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National Aerospace Laboratory NLR

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Abstract

In November 1996, the Erasmus bridge showed the phenomenon “rain/wind induced vibrations”. Under rain and windy weather the stay-cables started to vibrate. During this incident the bridge was closed for all traffic while the stay-cables were immediately connected to the bridge deck by poly-ethyleen ropes, whereafter the vibrations quickly died out.

A full-scale test campaign was initiated by the Public Works of Rotterdam to avoid cable vibrations in future, resulting in newly installed hydraulic dampers between cables and bridge deck. A preliminary monitoring system was installed to verify whether the dampers did not introduce negative effects on the vibrational behaviour of the bridge deck itself and to verify the vibrational amplitudes due to vortex excitation as predicted by wind tunnel tests. Up till now, no vibrations of any importance have been reported.

Contents

(16 pages in total)

1 Introduction

In the morning of November 4th, 1996, the stay-cables of the Erasmus bridge located in Rotterdam (the Netherlands) started to vibrate under rain and windy weather. Cable vibrations were observed with amplitudes of around 2 to 3 times the diameter (0.2 m) being 0.5 to 0.7 m mostly in the second mode shape. The bridge deck showed minor vibrations with amplitudes of around 25 mm. Precautions were taken by mounting poly-ethyleen ropes between the cables and bridge deck whereafter the vibrations quickly died out.

After intensive discussions, also with experts from abroad, it became clear that the Erasmus bridge, like many other bridges worldwide, was subjected to the phenomenom “wind/rain induced vibrations”. Although the main span of the bridge is of a medium size (285 m), the longest cables have a length of nearly 300 m, because the bridge has only one pylon (Fig. 1). Obviously the cables, having a smooth poly-ethyleen casing, became vulnerable to the specific excitation mechanism by which water rivulets, as observed on the cables, play an important role (Matsumoto 1992 and Hikami 1988).

As the original installed dampers, mainly designed to supress vortex excitation, were not capable of preventing unstable behaviour of the cables, a full-scale test campaign was initiated by the Public Works of Rotterdam in order to find a solution to prevent cable vibrations in the future.

This resulted, within one year, in the installation of new hydraulic dampers between cables and bridge deck. Simultaneously, equipment was installed to monitor the behaviour of the bridge deck on the long term.

This paper summarizes the design of the dampers, presents some significant results from full-scale testing and deals with the general idea that monitoring becomes more and more important to understand the behaviour of civil structures on the long term.

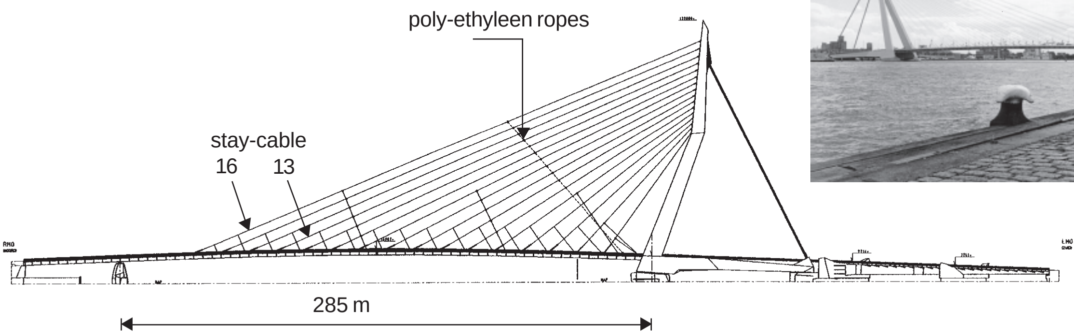


Fig. 1 The Erasmus Bridge

2 Preventing cable vibrations

In the open literature, a number of measures are described to prevent cable vibrations under rain and windy conditions (Fig. 2). Aerodynamic means such as surface roughness or protuberances on the casing as a precaution were not selected by the Public Works of Rotterdam, as the effectiveness of such means is still under discussion (Flamand 1994 and Matsumoto 1990) e.g. under circumstances like for example the occurrence of ice accretion. Furthermore taking such measures on existing cables will result in high costs, or it may even to be impossible to modify the outer shape from a practical point of view.

From discussions by the Public Works with Japanese specialists, it was concluded that increasing of the structural damping by hydraulic dampers was the most promising counter-measure to control cable vibrations.

For that reason, a test-campaign was initiated to verify the design of prototype dampers which have to increase the structural damping of the cables to at least 0.8% c/cr (percentage of critical damping) within a frequency range up to 3 Hz. Above this frequency (being the sixth or seventh vibration mode) no cable vibrations are reported in literature. This is due to the fact that the required high wind speeds will blow water rivulets away from the cable casing and consequently eliminate the excitation mechanism.

