Effects of Violent Vibrations of Cables on Dynamic Behaviour of Cable-stayed Bridges: Rehabilitation of Dubrovnik Bridge

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ABSTRACT: Shortly after the opening of Dubrovnik (Croatia) cable-stayed bridge in 2002 the violent vibrations of the bridge deck, pylon and cables in a strong wind "Bora" combined with light rain were observed about twelve times a year. This paper analyses effect of violent vibrations of cables and their influence on the overall dynamic behaviour of cable-stayed bridge. Firstly, the vibration of a massive cable which is connected to a vibrating cable-stayed bridge deck and pylon is presented analytically. Equations describing a change of the cable's force and its eigenvalues due to the cable’s undamped and damped vibrations are presented. Finally, the measures for Dubrovnik bridge rehabilitation are presented in a way of suppressing cables vibrations by devices. After installing the Maurer Adaptive Cable Dampers (ACD) in cables, regular long distance monitoring of the Dubrovnik cable-stayed bridge behaviour was established.

1 INTRODUCTION

The most violent vibrations of Dubrovnik bridge cables, especially the longest stay cable were observed on March 2005 when wind speed was 22-24 m/s causing the longest cable amplitude about 2.5 m. Cables vibration amplitudes were so large that two pairs of stay cables hit the light posts, broke them down with heavy damage incurred to parts of HDPE protective pipes. These large amplitudes were accompanied by very disturbing rattling noise caused by fretting of the steel strands as they moved against each other. The superstructure also moved in combine torsion-sway and bending modes. Quite a few pretensioned M-24 high-strength bolts at the cable to pylon anchorages connecting the steel tubes placed around HDPE pipes to allow installation of elastomeric bearings were broken and fell down on the bridge deck. Before the rehabilitation of the bridge another heavy damp snow storm hits the bridge causing violent vibrations of six longest stay cables of the main span, also accompanied by considerable movements of the superstructure (Savor et. al., 2006). This frighten event was even presented on Croatian national TV being a top daily news. Vibration amplitudes of stay cables were even greater comparing with amplitudes in the previous event. This occurrence cannot be attributed to rain-wind inducement but to the galloping of stay cables as well, because of incensement of wet-snow deposits on the wind side of stay cables which changed their circular cross-section.

The behaviour of cable-stayed bridges has been studied in many aspects, both in linear field (Tuladhar et. al., 1995, Au et. al., 2001, Gattulli and Lepidi 2007) and in nonlinear field (Ali and
Abdel-Ghaffar 1995, Gatulli at. al., 2002, Gatulli and Lepidi 2003, Virlogeux, M. (2005), some of them supported by experimental verification. The aim of this paper is neither to elaborate developments and advanced methods in analysis of cable-stayed bridges nor to elaborate further developments of existing methods but to focus on the causes of behaviour of one typical cable-stayed bridge in a moderate or strong Bora wind caused by violent vibrations of its cables and finding the explanation for such behaviour.

Starossek (1994) reviewed the history for cable vibrations starting from the 18th century with d’Alembert and Lagrange’s solution for the linear vibrations of an inextensible massless string to the Tonis (1989) nonlinear cable dynamics.

The problem of dynamic interaction between cables and other structural elements (cable-stayed bridge superstructure or pylon, antenna mast, etc.) demands research of boundary induced cable vibrations. This dynamic interaction was firstly a topic of research of several authors such as: Davenport and Steels (1965), Hartmann and Davenport (1966). More refined theories were presented by other authors and a complete linear theory of boundary-induced vibration of a damped cable was given by Veletsos and Darbre (1983) and Starossek (1991). Finally, dynamic excitation mechanisms and dynamic interaction between cables and other structural elements in bridge engineering were presented by Miyata, et.al. (1998), Causevic (2006, 2009), Caetano, et.al. (2008, 2000, 2000).

The differential equation of motion for parabolic cable and the value of a massive guy cable force \(dT\) due to cable’s undamped vibrations are given in the following forms (Davenport and Steels 1965):

\[
\frac{w}{g} \left[ \ddot{y}(x,t) + \frac{x}{s} \dot{y}(t) \sin \alpha \right] + c \left[ \ddot{y}(x,t) + \frac{x}{s} \dot{y}(t) \sin \alpha \right] = T \cdot \eta''(x,t) + dT \cdot y''(x) \tag{1}
\]

\[
dT = \frac{AE}{s} \cos \alpha \left[ \dot{y}(t) + \frac{w}{T} \int_0^s \eta(x,t) \, dx \right] \tag{2}
\]

where \(y'' = \frac{w}{T} \cos \alpha\) presents the basic differential equation of cable’s equilibrium; \(A\) is cross sectional area of cable; \(E\) modulus of elasticity of cable; \(w\) weight per unit length of cable; \(\alpha\) is the angle of cable’s inclination; \(\eta(x,t)\) is cable’s displacements on the occasion of its transversal vibrations; \(T\) is the static force in cable before the beginning of its vibrations; \(\dot{y}\) is the displacement of one end of cable during vibrations of the structure as the whole; \(s\) is the length of cable’s chord; and \(g\) is the acceleration of gravity, Fig.1.

