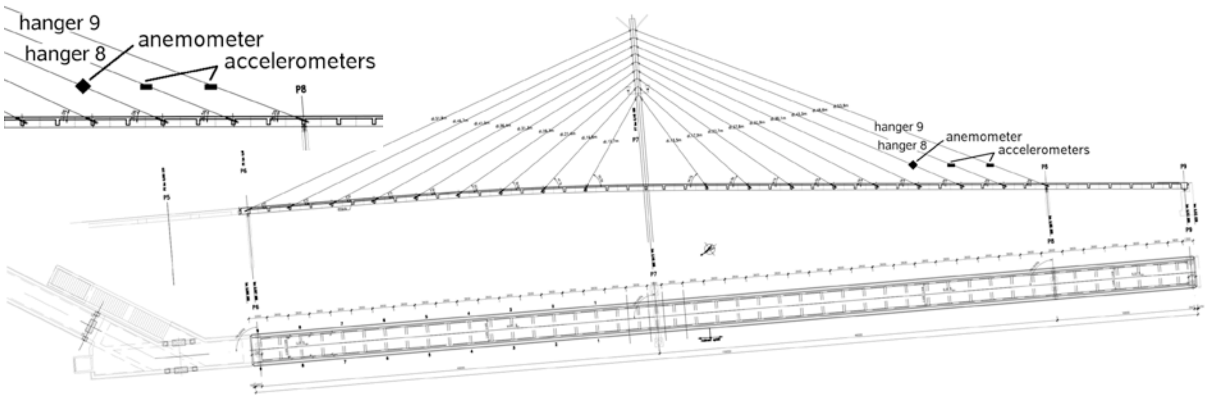

Fig. 1: Side and ground view of the footbridge.

3. Vibration measurement of hangers

The vibrations of the 9th and 8th left hangers (see Fig. 1) appeared at the relatively low wind speed (up to 3 m/s) when the wind direction was perpendicular to the footbridge axis.

Two pairs of ENDEVCO accelerometers type 86 measured dynamic response of the 9th and 8th hangers. The sensor distance from the hanger anchorage into the deck, projected onto the horizontal level, was 5 m. Tab. 1 shows the results of 60 s records for 9th and 8th hangers during the wind excitation.

Tab. 1: Forced frequency of hangers [Hz] and expected shapes of vibrations.

excitation	frequency [Hz]/ mode No. 9th hanger vertical	frequency [Hz]/ mode No. 9th hanger horizontal	frequency [Hz]/ mode No. 8th hanger vertical	frequency [Hz] / mode No. 8th hanger horizontal
wind	12.8/6	10.01/5; 12.08/6; 14.10/7	9.70/4; 12.26/5; 17.29/7	9.68/4; 10.13/4; 12.19/5; 17.29/7
wind	12.08/6; 16.26/8	10.01/5; 12.08/6; 16.26/8	9.70/4; 12.30/5; 14.80/6; 17.29/7	9.70/4; 12.30/5; 14.80/6; 17.29/7

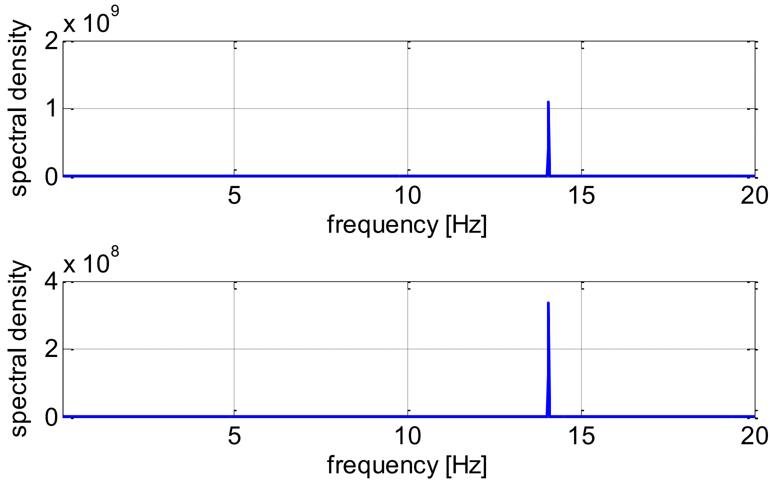

Fig. 2: Power spectral densities in vertical and horizontal directions of 9th hangers.

