



Characterisation and assessment of damage in cable structures

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Abstract

The paper discusses the main characteristics of some relevant cable structures and demonstrates that, generally, damage occurring in cables is best assessed using local measurements, including the installed force and mechanical properties. A review of techniques to evaluate force and mechanical properties is then illustrated with examples from previous experience. A specific focus is given to emerging methods for characterising those properties based on wave decomposition and propagation characteristics to enhance their potential in identifying damage.

Keywords Cable structures · Damage detection · Vibration measurements · Wave propagation · Structural health monitoring

1 Introduction

Cable supported structures are among the ones with the most complex structural behaviour, given the variety of applications involving combinations of members with the stiffness of different orders of magnitude, usually with a significant number of cables, which can be tensioned at different stress levels, resulting, therefore, in various degrees of geometric nonlinearity. The flexibility of cable structures, combined with the typical low damping, is the origin of a significant proneness to vibrations induced by environmental and operational loads, which may lead to damage. This is particularly relevant in long-span bridges. Cable-stayed and suspension bridges have been widely constructed in Europe since the 1960s, integrating today's vast networks of transportation infrastructures. In many of those structures, an accelerated deterioration has been observed caused by higher than expected traffic demands and sometimes by the malfunctioning of innovative components, resulting in partial failures, localised damage, vibrations and excessive deformations, in many cases requiring earlier than predicted retrofitting [1–4].

Regarding overhead transmission lines (OHL), a high standardisation has been achieved in their design, which employs vast spans today. However, limitations in

monitoring the condition of active lines explain a still considerable lack of knowledge regarding their structural behaviour, and therefore, it is not uncommon for the occurrence of damage in conductors, insulators, as well as in towers, with the consequent disturbances in the distribution of electricity and significant economic repercussions.

Despite the stated complexity of behaviour and the recognised difficulty in assessing the condition of these structures, there is evidence of the insufficiency of the existing inspection and maintenance plans in guaranteeing the good performance of cable-supported structures. However, it should be mentioned that the continuous unattended remote monitoring of structures was not possible or cost-effective until the last 10–15 years [4]. Visual inspection played a significant role in managing long-span bridges, complemented with programmed actions or specific measurements [5]; the same occurs for transmission lines, where particular devices have been available for a few decades, enabling the temporary and limited monitoring of active lines. The evolution in computational power and data transmission led to a considerable increase in capacity. Thus, monitoring systems measuring variable quantities became a common practice and an integrated component of bridge construction and management [6–10]. However, even today, cables are scarcely instrumented, and monitoring their behaviour is often outside the scope of implemented monitoring systems. This is also a consequence of the difficulty in obtaining information on the condition of members that exist in a high number (stay cables and hangers) from a necessarily limited number of

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sensors, and it is also due to the actual difficulty in extracting information of relevance regarding the cable condition from existing assessment techniques. Nevertheless, some cases where cables have been instrumented should be mentioned [7, 10].

Focusing specifically on the particular characteristics of cables from cable-supported bridges and transmission lines and the characteristics of their damaged state, this paper explores the potential of existing instrumentation and monitoring techniques to assess their condition. This discussion is supported by particular case studies from the author's experience. It includes the identification of ranges of variations of dynamic properties in different structures due to damage and the presentation of techniques presently available or under development for the characterisation of mechanical properties of tensioned cables.

2 Characterisation of cable structures

To understand a cable structure's behaviour and assess the corresponding condition, it is necessary to characterise the importance of nonlinearities and the range of relevant dynamic effects. Understanding how a damaged condition translates into measurable information is also essential. This section addresses these aspects by identifying the range of pertinent cable variation and structure properties and their influence on the cable condition considering different applications.

Cable structures are generally formed by associating two different substructures designated as primary and secondary sub-systems. The primary subsystem (the deck and towers, in the case of a bridge) is the part of the structure characterised by higher stiffness and massiveness. At the same time, the secondary sub-system, represented by the supporting cables, has a much smaller stiffness and mass. The two sub-systems interact, leading to complex dynamic behaviour. Depending on the relative importance of the cable-structure sub-systems, different static and dynamic characteristics can be found, which are different for cable-stayed, suspension bridges and even transmission lines.

2.1 Cable-stayed bridges

In cable-stayed bridges, the span and the characteristics of deck and towers govern the structure dynamics. Although geometric nonlinearity is relevant from the construction point of view, it can generally be assumed that the structure behaves linearly about the dead load configuration, at least for the short and medium spans no greater than 500 m. Regarding the cables, these are tensioned to at least 25–30% of the ultimate resistance, which contributes to their classification as highly tensioned members and their frequent

treatment as tensioned strings. In reality, very long and short cables depart from this consideration, as discussed in the next section.

A second distinctive characteristic of cable-stayed bridges is the significant number of stay cables, in the case of the bridges of the so-called second generation, in opposition to the small number of cables of former cable-stayed bridges [11]. This leads to high redundancy, which is undoubtedly beneficial from the point of view of structural safety but can eventually bring unfavourable energy transfer between the structure and the cables in their response to dynamic loads, resulting in important cable vibrations. This characteristic is well illustrated in the case of the Gadiana cable-stayed bridge, located in Portugal, with a central span of 324 m (Fig. 1), consisting of a prestressed concrete deck suspended from 128 stay cables spaced at 9 m at the deck anchorages.

The fundamental vibration mode of this bridge has a natural frequency of 0.391 Hz, and the first cable modes have frequencies in the range of 0.781–2.969 Hz [12, 13]. From an ambient vibration test based on records of acceleration measured along the deck, singular values of the power spectra have been obtained, which are represented in Fig. 2, together with a colour map of the frequency content of the measured acceleration during the test. Superimposed on this colour map are marks with the first natural frequency of all stay cables. This superposition evidences the frequency ranges, where resonance effects could occur considering only first-order modes. Moreover, the colour map can be divided into two areas, one corresponding to the setups 1–12 and the other corresponding to 13–30. The numbering of the setups follows the temporal sequence of data acquisition during 1 day of measurements. For the second region, corresponding to the second part of the day, a stronger response is observed in the first two vibration modes, which coincides with the increase of the wind velocity. Besides that, some new vibration modes appear in the interval of 0.5–1 Hz, which are consistent with the frequency of some of the longest cables. Indeed, during that part of the day, significant vibrations were observed in many of the cables, particularly relevant to the four longest cables and some of the shortest cables. The fact that the colour map was obtained from deck measurements evidences this transfer of energy from the cables to the deck, which can also occur in the opposite sense, for example, due to the excitation of the deck by traffic loads. Although no relevant vibrations were generally observed in the deck, some of the shortest cables vibrated strongly when the traffic on the bridge was heavier, and at least one strand from a cable was found broken 25 years after the bridge's construction.

This specific accident was widely covered in the news in 2016 as the strand was observed hanging from the top anchorage. Still, no visual information exists in many other bridges, particularly when external sheathing is used. The

Fig. 1 International Guadiana Bridge (1991)

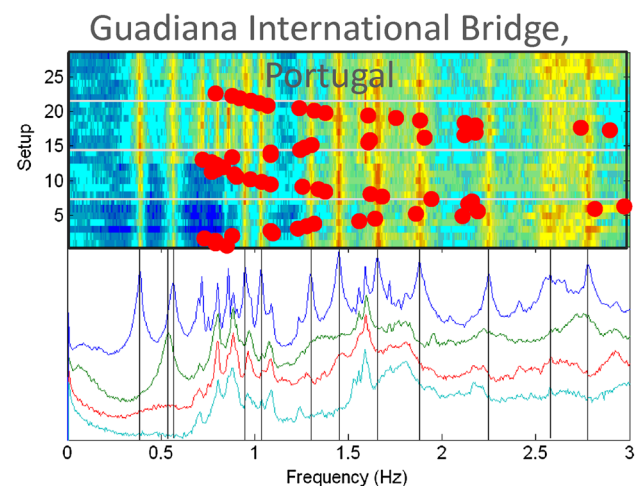
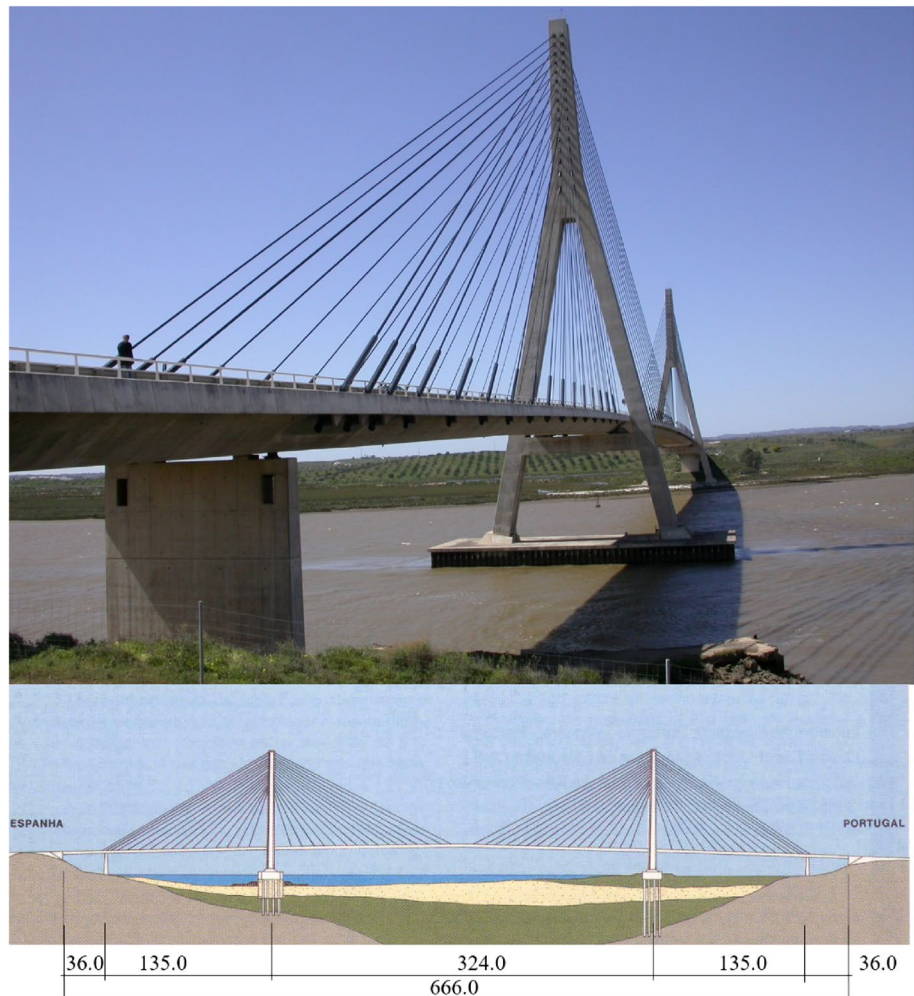