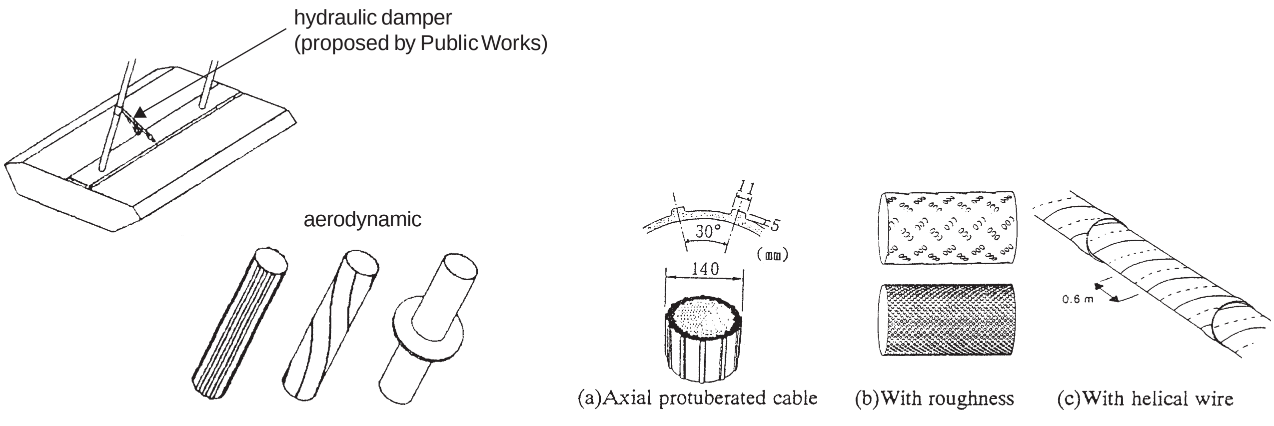


Fig. 2 Aerodynamic counter-measures to prevent cable vibrations

3 Design of the dampers

The original dampers were chosen in accordance with the recommendations of the Eurocode to suppress vibrations by vortex shedding. Chosen were automobile type shock absorbers. Through a mechanical linkage formed by the anti-vandalism tube, the point of application of the damping force was about 6.5 m from the end point of the stays.
As already mentioned the amount of damping of this system, although sufficient to suppress resonance due to vortex shedding, was too low to suppress vibrations by the rain/wind phenomena. Extensive Finite Element calculations showed that the dampers should have a damping coefficient of 30 to 40 kNms^{-1} in order to reach the intended level of damping.
The new damping system should not only fulfil damping requirements; quite important were reliability, maintenance and aesthetical viewpoints.

The position of the dampers with respect to the distance of the fixed end point of the stay was governed by the consideration that the dampers should be effective up to 3 Hz. For the longest stay this would mean up to seven eigen frequencies and a maximum distance of the damper of about 20 m. From an aesthetic viewpoint the damper should be as near to the fixed end point as possible. Further, the design should give the impression that the dampers were a logical part of the stay system and not an addition.
For structural reasons there should be two dampers on each stay. A consequence of the placement of the dampers near the end point is that high damping rates influence not only the amplitudes but the vibration modes as well. From the design curve (Fig. 3) it can be inferred that only at low damping rates the damper is effective for the full 100%. In our case the damper characteristics are located on the top of the design curve. The effect of the dampers was confirmed by a finite element simulation with the DIANA programme. It was clearly demonstrated that a vibrating stay could be damped sufficiently.

