WIND INDUCED CABLE VIBRATION
OF RION ANTIRION BRIDGE "CHARILAOS TRIKOUPIS"

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ABSTRACT: The wind induced cable vibration was thoroughly investigated during design in order to provide suitable alleviation measures. The structural response analysis, as recorded by the monitoring system, concluded to the most appropriate technical improvement. Dampers installation improved the response of the stays, as indicated by commissioning tests and actual recordings.

KEY WORDS: External dampers; Intrinsic Structural Damping; Rion Antirion Bridge; Stay Cable Vibrations; Structural Health Monitoring.

1 INTRODUCTION

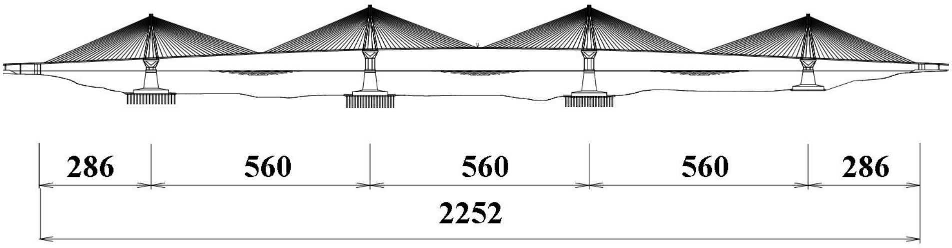

Figure 1. Rion Antirion Bridge elevation

Rion Antirion "Charilaos Trikoupis" bridge is a 5 span cable-stayed bridge joining Continental Greece with Peloponnese. The continuous composite deck has total length of 2252 m with three main spans of 560 m and side spans of 286 m. It is suspended by 4 concrete pylons with total height of 189 up to 227 m through 368 cables with total length from 79 up to 295 m. At each far end of the deck, a steel rotating frame (RF) supports the structure allowing longitudinal movement that is accommodated by special designed expansion joint. Furthermore, at pylon and RF location, the deck is transversally restrained through a fusing steel element that releases the deck when the transverse load,

on each element, exceeds ±10.500/±3.400 kN (pylon/abutment). Their capacity is based on wind ultimate design loads. In case of moderate/strong earthquakes, the deck is released and the induced energy is dissipated through viscous dampers located close to fuse elements.
The detailed design of the superstructure against wind induced vibration was very important in order to anticipate possible aerodynamic phenomena that could lead to instability of both deck and cables. Especially, the design of cables where aerodynamic problems can occur due to the very low intrinsic structural damping resulting from high tension, which is a common feature of cable stayed bridges. The theoretical studies needed to be complemented by actual measurements and observations (especially the first years of operation) that were provided by the Structural Health Monitoring system and visual inspections respectively. The analysis of actual strong wind events gave significant insight of structural behaviour allowing a better assessment of the possible risk that had to be compensated.

2 CABLE DESIGN AGAINST WIND INDUCED VIBRATION

Rion Antirion Bridge includes a large number of different cable stays. Basic dynamic properties are presented for 4 characteristic cables. The effects of various aerodynamic phenomena on the cable response as well as the necessary actions that need to be taken for mitigation of expected vibration are reviewed.

2.1 Cable dynamics

The modes in the horizontal (transverse) plane are sinusoidal with frequency related to the tension load T, the linear mass m and length L according to the approximate classical Eq.(1), [1]:

$$n_k = \frac{k}{2L}\sqrt{\frac{T}{m}} \quad (1)$$

Due to sag effect, the first mode in vertical plane is close to sinusoidal around the equilibrium shape with frequency equal to Eq.(2):

$$n_1 = \frac{1}{2L}\sqrt{\frac{T\left(1+\frac{EA}{T}\frac{\pi^2 s^2}{2L^2}\right)}{m}} \quad (2)$$

where: E is the Young modulus
A is the cross section area
s is the vertical sag of the cable.

The sag close to mid span is given by eq. (3):

$$s = \frac{4mgL^2 \cos a}{\pi^2 T} \tag{3}$$

where: g is the gravity acceleration
α is the mean cable inclination

The characteristics of the 4 representative cables are presented to the next table.

Table 1. Main characteristics of representative cables

Cable	No of Strands	Length (m)	Area (cm^2)	Mass (kg/m)	Tension (kN)	Inclination (deg)	1^{st} Vertical Frequency (Hz)	1^{st} Horizontal Frequency (Hz)
C3S23	70	286.2	105.0	97.1	6023	20.5	0.532	0.435
C3S19	59	239.4	88.5	81.4	5316	23.0	0.605	0.533
C3S14	47	182.5	70.5	65.2	4063	28.5	0.741	0.684
C3S04	43	87.0	64.5	59.8	1980	70.0	1.066	1.046

A comparison between the expected frequencies of the deck (for various modes) and the expected 1^{st} natural frequency of the cables are illustrated in *Fig.2*, where it is clear that longer cables have common frequency range with higher deck modes.