Fig. 2 Singular values of the deck acceleration records. Colour map of singular values and first natural frequencies of stay cables

question is whether the monitoring of the bridge or of that particular cable would enable the detection of such rupture.

To respond to this question, an exercise is made using the numerical model of the Guadiana Bridge, where the area of one of the 4th longest cables in the central span is reduced to 97% and 91% of the initial area, corresponding to the loss of one and four strands, respectively. The forces in the stay cables, the bridge’s natural frequencies, and the corresponding deviations to the undamaged condition are calculated and represented in Figs. 3 and 4, respectively.

The analysis of these figures shows that the loss of one strand in one cable leads to a variation of force in that cable of 3%, which can hardly be detected from vibration measurement, as this percentage is within the measurement error interval. As for the other cables, force variation is lower than 1%. Figure 3 also shows that a more significant section loss of about 10% (4 out of 37 strands) leads to a variation of cable force of 10%, which can be detected from vibration measurement of the force in that cable. It can also be concluded that no relevant force variation occurs for the other cables, including the adjacent cables.

Regarding the natural frequencies of the bridge, the analysis of Fig. 4 shows that even for a 10% loss of the cable

Fig. 3 Force distribution in stay cables of Guadiana Bridge for the undamaged and damaged condition of one stay cable. Relative difference to undamaged condition

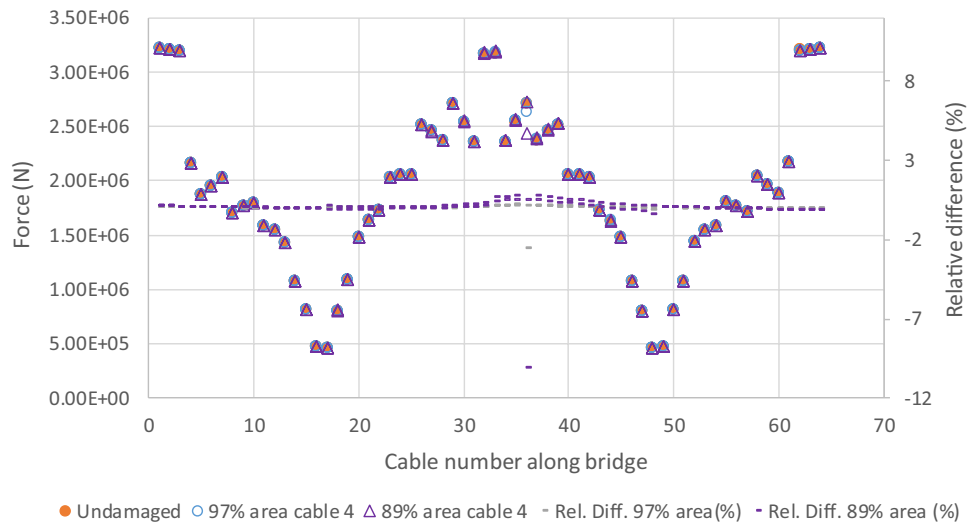
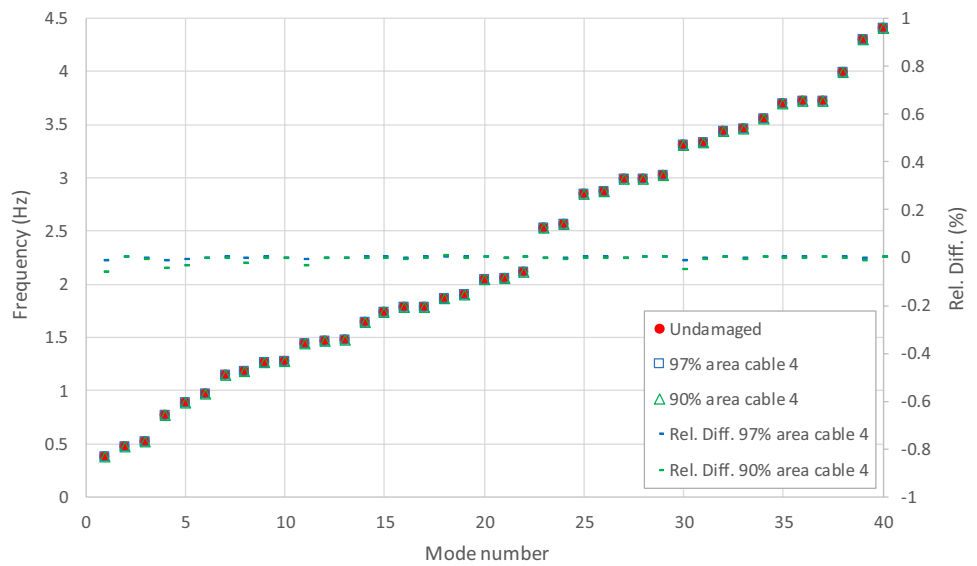


Fig. 4 Natural frequencies of Guadiana Bridge for the undamaged and damaged condition of one stay cable. Relative difference to undamaged condition



area, these change less than 0.2%. Therefore, using natural frequencies as global indicators of damage would not enable the identification of the cable loss of 10% of the area.

Knowing that vibration modes are more sensitive to damage than natural frequencies, an investigation of a damage index (DI) was developed for the Guadiana Bridge based on the modal flexibility proposed formerly by Pandey and Biswas [14] and investigated by other authors [15, 16]. Accordingly, the modal flexibility matrix in the undamaged $[F]$ and damaged $[F^d]$ conditions can be obtained from the mass normalised vibration mode matrices, $[\Phi]$ and $[\Phi^d]$, as

$$\begin{aligned}
 [F] &= [\Phi] \cdot [1/\omega^2] \cdot [\Phi]^T \\
 [F^d] &= [\Phi^d] \cdot [1/\omega_d^2] \cdot [\Phi^d]^T,
 \end{aligned}
 \tag{1}$$

where $[1/\omega^2]$ and $[1/\omega_d^2]$ are diagonal matrices ($n \times n$) containing the reciprocal of the squared circular frequencies of the first n vibration modes.

Representing by $[\Delta F]$ the matrix obtained from the difference between the damaged and undamaged modal flexibility matrices

$$[\Delta F] = [F^d] - [F],
 \tag{2}$$

the damage index (DI) is defined as a vector with a number of lines m equal to the number of degrees of freedom, whose components DI_i are determined from the ratio between the maximum of the absolute values of the modal flexibility variation components and the maximum of the absolute values of the modal flexibilities of the undamaged structure:

$$DI_i = \frac{\max | \Delta F_{ij} |_{j=1, \dots, m}}{\max | F_{ij} |_{j=1, \dots, m}} \quad (3)$$

Although the values of the DI do not reflect specific amounts of damage, their variation may indicate such occurrence. This can be observed in the representation of Fig. 5 of the DI obtained for the Guadiana Bridge model using the first 12 vibration modes (including bending and torsional modes) in the conditions of one and four strands loss in one cable, and assuming no variation of the mass matrix and no noise effects added. This figure shows that the DI increases at the deck nodes close to the anchorage of the damaged cable (indicated by a vertical line in Fig. 5), with a maximum at that section. It is also noted that the DI variation is more significant for the greater damage.

