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Damage prevention and aerodynamics of cable-stayed bridges in heavy snowstorms: A case study

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Abstract. This paper begins by analyzing cable vibrations due to external excitations and their effects on the overall dynamic behavior of cable-stayed bridges. It is concluded that if the natural frequency of a cable approaches any natural frequency of the bridge, the cable loses its rigidity and functionality. The results of this analysis explain the phenomenon that occurred on the Dubrovnik Bridge in Croatia during a storm and measures for its retrofit. A field test was conducted before the bridge was opened to traffic. It was concluded: "The Bridge excited unpleasant transverse superstructure vibration with the frequency of approximately 0.470 Hz. Hence, it seems possible that a pair of stays vibrating in phase may excite deck vibrations". Soon after this Bridge opened, a storm dumped heavy damp snow in the area, causing the six longest cable stay pairs of the main span to undergo large-amplitude vibrations, and the superstructure underwent considerable displacements in combined torsion-sway and bending modes. This necessitated rehabilitation measures for the Bridge including devices to suppress the large-amplitude vibrations of cables. The rehabilitation and monitoring of the Bridge are also presented here.

Keywords: adaptive cable damper (ACD); aerodynamics; cable-stayed bridge; monitoring; rehabilitation; vibrations

1. Introduction

A new generation of large-span cable-stayed bridges around the world (Sutong, China; Russian Island, Russia) has been designed given sufficient knowledge of phenomena such as resistance of bridges to extreme loads from wind, earthquakes, etc. Many aspects of cable-stayed bridge behavior have been studied in the past with linear (Au *et al.* 2001, Gattulli and Lepidi 2007) and nonlinear interactions between cables and a bridge (Gattulli and Lepidi 2003, Tabejieu *et al.* 2016), and some studies were supported by experimental verification (Bayraktar *et al.* 2017).

Starossek (1994), Virlogeux (2007) reviewed the history of cable vibrations, starting from the 18th century and the d'Alembert and Lagrange solution for the linear vibrations of an inextensible massless string, to work on nonlinear cable dynamics, Tonis (1989), Davenport and Steels (1965).

The problem of dynamic interactions between cables and a bridge demands boundary research on induced cable vibrations. This dynamic interaction was initially the topic of research of several authors (Davenport and Steels 1965, Hartmann and Davenport 1966). A more refined linear theory of boundary-induced vibration of a damped cable was given by Veletsos and Darbre (1983) and Starossek (1991).

The retrofitting and monitoring of certain phenomena on a bridge can be established in two situations:

- When these phenomena prove to be a problem for the functioning of the structure, as in the case of the Dubrovnik Bridge, where some cable features were insufficient for the functioning of the bridge in cases of extreme unexpected events (Savor *et al.* 2006),
- When a cable exhibits certain long-term time-dependent changes, i.e., when time-dependent effects of construction materials on the dynamic properties of cable-stayed bridges are possible (Au and Si 2012).

There is a variety of aspects related to cable-stayed bridge behavior, such as the effect of soil properties on cable-stayed bidges and the characteristics of base-isolated cable-stayed bridges under either strong wind or ground motion (Javanmardi *et al.* 2017); aerodynamics and flutter phenomena (Sukamta *et al*. 2017, Sham and Wyatt 2016); dynamic response of cable-stayed bridges under blast loads (Hashemi *et al.* 2016); wind tunnel tests on cable-stayed bridge decks or towers (Sun *et al.* 2016, Rosa *et al*. 2014), etc.

This paper neither aims to elaborate on the developments in the dynamic analysis of a cable-stayed bridge nor to examine further developments of existing methods. Instead, a simple procedure on how to avoid resonance of any cable within a bridge structure is presented in this paper. It also describes the behavior of a cable-stayed bridge in a situation when this simple control has not been carried out, i.e., when it is necessary to rehabilitate the bridge after a snowstorm and to monitor the behavior of the bridge. This is the significance of the study presented in this paper.