The cable displacement in the direction of its chord will be considered as negligible in comparison to the displacement perpendicular to the chord, i.e. cable’s vibrations in the longitudinal plane of the bridge will be considered only. Eigenvalues of cable have been solved by using Fourier series, Hartmann and Davenport (1966), Causevic (2009), among other methods for solving the equation which describes free wire vibrations.

\[
\bar{V}_n(\alpha) = C_n \sin \frac{n\pi \alpha}{s} ; \quad n = 1,2,3,... \tag{3}
\]

\[
\omega_n = \frac{n\pi}{s} \sqrt{\frac{Tg}{w}} ; \quad n = 1,2,3,... \tag{4}
\]
The solution for damped cable vibrations was obtained by representing the deflected mode shape via a Fourier series of sinusoidal components and to solve the amplitude of each component by substituting into the governing differential equation. Complex algebra has been used. In the analysis viscous damping was considered, i.e., equivalent viscous damping from an aerodynamic source. As the result of a strong wind the cable adopted a slightly different equilibrium position because of the effect of the following drag force per unit length of cable:

$$\frac{1}{2} \rho_w C_D D V_w^2 \sin \alpha$$

$\rho_w$ is the air density, $C_D$ drag coefficient, $D$ indicates the cable diameter and $V_w$ denotes the mean wind velocity.

Damping is inducing an imaginary component $dT_{\text{imag}}$ and reducing the peaks in the real component $dT_{\text{real}}$ of the massive guy cable force due to its damped vibrations.

$$dT(\omega) = \sqrt{[dT_{\text{real}}]^2 + [dT_{\text{imag}}]^2}$$

Values $dT(\omega)$ are schematically presented in Fig. 2 being function of viscous damping ratio $\xi$. Significantly large values of the cable's force are noticeable if its eigenfrequencies $\omega$ approach the vicinity of any cable-stayed bridge eigenfrequency $\omega_i$ resulting with: $\Omega = \frac{\omega}{\omega_i} \equiv 1$, Fig. 2.

In Fig. 2 only phenomenology is presented. This is why the values on the vertical coordinate axe on Fig. 2 are omitted.
2 REHABILITATION OF DUBROVNIK (CROATIA) CABLE-STAYED BRIDGE

Dubrovnik cable-stayed bridge has total length 490.2 m (Savor et al. 2006); the longest span 304 m and two approaching spans, Fig. 3. Two approaching spans (87 m in total) and part of longest span (60 m) are prestressed concrete box girder. The remaining part of superstructure of the longest span is steel/reinforced concrete composite, comprising the steel grillage of two main girders 2 m deep, cross steel girders, steel braces and concrete deck plate of 25 cm thickness, Fig. 4. A-type reinforced concrete pylon is 141.5 m height. Cable stays are in modified fan type position (Fig. 3) with length from 73 m to 222 m. Cable-stays in two inclined planes are spaced at 20 m.

The growth of cable-stayed bridge spans nowadays leads to an increasing length of stay cables. In case of Dubrovnik Bridge the longest cable has length 222 m. As longer the cables as more sensitive they are to dynamic excitation either from the supported structures (bridge deck, pylon) or wind. Small value of inherent damping capacity of cable is not sufficient to eliminate cable vibrations. Moreover, with increasing cable length the number of possibly excited eigenmodes is also growing, so the capacity of passive damping may not be sufficient to protect the cables over the whole range of occurring vibrational modes. Vibrations seemed mainly in the cables inclined plane, which can be partly explained by the amount of internal damping coefficient having only half the value of the value for transverse vibrations, Virilogueus (2005), Savor et al. (2006).

Additionally to conventional integrated elastomeric or friction damper Maurer Söhne and EMPA Zürich developed a semi-active damping system based on viscous damping device which allows an independent and real-time reaction of the damping device to the occurring vibrations.

Figure 2. Values of massive guy cable force $dT(\omega)$ for various damping ratio.
vibrations, Fig. 5. Terminological explanation for passive and semi-active devices is presented by Weber et. al. (2006), Magnuson (2011).

