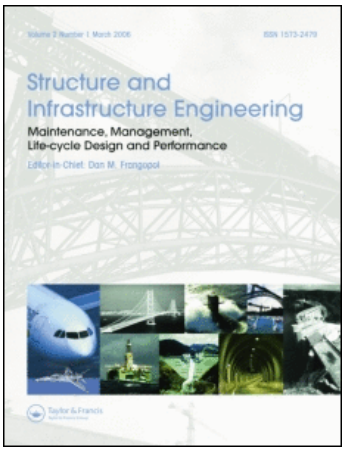

Structure and Infrastructure Engineering
Publication details, including instructions for authors and subscription information:
http://www.informaworld.com/smpp/title~content=t713683556

Rain-wind-induced cable vibrations in the Alamillo cable-stayed bridge (Sevilla, Spain). Assessment and remedial action

Joan R. Casas [a]; Angel C. Aparicio [a]
[a] Civil Engineering Department, Technical University of Catalonia, Barcelona, Spain

First Published on: 09 July 2009

To cite this Article Casas, Joan R. and Aparicio, Angel C.(2009)'Rain-wind-induced cable vibrations in the Alamillo cable-stayed bridge (Sevilla, Spain). Assessment and remedial action',Structure and Infrastructure Engineering,99999:1,
To link to this Article: DOI: 10.1080/15732470903068607
URL: http://dx.doi.org/10.1080/15732470903068607

Structure and Infrastructure Engineering
2009, 1–8, i*First* article

Taylor & Francis
Taylor & Francis Group

Rain–wind-induced cable vibrations in the Alamillo cable-stayed bridge (Sevilla, Spain). Assessment and remedial action

Joan R. Casas* and Angel C. Aparicio

Technical University of Catalonia, Civil Engineering Department, UPC, Jordi Girona 1–3, Campus Nord, Modulo C1, 08034 Barcelona, Spain

(*Received 31 March 2008; final version received 27 May 2009*)

Rain combined with wind action provoking vibration has been observed in the longest stays of the Alamillo cable-stayed bridge in Sevilla (Spain). The maximum displacements observed are in the order of magnitude of 0.5 m and have caused discomfort problems to the pedestrian circulation across the bridge. The paper shows the analytical and experimental studies carried out to analyse the possible solutions and also the steps developed to stop the vibration phenomenon. In 2004, a dynamic test in the cables was envisaged to measure the actual damping in the cables. It became clear that damping in the cables was very low and the solution required the installation of dampers. The damping devices had to be in accordance with the relevant aesthetic constraints of the structure. The installation of the damping devices took place during 2007. After the installation of the damping devices, a new dynamic test was carried out in February of 2008 to check if the level of damping introduced by the dampers was appropriate. During the tests performed in 2004, the natural frequencies of vibration were also obtained at the same time than damping. The comparison between the two sets of natural frequencies (1992 and 2004) allow to extract important conclusions about the correct performance and condition state of the bridge as well as the evolution of forces in the cables after 12 years of construction due to creep in the tower.

Keywords: cable vibration; cable-stayed bridge; damping; dynamic test

1. Introduction

The Alamillo cable-stayed bridge (Figure 1) was open to traffic in 1992 during the EXPO-92 world exhibition in Sevilla (Spain). The span length is 200 m and the deck is composed of a hexagonal steel box girder of 4.40 m depth. The width of the steel box is variable, and every 4 m, two lateral cantilevers 13.20 m in width, formed of steel ribs, support a reinforced concrete slab of 23 cm thickness, forming the traffic carriageway. The pedestrians pass over the upper flange of the steel box. The deck is connected to the pylon via 13 pairs of parallel stays, each pair consisting of 60 strands that are15.24 mm in diameter, except for the last pair, which are 292 m in length and which consist of 45 strands that are 15.24 mm in diameter. The pylon is a composite (steel–concrete) structure. Its variable cross section contains a circular void, 4 m in diameter up to a level of 76.15 m, reduced to 2 m diameter from the 76.15 m level to the 132.25 m level. The total height of the pylon from the pedestal base (7.0 m level) to the crown is 134.25 m. The pylon has an inclination of 32° to the vertical, which makes it possible to balance the forces in the cable stays without the use of back-stays. Both structures, pylon and deck, are embedded in a base that, in turn, is supported by 54 piles, 2 m in diameter and 47.5 m long (Aparicio and Casas 1997).