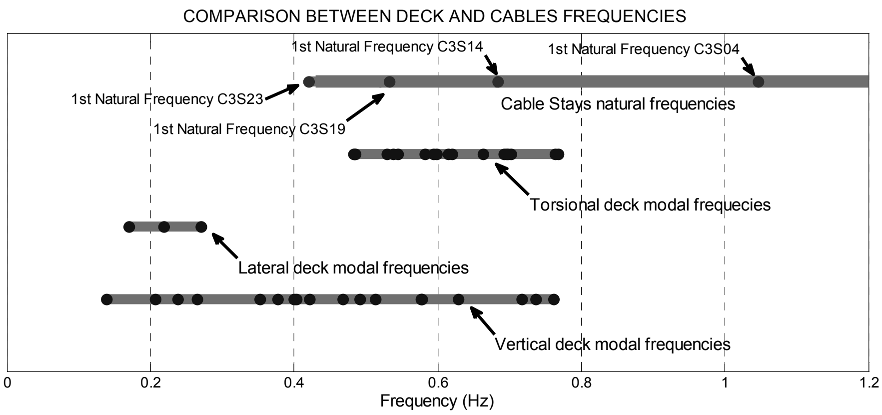

Figure 2. Comparison of deck with 1^{st} cable stay frequencies

An important fact regarding cable stays is the very low structural damping ξ_s. For the Rion Antirion Bridge it was estimated that ξ_s varies for long to short cables according the Eq.(4):

$$\xi_s = -6 \cdot 10^{-4} \cdot L + 0.24 \tag{4}$$

leading to ξ_s=0.068% (longer) up to ξ_s=0.193% (shorter).
However, along with the structural damping it should be introduced the high wind speed aerodynamic damping ξ_α that is proportional to the wind velocity U (when specific aerodynamic phenomena are absent) and is calculated through Eq.(5.1) for modes parallel and Eq.(5.2) for modes perpendicular to wind direction.

$$\xi_a = \frac{\rho U D C_d}{4\pi m n_k} \tag{5.1}$$

$$\xi_a = \frac{\rho U D C_d}{8\pi m n_k} \tag{5.2}$$

where: ρ is the air density
D is the cable diameter
C_d is the drag coefficient

For high wind speed (above 15 m/sec) it's worth mentioning that the aerodynamic damping is prevailing and in particular for 30 m/sec the ξ_α is 5 to 12 times the ξ_s.

2.2 Parametric excitation and buffeting

This is one of the most important phenomena for stay cable vibrations in Rion Antirion Bridge since for a wide number of cases the 1st natural frequency of the cables is in the same range with higher deck mode frequencies, as already illustrated in *Fig.2*.
For the estimation of the vibration amplitude of the cables, it is important to calculate the response of the deck and the pylons (where the cables are anchored), for different cases of wind speed. This was performed after deck buffeting analysis that included 15 wind cases. From these, only five cases were studied, regarding the excitation of the cables, plus the extreme wind speed case:

- No1 U(m/s)=5.90 (corresponding to the lowest damping of mode 1)
- No3 U(m/s)=10.0 (corresponding to the lowest damping of mode 5)
- No6 U(m/s)=15.9 (corresponding to the lowest damping of mode 9)
- No10 U(m/s)=19.7 (corresponding to the lowest damp. of mode 13)
- No15 U(m/s)=21.7 (corresponding to the lowest damp. of mode 18)
- Max U(m/s)=50.0 (corresponding to extreme wind speed)

Furthermore, the direct impact of wind buffeting to the cables was calculated in order to estimate the overall amplitude. The configuration is illustrated in *Fig.3*.

The analysis of the response subjected to deck excitation and buffeting indicated that the vibration amplitude, even for moderate winds (15.9 m/s) was quite high (more than 800 mm) especially for long cables (#16 and above). This is mainly due to parametric excitation, while buffeting influence can be neglected. The addition of structural damping (reaching δ=3%) through dampers significantly limits the vibration amplitude in about 300 mm. Even if dampers significantly improve the cable performance, it should be mentioned that only the shifting of cables modal frequency using cross-ties would be fully efficient.
The above results were incorporated to the cable configuration with the necessary adaptations in order to make feasible the installation of External Hydraulic Dampers (EHD) for cables #11 to #23 and/or Internal Hydraulic Dampers (IHD) for cables #1 to#10 and cross ties, if necessary. The installation of aforementioned dampers would be implemented if the actual behavior of the cables to strong winds would not be satisfactory. However, the suitable provisions such as anchors points on deck and cables had been taken into consideration during design/construction. The detailed design of the improvements should be reviewed and finalized by incorporating the data recorded from the Structural Health Monitoring system.