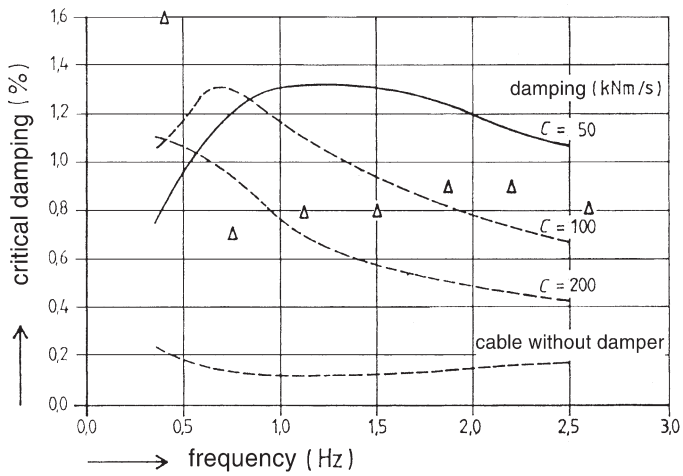


Fig. 3 Calculated and measured (Δ) damping values

Several types of dampers were studied, in particular visco-elastic and hydraulic systems. Structural requirements dictated dampers with a reasonably large displacement. On account of the traffic loads on the bridge the sag of the longest stays is diminishing from about 2.5 m in the dead load situation to 1.2 m under full load. This means that the observed displacements during the vibration are about half of the displacements under live load. Higher stressing of the stays is physically impossible because the force in the stay is directly related to the mass of the bridge section it is supporting. From this viewpoint hydraulic dampers were the best solution. Further, these dampers are highly reliable, as is indicated by their use in railway stock. Physical requirements were that the dampers should be as linear in behaviour as possible and should also be working at very low amplitudes (less than 1 mm).

Because the design of this type of dampers is a very specialised job requiring a lot of experience, two manufacturers were invited to design and deliver prototypes for testing on the bridge. The selected firms were LISEGA (Germany) and KONI (the Netherlands). LISEGA had experience in the production of prototype dampers for the supression of earthquake effects on bridges, and KONI is specialised in all types of dampers ranging from passenger cars to railway cars and even heavier ones. Both firms produced dampers which were remarkable linear in behaviour in factory tests. Internally this was achieved by a combination of orifices and spring loaded valves. During testing on the bridge some effects appeared which were not observed during factory testing.
At high amplitudes the dampers showed not to be as effective as was expected. Analysis of the test results indicated that play in the connection between damper and stay had a reducing effect. By incorporating a tension spring in the system which pre-stressed the connection of the damper to the stay this problem was solved. In the final design the spring forms an integral part of the damper. Reducing the play in the connection by using a clamp without play was not considered practical because the high contact pressure involved would surely damage the protective coating of the strands from which the stay is constructed.
A special problem was observed at low amplitudes. Cable damping was then reduced. It was concluded that this was caused by the internal friction on seals inside the damper. By modifying this part this problem was solved too.

For the final selection of the manufacturer technical reasons did not play a role. Both designs functioned equally well. Decisions were mainly governed by the practical point that the dampers should be placed before the 4th of November 1997, one year after the problems started. KONI could fulfil this requirement (Fig. 4).

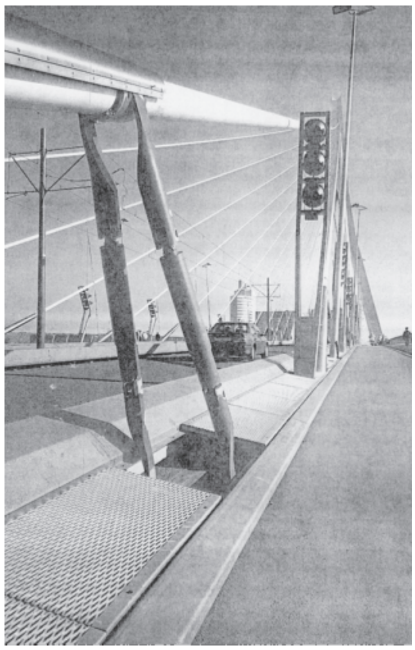


Fig. 4 Actual damper system

4 Full-scale tests

Full-scale testing of the cables was performed by means of a small hydraulic actuator owned by TNO Building and Construction Research (the Netherlands). The actuator was fitted to the cable of concern by a relatively soft helical spring (Fig. 5).
Instrumentation consisted of a force transducer, several displacement transducers (for the lowest frequency) and one piëzo-resistive accelerometer on the cable itself (for the higher frequencies).