After the construction of the bridge, a dynamic test was carried out by measuring the acceleration at several points in the tower and deck during the passage of two trucks over the undisturbed and disturbed (by an obstacle) pavement. Natural frequencies, damping and mode shapes of the pylon and deck were obtained (Aparicio and Casas 1992, Casas 1995). A dynamic test was also carried out in the stays to derive the final force in the cables from the natural frequencies obtained in their free-damped vibration (Casas 1994). Although, at that time, it was already known that, for the vibration phenomena in cables due to the combined action of wind and rain, during the dynamic test performed in 1992, the damping level in the stays was not measured because definitive criteria to avoid the problem of damping requirements was not yet available.

2. The problem

After some years in operation, rain-induced vibration of the longest cables (cables 9 to 13) appeared during

*Corresponding author. Email: joan.ramon.casas@upc.edu

ISSN 1573-2479 print/ISSN 1744-8980 online

DOI: 10.1080/15732470903068607
http://www.informaworld.com

rainy periods with moderate wind speeds of the order of 15 to 20 m/s. This phenomenon has been observed in the last years in other cable-stayed bridges (FHWA 2007). In the case of the Alamillo bridge, the maximum transversal deflections are of the order of 0.5 m for cables with a length between dampers in the range of 200 m (cable 10) up to 260 m (cable 13). As observed, in the longest cables (cables 10 to 13), the vibration was mainly in the second vibration mode. This large deflection causes fear to pedestrians crossing the bridge, and can lead to fatigue problems in the near future. In fact, in FIB (2003), it is recommended to limit the amplitude of the cable vibration for the first mode of vibration to $L/2000$ (where L is the chord length of the stay cable) in order to avoid problems in the stays. The Eurocode EN 2006 (CEN 2006) states that, for user comfort and safety, the amplitude of vibrations should not exceed the value $L/500$ for a moderate wind velocity of 15 m/s. More recently, in the USA, a maximum amplitude of the order of the magnitude of the cable diameter is allowed (FHWA 2007). Clearly, the recorded vibration amplitudes far exceeded the existing recommendations, and so fatigue safety is jeopardised.

Figure 1. View of the Alamillo bridge.

3. Analysis of the vibration problem

Because the wind–rain-induced vibration phenomena is highly dependent on the damping level of the cables, a dynamic test of all stays was carried out in 2004 to measure the level of critical damping in each stay as the first step to analyse and attempt to solve the problem. In the dynamic test, the excitation of the cable was achieved by hanging on the cable and moving it upward and downward with a frequency close to the natural frequency of the cable (see Figure 2). Once the amplitude of the vibration was large enough, the cable was released and then the free-damped vibration of the cable was recorded with a piezoelectric-type accelerometer that was located at the lower part of the cable (Figure 2). The acceleration records (Figure 3), after being conditioned by an accelerometer preamplifier and recorded on a portable computer were then used to obtain the natural frequencies via fast Fourier transforms (FFTs), as presented in Figure 4. Damping ratios were also obtained. The method used to derive damping was the logarithmic decrement using the free-damped