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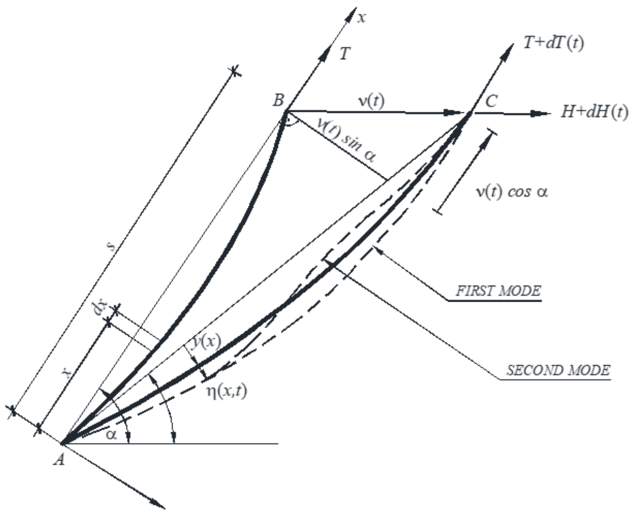

Fig. 1 Mode shapes of an inclined massive cable

2. Influence of cable vibrations on the dynamic behavior of a cable-stayed bridge

The differential equation of motion for a parabolic cable and the value of a massive guy cable force due to undamped cable vibrations dT are given in the following forms (see Appendix)

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

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

The symbols presented in Fig. 1 and Eqs. (1)-(2) are:
A is the cross-sectional area of the cable;
E is the modulus of elasticity of the cable;
w is the weight per unit length of the cable;
α is the angle of cable inclination;
$\eta(x,t)$ is the cable displacement on the occasion of its transversal vibrations;
T (Tension) is the static force in the cable before the beginning of vibrations;
dT is the value of a massive guy cable tension force due to cable undamped vibrations;
v is the displacement of one end of the cable during the cable-stayed bridge vibrations;
s is the length of the cable chord;
c is the viscous damping, i.e., the equivalent viscous damping from the aerodynamic source (a more refined theory on cable damping is presented in Weber and Distl (2015a), Weber and Distl (2015b);
g is the acceleration of gravity;
and $y''=\frac{w}{T}\cos\alpha$ is the basic differential equation of cable equilibrium.

The cable displacement in the direction of its chord is considered negligible in comparison to the displacement perpendicular to the chord. Cable vibrations only in the longitudinal plane of the bridge are considered.

The eigenvalues of the cable $(\bar{V}_n(x);\omega_n)$ are solved using Fourier series (Davenport and Steels 1965, Hartmann and Davenport 1966) and the usual methods for solving the equation that describes free-wire vibrations