Although very promising, this representation could not be obtained experimentally, as only a few sections along

the deck usually are instrumented. Furthermore, it is known that the accuracy of experimentally identified mode shapes is lower than that of natural frequencies due to noise present in the measurements, which could compromise the capacity to identify small levels of damage. It is nevertheless relevant to show the result of the truncation to seven degrees of freedom shown in Fig. 6, still not introducing noise effects. Although the instrumented sections do not correspond to the anchorage section of the damaged cable, the DI variations would indicate such occurrence.

It is interesting to repeat this analysis considering a cable-stayed bridge of the so-called first-generation [11], where stay cables are employed in a small number. In Portugal, the cable-stayed bridge at Figueira da Foz is an example. The bridge, represented in Fig. 7, has a main span of 225 m suspended from a total of 12 cables continuous over saddles on the towers and anchored at a 30 m distance on the deck. A short, simply supported span of 30 m at the central part

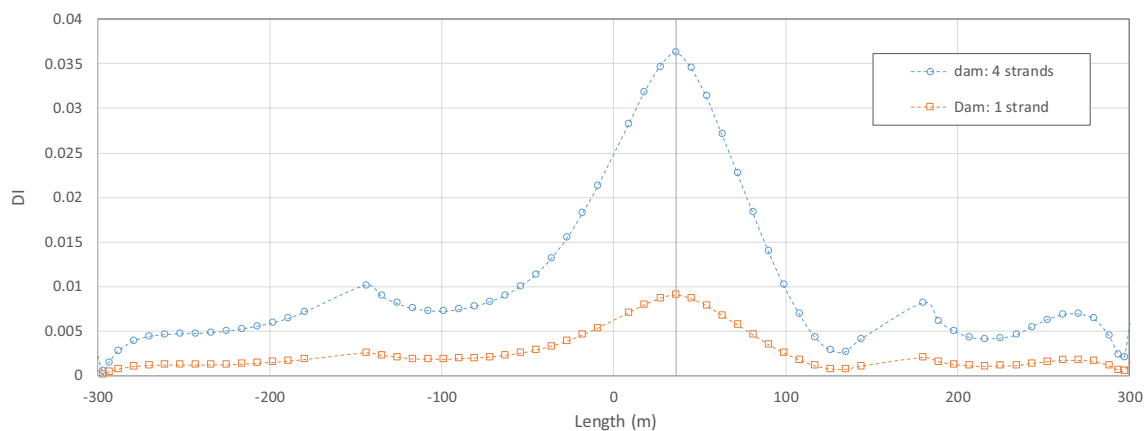


Fig. 5 DI calculated for the Guadiana Bridge, assuming the loss of one or four strands at one cable

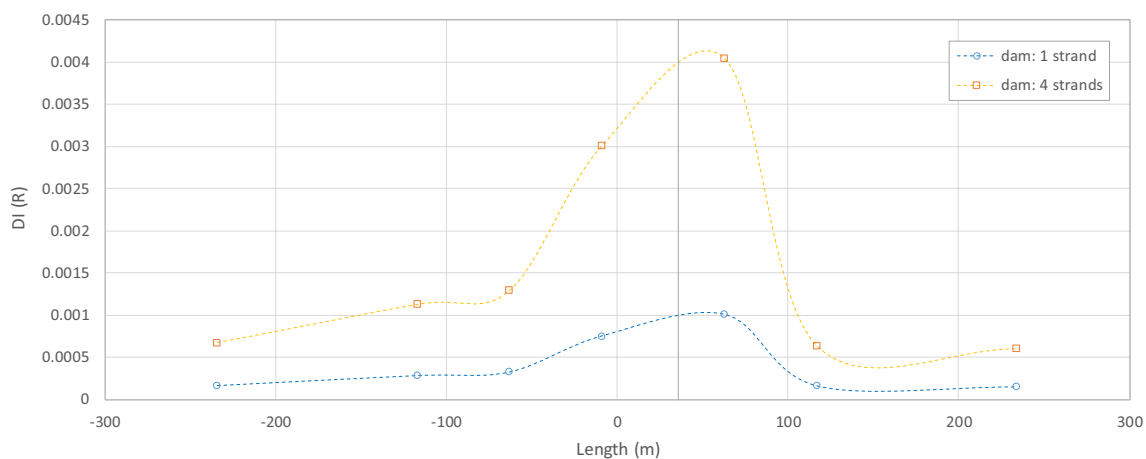
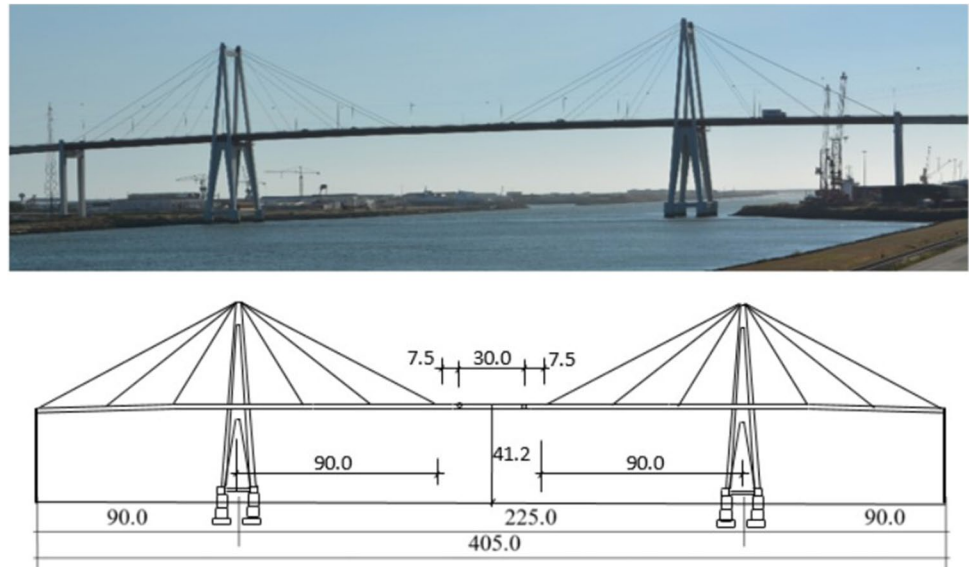


Fig. 6 DI calculated for the Guadiana Bridge assuming the loss of one or four strands at one cable and a small number of instrumented points

Fig. 7 Figueira da Foz Bridge (1984)



of the bridge renders this structure extremely flexible and results in a small degree of redundancy.

Compared to the Guadiana Bridge, the Figueira da Foz Bridge exhibits a similar interval of natural frequencies, with a first vertical vibration mode at 0.50 Hz. As only three different cable lengths are involved, a narrower interval of potential resonance in the first cable mode exists, corresponding to 1.2–2.2 Hz.

To understand how cable damage translates into the structural behaviour, the installed force is represented in Fig. 8 for the original condition without damage, and also considering 5% and 10% reduction of the section of a medium length cable (cable 5, see Fig. 7) of the central span. The relative deviations of cable forces are also represented. The resulting natural frequencies for the three conditions are illustrated

in Fig. 8, together with the variations from the undamaged condition.

Figure 8 shows that, compared to the Guadiana Bridge, the damage in the medium length stay cable leads to a lower variation of the force in this cable, reaching 5% for a 10% loss of the section, although affecting slightly more the adjacent cables. The local measurement of force would likely and hardly enable to detection of a cable loss of 10% area by comparison with results from a previous measurement. As for the natural frequencies, it can be observed in Fig. 9 that the variations observed concerning the undamaged condition are within 1% and, therefore, would not be identifiable from monitoring the bridge deck and towers. It remains a conclusion that the monitoring of bridge frequencies does not provide sufficient evidence of cable damage.

Fig. 8 Force distribution in the stay cables of Figueira da Foz Bridge for one stay cable's undamaged and damaged condition. Relative difference to undamaged condition