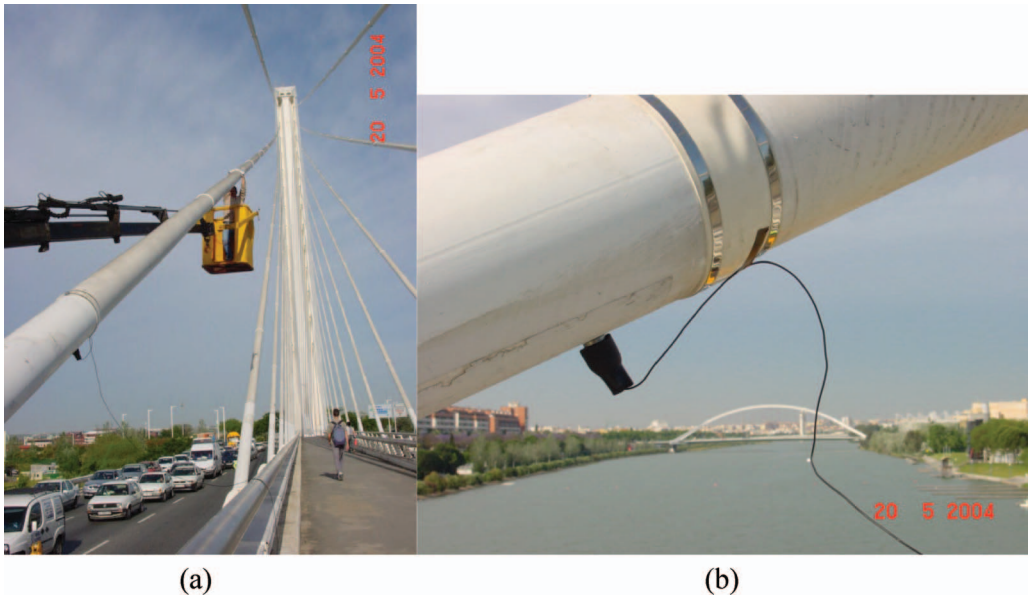

Figure 2. (a) Excitation by hanging from a crane and (b) view of the accelerometer attached to the cable.

vibration part of the total recorded signal. Because the total acceleration signal has contributions from several vibration modes, first the signal was divided into the different mode components by selective digital filtering and then, the logarithmic decrement and the percentage of critical damping (damping ratio ζ), corresponding to each vibration mode, was obtained, as presented in Table 1.

It should be mentioned that not all recorded signals were valid to apply the method. In fact, in the short

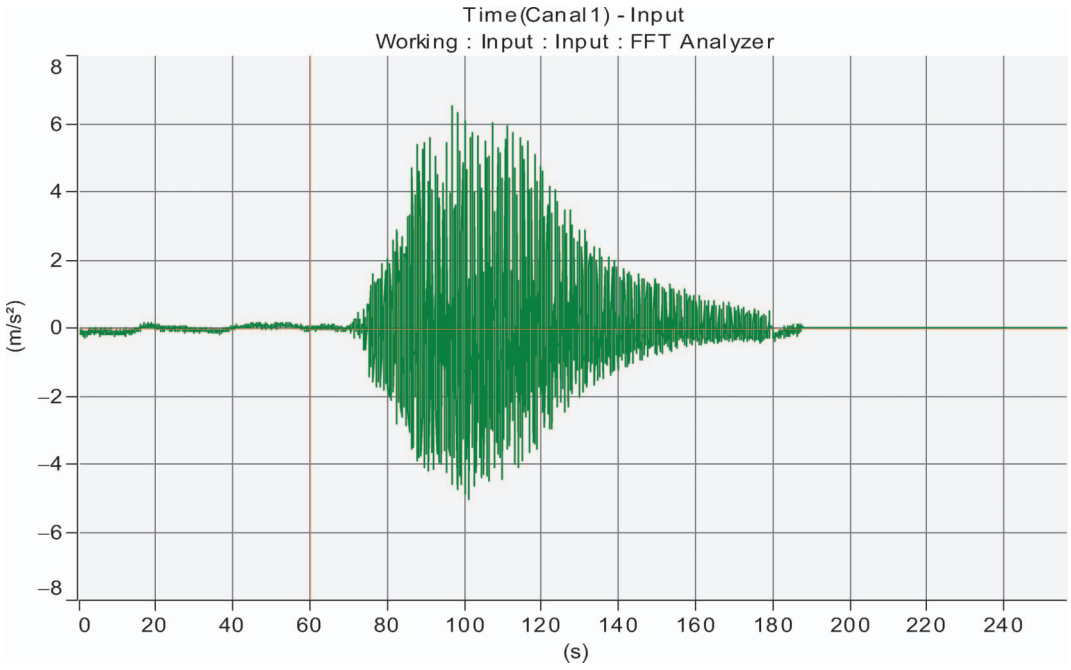

Figure 3. Acceleration record of cable 6L. The forced and the free-damped parts should be noted.