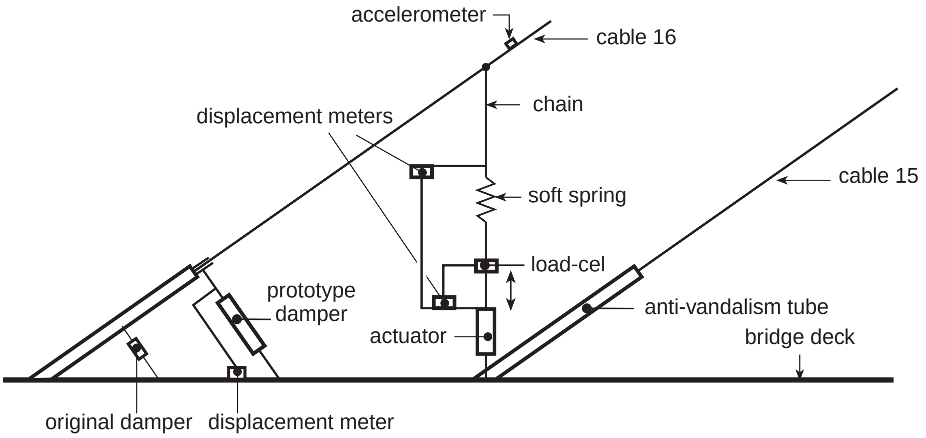


Fig. 5 Hydraulic Actuator System

Excitation of the cable was of a sinusoïdal type by which the well-known 90 degrees phase criterion was applied to set the cable purely in one of its natural frequencies. When a steady amplitude was obtained, the excitation was switched off resulting in a decay from which the damping (logaritmic decrement) was estimated by a quick-look method (NLR) and a curve-fit procedure (TNO). Upon agreement about the damping factor, the excitation was started again and the next natural frequency was adjusted.

The test programme looked simple: first some representative cables (nr 13 and 16, western side) without any damping devices should be subjected to sine excitation whereafter prototype dampers were installed and the sequence of excitation and decaying should be repeated. However, a number of practical inconveniences were encountered during the tests, mostly caused by traffic.

4.1 Practical considerations

As adjustment of the 90 degrees phase crititerion and reaching steady amplitude of the cable took some time (around five minutes), traffic was continuously a source of inconvenience. Passage of especially heavy vehicles did move the bridge deck, changed the cable tension to a certain amount and with that the cable's natural frequency. In such circumstances the phase criterion could hardly be maintained especially for the lowest frequency of the cables without damper devices. This resulted mostly in conflicting decays as shown in Fig. 6. Two attemps with cable 13 are shown at a natural frenquency of f = 0.395 Hz. Neither the first nor the the second attempt gave a satisfactory estimation of the damping factor.
Another inconvenience was the excitation at higher frequencies (f > 1.2 Hz) of the cables without damping devices. As the modal mass, as well as the damping, is substantially lower at higher mode shapes, a small sinusoïdal excitation resulted in large amplitudes fully out of control, so it was decided to cancel the tests at higher frequencies.

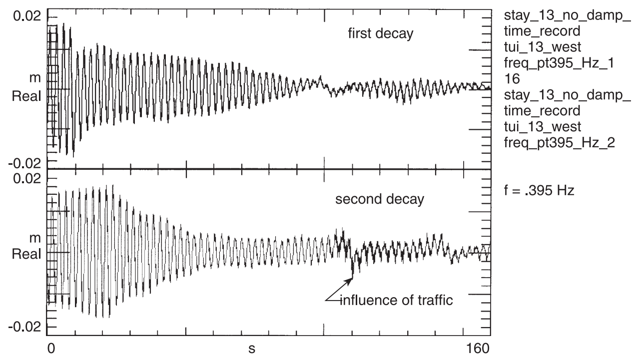


Fig. 6 Conflicting decay's of cable 13 without damping devices

The same test procedures were applied once the prototype damper was installed; first at cable 13, later on at cable 16. In first instance the dampers failed as they did not fulfil the expectation of 0.8% damping. The reason was a certain amount of play between cable strings and the poly-ethyleen casing to which the damper was connected. This shortcoming was quickly solved by mounting a helical spring between cable casing and bridge deck. A pre-tension of around 6500N was sufficient to eliminate the play and resulted in a satisfactory damping behaviour as function of the cable frequencies (next section).

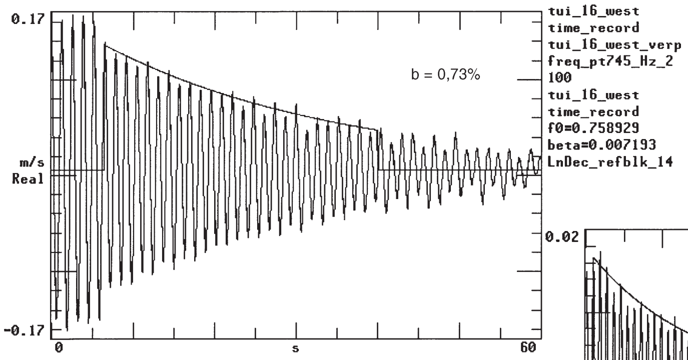
4.2 Results of damping measurements on cables 13 and 16

