Structural Engineering Documents

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Elsa de Sá Caetano

Cable Vibrations in Cable-Stayed Bridges

International Association for Bridge and Structural Engineering IABSE
Association Internationale des Ponts et Charpentes AIPC
Internationale Vereinigung für Brückenbau und Hochbau IVBH

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ISBN 978-3-85748-115-4

Publisher:
IABSE-AIPC-IVBH
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CH-8093 Zürich, Switzerland

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Preface

The numerous episodes of cable vibration exhibited by different types of cable-stayed bridges all over the world have posed a strong demand to understand and control the involved phenomena. Intensive studies have been conducted during the last two decades, serving directly the designers' needs and at the same time framed by academic research projects funded by national science institutions and authorities interested in producing guidelines and codes for the safe design and assessment of their structures.

The present book provides a comprehensive survey on the existing and published knowledge related to cable vibrations, focusing on the governing phenomena, on the methodologies to assess their effects and on the design of control devices.

The evident need to convert research results into practice and to learn from practical experience motivated the invitation of experienced designers and consultants to present a contribution from particular cases in which they participated. I would like to deeply acknowledge the extremely interesting and enriching reports provided by Drs. Yves Bournand (VSL International), Chris Geurts (TNO), Carl Hansvold (Johs. Holt), Allan Larsen (Cowi) and Randall Poston (WDP & Associates).

I would also like to express my gratitude to the support and material provided by many researchers from all over the world, expecting that I can honour their work. In particular, I would like to refer to Professor Yozo Fujino, Dr. Hoang Nam, Professor Masaru Matsumoto and Professor Hiroshi Tanaka who carefully reviewed the manuscript and supplied important research reports. I also thank Professors Hiroki Yamaguchi and Steen Krenk, Drs. Shaohong Cheng, Olivier Flamand, Christian Cremona, and António Pinto da Costa and Mr. Craig Winters and Philippe Duflot for publications and images provided.

Finally, I want to thank IABSE for providing the opportunity and the support to publish this book, and Dr. Geoff Taplin, Chair of the Editorial Board, for his personal commitment to the success of this publication.

Porto, August 2007

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Table of Contents

	Page
1 General	1
2 Organisation of the Text	3
3 Brief History of Cable-Stayed Bridge Construction	5
4 Vibration Phenomena Directly Induced by Wind and Rain	13
4.1 Wind Loads on Stay Cables	13
4.1.1 Fixed cylinder immersed in smooth flow	14
4.1.2 Fixed cylinder immersed in turbulent flow	17
4.1.3 Moving cylinder immersed in turbulent flow	19
4.1.4 Linearised equations of motion	21
4.2 Buffeting	22
4.3 Vortex-shedding	23
4.3.1 Fundamental characteristics	23
4.3.2 Amplitude of oscillations	25
4.4 Galloping	29
4.4.1 Fundamentals	29
4.4.2 Prediction and control measures	32
4.5 Wake Effects	33
4.5.1 Resonant buffeting	34
4.5.2 Vortex resonance	34
4.5.3 Interference effects	35
4.5.3.1 Vortex resonance effects	36
4.5.3.2 Galloping	36
4.5.3.3 Interference galloping of free cables	37
4.5.3.4 Interference effects in stranded cables	38
4.6 Rain-wind Induced Vibrations	39
4.6.1 Identification of the phenomenon	39
4.6.2 Experimental observations	40
4.6.3 Analytical and design models	44
4.6.3.1 Analytical model from Yamaguchi	44
4.6.3.2 Analytical model of Peil and Nahrath	48
4.6.3.3 Design model of Geurts and van Staalduinen	49
4.6.4 Mechanisms of instability	50
4.6.4.1 Conventional Karman vortex excitation	50
4.6.4.2 Galloping instability	51
4.6.4.3 High speed vortex excitation	51
4.6.5 Other variables to rain-wind induced oscillations	52
4.6.6 Practical cases of occurrence of rain-wind vibration and prevention measures	52
4.7 Drag Crisis	54
5 Indirect Excitation	55
5.1 General	55
5.2 External Excitation	55
5.2.1 Linear model	56
5.2.2 Linearity of response of current stays	58
5.2.3 Non-linear model	59

5.3 Parametric Excitation 63
5.3.1 General equations 63
5.3.2 Application to a stay cable 66
5.3.3 Practical occurrence of external/parametric excitation 67
5.4 Cable-structure Interaction 69