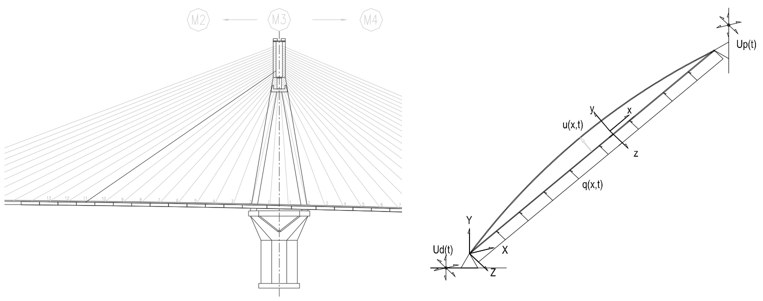

Figure 3. Configuration model for cable response calculation

2.3 Galloping

This kind of instability, usually perpendicular to the wind, is well known for slender structure whose cross section presents a strong negative slope for the lift coefficient C_L for some wind directions α. However, for the circular sections, selected for Rion Antirion Bridge stay cables, the Den Hartog criterion Eq.(6)

$$\frac{dC_L}{da} + C_D < 0 \tag{6}$$

is not fulfilled and thus galloping cannot occur.
However, this might not be the case for specific conditions, for instance ice

accretion which modify the symmetric cross section of the cables.

2.4 Rain Wind induced vibrations

Rain-wind induced vibration is one of the most usual stability problems of inclined cables [2]. For moderate rain conditions and wind velocities (8 up to 15 m/sec), large amplitude vibrations can occur for different combinations of cable inclination and wind directions. The presence of two water rivulets with the upper one oscillating circumferentially, synchronously with the cable's motion, is one of the key points of this instability [3]. The water acts as a trigger for an instability called "dry cable vibration" making it stronger and more stable.

The most efficient and common alleviation method is the disorganization of the transition through critical Reynolds number with helical thread on the protective ducts. However, since the diameter of the duct used in Rion Antirion Bridge is larger than the previously experimentally studied ones, it was proposed a series of tests in order to verify the effectiveness on current situation in the Jules Verne climatic wind tunnel in Nantes.

The test was consisting in reproduction of instability for smooth High Density Polyethylene (HDPE) ducts, on a sectional model of Rion Antirion Bridge by examining various combinations of wind speed and cable inclination, and then evaluation of the cable response covered with helical threaded HDPE duct for the same parameter combination.

The good performance of the helical threaded HDPE was verified for all the cases examined and thus the risk of rain-wind induced vibrations for Rion Antirion Bridge cable stays was eliminated.

3 MONITORING SYSTEM

The Rion Antirion Bridge is equipped with a Structural Health Monitoring system that is oriented to provide useful information regarding the response of the structure to various environmental loads such as earthquakes and strong winds.

In particular for the evaluation of the cable stays response four different types of sensors are used:

- 13 3D Accelerometers on cables (at 10 m height from deck)
- 12 3D and 3 1D Accelerometers on deck (located close to mid spans)
- 16 Load cells on cable strands (at top anchorage)
- 2 Anemometers (M1-M2 and M3-M4, 6 m above deck)

The location of each instrument is presented in *Fig.4.*

Two main categories of data files are created:

- History files (0.5 sec averaged values recorded every 30sec, except wind speed and direction that are 2' & 10' average after February 2008)
- Dynamic files (High sampling frequency at 100 Hz with 60 sec duration)

The History files are created continuously, while the Dynamic files are recorded every 2 hours (Automatic) or when particular threshold is over passed (Alert).
All types of files (History/Automatic/Alert) are very useful in order to understand the actual bridge response and evaluate the effect of potential improvements.

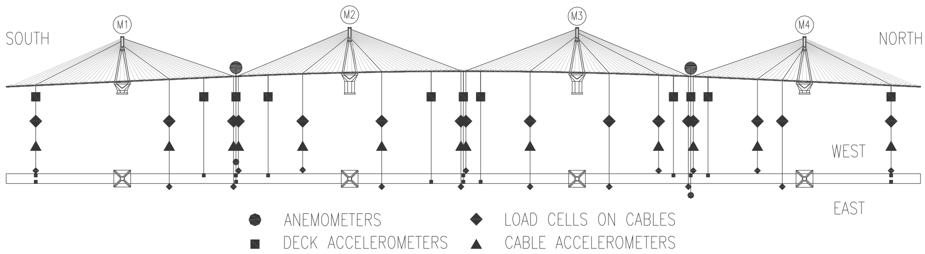

Figure 4. Location of structural monitoring sensors (cable relevant)

4 23RD OF JANUARY 2006 STRONG WIND EVENT

On 23rd of January 2006 a strong storm occurred in the vicinity of Rion Antirion Bridge. The main characteristics of the storm were the particularly eastern (120° clockwise from Bridge axis) strong winds (31.2 and 28.3 m/sec 2' average on M1M2 and M3M4 meteo stations) and the particularly low temperature (1.2°C). In *Fig.5* the 2' average wind speed and direction are provided.