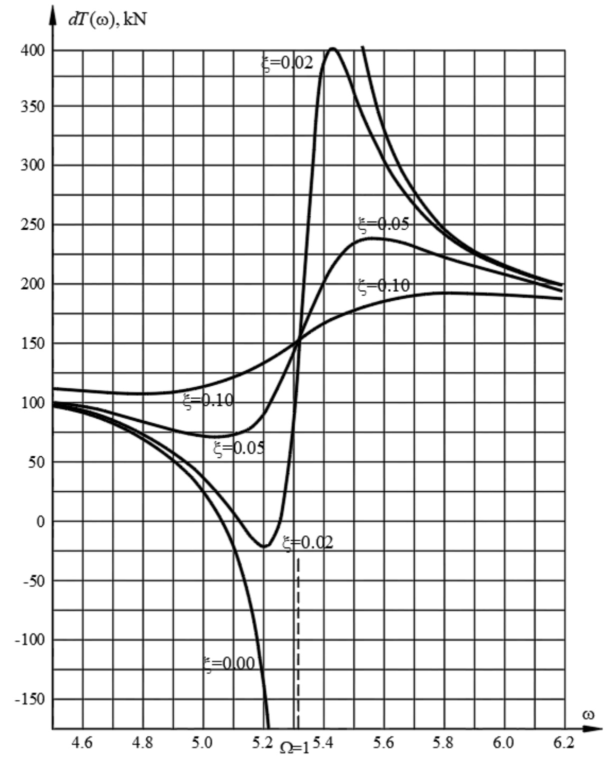

Fig. 2 Values of massive guy cable force $dT(\omega)$ due to cable vibrations for various damping ratios ξ for an arbitrary cable (ξ=0.02 obtained after rehabilitation of the Dubrovnik Bridge, Table 2)

$$\bar{V}_n(x)=C_n\sin\frac{n\pi x}{s}\quad ;\quad n=1,2,3,\ldots \quad (3)$$

$$\omega_n=\frac{n\pi}{s}\sqrt{\frac{Tg}{w}}\quad ;\quad n=1,2,3,\ldots \quad (4)$$

$$\Omega=\frac{\omega}{\omega_n}\quad ;\quad n=1,2,3,\ldots \quad (5)$$

The solution for the damped cable vibrations was obtained by representing the deflected mode shape via a Fourier series of sinusoidal components and solving for the amplitude of each component by substituting the modes into the governing differential equation (using complex algebra). In the analysis, viscous damping was considered, i.e., the equivalent viscous damping from an aerodynamic source.

Damping induces an imaginary component dT_{imag} and reduces the peaks in the real component dT_{real} of the massive guy cable force due to its damped vibrations.

$$T(\omega)=\sqrt{\left[dT_{real}\right]^2+\left[dT_{imag}\right]^2} \quad (6)$$

Values of $dT(\omega)$ are computed and schematically presented in Fig. 2 for an arbitrary cable as a function of the viscous damping ratio. Large changes in the cable force $dT(\omega)$ are observed when cable frequencies ω approach any natural frequency ω_n.

The focus here will be on analysing the cable deck and the cable pylon interactions. Even though the complex