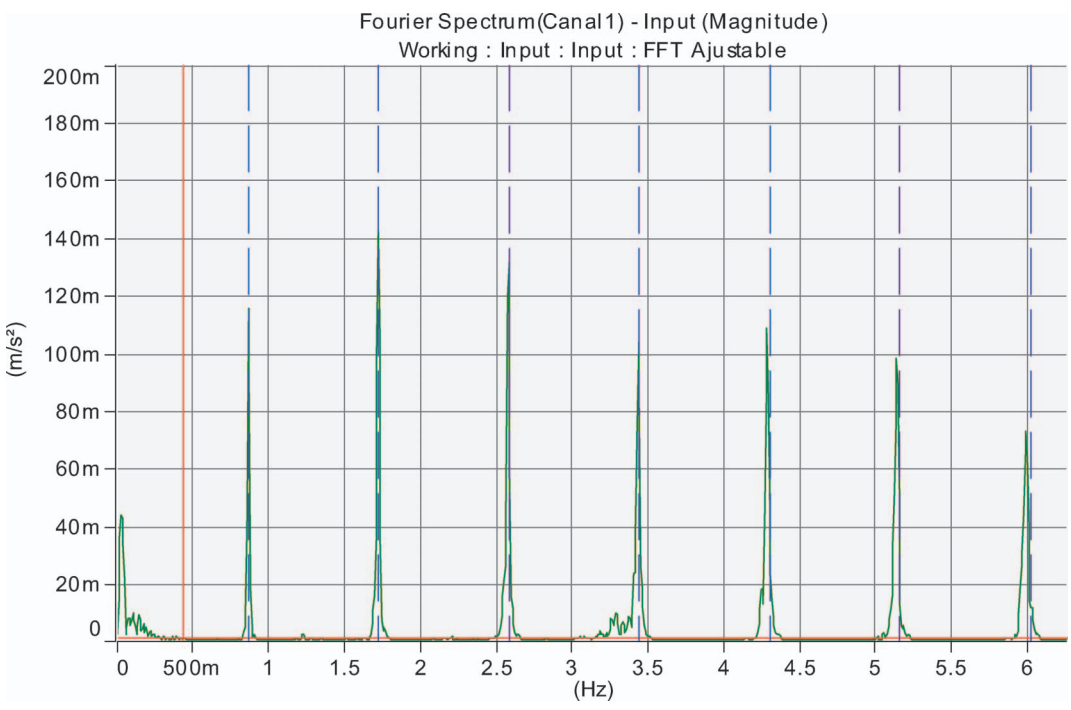

Figure 4. FFT of vibration recorded in accelerometer in cable number 6.

Table 1. Values of the percentage (in %) of critical damping deduced from the total and decomposed signals.

Cable	Total record	Decomposed record			
	ζ	ζ_1	ζ_2	ζ_3	ζ_4
1R	2.39	1.00	1.27	–	–
2L	1.24	1.18	0.59	–	–
5L	0.48	–	0.43	–	–
7L	0.38	–	0.19	0.19	0.18
8R	0.57	–	0.49	0.18	–
9L	0.35	–	0.32	0.16	–
10L	0.30	–	–	0.21	0.18
13R	0.24	–	0.16	0.05	0.05
13L	0.32	–	–	–	–

Cable notation: 1 refers to the shortest and 13 to the longest cables. The right (R) and left (L) is related to a pedestrian walking from the end of the deck towards the tower (see Figure 1).

cables, due to their transversal stiffness, it was very difficult to excite a vibration with sufficient amplitude to be measured with accuracy. Therefore, in Table 1, only the cables with the most reliable results are presented. Cable 1 is the shortest and 13 the longest one. As a result, the additional damping necessary for cables lower than 5 could not be decided upon. This is not an issue, as these cables did not show any problem of rain-induced vibrations, and their existing damping was not a problem. Also in Table 1, the results obtained by decomposition in the different vibration modes are compared with the rough estimates obtained using the complete signal. The results obtained show how the cables more prone to vibration and those with larger vibration amplitude are those with the lowest damping (cables 9 to 13).

As can be seen, the damping in each mode is lower than the damping obtained with the complete signal. Of course, the most accurate results are those obtained through signal decomposition. In addition, it can be deduced, from Table 1 and the FFT obtained for the different cables, that the highest contribution to the total vibration comes from vibration modes 2 and higher. The dominance of mode 2 in the vibration was also confirmed by visual observation (monitored by video) of the vibration of the cables during an episode of stormy weather with heavy rain and wind. FIB (2003) and CEN (1993) recommend a percentage of critical damping higher than 0.5% (logarithmic decrement higher than 0.03) in cables longer than 80 m to avoid the rain-induced vibration. As can be seen in Table 1, none of the long cables accomplishes this requirement.