4.2.1 Without damping devices

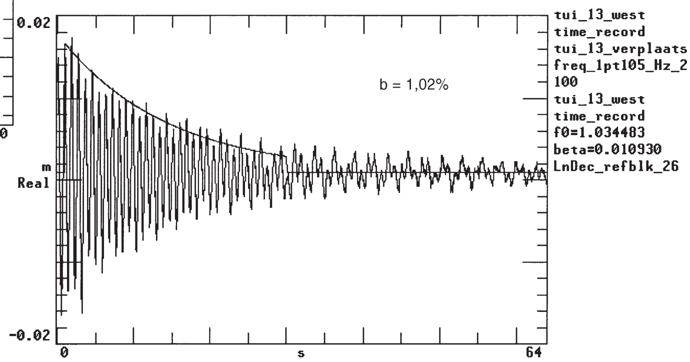
As already mentioned the frequency range in which the cables could be excited was very limited. From the decay's it was observed that the structural damping of the cables itself is very low (around 0.1% to 0.2% c/cr). At the lowest cable frequency of 0.4 Hz however, the damping tends to be higher. This can be explained by interaction with the bridge deck having a first bending frequency of f = 0.44 Hz with a damping of at least 0.6% c/cr. The latter value was obtained from monitoring of the bridge deck (section 5).

4.2.2 With new hydraulic dampers

Once the dampers were installed with pre-tension on the cables, decays such as shown in Fig. 7a and 7b were obtained. Besides a quick-look analysis on the bridge, a more detailed analysis was applied afterwards by means of a curve-fit procedure (LnDec) performed at NLR. Damping factors as a function of the natural frequency of the cables are presented in Fig. 8 from which it can be observed that the longest cable (Fig. 8a) does not fulfill the requirement of 0.8% damping for all frequencies. At some of the frequencies still play was detected which means that the pre-tension on that cable was probably too low. It is the expectation that increasing the pre-tension may result in higher damping values over the entire frequency range like those valid for stay cable 13 (Fig. 8b).

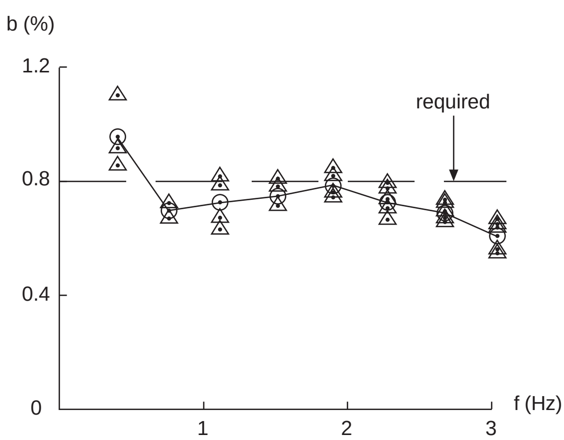


a) Cable 16 (f = 0,76 Hz)

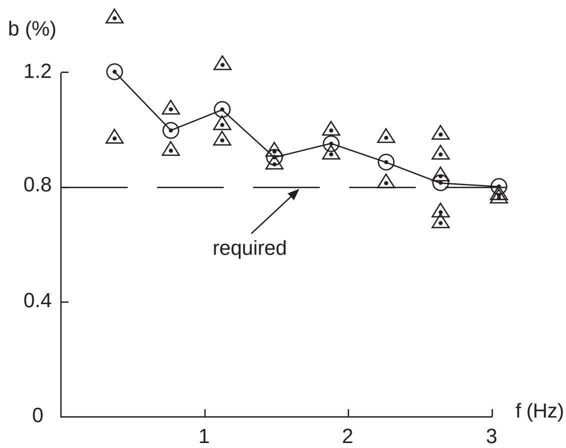


b) Cable 13 (f = 1,03 Hz)

Fig. 7 Decay of cable with prototype dampers and the curve-fit procedure (LnDec)



a) Cable 16 (west) with dampers (⊙ = average)



b) Cable 13 (west) with dampers (⊙ = average)

Fig. 8 Damping obtained with prototype dampers

5 Bridge deck monitoring

A preliminary monitoring system was in operation from July 1997 till November 1998, with the aim to verify wind tunnel predictions once vortex excitation occurs. This monitoring system was based on two accelerometers which were placed at a specific position (at a node of the mode shape of the second bending) in both main girders to eliminate vibrations other than those from the first bending and first torsion.
After narrow-band filtering (0.1 – 0.75 Hz), the accelerations were summed and subtracted. By applying appropriate weighing factors, the signals were representative for bending and torsional amplitudes with reference to the position of the hand railings. By a level detector, the crossings of pre-selected accelerations were counted. The data was converted to amplitudes and stored in the memory of the level detector.
A diagram of the monitoring equipment is presented in Fig. 9.