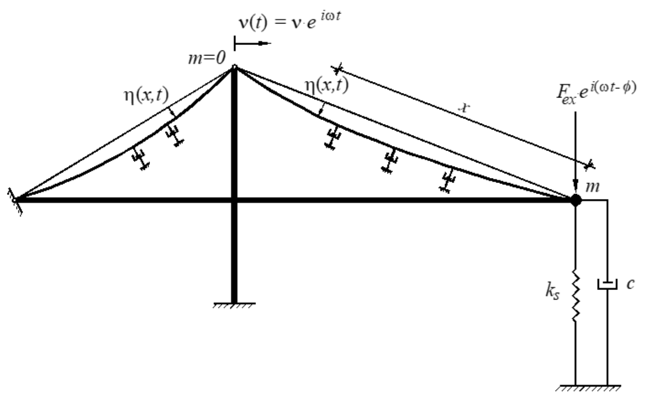

Fig. 3 Illustrative example

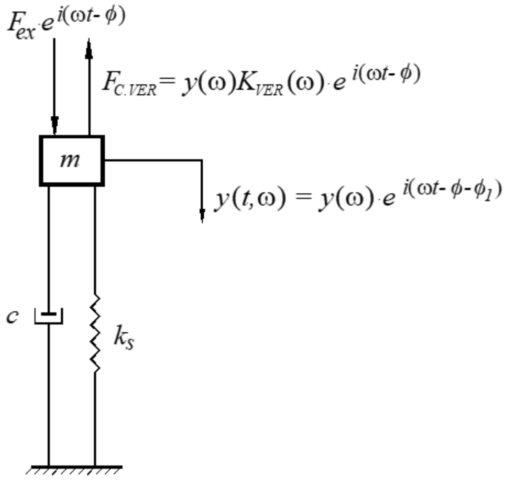

Fig. 4 Forced vibration of the discrete mass system presented in Fig. 3

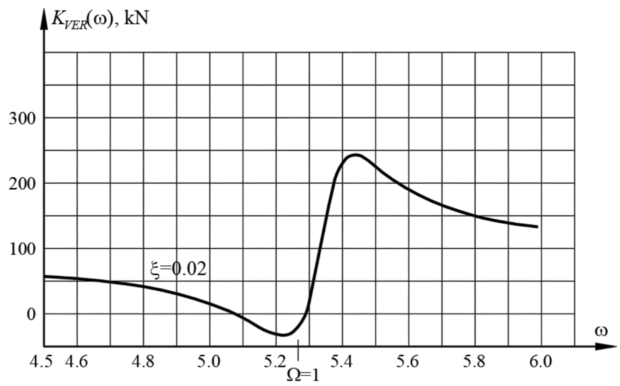

Fig. 5 Cable stiffness changes vs. bridge frequency

problem of modelling the interaction of a cable and a girder is already known, here it is presented in a simplified way to meet the needs of designers. This simplicity provides a basic understanding of the complex behaviour of structures with cables. As an illustration of the effect of cables on the dynamic behaviour of a structure with cables, a damped vibration of a simplified discrete system with one degree of freedom was analysed in Fig. 3. This simple discrete model is sufficient to describe phenomenology. The cable is connected to the discrete mass that is part of a bridge deck. The cable produces a force acting as an additional external force on a discrete model presented in Fig. 4

$$F_{C.VER} = y(\omega)K_{VER} \cdot e^{i(\omega t-\varphi)} \tag{7}$$

where

$y(\omega)$ presents the discrete mass deflection amplitude;

K_{VER} denotes the vertical component of the cable force for the unit displacement of the upper cable end, as shown in Figs. 1 and 3.

The results of the analysis of a simplified discrete system in Figs. 3-4 are presented in Fig. 5, where cable stiffness degradation is observed when its eigenfrequency approaches the eigenfrequency of the structure in Fig. 3, i.e., the bridge as a whole. The cable stiffness is suddenly reduced, and vibrations of a cable start with large amplitudes and nonlinear behaviour, enabling undesired large vertical deformation of the bridge deck, as shown in Fig. 5. This behaviour could also cause problems in cable-to-girder or cable-to-pylon connections.

Here, the presented phenomena were observed in the Dubrovnik Bridge, which will be explained later as the case study.

Cable vibrations with large amplitudes can also be caused by the so-called “wet cable phenomenon”. This phenomenon occurs when rain and low temperature (snow) cause a cable with smooth surfaces to lose its circular cross section. Indented cables were used to solve this problem in the Tatara cable-stayed bridge in Japan (Čaušević 2001).

The stiffness degradation K_{VER} presented in Fig. 5 is related to an arbitrary bridge.

Here, the presented analysis relates not only to cable-stayed bridges but also to other suspended structures with cables (antenna masts, towers, etc.). One example of a suspended structure, a double-column guyed tower, in strong wind is presented in Xiao *et al*. (2016).

3. Rehabilitation and monitoring of the Dubrovnik Bridge

In the previous part of this article, the consequences of cable resonance with the bridge were analytically described, while in this third part, it is shown that such resonance exists in the case of the Dubrovnik Bridge, and it was described how to eliminate this phenomenon of resonance using devices.

The Dubrovnik Bridge spans the Dubrovacka River Strait on the western entrance to Dubrovnik. The bridge has a total length of 490.2 m, consisting of a central span of 304 m and two approach spans, as illustrated in Fig. 6. Two approach spans (87 m in total) and part of the longest span (60 m in length) are made from prestressed concrete box girders. The remaining superstructure section of the longest span is a steel/reinforced concrete composite with an open cross section consisting of the steel grillage of the two main girders that are 2 m deep, steel cross girders, steel braces and a concrete deck plate that is 25 cm thick, as shown in Fig. 7. The cross section was defined after testing the bridge model in the wind tunnel (Sedlaček *et.al.* 1998). An “A”-type reinforced concrete pylon is 141.5 m high. Cable stays are in a modified fan-type position with lengths from 73 m