Comparing the values of the damping in Table 1 with those proposed by the Post-Tensioning Institute (PTI 2001) to avoid the rain–wind vibration, it is clear that most of the cables do not match the requirements. In PTI (2001), the requirement to avoid the rain–wind vibration is:

$$\frac{m\xi}{\rho D^2} > 10, \tag{1}$$

where m is the mass of the cable per unit length; ξ is the percentage of critical damping; ρ is the density of air; and D is the diameter of the external pipe.

In order to avoid excessive vibration by vortex shedding in the case of excitation only by wind (without rain), the following criteria is recommended (Cremona and Foucriat 2002):

$$\frac{m\xi}{\rho D^2} > 1.2. \tag{2}$$

In the Alamillo bridge, $D = 0.2$ m and the mass per unit length is 77 kg/m for cables 1 to 12 and 60 kg/m for cables 13. According to the damping values measured (Table 1), the longest cables do not follow Equation (1); therefore, this gives the explanation for the large deflections observed under the combined action of rain and wind. However, all cables accomplish Equation (2) and large vibrations due to the wind action only should not be expected. In fact, this has been observed during the service years of the bridge. The most prone stay to have such problems is number 13 (low mass and low damping). For this cable, Equation (2) is 1.91, very close to the proposed threshold. In fact, during the operation life of the bridge, some vibration phenomena have been observed in this cable, even at very low wind speeds and in the absence of rain.

As mentioned, during the tests in 1992, the natural frequencies in the cables were measured and the corresponding forces were derived. During the tests performed in 2004, the natural frequencies of vibration were also obtained at the same time as the damping. In Table 2, the natural frequencies, derived as the average of the frequencies of the lowest vibration modes, obtained in 1992 and 2004 for the cables on the right (R) and left (L) side, are shown. The comparison between the two sets of natural frequencies (1992 and 2004) allows important conclusions about the behaviour and performance of the bridge, as well as the evolution of the forces in the cables after 12 years of construction, to be made.

According to the values in Table 2 and the method proposed in Casas (1994) to derive the effective cable length and the force in the cable, based on the natural frequency, the values of the force in the stays after bridge completion and after 12 years in service were deduced, as shown in Table 3.

In Table 3, the actual forces are higher in the short cables (cables 1 to 7) and lower in the long cables (8 to 13). The highest variation in the cable force is of the order of 5% to 6%. However, the difference in the sum of the forces in all cables between 1992 and 2004 is only 0.36%. This extremely low difference, close to the precision limit of the instrumentation used, clearly shows two important facts. The first is that the modification in cable forces with time was due to an internal redistribution between the cables, most probably because of creep in the concrete tower. In fact, the finite-element calculations and structural analyses carried out during the bridge design excluded the possibility of these changes in cable forces because of a temperature effect. Second, it validates the experimental results obtained in the dynamic tests carried out in 1992 and 2004.

Apart from the free-vibration excitation of the cables, the vibration due to traffic was also measured. In order to get a significant level of vibration in the cables, a truck going over artificial unevenness in the pavement was immersed in the traffic flow (Figure 5).

In this case, the attachment of accelerometers in the bridge deck (Figure 6) allowed the vibration of the deck to be obtained, and the natural frequencies could be derived according to the methodology presented in Casas (1995), based on the analysis of the Fourier spectra obtained by FFT of the time signals. From the vibration records, the values in Table 4 were deduced. The values in Table 4 are also compared with the values obtained in the dynamic tests carried out in 1992 after bridge completion and before opening to the traffic (Aparicio and Casas 1992, Casas 1995).

The small differences of natural frequencies between years 1992 and 2004 indicate a good performance and the healthy condition of the bridge.

Table 2. Natural frequency (in Hz) of cables obtained in dynamic tests in 1992 and 2004.