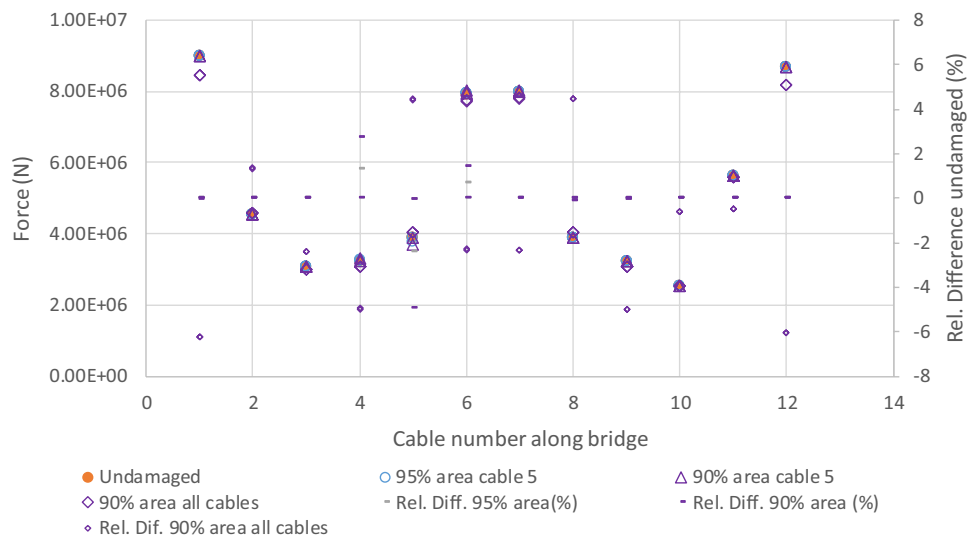
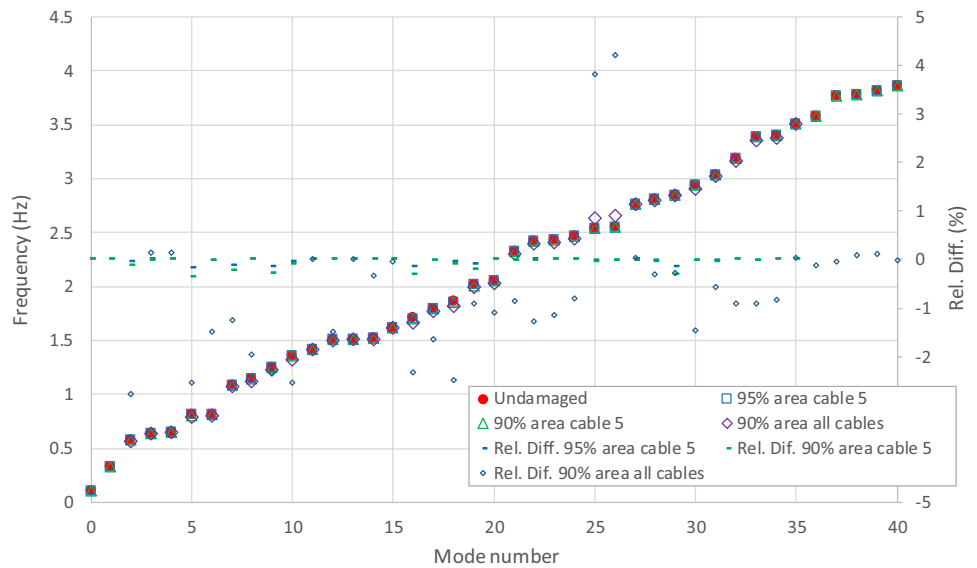


Fig. 9 Natural frequencies of Figueira da Foz Bridge for the undamaged and damaged condition of one stay cable. Relative difference to undamaged condition



As for the measurement of cable force using vibration measurement, it can be concluded that cable losses of at least 5% in the Guediana Bridge and 10% in the Figueira da Foz Bridges, respectively, would be needed to be detectable by comparison to a previous measurement. An even more dramatic reduction of 10% of the section of all cables has been simulated for the Figueira da Foz Bridge. The variation of force and frequency is represented in Figs. 8 and 9 and helps consolidate the conclusion that redistribution of force with a maximum of 6% for the longest cables would be detectable from force measurement at the cables, and again natural frequencies would suffer variations no greater than 4%, being of lower magnitude for deck modes, and, therefore, not allowing the detection of cable damage. On the contrary, a generalised reduction in the cable section of 10% would result in a maximum vertical displacement of 0.06 m at mid-span, which emphasises the importance of combining static

and dynamic measurements to identify damage in structures with a lower degree of redundancy.

Regarding the assessment of DI based on the modal flexibility, Fig. 10 shows that a similar pattern to the one described for the Guediana Bridge is obtained in the differentiation of the DI at the anchorage section of the damaged cable and the corresponding increase with the increase of damage in the cable. Again, noise, ambient effects and even other damage sources would contribute to masking this observation, as reported in [17].

2.2 Suspension bridges

Suspension bridges are usually characterised by a very low stiffness and long span. Another characteristic of these bridges is that the main cables work with relatively low stress. Consequently, they possess very low natural

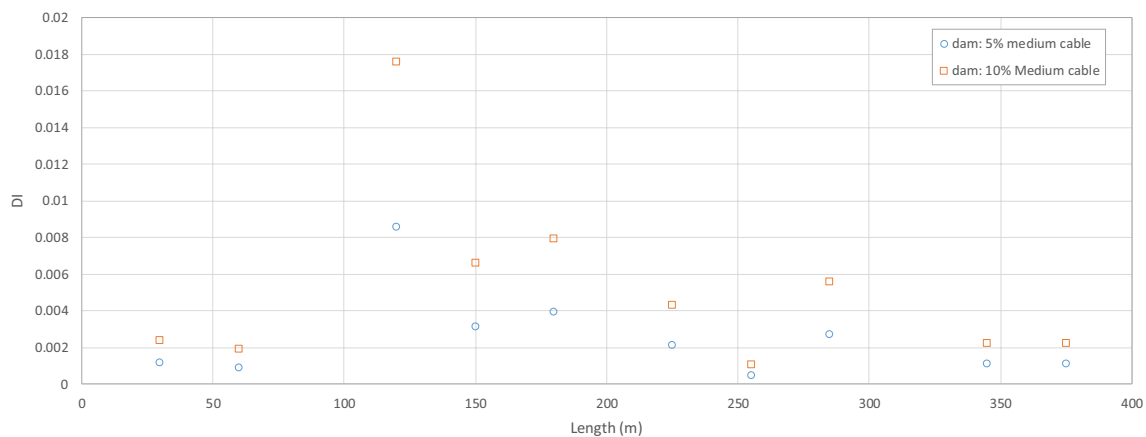


Fig. 10 DI calculated for the Figueira da Foz Bridge, assuming the loss of 5% and 10% area of one medium-length cable

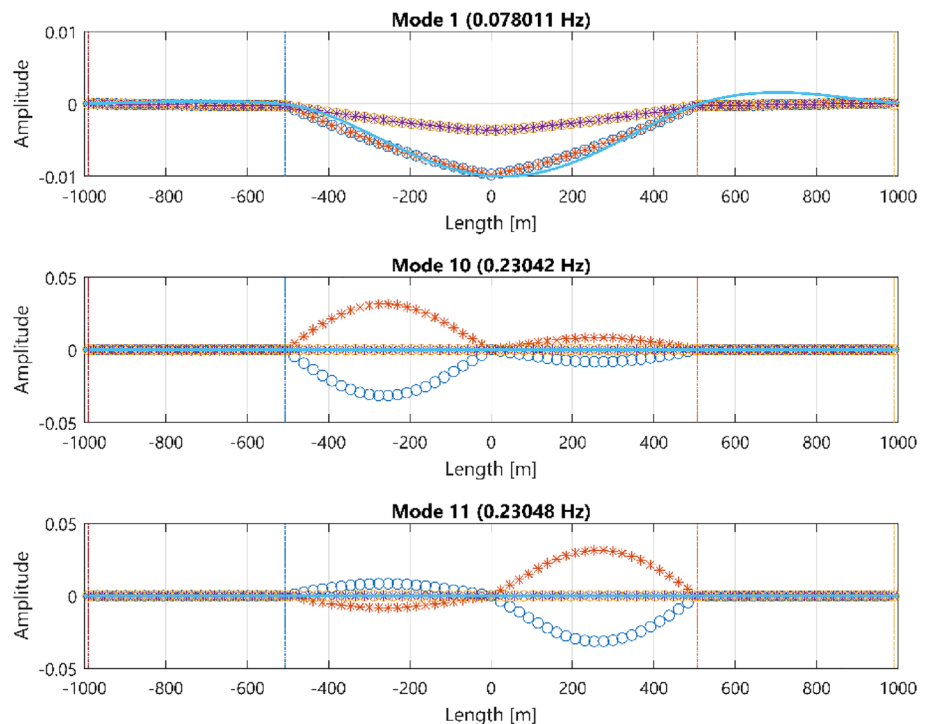
frequencies, in some cases very close to the limit sensitivity of the force balance type accelerometers typically used in identification tests [18]. Although geometric nonlinearity is of greater importance for these structures than for cable-stayed bridges, it is still a common approach to assume linearity of behaviour about the dead load configuration. In these bridges, hangers have varying lengths and approximately constant tension, therefore, varying natural frequencies. Given their function, the installed force is lower than in cables from cable-stayed bridges, so their diameter is also smaller. The range of hanger fundamental natural frequencies may be broad, considering the length varies from a few meters at the mid-span to a length similar to the tower height above the deck. However, resonances in fundamental modes of the bridge are less likely than in the case of cable-stayed bridges. As an example, reference is made to the 25th April suspension bridge, in Lisbon, with a main span of 1013 m, whose fundamental horizontal and vertical vibration modes have natural frequencies of 0.07 Hz and 0.116 Hz, respectively [19], and whose hanger fundamental frequencies vary from about 0.75 to 39 Hz. The last interval suggests the potential for vortex-shedding vibrations of some hangers, an aspect that has indeed been an issue in several suspension bridges [20, 21].

Regarding the main cables, the in-plane and out-of-plane vibrations are different given the restrictions introduced by the hangers. Therefore, vertical in-plane modes necessarily involve the combined participation of the main cables and the deck, leading to natural frequencies that are slightly

higher than those of the cable-only tensioned at the same force. Horizontal vibration modes, on the contrary, are a lot more complex, as they can include lateral deck displacement only, cable displacement only, with different combinations of the two edge cables configurations (in-phase, in opposition of phase, ...), and also combinations of deck and cables modal displacements. This characteristic results in a very complex identification problem, as many natural frequencies are present and very lowly spaced. For example, in the case of the 25th April Bridge, about 50 natural frequencies exist below 0.50 Hz.