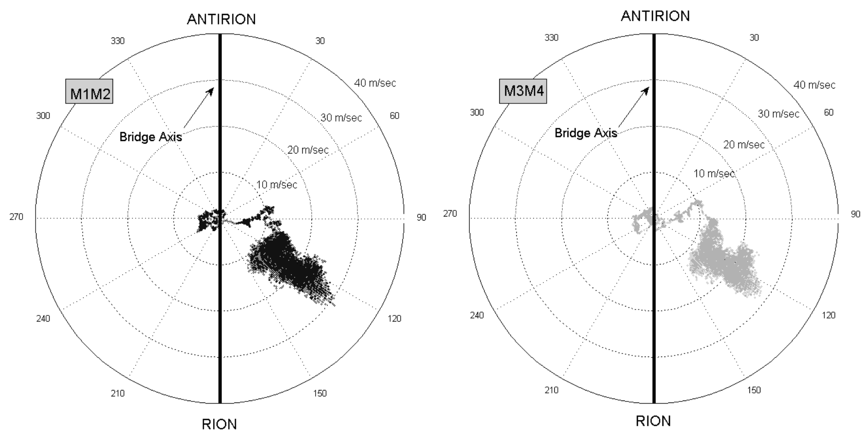

Figure 5. 2' average wind speed and direction graphs

During this event significant vibrations of the cables were observed, especially for intermediate and long cable stays (#16 and upper) the amplitude of which was exceeding ±2.0 m. Also due to low temperatures, ice formation was observed on several cables. The recorded response was calculated thanks to Alert files and is presented in the next paragraphs.
The large cable vibration enabled further analyses of the recorded data in order to optimize the design of the dampers for preventing similar vibration incidents in the future.

4.1 Deck vibration

In order to calculate the maximum displacement amplitude from acceleration time histories contained in Alert files, the following processing was applied:

- Mean removal
- band pass filtering “8th order Butterworth with corner frequencies 0.1 and 5 Hz”

The maximum vertical displacement amplitude at each sensor’s location is presented in the next table, and in *Fig.6*.

Table 2. Maximum vertical displacement amplitude at sensor location

Position	Accelerometer channel	Amplitude (cm)	Position	Accelerometer channel	Amplitude (cm)
M1S18E	E3 Z axis	10.23	M2M3W	D17 Z axis	15.62
M1S18W	E4 Z axis	9.26	M3S20W	E19 Z axis	14.01
M1N17E	E7 Z axis	10.21	M3N20E	E24 Z axis	14.32
M1M2W	E9 Z axis	11.62	M3M4E	D26 Z axis	15.51
M1M2E	D9 Z axis	13.75	M4S20E	E28 Z axis	11.94
M2S17E	E11 Z axis	16.03	M4N18W	E32 Z axis	10.65
M2N14W	E15 Z axis	13.44	M4N18E	E33 Z axis	10.66
M2M3E	E17 Z axis	15.21	-	-	-

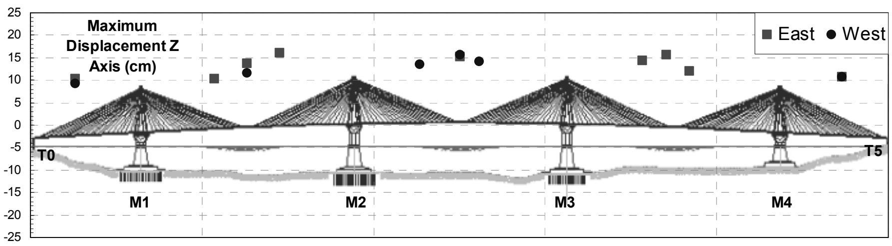

Figure 6. Maximum vertical displacement amplitude at sensors location

The frequency analysis of the acceleration time histories indicate that a large number of deck modes were participating to vibration but only few of them had significant displacement amplitude. In *Fig.7* the average normalized power spectral density [4] for acceleration and displacement time histories of all the deck sensors are presented and compared with the frequency band on the 1st natural mode of the cables.