Fig. 2 shows frequency analysis corresponding to the measurement recorded during the excessive vibrations (see Tab. 2),while Fig. 3 shows the time history of acceleration.

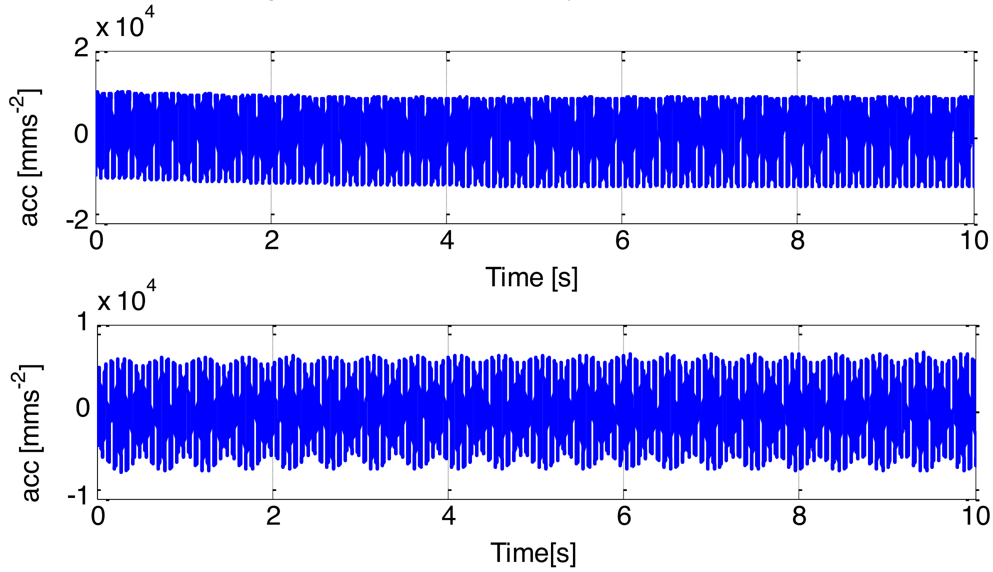

Fig. 3: Time history of the displacement of hanger 9 in the vertical and the horizontal directions.

Dynamic vertical response of the deck from the cable vibration was measured by the same accelerometer as the hanger, placed at the deck edge. In the still air periods, the 9th and 8th hangers were forced to vibrate in the vertical plane; thus, the effects of possible vandals activities were simulated. The measured signals were recorded using a DEWETRON computer type 2010 and analysed by the MATLAB software.

Tab. 2: Amplitudes v [mm] in sensor location for left and right hangers.

excitation	9th hangers		8th hangers	
	vertical	horizontal	vertical	horizontal
wind 1.75 ms^{-1}	4.6E-2	2.9E-2	2.3E-2	1.7E-2
wind 2.2 ms^{-1}	7.9E-2	2.5E-2	3.2E-2	1.6E-2
*wind 2.0 ms^{-1}	left: 1.9E-1 right: 1.36	left: 3.8E-2 right: 7.4E-1	–	–
wind 1.8 ms^{-1}	left: 8.8E-2 right: 1.45E-1	left: 3.3E-2 right: 6.6E-2	–	–
wind 1.75 ms^{-1}	left: 7.2E-2 right: 3.0E-1	left: 2.5E-2 right: 1.8E-1	–	–

Note: * The measurement with observed excessive vibrations.

Tab. 3: Frequencies of vertical vibrations of the footbridge deck.

excitation	frequency [Hz]	amplitude v_0 vibration [mm]
wind	<u>10.01</u>	≈9.2E-4
wind	1.75; 2.90; 10.01; <u>12.60;</u> 14.26	≈6.8E-4
wind	12.60; 14.00; <u>14.26</u>; 14.91	≈2.9E-3
wind	<u>12.30</u>; 12.60; 14.00	≈9.4E-4
wind	<u>1.75</u>; 12.60	1.5E-1

On Tab. 2, there are amplitudes (in mm) of vertical and horizontal movements of hangers in the sensor locations. Vertical means perpendicular to the catenary tangent, in the sensor location.