Considering the discussed aspects, identifying the force in the main cables becomes a complex problem if using the vibration method applied directly to the main cables. The technique of extracting the force from in-plane cable measurements would not be applicable, given the deck's participation in the global in-plane response. On the contrary, local vibration modes identified from horizontal measurements may be used in force estimation, but this requires simultaneous deck measurements to clearly separate local and global cable vibration modes and several instrumentation sections along the cable. This can be exemplified by representing some calculated vibration modes of the 25th April Bridge [22]. The first horizontal mode shown in Fig. 11 involves simultaneous vibration of the deck and cables in the central span and cannot be used to identify the cable force. On the contrary, a half span cable mode is observable, which could be used to identify force provided that the modal configuration would be identified from various measurements along

Fig. 11 Some horizontal vibration modes of 25th April Bridge. Representation of deck (solid line) and main cable (symbols, upstream and downstream) modal displacements



the cable. However, care should be taken regarding the possible superposition of very closely spaced frequencies associated with different combinations of the local modes of the upstream and downstream cables, as also shown in Fig. 11.

A second alternative regarding the force identification in the main cable consists of summing up the force identified in the hangers, which could be again obtained from vibration measurement, and introduced as input in analytical formulae relating cable tension with chord length, distributed force and sag. In this case, some difficulty might be felt in estimating force in the shortest hangers.

As for the identification of damage, studies have been made on an analytical basis [23], evidencing that the reduction of the cross section of the main cable would be more easily observed in symmetric than in anti-symmetric modes. However, given the length of the cable and the localised nature of typical damage, it has been concluded again by the author that relatively small and localised damage, for example leading to a 10% reduction of the cross section, would not lead to measurable variations of natural frequency, even of the local cable modes.

Considering the direct assessment of the modal flexibility of local lateral vibration modes of the main cable, the DI obtained by simulation of the cross-sectional reduction along two different extensions of 4 m and 22 m is shown in Fig. 12. Not including noise and environmental effects, the identification of damage seems more challenging than in the previous examples, although possible for the higher extension of degradation of the cable. Nevertheless, this assessment implies the need to monitor the cable for many sections. In this respect, it is relevant to mention the perspective of a combination of the DI with distributed cable measurements enabled by fibre optic sensors and algorithms of shape detection [24].

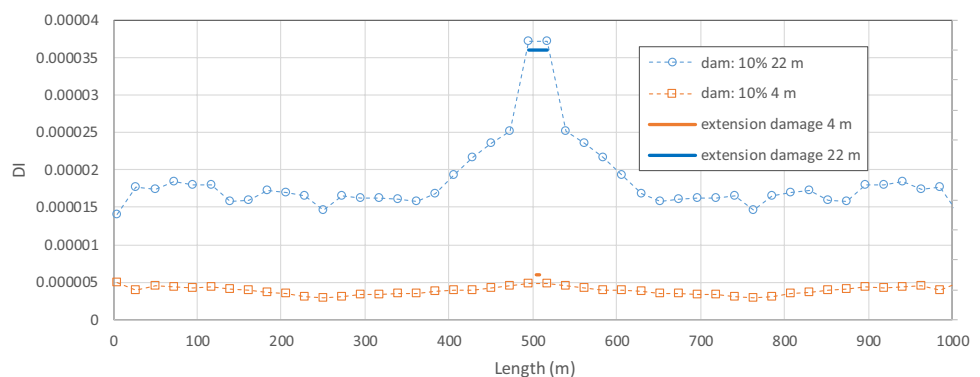
2.3 Overhead transmission lines

Overhead transmission lines are probably the most challenging structures to assess, considering the difficulty in measuring a line in tension and the complex dynamic

behaviour resulting from their geometric and material nonlinearity and the very lowly tensioned cable supported on flexible towers. This is further enhanced by the wide range of frequencies associated with the various associated vibrating phenomena.

In effect, cables from transmission lines have very low fundamental frequencies, significant sag, and high Irvine parameters [25], resulting in multiple cross-overs and the non-validity of the vibration chord formula to assess force. On the contrary, simplified formulae incorporating nonlinear geometric effects may be easily used to evaluate force and the corresponding variation, especially due to the thermal changes associated with electricity transport. It is stressed that measuring temperature and deformations at one or more sections along the cables using fibre optic sensors constitutes an essential tool for managing electric lines, as the current approach is to make an analytical-based estimation of temperature that may significantly differ from the site information. As for the dynamic behaviour, cables from transmission lines may exhibit very high amplitude vibrations at very low frequencies if affected by galloping due to ice accretion, for example, or by interference effects from adjacent cables, but also very high amplitude (although not visually observed) vibrations at very high frequencies, reaching 100 Hz, in this case, due to vortex-induced vibrations, here designated aeolian vibrations. While the first two vibration types are relatively rare and induce rather large stress cycles (which are nevertheless below the fatigue-relevant stresses, if appropriately designed), aeolian vibrations are extremely frequent. They are the origin of fatigue problems that result in broken wires close by the cable supports and damper clamps, broken insulators and, in the limit, the breakage of the cables. In effect, while the cables from cable-stayed bridges exhibit vortex-induced vibrations for typical mean wind velocities of 10–20 m/s (10-min average), the cables from transmission lines exhibit aeolian vibrations for wind velocities of 2–10 m/s, what is a result of their small diameter. Since these vibrations are associated with frequencies of 20–100 Hz, their experimental assessment requires high sampling frequencies.

Fig. 12 DI calculated for the 25th April Bridge cable, assuming the loss of 10% area at the mid-span along 5 m or 23 m



This aspect is illustrated with the measurements made on a de-activated span of a transmission line with a 595 m length [26]. The scheme of the SHM system installed in the line is represented in Fig. 13. It includes instrumentation of the top of one tower (P) with two accelerometers (longitudinal and transversal to the line) and with one sonic anemometer, and the instrumentation of one conductor with accelerometers in two sections (A and C), in-plane and out-of-plane, and with fibre optic sensors to measure strains and temperature in 5 sections (A to E).

With short interruptions due to maintenance and power shortage, the monitoring system was active for 2 years and collected data continuously between April 2017 and April 2019. In particular, from August 2018 to April 2019, Stockbridge dampers were installed in the line, so it was possible to assess and compare the response of the conductor with and without a vibration mitigation system.

The colour maps presented in Fig. 14 show the frequency content of the vertical acceleration of the conductor at section A (see Fig. 13) during the 2-year measurement period, where it can be observed that the frequency content ranges from 0 to 50 Hz, with the dominant contribution in the range 10–50 Hz. Figure 14 also presents a detailed frequency analysis of the same records in the range from 0 to 10.4 Hz, where faint, closely spaced bands evidence the numerous cable modes and the two more pronounced bands with the

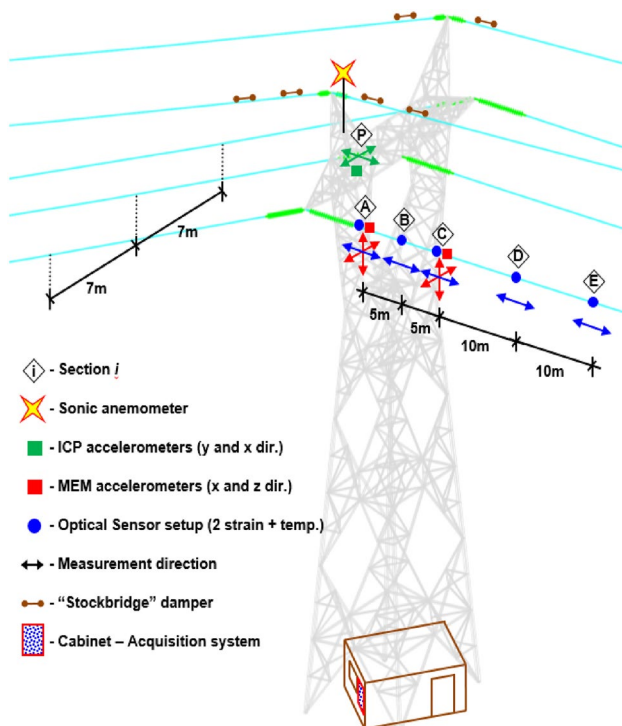


Fig. 13 Instrumentation of transmission line: scheme of the monitoring system

frequency of about to 1.7 Hz and 3.6 Hz evidence interaction effects with the tower frequencies.

The evidence of aeolian vibrations in this transmission line is shown in Fig. 15, where the 10-min RMS values of the acceleration recorded in the conductor at the sections A and C (see Fig. 13), in-plane and out-of-plane, are plotted against the mean 10-min wind velocity for the dominant wind directional sector with 30° angle, before and after installation of Stockbridge dampers. In these plots, the quadratic dependency of the conductor response with the wind velocity reflects the buffeting response, with a maximum rms value of about 1 m/s² for a maximum mean wind velocity of about 18 m/s. This response is not relevantly attenuated by the presence of the Stockbridge dampers. On the contrary, high rms amplitudes, reaching 3 m/s², can be observed for mean wind velocity in the range of 2–5 m/s, and these are attenuated more than three times by the presence of the Stockbridge dampers.