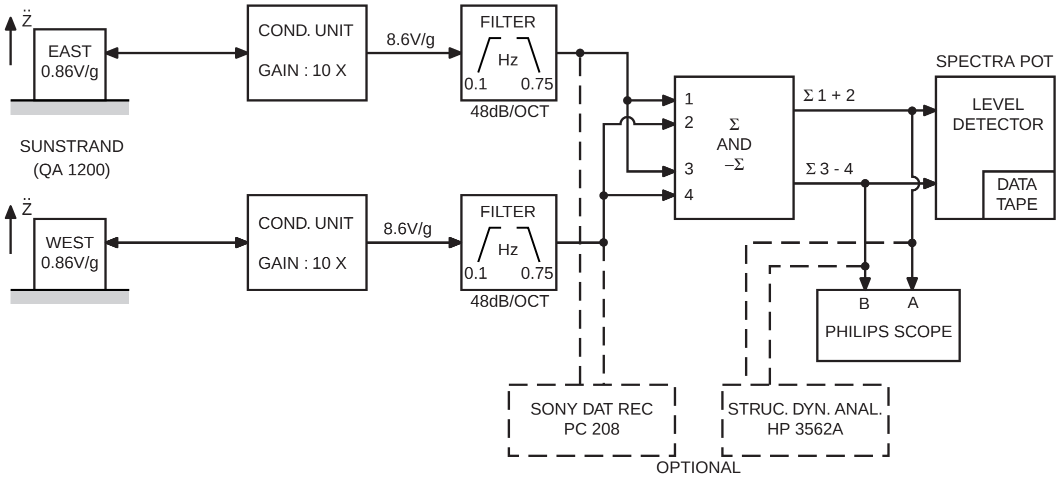


Fig. 9 Diagram of monitoring equipment

5.1 Results from monitoring

Results expressed as peak-to-peak amplitudes at the railing's position were obtained during the period July 1997 till November 1998. An example is shown in Fig. 10. Most of the time, the bridge deck behaves moderately, also at high wind speeds (up to 30 ms^{-1}). Only in the beginning of March 1998 some vortex excitation appeared to be present (after analysing the countings in more detail) resulting in a peak-to-peak amplitude of around 5 cm of the lowest bending mode. This amplitude is nearly half the value predicted from wind tunnel testing in a smooth flow. As turbulence will be present in practice, which counteracts vortex shedding, it is not the expectation that this bridge will respond with much higher amplitudes than already measured. The character of the bridge deck response is illustrated by Fig. 11. The time signals show that the response remains low, though there are time segments where the amplitude starts to rise but dies rapidly.