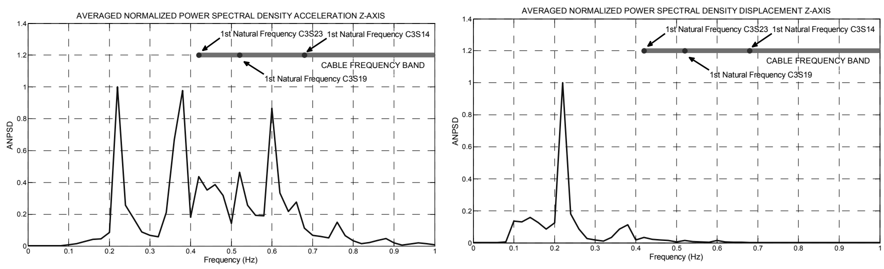

Figure 7. ANPSD for acceleration and displacement based on 02:00 24/01/2006 dynamic file

4.2 Cable vibration

In order to calculate the maximum displacement amplitude from acceleration time histories contained in the Alert files, the following processing was applied:

- Mean removal of Y and Z axis
- Frequency analysis of both Y and Z axis for identification of participating modes.
- Calculation of displacement time history (for both Y and Z axis) and decomposition into participating mode time histories $u_i(t)$ at accelerometer location L_a, $u_i(L_a,t)$, based on frequency content.
- Calculation of each modal coordinate response $q_i(t)$ based on Eq.(7), since $u_i(L_a,t)$ is known and $\Phi_i(L_a)$ is also known for cables and is described in Eq.(8).
- Calculation of maximum displacement for each location as an orthogonal composition of Y and Z displacement time histories for all participating modes according Eq.(9).

$$u_i(x,t) = \Phi_i(x) \cdot q_i(t) \tag{7}$$

$$\Phi_i(x) = Sin\left(i\pi \frac{x}{L}\right) \tag{8}$$

$$u_m(x) = \max\left(\sqrt{\sum_{i=k}^{n} u^2{}_{y,i}(x,t) + u^2{}_{z,i}(x,t)}\right) \tag{9}$$

The maximum amplitude of vibration U_m=max(u_m(x)) for all cables calculated for all Alert files are presented to Table 3 and *Fig.8*.

Table 3. Maximum vertical displacement amplitude

Position	Accelerometer channel	Amplitude (cm)	Position	Accelerometer channel	Amplitude (cm)
C1S18W	J4 Y and Z	227.4	C3S23W	J18 Y and Z	263.6
C1N10E	J6 Y and Z	168.4	C3S10E	J20 Y and Z	36.9
C1N23E	J8 Y and Z	241.9	C3N17W	J23 Y and Z	257.1
C2S23W	J10 Y and Z	207.1	C4S23W	J27 Y and Z	317.1
C2S10W	J12 Y and Z	91.3	C4S10W	J29 Y and Z	19.6
C2N07E	J14 Y and Z	57.3	C4N18W	J32 Y and Z	249.96
C2N23E	J16 Y and Z	255.5	-	-	-

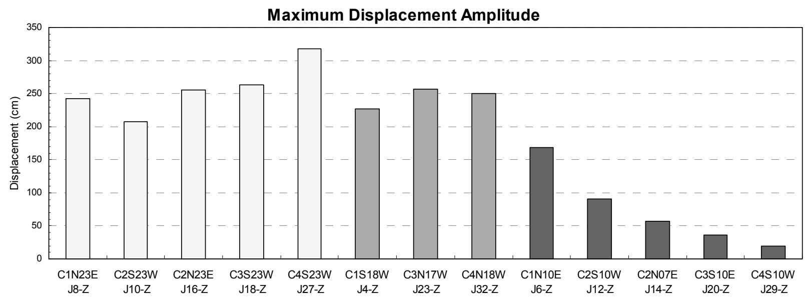

Figure 8. Maximum displacement amplitude of cable stays sorted per length

Despite the large amplitude of cable vibration, the respective loads were within SLS (50% of F_{GUTS}=265.5 kN) as presented hereunder.

Table 4. Maximum cable load

Position	Sensor	Maximum load (kN)	Percentage of F_{GUTS} (%)	Position	Sensor	Maximum load (kN)	Percentage of F_{GUTS} (%)
C1S18W	K4	103.9	39.1	C3S10E	K20	83.6	31.5
C1N10E	K6	94.7	35.7	C2N07E	K22	69.5	26.2
C1N23E	K8	104.8	39.5	C3N17W	K23	100.6	37.9
C2S23W	K10	95.8	36.1	C3N23E	K25	90.9	34.2
C2S10W	K12	91.3	34.4	C4S23W	K27	104.0	39.2
C2N07E	K14	71.1	26.8	C4S10W	K29	91.7	34.5
C2N23E	K16	103.5	39.0	C4N05E	K30	66.7	25.1
C3S23W	K18	102.6	38.6	C4N18W	K32	112.4	42.3

5 DESIGN AND IMPLEMENTATION OF TECHNICAL IMPROVEMENT

The data recorded from the monitoring system, and particularly the dynamic records were further processed in order to optimize the design of the required technical improvement for the mitigation of cable vibrations. Two possible solutions were available, dampers EHD and/or IHD and cross-ties, with the first being the most favorable one, since the implementation of cross-ties in RA Bridge do not shift all cable frequencies beyond higher deck modes frequencies.