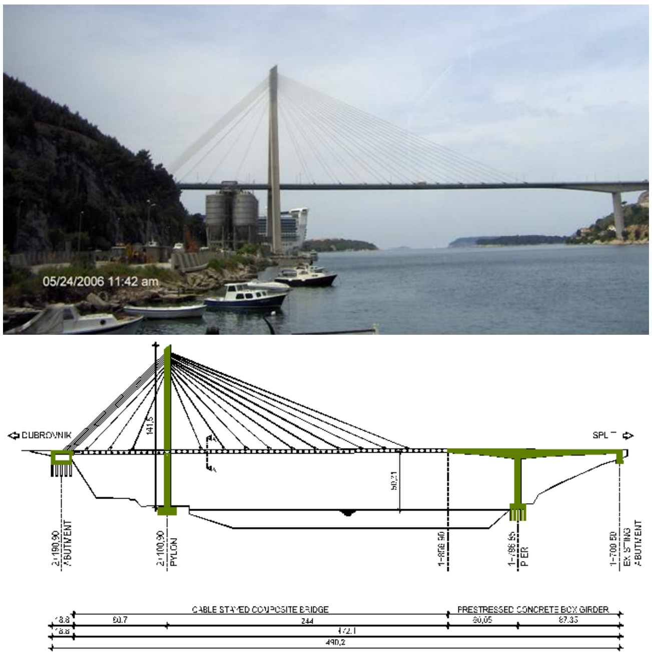

Fig. 6 Dubrovnik Bridge and its longitudinal section

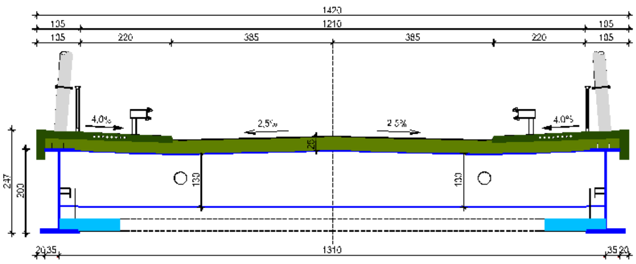
Fig. 7 Steel/reinforced concrete composite cross section of the bridge

to 222 m. Cable stays in two "in-plane" directions are spaced 20 m apart at the main girder level.

Shortly after the Bridge opened in 2002, violent vibrations of the bridge deck, pylon and cables in a Bora wind (specific for the Dalmatia region) with moderate wind speeds of 12.9-14.7 m/s combined with light rain were observed approximately twelve times a year. The most violent vibrations of the bridge cables, especially of the longest cable, were observed in March 2005 during a storm that dumped heavy damp snow when the wind speed was 22 - 24 m/s, causing the longest cable to vibrate with an amplitude of approximately 2.5 m (Savor *et.al.* 2006).

Before the bridge was opened to traffic, a field test was conducted as a 32-ton "zig-zag" truck crossed the bridge. It was noted that, "the Bridge excited unpleasant transverse superstructure vibrations with the frequency of approximately 0.470 Hz.

Hence, it seems possible that a pair of stays vibrating in phase may excite deck vibrations" (Savor *et.al.* 2006). The measured values of cable stay frequencies and cable damping for the two longest cables for the first two modes are presented in Table 1 (Savor *et.al.* 2006):

In accordance with values found in other bridges (Virlogeux 2007), the inherent structural cable damping is

Table 1 Frequencies and structural damping for the two longest cables before the bridge was opened to traffic