Tab. 3, there are frequencies of vertical vibrations of the deck edge near anchorage of the 7th left hanger, and sizes of deviations. Dominant frequency is underlined.

4. Forced vibration of hangers

The 9th and 8th hangers were forced to the rhythmical oscillations in the vertical plane. The excitation frequency was selected to correspond the frequency of the 1st modal shape of the hanger; in case of 9th hanger the frequency was $f_{(1)}$=2.03 Hz, in the case of 8th hanger the frequency was $f_{(1)}$=2.45 Hz.

Amplitude of the vertical vibration of the midpoint of the 9th hanger in the sensor location and in the vertical plane movement in the middle of the hanger end points is v_0= 62.15 mm. Similarly, in the horizontal direction, the amplitude was v_0= 23.8 mm. During the rhythmic forcing of the hanger, it was not possible to eliminate the horizontal component of the motion. Amplitude of the vertical vibration of the 8th hanger in the sensor location was $v_{0\exp}$= 13.55 mm. Then, the amplitude of vertical movement in the middle of the hanger end points is v_0= 38.34 mm. Similarly, in the horizontal direction v_0= 13.72 mm

From the analysis of response of the 9th and 8th hangers, footbridge section above the trackage, from the testimony of local citizens and from the video recording it is certain that the excitation is caused by the wind, the direction of which is perpendicular, or close to perpendicular, to the footbridge axis.

5. Selected aeroelastic effects

It has been concluded, from the analysis of the motion, that excessive vibration is not a wake galloping, since the hangers distance is too large, ($L > 3\ \text{m}, d = 0.054\ \text{m}$). This phenomenon would happen when the wind sped exceed 15 m/s. Parameter L is the distance between the cables. Also, the geometric configuration excludes the so-called jet-switch mechanism (Dye, 1965).

We can also exclude the rivulets from the analysis, because the above mentioned vibration was observed also in dry conditions and the relation between the frequency of vibration of this type and wind speed is dramatically different from the frequency of separated vortices corresponding to the Strouhal effect.

Hence, the effect under examination is caused by a periodical vortices separation behind a smooth cylinder with the circular cross-section. The influence of the surface smoothness is testimonied by the by the fact that for example hanger no. 9 vibrated with almost as double amplitude in the vertical direction as that one in the horizontal direction.

This is a typical effect of a periodical vortices shedding. In principle, the frequency of the separation is given by the Strouhal frequency:

$$f_{Sr} = S_r \cdot \frac{\bar{V}}{d} \tag{1}$$

where V is the mean value of speed of the wind, the direction of which is perpendicular to the roller axis, d is the roller diameter and 0.28 is Strouhal number S_r, (Anagnostopoulos, (2002), Pirner & Fischer, 2003) for the given Re number and intensity of turbulence. No higher frequencies have been considered for S_r = 0.4, 0.6.

Note: Reynolds numbers, characterising the circumfluence mode, are:

$$\text{Re}_{12,08} \doteq 0{,}13 \cdot 10^5 \ , \ \text{Re}_{14,10} \doteq 0{,}14 \cdot 10^5 \ , \ \text{Re}_{16,26} \doteq 0{,}16 \cdot 10^5 \ .$$

Hence, this mode is called "critical"; some authors call it "transitional" (Pirner & Fischer, 2003). For this mode, the specified Reynolds numbers and turbulence intensity was I_m = 9.1 % and S_r = 0.28.

The 9th hanger vibrated perpendicular to the wind direction with frequencies (see Table 1): 12.08; 14.10; 16.26 Hz.

$$\overline{V}_{12.08} = 2.31 \text{ m/s}, \ \overline{V}_{14.10} = 2.72 \text{ m/s}, \ \overline{V}_{16.26} = 3.13 \text{ m/s}.$$

Mean wind speeds correspond to those frequencies.