6 Control of Vibrations in Cable-Stayed Bridges 71
6.1 General 71
6.2 Vibration Control Systems 71
6.2.1 Aerodynamic control of vibrations 71
6.2.2 Structural control of vibrations 73
6.2.3 Mechanical control of vibrations 74
6.2.4 Active control—systems 77
6.2.4.1 Active aerodynamic appendages 77
6.2.4.2 Active mass dampers 77
6.2.4.3 Active tendon control 78
6.3 Design of an Optimal Passive Damper 78
6.3.1 General 78
6.3.2 State-of-the-art of research 79
6.3.3 Problem formulation 80
6.3.3.1 Taut cable 80
6.3.3.2 Shallow cable 85
6.3.3.3 Bending stiffness effects 90
6.3.3.4 Flexibility of the dampers or of the supports 94
6.3.3.5 Damper non-linearity 97
6.3.3.6 Combined effects of sag, bending stiffness and flexibility of damper supports 99
6.3.3.7 Combined effect of two dampers 100
6.3.4 Practical applications 103
6.3.4.1 Evaluation of maximum attainable damping ratio for a particular damper location 103
6.3.4.2 Specification of damper size to fulfil minimum damping requirements 107

7 Case Reports 109
7.1 Skarnsundet Bridge (Norway) 110
7.2 Puente Real Bridge (Badajoz) 112
7.3 Veterans Memorial and Fred Harman Bridge (Texas) 114
7.4 Erasmus Bridge (Rotterdam) 119
7.5 Kap Shui Mun Bridge (Hong Kong) 124
7.6 Oresundsbron (Denmark–Sweden) 128
7.7 Uddevallabron (Sweden) 131
7.8 Friction Damper Test 133

8 References 137

Appendix A 147
A.1 Objectives 147
A.2 Static Behaviour 147
A.2.1 General assumption: Elastic catenary 148

A.2.2 Elastic parabola 153
A.2.3 Numerical modelling 154
A.2.3.1 Linear model: Truss element 154
A.2.3.2 Linear model refinement: Equivalent modulus of elasticity 155
A.2.3.3 Linear model refinement: Multi-link approach 156
A.2.3.4 Non-linear model: Cable element 157
A.2.3.5 Comparative analysis for global study of a cable-stayed bridge 157

Appendix B 163

B.1 Objectives 163
B.2 Linear Theory of vibrations of horizontal—cables 163
B.2.1 Basic assumptions and equilibrium equations 163
B.2.2 Natural frequencies and modal shapes 164
B.2.2.1 Out-of-plane motion 164
B.2.2.2 In-plane motion 164
B.3 Linear Theory of Vibrations of Inclined Cables 168
B.3.1 Simplified approach 168
B.3.2 Asymptotic approach 169
B.4 Bending Stiffness Effects 173
B.4.1 Taut string approach 173
B.4.2 Simplified sagged cable approach 174

Appendix C 177

C.1 General 177
C.2 Methods of Force Assessment 177
C.2.1 Direct measurement of stress in tensioning jacks 177
C.2.2 Application of ring load cells or of strain gauges in strands 177
C.2.3 Measurement of cable elongation 178
C.2.4 Topographic survey 179
C.2.5 Vibration method 179
C.3 Force and Damping Assessment Based on the Vibration Method 179
C.3.1 Vibrating chord theory 179
C.3.2 Bending and sag effects 180
C.3.3 Measurement of cable frequencies 181
C.3.4 Estimation of cable damping 182
C.3.5 Practical application 184

1. General

Cable-stayed bridges were established as a new category among the classical types of bridges in the second half of the twentieth century. Their relative stiffness when compared to suspension bridges, self-balancing characteristics adequate for construction in weak soil and offering an economically advantageous solution for moderate to large spans, and notable aesthetic qualities, led to an enormous increase in the construction of these structures all over the world. This expansion was naturally accompanied by significant technological developments that led to progressively larger and more slender prototypes. Cable-stayed bridges are subjected to a variety of dynamic loads like traffic, wind, pedestrians and seismic loads, which are rather complex to characterise. Considering in addition that stay cables constitute very flexible structural elements generally characterised by small damping coefficients, it is understandable that these elements are very prone to vibrations.