Cable	*f* (right 1992)	*f* (right 2004)	*f* (left 1992)	*f* (left 2004)
1	2.276	2.189	2.195	2.317
2	1.722	1.704	1.702	1.755
3	1.429	1.433	1.430	1.453
4	1.119	1.141	1.123	1.133
5	0.965	0.995	0.981	0.975
6	0.849	0.862	0.854	0.859
7	0.761	0.777	0.775	0.759
8	0.685	0.692	0.693	0.684
9	0.619	0.621	0.624	0.622
10	0.555	0.550	0.553	0.535
11	0.501	0.492	0.507	0.489
12	0.452	0.445	0.453	0.435
13	0.501	0.482	0.498	0.489

Table 3. Cable forces (in kN) in 1992 and after 12 years in service.

Cables	Force (1992)	Force (2004)
1	5778	5833
2	5847	5973
3	6112	6228
4	5307	5461
5	5405	5537
6	5082	5189
7	5239	5249
8	5121	5101
9	5013	5018
10	4640	4447
11	4562	4309
12	4199	3964
13	4365	4127
Total	66680	66436

4. Proposed remedial and final solution for rain-induced vibration

Possible solutions to stop the rain–wind-induced vibration in cables are:

(1) use of stay pipes manufactured with some geometrical ribs or dimples on the external surface;
(2) interconnecting cables using additional stays that modify the frequency of vibration of the cables; and
(3) installation of damping devices to increase the low damping of the cables.

Solution (1) is very well suited in the case of new construction, but presents economical and technical

Figure 5. Excitation of deck and cables by traffic and truck over an obstacle.

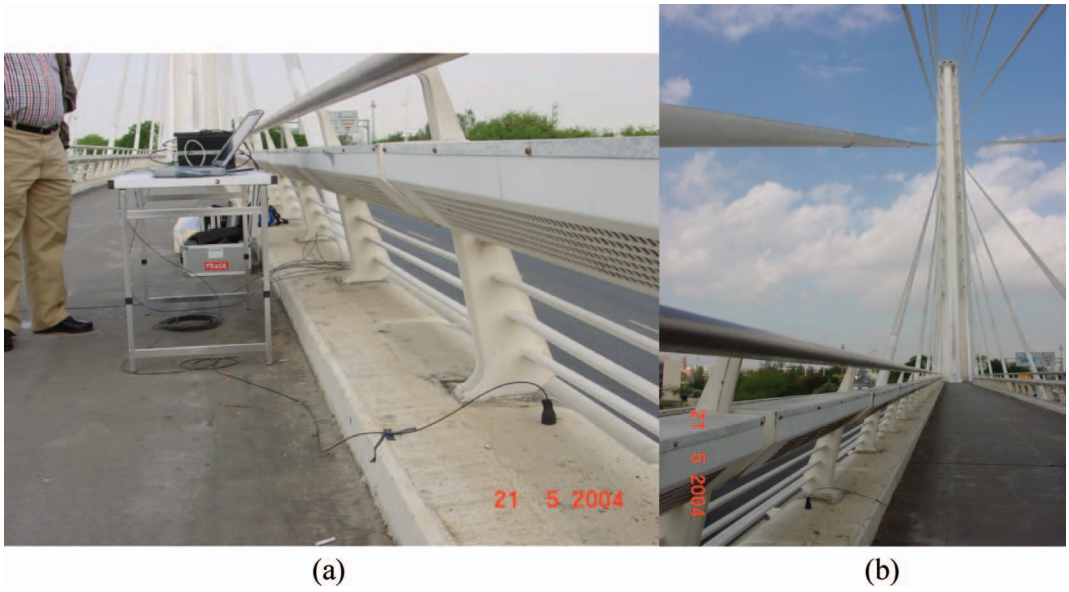

Figure 6. (a) Eccentric situation to record flexural and torsional vibrations and (b) the accelerometer in the deck.

Table 4. Natural frequencies (in Hz) in the deck. Comparison of values after 12 years in service.

Vibration mode	Year 2004	Year 1992
Longitudinal bending 1	0.42	0.40
Longitudinal bending 2	0.66	0.66
Longitudinal bending 3	1.21	1.205
Torsion 1	1.172	1.155
Longitudinal bending 4	2.180	2.155
Torsion 2	–	2.295
Longitudinal bending 5	2.82	2.78

problems when applied to existing cables. Solution (2) is not suitable in the present case due to aesthetic reasons and difficulties in accessibility and maintenance. Solution (3) seems to be the most recommended. The next point is to calculate the damping necessary to stop the problem.