Stay No.	Mode 1		Mode 2	
	f (Hz)	ξ	f (Hz)	ξ
1	0.647	0.0017	1.271	0.001
2	0.626	0.0017	1.212	0.0009

low and must be augmented by using devices. Nevertheless, the stay frequency of 0.626 Hz is in the domain of resonance to the transverse superstructure vibration frequency of 0.470 Hz. The superstructure vibration frequency-to-stay frequency ratio triggered to the domain 1.250>r>0.750 should be avoided. For the Dubrovnik Bridge, the frequency ratio r has the following value

$$r = \frac{0.470}{0.626} = 0.751 \tag{8}$$

Before the bridge was rehabilitated, another strong, wet snowstorm occurred in the area in May 2006. Wet snow was deposited on the windward side of the cable, causing the six longest pairs of main span cables to undergo strong vibrations accompanied by significant superstructure movements in combined torsion and bending modes (Savor *et al.* 2006). This frightening event was recorded on Croatian national television in major daily news programs. The cable vibration amplitudes were even higher than the amplitudes that occurred in previous events.

The conclusions based on this event were as follows:

• The cables excited the vibration of the superstructure due to the proximity of the eigenfrequencies of the cable and the bridge;

• Inherent structural damping in the existing stay cables was insufficient.

The vibrations of the cables were mainly in-plane (Virlogeux 2007). Adjustable cable dampers based on the latest magnetorheological (MR) technology (Weber *et. al.* 2006, Magnuson *et. al.* 2011, Khan 2012, Weber and Maślanka 2014, Gkatzogias and Kappos 2016) were installed on the bridge to augment damping to up to 2.1% of the critical damping of most affected cable stays (Fig. 8).

Using a magnetorheological damping fluid in a viscous damping device combines the benefits of passive and active systems. When the cable undergoes large-amplitude vibrations in a gust of wind, it creates a magnetic field inside the damper that attracts metal particles, increasing the viscosity of the fluid inside the damper and increasing the damping force (Fig. 8). These devices were used in the rehabilitation of the Dubrovnik Bridge.

In addition, an adaptive cable damper (ACD) based on viscous damping is designed and implemented in the Dubrovnik Bridge (Weber *et. al.* 2007). This device is a magnetorheological (MR) damper, which allows an independent and real-time reaction of the damping device to the vibrations.

The new generation of ACD-MR dampers was installed (Figs. 9-10). These dampers can provide continuously adjustable damping and are frequency/mode insensitive. The MR damper provides considerable damping even in its passive mode. The reduction effect of MR dampers increases with increasing current up to a certain

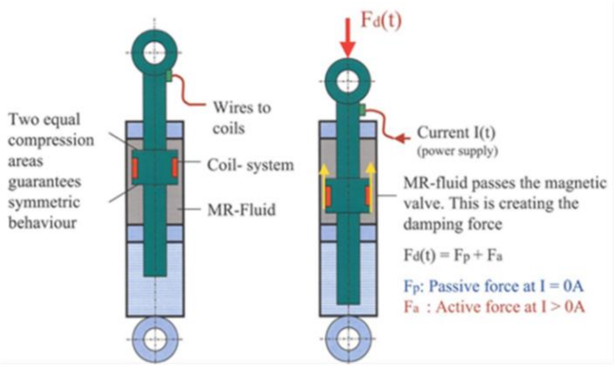

Fig. 8 Magnetorheological (MR) fluid and a magnetic valve create the damping forces

Fig. 9 Adaptive cable damper (ACD) installed after the May 2006 snowstorm (18 ACD-65 for 18 single cables)

level, but it is most effective for resonant vibration cases (Savor *et al.* 2006).

Computations for the dampers were performed by the damper supplier, and the efficiency of the proposed dampers was verified. The results clearly exemplify the beneficial effect of dampers on vertical displacement reduction. The structural damping was calculated according to the FHWA (2005) sponsored study for damped stay cables 1 and 2 (Table 2), with the damping coefficient c taken from the producer-provided specifications (Table 2).