6. Examination of excessive vibrations using numerical simulation

To identify an origin of the unstable behaviour of the selected hanger and to investigate the possibilities to suppress excessive vibrations, two-dimensional numerical model adapted for solving the fluid-structure interaction has been defined. This method employs three boundary value problems respecting the Newton's fluid, elastodynamic equations and equations of the Lagrangian-Eulerian description of motion. A circular cross-section with adequate dimensions $d = 0.054\,\text{m}$ is free oscillating in vertical and horizontal direction x and y, respectively, with specific frequencies. Each of the degrees of freedom is proportionally damped with respect to the velocity. Furthermore, it is assumed that the hanger is subjected to the wind action with a constant velocity profile. Due to the primitive variable formulation of the Navier-Stokes equations, no turbulence is given on the inlet boundary. This does not correspond to the experiment, wherein turbulence components are always presented, and therefore certain discrepancy in the results may be expected.
Simulations have been performed with COMSOL[R] software. This partly-open computation package solves an arbitrary weak-form of integral-differential equations with boundary conditions (Pospíšil et al., 2006). It makes a possible to define the modified Navier-Stokes equations for incompressible fluid, see Tezduyar et al., (1992).

6.1. Strouhal number determination on stationary hanger

The Aerodynamic effects of the vortex separation on a structure are studied when the body is considered immovable (Simiu & Scanlan, 1996). This analysis provides fundamental data about aerodynamic forces acting on the body, which are, in general, function of the time. Drag, lift or moment coefficients can be established as well as their periodic character.

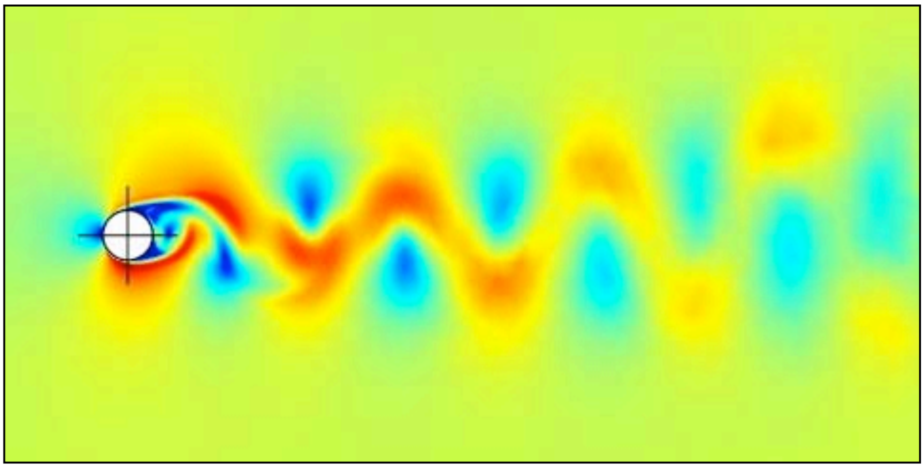

Fig. 4: Instantaneous velocity field around the hanger in the smooth flow regime at 2 m/s.

In Fig. 4 the velocity field in the immediate neighbourhood of the section is illustrated. The Kármán vortex street behind the hanger is evident. The wind velocity is $V = 2\,\text{m/s}$ and the fluid characteristics correspond to the air at 20 degrees of Celsius. Periodic flow pattern around the stationary section generates a loading in the vertical and horizontal plane, as demonstrated by the time histories diagrams in Fig. 5. Both force components are noticeable periodic with dominant frequencies. They are predicted by a spectral analysis as displayed in Fig. 6. For the vortex shedding type of instability, the Strouhal number S_r related to the vertical component is crucial. The right hand side figure shows one dominant normalised frequency $fd/V = 0.2265$ corresponding to S_r.