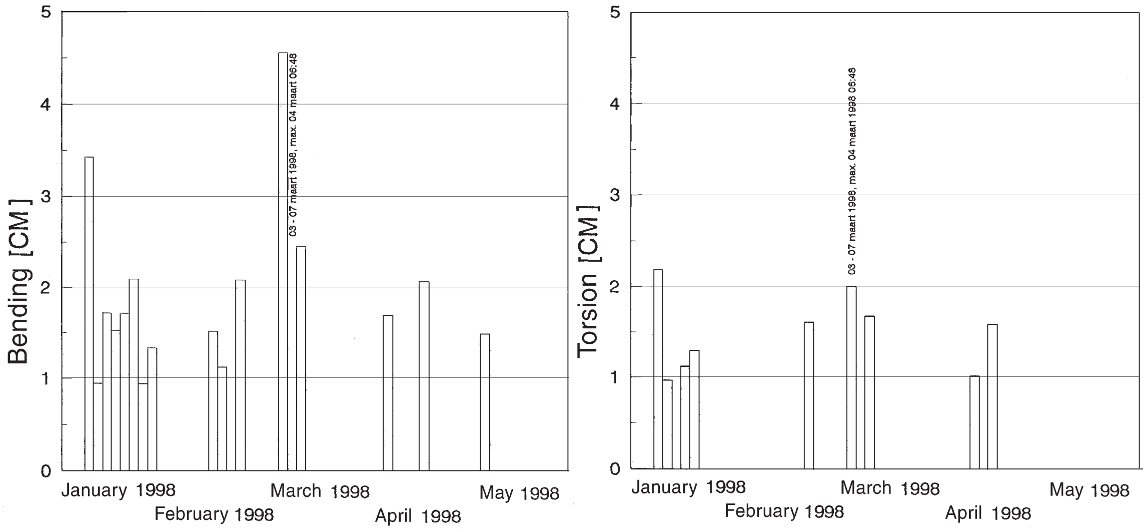
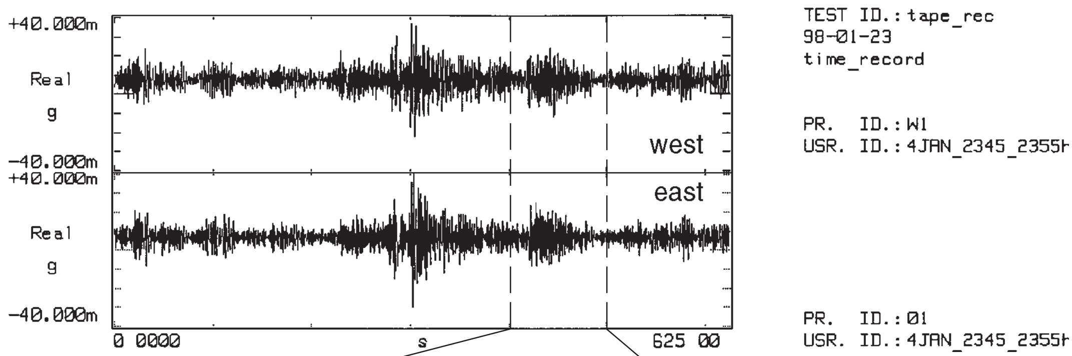
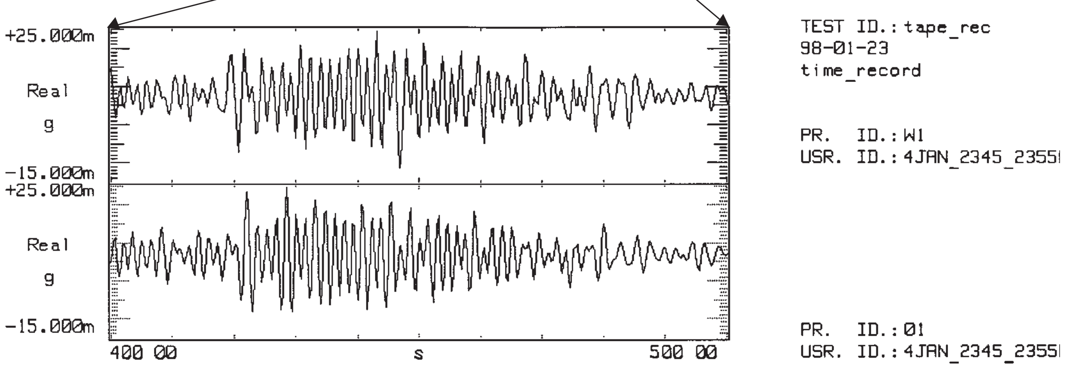


Fig. 10 Results of monitoring (amplitudes peak to peak) at the position of railing



a) During 10 minutes



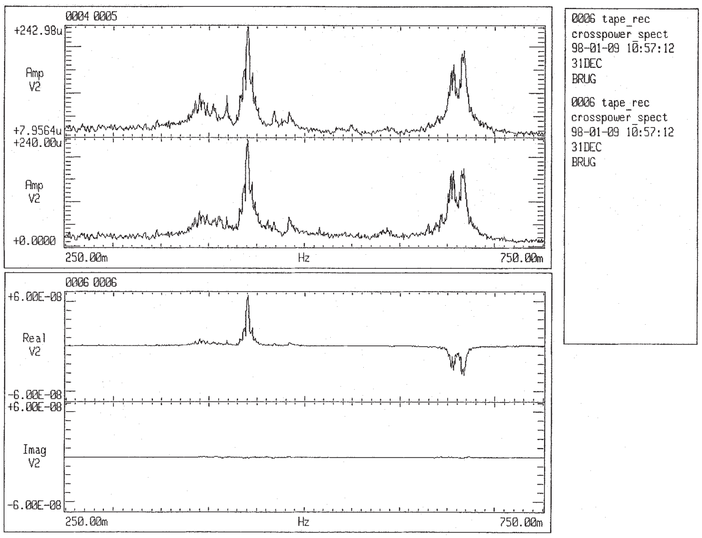
b) During 100 seconds (400 - 500 s)

Fig. 11 Example of time signals of accelerometers (east and west)

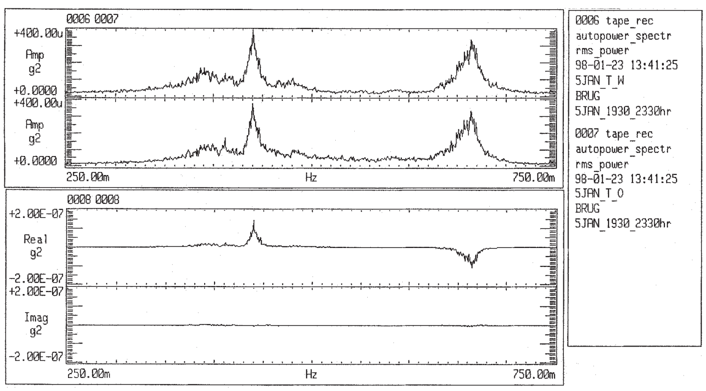
5.2 Additional data reduction

At the end of 1997, the monitoring system was extended with a digital/analog recorder and a dual-channel dynamic signal analyser because of the large variety in wind conditions expected at that time. Frequency spectra of the bridge deck accelerations were obtained after sampling the time signals up to 190 hours continuously.