The recommendations of FIB (2003) and CEN (1993) require a percentage of critical damping higher than 0.5% in cables longer than 80 m. According to the recommendations of the PTI (Equation (1)), and the cable characteristics of the Alamillo bridge, the minimum percentage of critical damping to avoid rain–wind-induced vibration is 0.64% for cables 1 to 12 and 0.82% for cable 13. From the results in Table 1, it is clear that only cables lower than number 5 meet this requirement, and therefore additional damping is needed to solve the problem. According to the actual values of damping in the cables, the minimum additional damping that should be provided by the damping system to be installed in each cable is presented in Table 5. Dampers had to be installed from cables 5 up to 13.

The solution of additional dampers was finally chosen and was installed in the bridge during 2007. The dampers are of the magneto-rheological type. The damping values in Table 5 should be guaranteed for a maximum distance of the damper to the anchorage lower than 3% of the cable length. Due to aesthetic constraints, one of the main requirements in the case of external dampers is that the location of the damper should not be above the pedestrian handrail. As a general rule, the damping devices to be installed should have a minimum visual impact on the global appearance of the bridge. As can be seen in Figure 7, where one of the installed dampers can be seen, these requirements were successfully fulfilled, and the dampers are hardly noticeable.

To check that the required damping is provided by the dampers, a new dynamic test in the cables was carried out in February 2008. The cables were again excited by hanging, as during the tests in 1992 and 2004. However, in this case, due to the presence of the dampers, it was much more difficult to excite the cables, and the amplitude of the recorded acceleration was clearly lower than that obtained in the test in 2004 (see Table 6). In Table 6, the vibration amplitudes and the damping results (percentage of critical damping) for each cable and the first three modes of vibration are shown and compared with the results from the tests in 2004. Only for those cables where suitable records for analysis were obtained. The natural frequencies measured in the 2008 test were very similar to the ones obtained in 2004. The slight variation

Table 5. Minimum damping required in the cables.

Cable	Minimum additional damping ratio
5	0.48
6	0.48
7	0.48
8	0.48
9	0.48
10	0.64
11	0.64
12	0.64
13	0.80

Figure 7. View of the dampers.

cannot be accurately related to any change in the cable forces.

In all cases, the damping obtained after the placement of the dampers is much higher, despite the lower amplitude vibration. In the case of the damped cables, the value of the damping is highly dependent on the amplitude of vibration, as expected for a magneto-rheological damper. In fact, for the case of cable 13R, for example, for acceleration levels between 0.15 and 0.4 m/s^2, damping in the second vibration mode is 0.41%, whereas for amplitudes between 0.4 and 0.5 m/s^2, the value is 0.62%. For levels above 0.5 m/s^2 and up to 1 m/s^2, the measured damping is 1.19%. These results confirm the non-linear behaviour of the dampers.

In cables 12, 11 and 10L, the minimum required damping is 0.64% (see Table 5). The measured damping is slightly lower (around 0.57%). According to the explanation given for cable 13, this low damping can be due to the low amplitude of acceleration obtained in the test. A similar behaviour appears in cable 6R.

Table 6. Percentage of critical damping and acceleration amplitudes in the cables in the dynamic tests in 2004 and 2008.

	Without dampers (2004)				With dampers (2008)			
Cable	ζ_1 (%)	ζ_2 (%)	ζ_3 (%)	A (m/s^2)	ζ_1 (%)	ζ_2 (%)	ζ_3 (%)	A (m/s^2)
5L	–	0.43	–	2.1	–	0.56	–	0.7
5R	–	–	–		0.70	0.64	0.49	1.2
6L	–	–	–		0.99	1.00	0.64	1.9
6R	–	–	–		–	0.37	–	0.5
7L	–	0.19	0.19	6.0	0.64	0.65	0.43	1.0
7R	–	–	–		–	–	–	*
8L	–	–	–		–	–	0.51	0.9
8R	–	0.49	0.18	5.0	–	–	0.59	1.0
9L	–	0.32	0.16	1.0	–	0.56	–	1.0
9R	–	–	–		–	–	–	*
10L	–	–	0.21	4.0	–	0.48	–	0.3
10R	–	–	–		–	0.65	–	0.5
11L	–	–	–		–	0.57	–	0.8
11R	–	–	–		–	–	–	*
12L	–	–	–		–	0.57	0.51	0.7
12R	–	–	–		–	0.51	0.21	1.0
13L	–	–	–		–	0.91	0.38	1.0
13R	–	0.16	0.05	4.0	–	1.19	0.33	1.0

*Signal not adequate for analysis, R = right, L = left.