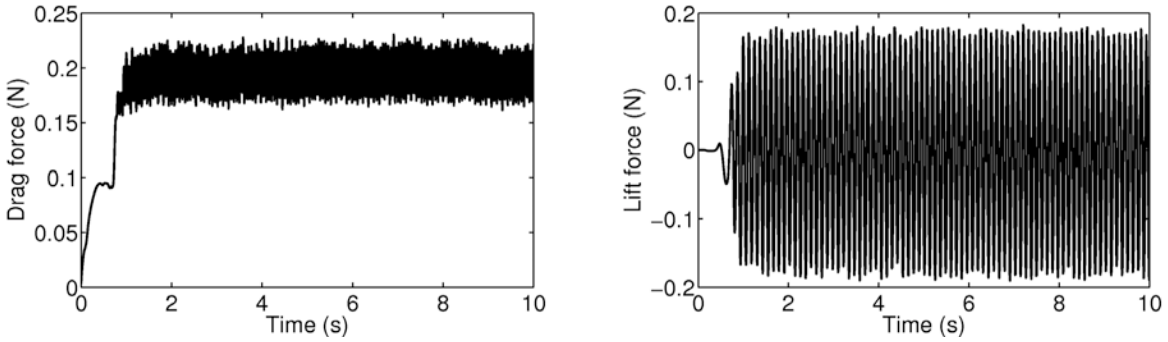

Fig. 5: Time histories of drag (left) and lift (right) force component.

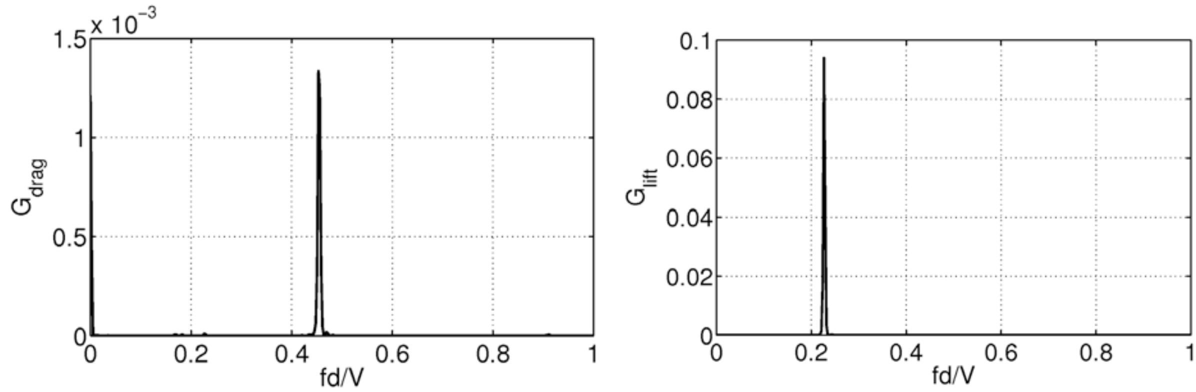

Fig. 6: Normalised power spectral density of drag (left) and lift (right) force component.

6.2. Figures and tables (subsection)

Elastic mounting of the circular section is modelled by fictitious supports. For the both degrees of freedom reaction force *F* is established in its centre representing a spring-mounting with the linear characteristic. This force depends on the body displacement and the spring stiffness *k*. Analogously; the net damping effect is applied being a function of hanger response velocity and the damping parameter δ .

The vertical and horizontal frequencies of the hanger are identical and they correspond to the seventh natural frequency $f_{(7)}$=14.10 Hz, see Tab. 1. This is achieved by the appropriate combination of the mass of the section and the spring stiffness *k*. Damping parameter is selected equally for both degrees of freedom as empirical value for steel structures; $\delta = 0.02$.

When the air is passing the circular section, the vortex development in the wind is generated acting as a powerful force on the section surface and the loading of the body itself. Provided the vortices separation frequency is relatively close to one of the natural frequencies of the hangers, harmonic fluid-induced oscillations with high amplitudes may occur. In terms of the geometric, modal and fluid characteristics, the critical wind velocity associated with the vortex-induced instability onset can be determined. Considering S_r obtained in the previous section and the natural frequency of the hanger $f_{(7)}$, an estimation of the instability onset can be stated as:

$$V = f_{(7)} \cdot d / S_r = 14.10 \cdot 0.054 / 0.226 = 3.37\ \mathrm{m}/s$$

Dynamic response of the circular section at this wind velocity is presented in Fig. 7. It is evident that for t > 0.5 s, (after vanishing fluid transient behaviour), the vibrations in the cross-wind direction increased up to the value of almost 2 mm in the steady state.