An interesting result for the torsional behaviour of the bridge deck is presented in Fig. 12. A measurement of 72 hours continuously during which no wind was present showed two closely spaced frequency peaks of f = 0.65 Hz and f = 0.67 Hz. Both peaks are torsional modes as the real part of the cross-spectrum is negative at those frequencies. The cause of such a behaviour is not fully understood as computations only showed one torsional frequency of f = 0.69 Hz. A reason could be the characteristics of passing vehicles possibly working as a tuned damper. With wind and traffic only one torsional frequency is coming up (f = 0.66 Hz) just between the two peaks mentioned before.



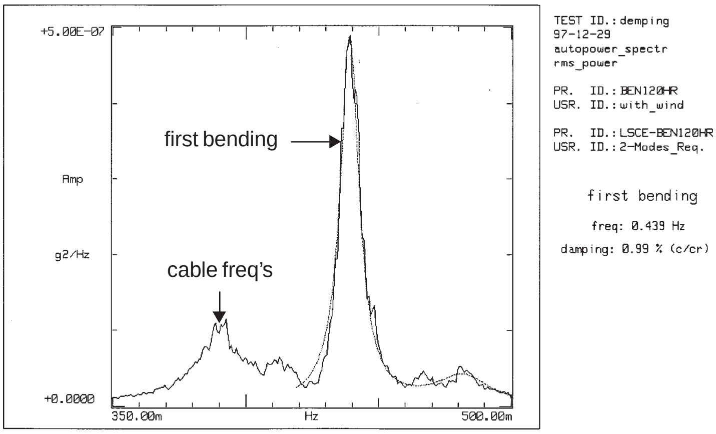
a) Excitation by traffic (no wind)



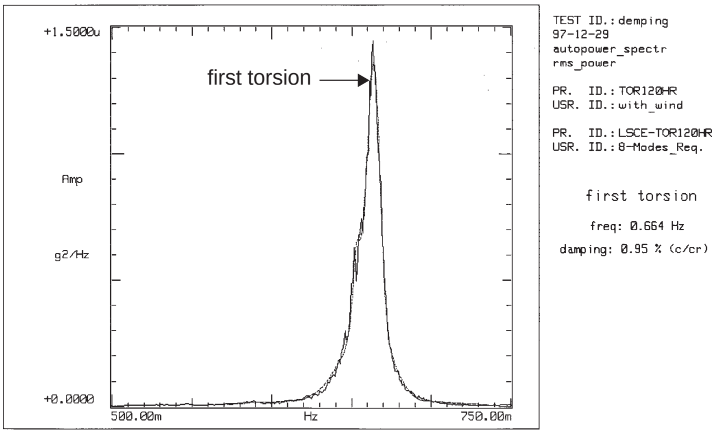
b) Excitation by traffic and wind ($\approx$ 12 ms^{-1})

Fig. 12 Cross spectra between signals of east/west accelerometer

Finally two examples of spectra (after a measurement of 120 hours) from which the damping factors are derived, are presented in Fig. 13. A curve-fit procedure was applied from which a damping factor of around 0.9 to 1.0% (c/cr) was obtained for both the lowest bending and torsion mode of the bridge. At that time excitation took place by traffic and wind of 10-12 ms^{-1} from the south, more or less perpendicular to the bridge.



a) First bending



b) First torsion

Fig. 13 Determination of damping (c/cr) by a curve-fit after 120 hrs sampling (excitation: traffic and wind)

6 Concluding remarks

From full-scale tests on the Erasmus bridge it is concluded that a bridge already in use should be closed for all traffic while one is carrying out damping measurements on the stay-cables. Although the length of this bridge is of a medium size, monitoring of such a structure remains important according to the results from a spectral analysis. Further the monitoring showed that this bridge deck is not vulnerable for vortex excitation. In general vibrational amplitudes are moderate, and lower than predicted from wind tunnel testing. Turbulence at the site might be a reason as it counteracts vortex excitation. Also at high wind speeds up to some 30 ms^{-1}, perpendicular to the bridge, no vibrations of any importance were reported up till now.

7 References

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