Since the wind velocity range of 2–5 m/s occurs every day, aeolian vibrations are very frequently present in transmission lines. In the current line, it was concluded that they would appear for about 30% of the daytime. Therefore, it is necessary to assess the fatigue resistance of the cables and ensure that dampers are added to guarantee the design service life. CIGRÉ has developed a specific procedure for this purpose [27], which estimates the lifetime of a line from the measurement of vibration cycles and the construction of stress variation histograms. The characterisation of the line condition is then based on the instrumentation using an autonomous device mounted on the cable close by the fixation, which comprehends an LVDT transducer, a battery and a memory disk. Due to battery and memory limitations, this device is usually programmed to record the relative displacement of the cable during 10 s on every period of 15 min for 3 months, and the cable lifetime is estimated from the extrapolation of the data collected. The application to the transmission line conductor above described using both accelerations and strains [28] has shown that this short duration measurement may lead to unconservative estimates of the cable lifetime.

3 Techniques for identification of damage in cables

From the previous section, it could be concluded that small amounts of damage in cables can very hardly be identified from the variation of global parameters of the structure as natural frequencies. Although the identification of vibration modes may be used in conjunction with numerical models to detect damage, more local methods, for example, enabling the assessment of the installed force, appear more effective.

Fig. 14 Colour map of in-plane acceleration spectra at section A for 1-h records from April 2017 to April 2019. Representation in the range 0–125 Hz (top) and 0–10.4 Hz (bottom)

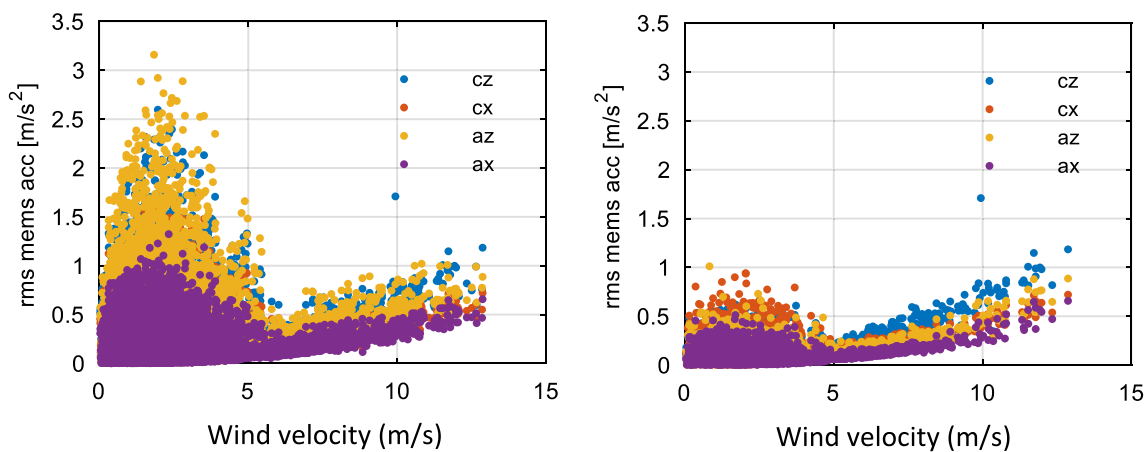
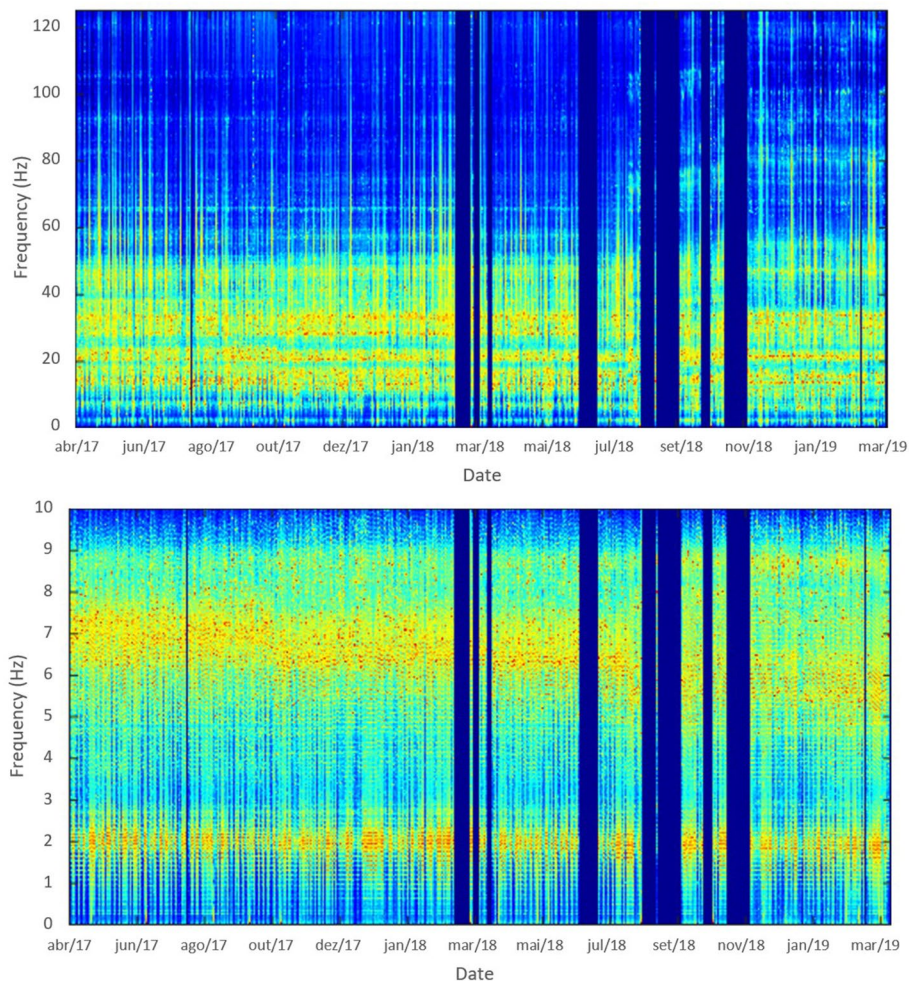


Fig. 15 RMS values of acceleration of conductor vs mean wind velocity, before (left) and after (right) installation of Stockbridge dampers

This section discusses different techniques to assess force from vibration measurements and introduces a new method to identify damage incorporating the bending stiffness and shear coefficient characterisation.

3.1 Vibrating chord theory and derivate methods

In the original vibrating chord theory, the natural frequency of a cable is related to the installed tension by an explicit

formula depending on the distributed mass and the free length of the cable. The validity of the application of this formula has been established for relatively long, highly tensioned cables as long as the corresponding first five natural frequencies vary linearly with their order [29]. The simplicity of the method has led to a wide application in construction and inspection surveys [12, 30, 31]. With some improvements, this method has also been applied to more complex cases, involving, for example, very long sagged cables, short, stiff cables, low tensioned cables and cables anchored on flexible supports. For these cases, specific formulae based on simplified analytical solutions have been used [32, 33], as well as numerical formulations allowing the identification of various cable parameters from the measurement of sets of natural frequencies and particular assumptions respecting the boundary conditions [34–37].

Nevertheless, it is noted that, while these formulations adequately represent the dynamic characteristics of cables for ranges of dispersive behaviour, there remains uncertainty for the definition of some parameters. These are the exact degree of constraint at the anchorages, the bending stiffness (that depends on the cable deformation and tension), and eventually, the exact cable length. Therefore, it is highly relevant to define all estimates of cable force by confidence intervals, considering the particular uncertainties of the problem under analysis and the intervals of variation of the various quantities involved in the cable estimation.

At the same time, and given that experimental techniques have immensely improved in recent years, it is possible to obtain very accurate estimates of cable frequencies for many vibration modes other than the first two. This allows the simultaneous identification of force and some governing

parameters using optimisation criteria and curve-fitting techniques [30, 36, 37].

From another perspective, also considering the progress in the numerical modelling of complex structures, finite element models have been combined with experimental testing and identification techniques to address simultaneously the problems of non-ideal support and unknown bending stiffness [38]. This was the case with the force assessment at different cables of the London 2012 Olympic stadium roof, which involved cables with spans of 5–35 m in length, diameters of 25 m to 80 mm and varying stress levels [39]. Figure 16 shows schematically the disposition of these cables. The relative length of the sockets of the radial cables was one of the aspects that deserved a specific consideration in demanding an adequate model of the end constraints. Figure 17 shows different mechanical conditions simulated with numerical models that were defined and adjusted based on the measured natural frequencies, and the validation of the final adjusted model was made from the identification of vibration modes, as shown in Fig. 18.

In a similar exercise, different propositions have been made by other authors to use identified mode shapes and the position of their nodes to determine an equivalent free length of the cable [40, 41] and improve the characterisation of end conditions. In this particular aspect and when short cables are the target of the analysis, reference is made to the interest in exploring a varying length according to the vibration mode order, considering that the different participation of different order modes in the global response may be associated with different end constraints.