The horizontal response component is shown in Fig. 8. It is of several orders lower than the amplitude for the vertical oscillations. According to its oscillation character it can be concluded that a force vibration regime occurred however, no frequency correlation between the oscillating mass and aerodynamic forces was observed.

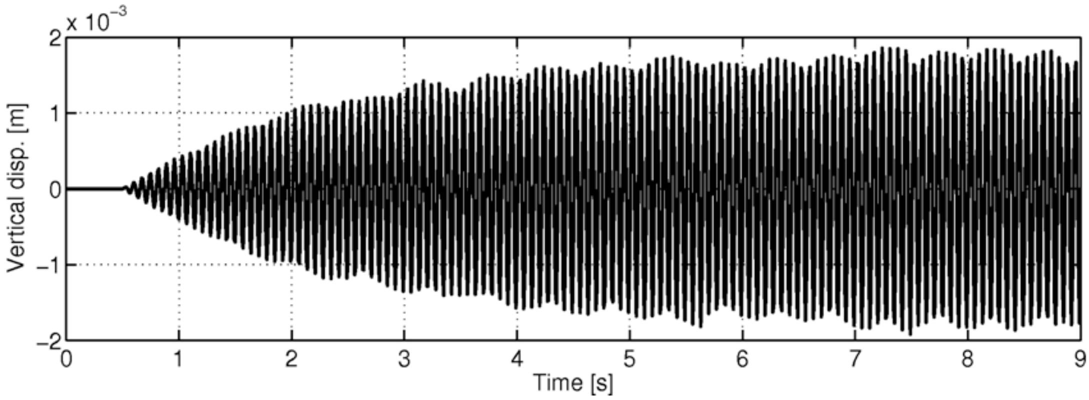

Fig. 7: Cross-wind response time-histories of the hanger 9 at $V = 3.37$ m/s .

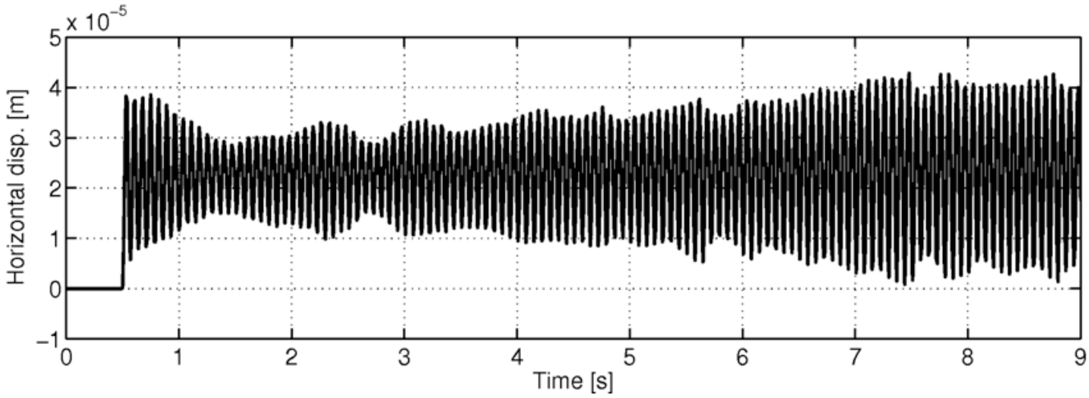

Fig. 8: Along-wind response time-histories of the hanger 9 at $V = 3.37$ m/s .

Comparing the experimentally obtained responses with the numerical ones, a good agreement is reached only for the vertical amplitudes. The numerical simulation does not cover well the spatial behaviour of the hanger along the elliptic path as it has been observed in the experiment.

Analogous simulation has been performed supposing a lower wind velocity $V = 2.70$ m/s . This value agrees with the experimental wind measurement in the anemometer location. In addition to that an initial excitation is applied to the hanger which establishes favourable conditions to start oscillating. This procedure simulates the case of lock-in mechanism when a part of the hanger (at the top of the pylon, e.g.) is subjected to the vortex shedding instability while the remaining part is placed in the wind with lower velocity. The dynamic behaviour of the hanger under such conditions is depicted in Fig. 9. From the figure it follows that the system does not maintain the stable vibrations and the amplitudes are decaying. In fact, it proves that even though the effective length of a hanger with generation of the flow-induced oscillations is limited, excessive vibrations of the whole hanger may arise and sustain for relatively wide range of the wind speeds.