By using a magneto-rheological damping fluid in viscous damping device, the advantages of passive and active systems are combined. When cable vibrates in a gusty wind with violent vibrations and large amplitudes, inside the damper the magnetic field is created (Fig. 6) attracting metal particles in Magneto-Rheological (MR) fluid, i.e. enlarging viscosity of fluid inside damper and as the consequence increasing the damping force, Fig. 6. These devices have been used in rehabilitation of Dubrovnik Bridge.

Figure 3. Dubrovnik cable-stayed bridge and its longitudinal section.

Figure 4. Steel/reinforced concrete composite cross-section of Dubrovnik cable-stayed bridge.
Figure 5. Comparison of vibration decay in cable for: (a) no damping, (b) passive damper, (c) adaptive damper producing a faster decay of deformation $d$.

Figure 6. Magneto-Rheological (MR) fluid and magnetic valve creating the damping force.

Because of moderate to strong wind the cables of Dubrovnik cable-stayed bridge (Fig. 3) experienced large amplitudes. Originally designed and erected cables of Dubrovnik cable-stayed bridge contain inherent damping ratios less than 0.4%. For suppressing large amplitudes in moderate to strong wind the bridge rehabilitation was made by using the Maurer dampers (Fig. 6) creating up to 1.6% of critical damping in the semi-active behaviour of dampers, Fig. 7.

It is very important for proper cable behaviour in a strong wind that amplitudes of cable vibrations must be reduced as fast as possible, which can be achieved by inducing damping in cable by Adaptive Cable Dampers (ACD). Having in cable $\xi \geq 1.0$ is enough to fulfil the need for suppressing the problem of cables nonlinear behaviour (Fig. 6, Fig. 7, Fig. 8 and Fig. 9).
Figure 7. Increase of damping ratio by using Adaptive Cable Damper System.

Figure 8. Maurer Adaptive Cable Damper (ACD) installed during 2006 (18 ACD-65 for 18 single cables).

Figure 9. Maurer Adaptive Cable Damper (ACD) installed during 2006 (2 ACD-140 clusters each for 6 back-stays).
After installing Adaptive Cable Dampers (ACD) in rehabilitation for longest cables whose locations are presented in Fig. 10 and Fig. 11, regular long distance monitoring and remote control of Dubrovnik cable-stayed bridge behaviour was organized.

![Location of ACD dampers and Control system](image)

**Figure 10.** Locations of Adaptive Cable Dampers (ACD) in red: Total number of ACD is 18 for single cables (Fig. 8) and 2 for cluster of cables (Fig. 9).

![Cabling System in monitoring](image)

**Figure 11.** Cabling System in monitoring.

### 3 CONCLUSIONS

As the result of the analysis presented in this paper the following is concluded:

- When the cable-stayed bridge has eigenfrequency close to the eigenfrequency of any cable, this cable produces considerable kinetic energy and its behaviour starts with large amplitudes. This energy has to be considered when eigenvalues of the bridge are analysed.

This conclusion has been observed firstly during the laboratory dynamic testing of the Argentina cable-stayed bridge model made by Oberti (1972): “During the test the motion of the bridge was very much disturbed by the vibrations of the cables that happened to collide during...”
their motion. A test for the determination of the mode shapes has been accomplished using extremely low level of exciting forces so as to avoid this kind of problem”. Oberti (1972) was the first who pointed on existing of the problem and he avoided these phenomena by accomplishing testing by low level of exciting forces. In the first part of this paper the causes for violent cable-stayed bridge vibrations are explained analytically.

- The response of a cable-stayed bridge to a general type of external dynamic loading will be disturbed by the vibration of cables, especially when the eigenfrequencies of the bridge structure approach the vicinity of the eigenfrequency of any of the cables. In such a case the cable's stiffness will be suddenly reduced and cable will start vibrating with large amplitudes and nonlinear behaviour enabling undesired large vertical and torsional deformation of the bridge deck.

- The eigenfrequencies of the bridge itself should not approach the vicinity of natural frequencies of any of the cable-stayed bridge cable. To fulfil this demand is an important and difficult task, especially in the design of the fan-type cable-stayed bridge because of a great number of different cables on the bridge, bearing in mind that a cable's eigenfrequency depends not only on the cable’s length but also on the weight of the cable and its static force. This increases the probability of cable resonance with bridge.

- In the case of a cable resonance with a bridge the cable's behaviour starts with nonlinear vibrations and large amplitudes and the cable loses its stiffness and its function, i.e. degree of redundancy of the bridge will be reduced. In this situation instead of n cables there will be (n-1) cables on the bridge, the forces in other cables will suddenly change their values and the bridge deck overcomes larger vertical deformations, which is undesirable.

- To suppress the cable’s violent behaviour the use of the Maurer Adaptive Cable Dampers (ACD) when needed is recommended.

- Having in mind case study presented in this paper torsional more stiff cross-sections for bridge girder is recommended.

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