This aspect can be illustrated by the cables of a cable-stayed bridge on Madeira Island [38]. With a central span

Fig. 16 Partial representation of assessed cables at the 2012 London Olympic stadium roof [39]

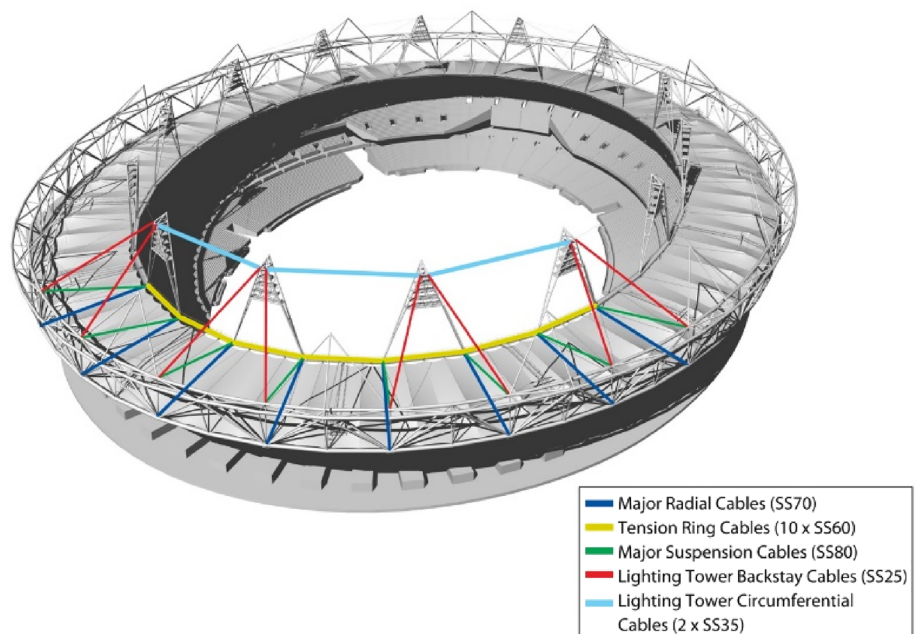
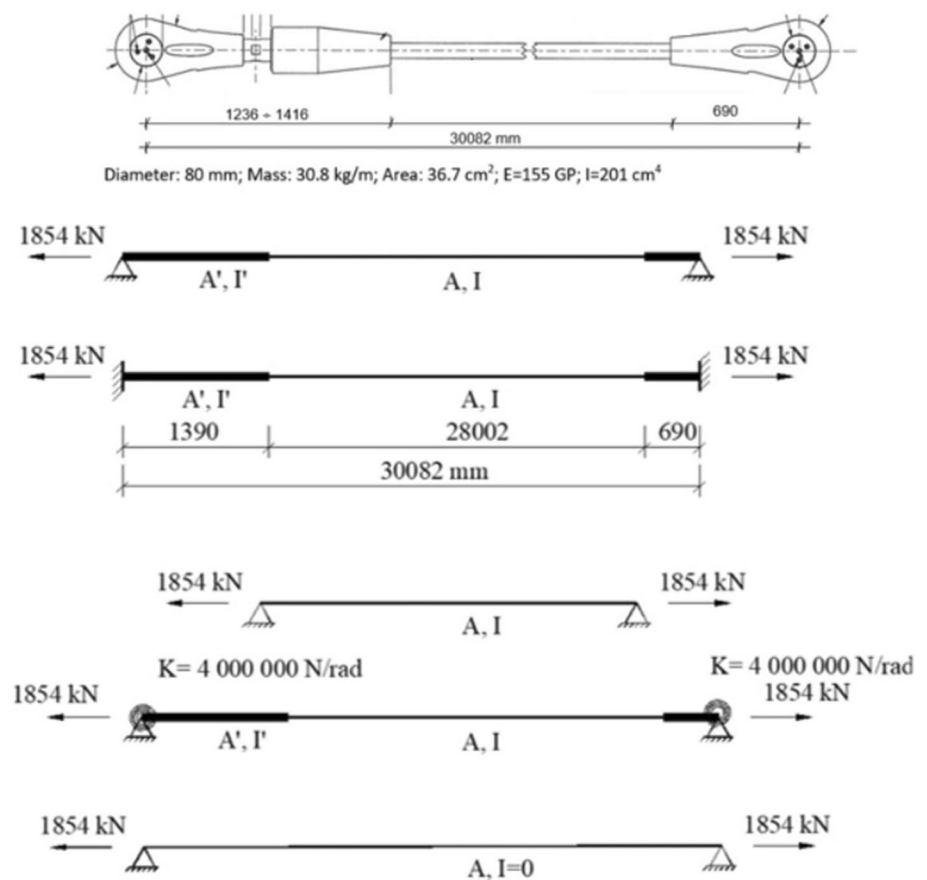


Fig. 17 Example of force assessment in cable with complex supports. Different models for numerical calculation



of 92 m and two side spans of 46 m (Fig. 19), the viaduct was the object of an inspection and force was assessed in all cables, which have lengths of 18–49 m.

For some of the cables, it was observed that the numerical fitting of a finite element model using the measured natural frequencies implied that different lengths needed to be used. Iterative variations of the cable length, even including bending stiffness effects, have then been simulated to better approximate the measured frequencies. A pattern was observed consisting of a first measured frequency systematically higher than the numerical and higher order frequencies systematically lower than corresponding numerical values for the matched force for each length tested. This can be observed for an intermediate cable with calculated and identified natural frequencies systematised in Table 1. It was concluded that neoprene guides inside the pipes could constrain vibration and act as simple supports. The modelling of the shorter span, with results designated by (3) in Table 1, indicates that it is only valid for the first frequency. This is probably due to the vibration amplitude in the first mode being greater than that of higher modes. The contact with the neoprene guides would lead to a shorter cable span, while for higher modes smaller vibration amplitude would not imply such span reduction.

3.2 Vibration propagation methods

In addition to the many studies conducted for the assessment of cable force based on modal approaches, a new line of research is gaining importance in recent years, focusing on the decomposition of the vibration modes in their wave components and on the consideration of their independency on the end constraints to improve the estimation of force [42].

More recently, the variation of these wave components as a consequence of damage has been explored from the perspective of the detection and identification of cable damage [43]. The idea is that if a localised excitation is applied at a given cable section, longitudinal and transverse waves will be generated, which will propagate along the cable. Two distinct methodologies are presented in the literature to identify these waves: the one described in [42] demands the instrumentation of at least five sections (to fully characterise the complex boundary conditions of the considered segment) [43]. The second methodology [44] only requires two sensors and, using a wavelet transform, extracts these properties from the direct non-reflected propagation waves from the impact location, eliminating the effect of non-ideal conditions at the interfaces. Given the practical applicability, this methodology has been used in one-dimensional media, such as beams or rails [43].

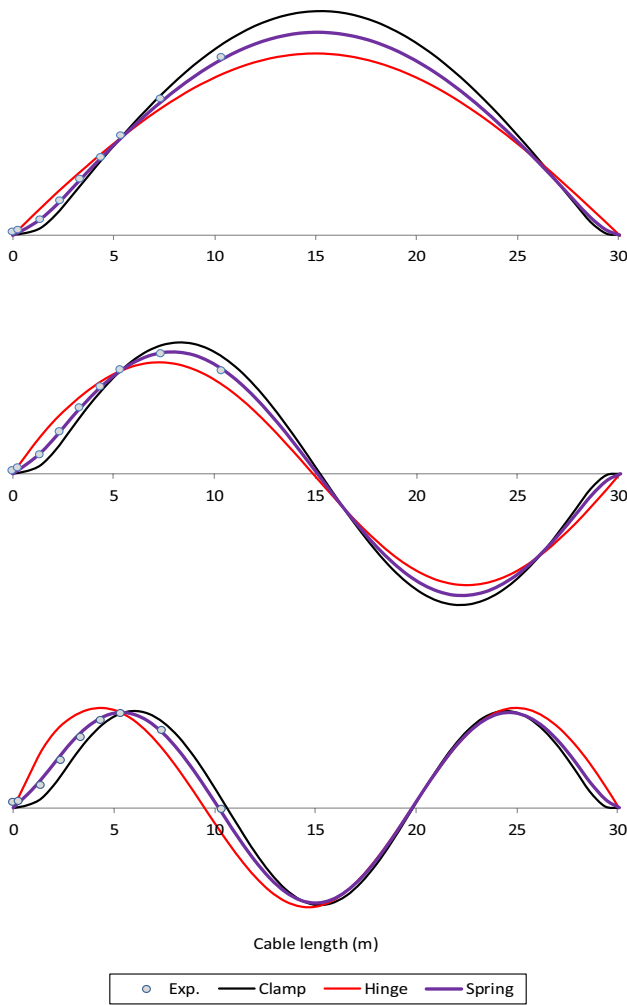


Fig. 18 Experimental validation of numerical model from mode shape identification [39]



Fig. 19 Viaduct of Comboio, Madeira Island

Table 1 Study of length and EI influence on cable frequency

Mode no	Natural frequency (Hz)			
	EI=0, L (1)	EI=31I _{st} , L (2)	Exp	EI=31I _{st} , L' (3)
1	4.229	4.2478	4.4434	4.4159
2	8.4577	8.4948	8.4277	8.8311
3	12.689	12.743	12.7539	13.248
4	16.924	16.993	16.8262	17.667

$L=33.49$ m; $m=50.85$ kg/m; no. strands: 37; Area: 51.8cm^2 ; $E=196.5\text{GPa}$; $T=4080$ kN; $\epsilon=4.0e-3$; $L'=32.82$ m

The values in bold correspond to the best numerical fit to experimental data

The properties of reflected waves due to discontinuities associated with cracks and wire breaks have also been used in the context of guided ultrasonic wave propagations methods employed to identify damage in cables from suspension bridges [45] and transmission lines [46].