In accordance with the values of structural damping in cables of other cable-stayed bridges (Virlogeux 2007), the augmentation of damping after installing MR dampers is a proper measure of Dubrovnik Bridge rehabilitation.

Moderate or strong winds caused the cables of the Dubrovnik Bridge to experience large amplitude vibrations. Originally designed and installed bridge cables have an inherent damping ratio of less than 0.2% (Table 1). To suppress large-amplitude vibrations under moderate to high wind, bridge rehabilitation was performed using MR dampers, generating up to 2.1% of critical damping in semi active dampers, as presented in Table 2 and Figs. 9-10.

Following the installation of adaptive cable dampers (ACDs) (Figs. 9-10), regular long-distance monitoring and remote control of the Dubrovnik Bridge were established (Fig. 11).

The Dubrovnik Bridge was among the first cable-stayed bridges to be designed with magnetorheological (MR) dampers, so it served as a prototype (Weber *et. al* 2007).

Fig. 10 Adaptive cable damper (ACD) installed after the May 2006 snowstorm (2 ACD-140 clusters each for 6 back stays)

Table 2 Frequencies and structural damping for the two longest cables obtained after installing MR dampers

Stay No.	Mode 1		Mode 2	
	f (Hz)	ξ	f (Hz)	ξ
1	0.647	0.020	1.271	0.018
2	0.626	0.021	1.212	0.018

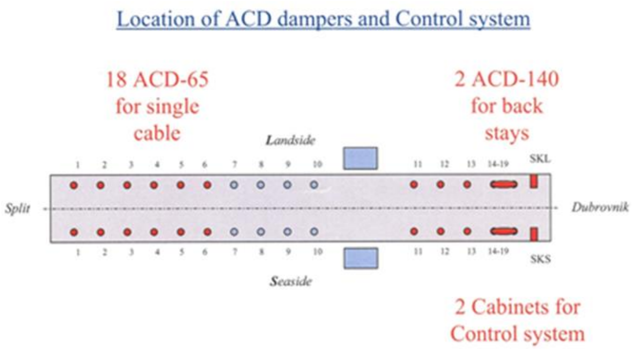

Fig. 11 Locations (in red) of adaptive cable dampers (ACDs): The total number of ACDs is 18 for single cables (Fig. 9) and 2 for clusters of cables (Fig. 10)

The same types of dampers were later installed in some of the world's longest cable-stayed bridges, including the Sutong Bridge in China (1088 m span) and the Russky Bridge in Russia (1104 m span).

4. Conclusions

The response of a cable-stayed bridge to general types of external dynamic excitations is affected by the cable vibrations when the natural frequency of the bridge approaches the natural frequency of any cable. In such a case, the stiffness of the cable decreases sharply (Fig. 5), i.e., the cable initiates large-amplitude vibrations and nonlinear behavior, allowing unwanted large vertical and torsional deformation of the bridge deck.

The natural frequencies of the bridge should not approach the natural frequency of any bridge cable. Meeting this requirement is an important task, especially in the design of a fan-type cable bridge due to the large number of different cables on the bridge, bearing in mind that the natural frequency of a cable depends on its length, weight and static force, Eq. (4). This increases the probability of resonance of the cable and the bridge.

When the natural frequency of a cable approaches any natural frequency of the bridge, the cable loses its rigidity and functionality, i.e., the degree of redundancy of the bridge decreases. In this situation, instead of n cables in full carrying capacity, there will be (n-1) cables on the bridge; the values of forces in other cables change, and the deck of the bridge is exposed to a possible increase in vertical deformations. A simple procedure to avoid resonance of any cable with a bridge structure is presented in this paper.

Adjustable cable dampers based on MR technology were installed on the Bridge to raise the low structural damping of the most affected stay cables and thus mitigate the cable stay vibrations. MR dampers were installed on the Bridge to augment the damping ratio by up to 2.1% of the critical damping of the most affected cables. Originally designed and installed bridge cables have an inherent damping ratio of less than 0.2%. This augmentation in cable damping was sufficient to retrofit the Dubrovnik Bridge.