5.1 Design parameters

During design phase, it was investigated if the EHD system is efficient enough to mitigate the cable vibrations as they were observed and recorded by the monitoring system of Rion Antirion Bridge. Thus the main question was how much damping is required to be added for minimizing the cable vibration.
Three different excitation scenarios were investigated:

- Resonance and parametric excitation
- Iced cable galloping
- Dry inclined cable galloping

Initially it was investigated the damping ratio that is required in order to avoid resonance and parametric excitation. The damping was calculated for different length of cables with basic criterion the limitation of the vibration amplitude to one diameter, when input excitation was described by an envelope frequency function Eq.(10) that was calculated from 23rd of January 2006 wind event.
The results are presented to *Fig.9* and indicate that ξ=1% of total structural damping is required from cables with length between 100 and 250 m and ξ=1.5 % for longer cables. No additional damping was required for short cables with length less than 100m.

$$V(f) = 1 + \frac{3}{f^2} \text{ (mm)} \tag{10}$$

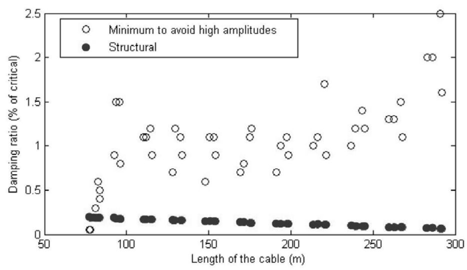

Figure 9. Required damping ratio for vibration mitigation

For the selected damping ratio (1.0%/1.5% for intermediate/long, and no additional damping for short cables L<100m) the critical wind velocity was calculated for both ice [5] and dry inclined galloping. *Fig.10* summarizes these results.

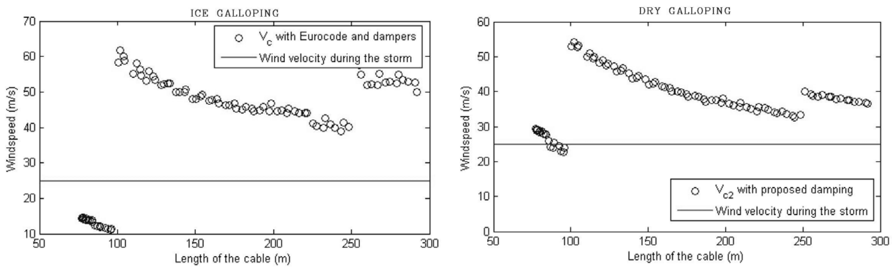

Figure 10. Critical wind speed for ice and dry galloping

The required damping ratio includes the aerodynamic damping and thus the required structural damping ratio is significantly lower. The selected damping system was designed in order to guarantee 4% logarithmic decrement for all cables above #11. The general arrangement is illustrated to *Fig.11*.

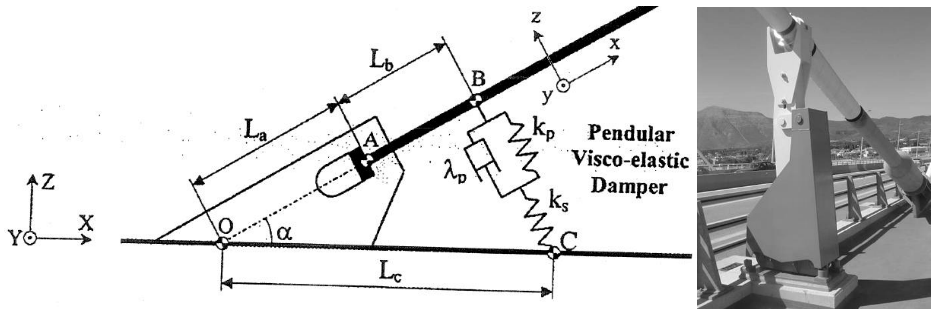

Figure 11. General arrangement of dampers for cables #11 and above and actual implementation (C4N23E)

5.2 Installation and commissioning test

The installation of 208 dampers was performed in the first semester of 2007. In order to verify the proper functioning of damper commissioning tests were performed on 6 different cables. The aim was to excite the 1st natural mode of each cable and calculate the logarithmic decrement from measured acceleration time histories, before and after damper installation. For the tests a mobile temporary acquisition system was used. The results of the commissioning tests are summarized in Table 5.