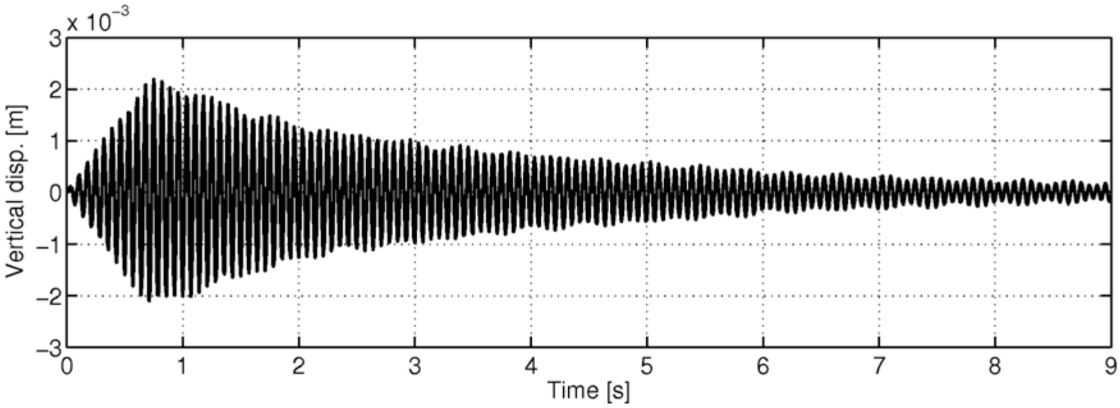

Fig. 9: Cross-wind response time-histories of the hanger 9 at $V = 2.70$ m/s *under an initial excitation.*

7. Conclusion: proposal on elimination (or reduction) of the hanger vibrations

Hangers 9 and 8 in the footbridge section above the bridge deck vibrate with higher shapes and clearly visible amplitudes under the load of the wind as slow as 1.75 to 2 m/s, if its direction is perpendicular to the footbridge axis.

Vibrations of the cables under the wind load are results from the periodical separation of vortices behind the hanger with circular cross-section (Strouhal vibrations, Kármán vortex path). Theoretical calculation has indicated that:

Vibrations of the cables caused by wind load on the deck can be neglected.

The main cause of the unacceptable effect is the smoothness of the PE pipe surface affecting the flow regime.

The vibrations of the 9^{th} and 8^{th} hangers caused by wind do not pose a threat to the footbridge structure (or cables, deck or pylon), so it would be possible to keep it in the original state.

However, the motion of the cables could lead to the discomfort or fear of some pedestrians and, in the case of a larger amount of people on the footbridge, even panic. Therefore it is necessary to reduce the vibrations to the indistinctive level, or eliminate them.

In this case, there are three possibilities:

- To equip the 9^{th} and 8^{th} hangers with Stockbridge dampers (see Anagnostopoulos, (2002), the dampers there are doubled; in our case, single ones will do).
- Interlocking, by means of an additional cable, the 9^{th} and 8^{th}, perhaps also the 7^{th}, hangers and anchor them into an anchorage "block".
- Changing the flow regime by coarsening the hanger surface through covering with (sticking) grains with diameter up to 5 mm onto the upper half of the circular cross-section.

The first possibility affects negatively the footbridge architecture. Despite that, the necessary masses of the dampers for the 9^{th} hanger are as follows: Total mass of 3 cables is 171.27 kg. The recommended ballast mass is 1/10 to 1/20 of the total cable mass, i.e., in the more efficient case, $m_{abs,tot}$ B~ 17 kg.

The 9^{th} hanger vibrates in the 6^{th}, 7^{th} and 8^{th} shape when subjected to the wind. The number of the vibration mode is identical with the number of antinodes and hence with the number of Stockbridge dampers.

The ballast mass, providing they are made of steel, would not exceed the hanger diameter in order to maintain the visual aspects of the footbridge.

Acknowledgement

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