5. Conclusions

The measurement of actual damping in the cables of the Alamillo bridge, carried out in 2004, showed that the rain–wind-induced vibrations observed in several cables of the bridge were due to low damping provided by the internal dampers supplied originally with the cable. The proposed solution to stop the problem was to install damping devices in the cables most prone to be affected by the phenomena (cables 5 to 13). The correct placement of the dampers and their damping characteristics are of high importance due to the aesthetic constraints that the bridge presents. The measurement of damping also allowed the calculation of the damping that should be provided by the damping devices to be installed, which is in the range of 0.48% to 0.8% (percentage of critical damping). The dampers were installed during the year 2007, and the dynamic tests carried out in the cables at the beginning of 2008 showed that the amount of damping provided by the dampers seems to be sufficient to stop the problem of rain–wind-induced vibration. In fact, no vibration problems have been detected up to now.

In the dynamic tests carried out in 2004, the natural frequencies in the cables were also monitored and compared to those deduced from the dynamic tests after construction. Despite differences in natural frequencies in specific cables of the order of 6%, the difference in the sum of forces in all cables between

1992 and 2004 is only 0.36%. This extremely low difference, close to the precision limit of the instrumentation used, clearly shows that the modification in cable forces with time has been due to an internal redistribution of forces between the cables, most probably because of creep in the concrete tower. The values obtained confirm that the cables are performing as expected during design, and no evident loss of efficiency due to corrosion or other degradation processes is observed. The vibration records in the deck due to the normal traffic on the bridge indicate a similar conclusion in the case of the bridge deck. Therefore, a healthy condition of the bridge can be derived from the dynamic tests carried out, after more than 16 years in full service.

Acknowledgements

The work presented in this paper has been developed under contract C-05602 between the Technical University of Catalunya (Bridge Department) and the Regional Government of Andalucia (Junta de Andalucia). The authors want to acknowledge the technical and logistic support provided by Dr Carlos Alonso and Mr José Alvarez from the Road Directorate of the Junta de Andalucia during the execution of the tests and the support from Spanish Ministry of Education through project BIA-2007-28685-E.

References

Aparicio, A.C. and Casas, J.R., 1992. *Final report of the dynamic tests carried out in the Alamillo bridge* (in Spanish). Spain: UPC-Regional Government of Andalucia.

Aparicio, A.C. and Casas, J.R., 1997. The Alamillo cable-stayed bridge: special issues faced in the analysis and construction. *Proceedings of the Institution of Civil Engineers, Structures and Buildings*, 122, 432–450.

Casas, J.R., 1994. A combined method for measuring cable forces: the cable-stayed Alamillo bridge, Spain. *Structural Engineering International*, 4 (4), 235–240.

Casas, J.R., 1995. Full-scale dynamic testing of the Alamillo cable-stayed bridge in Sevilla (Spain). *Earthquake Engineering and Structural Dynamics*, 24 (1), 35–51.

Cremona, C. and Foucriat, J.C., 2002. *Comportement au vent des ponts*. Paris: AFGC – Pressess de lEcole Nationale des Ponts et Chaussées.

European Committee for Standardisation (CEN), 2006. *EN 1993-1-11, Eurocode 3: Design of steel structures. Part 1.11: design of structures with tension components*. Brussels: CEN.

Federal Highway Administration (FHWA), 2007. *Wind-induced vibration of stay cables*. Publication No. FHWA-HRT-05-083. McLean, Virginia: FHWA.

Federation International du Beton (FIB), 2003. *Recommendations for the acceptance of stay cable systems using prestressing steels*. Lausanne: FIB.

Post-Tensioning Institute (PTI), 2001. *Recommendations for stay cable design, testing and installation*. Phoenix: PTI.