3.3 New approach for damage assessment in cables

Focusing on the propagation of transverse waves and the relation between the velocity of propagation and the characteristics of the propagating means, such as force, bending stiffness and shear coefficients, a new approach to detect damage is being developed, which takes advantage of modern instrumentation and of the capacity to sample at very high frequencies [47, 48].

The developed method directly measures the propagation velocity of transverse waves generated from an impulse between two relatively close sections of a cable. The technique can be illustrated with the scheme of Fig. 20 and was first applied to a cable from a small cable-stayed bridge of 9.1 m, formed from a solid bar with a diameter of 28.6 mm made of stainless steel [47].

The idea is that, once an impact is applied at a section of the cable, a transverse signal propagates longitudinally and successively reflects at the cable ends. The signal has a wide frequency content. If the cable bending stiffness were null, the velocity of propagation of the different harmonics composing the transverse response would occur at a constant velocity $c_t = \sqrt{T/\mu}$, where T is cable tension and μ is the mass per unit length. Non-null bending stiffness EI results in a so-called dispersive behaviour, where different harmonics propagate with varying velocity. Using an Euler–Bernoulli approach, a group velocity $c'_{gr}(\omega)$ is defined as a function of c_t and a parameter $c_f = \sqrt{EI/\mu}$ as [47]

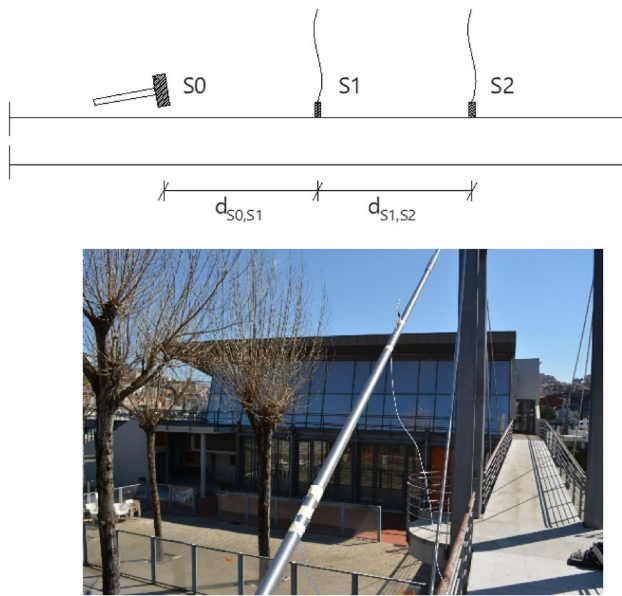


Fig. 20 New technique to identify cable properties by measuring the propagation velocity of transverse waves. Application to a cable from a small cable-stayed bridge

$$c'_{gt}(\omega) = \sqrt{\frac{2c_t^4 + 8c_f^2\omega^2}{c_t^2 + \sqrt{c_t^4 + 4c_f^2\omega^2}}}, \tag{4}$$

where ω represents the circular frequency.

Using the Timoshenko model, a more complex expression for the velocity of propagation is obtained [47] which asymptote is

$$\sqrt{c_t^2 + GAK/\mu}, \tag{5}$$

depending on the taut string velocity of propagation c_t , on the distortion modulus G , the cross-sectional area A , the shear coefficient of the cross section K , and μ .

Then, once the velocity of transverse wave propagation is determined as a function of ω , the dispersive relation (4) can be constructed by fitting the experimental curves, enabling the identification of the cable parameters T and EI in case the Euler–Bernoulli model applies. The asymptote (5) of the dispersion relation enables the identification of K . In all cases, no assumptions regarding the cable boundary conditions are made.

The application of a time–frequency analysis based on the wavelet transforms to the records collected in the sections S1 and S2 enables to identify the time arrival of each frequency wave, as illustrated by the scalogram in Fig. 21. It is noted that only the first passage of the wave between the two instrumented sections is of interest, to avoid multiple reflections and perturbation of the collected signals.

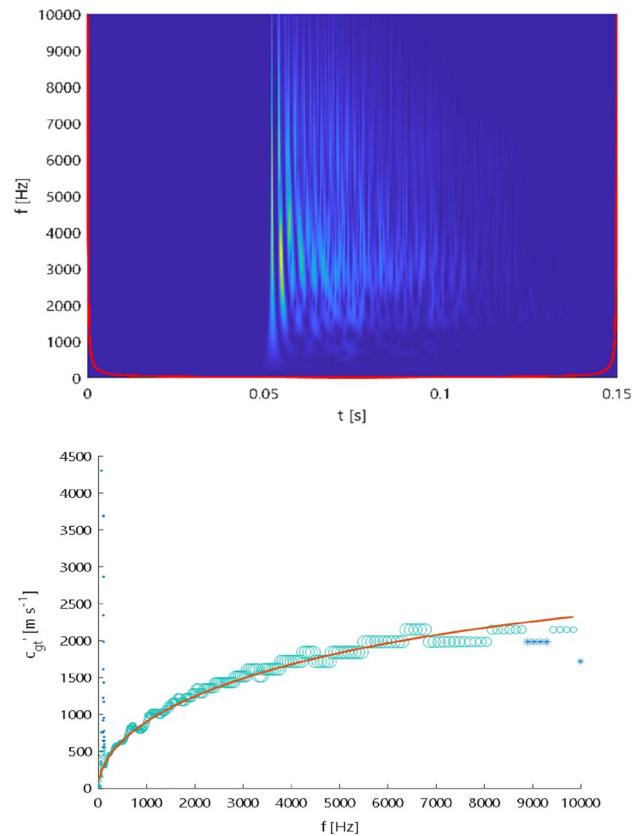


Fig. 21 Assessment of dispersion relations in a cable: scalogram of the response at S1 (top); experimental and fitted dispersion relation

The experimentally built dispersion relation for the exemplified cable is shown in Fig. 21 together with the fitted curve.

As can be observed in Fig. 21, although the relevant natural frequencies of the cable are below 10 Hz, the velocity of propagation can reach about 2000 m/s at a frequency of 10 kHz. To accurately characterise this velocity, it is necessary to sample at very high frequencies, at least 2 kHz, to define the dispersive relation in a range of 800 Hz.

The studies developed in [47] and the application developed in [48] evidence that the fitted dispersive relations have different sensitivities to the various identified parameters. The installed force, for example, is only sensitive in the low-frequency range, where the frequency resolution is relatively poor due to the short duration of the records associated with an impact excitation. This compromises the accuracy of the identification of force and demands for combined identification using the identified group velocity and the natural frequencies used in the vibrating-chord-derived formula. On the contrary, the flexural stiffness and the shear rigidity are very sensitive in the high-frequency range. Therefore, it is expected that high-quality estimates of these parameters can be obtained. Moreover, this

suggests that these parameters constitute damage-sensitive features. Going back to the initial sections of this paper, it is noted that the breaking of a wire in a multi-wire section does not significantly alter the installed axial force, as there is redistribution of stresses to adjacent wires but changes the local stiffness properties of the cable section are more evident.

To summarise, assuming a suitable analytical or numerical model for the cable, the experimentally calculated velocities of wave propagation can be used in the inverse problem of estimating cable parameters, such as the axial force and the bending or shear stiffness. The latter possibility is of paramount importance, since local evaluations of these properties are not possible using traditional techniques. Still, research is needed to validate the results achieved with site experiments.

4 Conclusions

The paper focuses on the characteristics of cables from different cable structures, discussing specific aspects of their dynamic behaviour and introducing some sensitivity studies to demonstrate that a small percentage of cable damage is generally not translated into parameters that are readily observable in global characteristics of the structure, such as natural frequencies and mode shapes. The use of techniques based on the modal flexibility may contribute to identifying the presence of damage in cables, although requiring robust numerical models and monitoring several sections along the bridge or cables. In general, to detect slight variations of force, local measurements of force need to be made. A review is then presented of techniques available to identify cable force, and the limitations of the most common techniques regarding uncertainty in the definition of boundary conditions are discussed. Reference is then made to a new generation of methods based on the decomposition and propagation of waves, which enable the identification of mechanical characteristics, such as flexural and shear stiffness, quantities that may constitute damage-sensitive features. In particular, a new method under development by the author and her research team is briefly described. Although validation of this method is still needed, it opens new perspectives in assessing cable damage.

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