Table 5. Commissioning test results

Position	Logarithmic decrement w/o damper	Logarithmic decrement with damper	Position	Logarithmic decrement w/o damper	Logarithmic decrement with damper
C2S11W	1.79 %	6.42 %	C2S16W	1.58 %	4.99 %
C2S12W	2.06 %	6.15 %	C2S19W	1.08 %	6.01 %
C2S14W	1.53 %	5.31 %	C2S22W	1.82 %	5.35 %

The commissioning tests led to the acceptance of the installed EHD dampers as an efficient tool for cable stay vibration mitigation.

6 8TH OF MARCH 2010 STRONG WIND EVENT

Three years after the installation of external dampers, the most severe wind storm during Bridge operation period occurred. This event provided an excellent opportunity to verify the overall behaviour of the cable stays equipped with dampers.

The main characteristics of the storm were eastern (100° clockwise from Bridge axis) strong winds (35.4 and 30.7 m/sec 10' average on M1M2 and M3M4 meteo stations) and low temperature (6.5°C). In *Fig.12*, the 10' average wind speed and direction are presented.

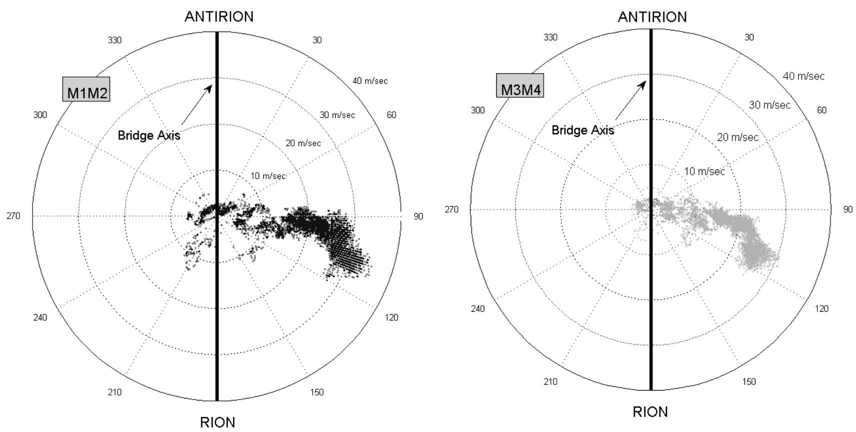

Figure 12. 10' average wind speed and direction graphs

During this event no significant vibrations of the cables were observed. The response of the deck had similar frequency content but higher vibration amplitudes compared with 2006 event, as expected. No ice formation was

observed during 2010 event.
For comparison with 2006 event, the same treatment of data (deck/cable accelerometers and load on cables) was performed and the results are presented hereunder.

6.1 Deck vibration

The maximum vertical displacement amplitude at each sensor's location is presented in the Table 6, and in *Fig.13*.

Table 6. Maximum vertical displacement amplitude at sensor location

Position	Accelerometer channel	Amplitude (cm)	Position	Accelerometer channel	Amplitude (cm)
M1S18E	E3 Z axis	14.75	M2M3W	D17 Z axis	18.59
M1S18W	E4 Z axis	14.45	M3S20W	E19 Z axis	18.76
M1N17E	E7 Z axis	14.60	M3N20E	E24 Z axis	17.78
M1M2W	E9 Z axis	18.47	M3M4E	D26 Z axis	14.40
M1M2E	D9 Z axis	18.79	M4S20E	E28 Z axis	14.10
M2S17E	E11 Z axis	16.79	M4N18W	E32 Z axis	12.48
M2N14W	E15 Z axis	15.00	M4N18E	E33 Z axis	12.99
M2M3E	E17 Z axis	19.75	-	-	-

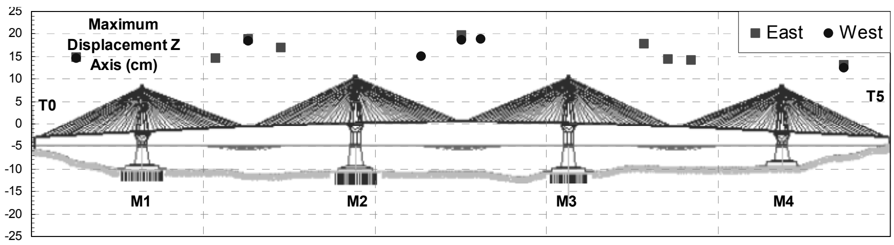

Figure 13. Maximum vertical displacement amplitude at sensor location

In *Fig.14* the average normalized power spectra density for acceleration and displacement time histories of all the deck sensors are presented and compared with the frequency band on the 1st natural mode of the cables.