Preliminary results of this research were presented at a conference in South Korea (Čaušević and Bulić 2015), and very useful feedback from the participants of that conference was considered when writing this paper.

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Appendix

Considerations for weightless wire

When the top of a taut, weightless (therefore without sag) inclined wire is deflected horizontally by a small amount v, and the cable is stretched by an amount $vcos\alpha$, where α is the angle of inclination. This causes an increase in cable tension, dT (over and above some initial prestress value T), of an amount:

$$dT = \frac{AEv\cos\alpha}{s}$$

where s is the length of the wire and A is its cross-sectional area.

The resulting guy modulus (the so-called "taut wire" modulus) is

$$\kappa = \frac{dT\cos\alpha}{v} = \frac{AEv\cos^2\alpha}{s} \tag{a}$$

Considerations for heavy cables

For heavy cables, Eq. (a) is inexact because of the effects of sag, as displayed in Fig. 1. Strictly speaking, the heavy cable will hang in an arc as a catenary. However, as a first approximation, a shallow parabolic arc can represent the cable with its axes perpendicular to the chord joining the cable ends, as shown in Fig. 1. The weight of the large cable-stayed bridge cable is a considerable percentage of a safe operating tension in the cable, so the cable is considered "heavy." The cable is no longer taut and sags appreciably. The elastic stretches of the cable accounts for only a part of the overall extension of the cable under a load. The remainder is the result of changes in the cable-sag geometry.

To derive the motion equation of the heavy cable referred to in Fig. 1, we define the coordinates in a way that, during vibrations, the upper end moves horizontally for an amount, which initially is assumed in the plane of the set of bridge cables. In the equilibrium position AB, the deflection of an element of the guy at distance x is y (see Fig. 1). During pylon deflection, chord AB rotates to the new position AC, and the instantaneous deflection of the same element measured from the chord in this new position is now defined as $y + \eta(t)$. The total normal acceleration of this element is

$$\frac{x}{s}\sin\alpha\frac{\partial^2 v(t)}{\partial t^2} + \frac{\partial^2\eta(t)}{\partial t^2} \tag{b}$$

The first part of the above expression is the result of the chord. The velocity of the element has a similar expression, with $\frac{\partial}{\partial t}$ replacing $\frac{\partial^2}{\partial t^2}$, and the equation of motion for the cable element, taking damping into account, can be written as

$$\frac{w}{g}\left(\frac{x}{s}\sin\alpha\frac{\partial^2 v(t)}{\partial t^2} + \frac{\partial^2\eta(t)}{\partial t^2}\right) + c\left(\frac{x}{s}\sin\alpha\frac{\partial v(t)}{\partial t} + \frac{\partial\eta(t)}{\partial t}\right) = T\cdot\eta''(t) + dT\cdot y'' \tag{c}$$

In Eq. (c), a second-order term $dT(t)\cdot\eta''(t)$ has been neglected.

The deflection of the guy top causes an increase in the overall chord length. This must be accounted for by the elastic stretch under the additional tension dT and the change in arch length of the guy. If the deflection $\eta(t)$ is small compared to cable sag y, the change in arc length is given by

$$\int_0^s y'' \cdot \eta(t) dx \tag{d}$$

The longitudinal displacement of the upper end of the guy can then be accounted for by

$$v(t) \cos\alpha = \frac{dT \cdot s}{AE} + \int_0^s y'' \cdot \eta(t) dx \tag{e}$$

If the guy is assumed to be parabolic, then

$$y'' = -\frac{w}{T} \cos\alpha \tag{f}$$

Therefore, the additional tension dT due to undamped cable vibrations is given in the following form

$$dT = \frac{AE}{s} \cos\alpha \left[v(t) + \frac{w}{T} \int_0^s \eta(t) dx \right] \tag{g}$$