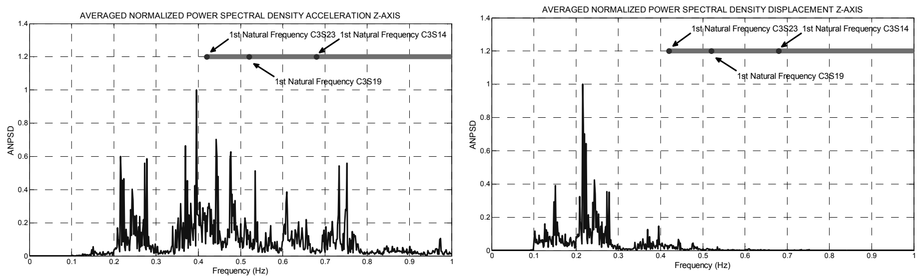

Figure 14. ANPSD for acceleration and displacement based on 07:15 08/03/2010 Alert file

6.2 Cable vibration

The maximum amplitude of vibration $U_m = \max(u_m(t))$ for all cables calculated for all Alert files are presented to Table 7 and *Fig.15*.

Table 7. Maximum vertical displacement amplitude

Position	Accelerometer channel	Amplitude (cm)	Position	Accelerometer channel	Amplitude (cm)
C1S18W	J4 Y and Z	29.5	C3S23W	J18 Y and Z	23.5
C1N10E	J6 Y and Z	15.5	C3S10E	J20 Y and Z	20.8
C1N23E	J8 Y and Z	33.3	C3N17W	J23 Y and Z	30.2
C2S23W	J10 Y and Z	40.4	C4S23W	J27 Y and Z	25.5
C2S10W	J12 Y and Z	22.1	C4S10W	J29 Y and Z	12.5
C2N07E	J14 Y and Z	14.6	C4N18W	J32 Y and Z	27.3
C2N23E	J16 Y and Z	24.5	-	-	-

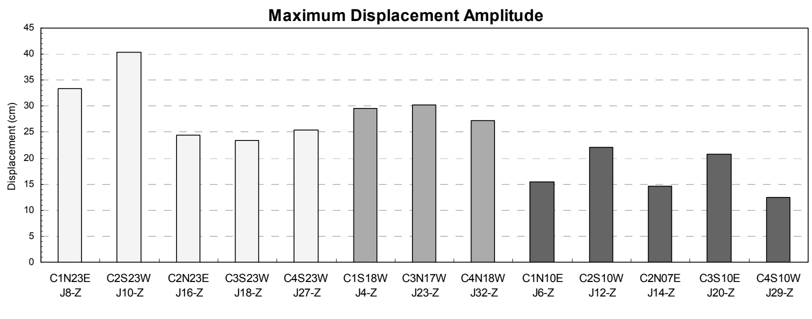

Figure 15. Maximum displacement amplitude of cable stays sorted per length

The maximum load of the cables was within SLS (50% of F_{GUTS}=265.5 kN) as presented hereunder.

Table 8. Maximum cable load

Position	Sensor	Maximum load (kN)	Percentage of F_{GUTS} (%)	Position	Sensor	Maximum load (kN)	Percentage of F_{GUTS} (%)
C1S18W	K4	96.0	36.2	C3S10E	K20	77.6	29.2
C1N10E	K6	86.4	32.5	C2N07E	K22	64.6	24.3
C1N23E	K8	91.8	34.6	C3N17W	K23	87.0	32.8
C2S23W	K10	85.8	32.3	C3N23E	K25	77.8	29.3
C2S10W	K12	84.7	31.9	C4S23W	K27	87.8	33.1
C2N07E	K14	64.5	24.3	C4S10W	K29	89.2	33.6
C2N23E	K16	88.6	33.4	C4N05E	K30	60.8	22.9
C3S23W	K18	87.0	32.8	C4N18W	K32	101.2	38.1

7 CONCLUSIONS

The design against wind induced vibrations of slender structures, such as cable-stays, is a high importance issue regarding safety and user comfort, especially for important infrastructures such as the Rion Antirion Bridge. Several studies (theoretical and experimental) were performed during design phase in order to minimize uncertainties regarding cable stay vibration. Additionally, proper adaptations on the deck and cables were performed during design/construction phase for easy implementation of potential mitigation measures. The analysis of the actual structural response recorded through the Monitoring system of the Bridge due to a strong wind event in 2006 gave the opportunity to optimize the required technical improvement, in this case external dampers on the intermediate and long cables. The efficiency of adopted improvement was verified through commissioning tests. Three years after the implementation the most severe wind storm stroke the Rion Antirion Bridge. Nevertheless, the response of the cable stays to that excitation was limited, even though the deck vibration was more intense than in 2006, as expected.

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