The former reports on cable vibrations refer to the Brotonne bridge in France, and date back to 1976 [1]. According to Stiemer *et al*. [2], the vibrations were so large that the longest cables, which were almost parallel and 1957 *mm* apart at the centre, hit each other. Although the source of these oscillations was not understood at that time, the problem was solved by mounting viscous dampers close to the deck anchorages. Many vibration problems have been detected ever since in bridges all over the world, which have motivated intensive research on the various phenomena. The first studies on cable vibrations date back to the 1980s and, although the full understanding of the governing mechanisms of some complex phenomena has not been achieved yet, important information concerning the various sources of vibrations and possible methodologies for prediction and control, that are of wide interest to bridge designers, can be found in the literature.

Taking these aspects into consideration, it is the purpose of this book to systematise the most recent understanding of the various phenomena of cable vibration, and to present, whenever possible, methodologies for the prediction of potential phenomena and for an efficient control of occurrences. This information will be complemented with the presentation of practical case studies by recognised bridge designers and researchers. In the Appendices, the most relevant aspects of the dynamics of stay cables are discussed.

2. Organisation of the Text

The current publication is divided into seven Chapters, with the following content:

- Chapters 1 and 2 introduce the problem of cable vibration in cable-stayed bridges and present the objectives and organisation of the publication.
- Chapter 3 presents a brief overview of the history of cable-stayed bridges.
- Chapter 4 describes the cable vibration phenomena that are induced by direct action of wind and/or rain, namely: buffeting, vortex-shedding, galloping, wake interference, rain–wind vibration and recently identified phenomena.
- Chapter 5 presents indirect excitation phenomena, i.e. phenomena of cable vibration that are induced in the cable by the movement of its anchorages. For convenience of study, these are separated into two groups, the so-called external excitation and the parametric excitation phenomena, which are introduced considering the study of the stay cable individually. The dynamic cable–structure interaction is also discussed in this Chapter.
- Chapter 6 is dedicated to the description of damping systems for stay cables and to the presentation of a simple methodology for the design of passive dampers.
- Chapter 7 has been developed by bridge designers recognised worldwide, reporting on specific vibration problems in particular bridges and the solutions implemented.

The text is complemented by three Appendices where theoretical and practical issues that support cable vibration studies are presented. They are:

- Description of the deformational characteristics of stay cables (Appendix A).
- Presentation of the fundamentals of cable dynamics (Appendix B).
- Reference to experimental methodologies for practical assessment of cable force and damping (Appendix C).

3. Brief History of Cable-Stayed Bridge Construction

The concept of supporting a bridge girder with inclined stays was introduced in the seventeenth century, with former application proposals attributed to Verantius (Italy, 1617) and Löscher (Germany, 1784) [3, 6].

Several stay applications in the form of iron chains and wire ropes were referred to in the literature throughout the nineteenth century. The lack of a full understanding of the structural behaviour of cable bridges and the technological limitations of the time have resulted in some failures and in the consideration that bridges with inclined stays were too flexible and unsafe, the consequence being the preference for suspension bridges.

During the second half of the nineteenth century, several suspension bridges were constructed. They employed additional inclined cables to stiffen the bridge structure and provide stability against wind. The famous Brooklyn bridge (*Fig. 3.1*), designed by Roebling and completed in 1883, having a total length of 1059.9 *m* and a main span of 486.5 *m*, is the major example of this hybrid type of cable bridge and represents a milestone in the history of bridge construction. Of particular note is the use, for the first time in a large construction, of steel for cable wire instead of iron, at a time when this new material (with twice the strength of iron) was only used for construction of railroads in England.

The development of modern cable-stayed bridges dates from the second half of the twentieth century, and is largely due to Dischinger, who realised that higher stiffness and stability could be achieved with high strength, pre-stressed cables [4]. Combining these properties with the calculation facilities introduced by the use of computers, systematic structural analysis became possible, leading to a rational design and permanent control of cable forces, and thereby to an efficient design for these structures.

The Strömsund bridge in Sweden, opened to traffic in 1956, is generally considered the first cable-stayed bridge of the modern era [5]. As may be seen from *Fig. 3.2*, the bridge is composed of three spans and has a total length of 332 *m*, and a main span of 182.6 *m*, the steel and concrete deck being suspended from each pylon by four pairs of diagonal stay cables.

For technical, economic and aesthetic reasons, cable-stayed bridges, that became very popular in Germany during post-war reconstruction, have gained increasing importance, and their use has quickly spread all over the world, with a multitude of innovative solutions.

It is interesting to note that the design of cable-stayed bridges underwent considerable evolution in a short period of less than 50 years. Mathivat [8], identified three so-called *generations* of cable-stayed bridges that characterise the major developments.

The bridges of the *first generation*, like the Strömsund bridge, used a limited number of stay cables, usually between two and six pairs of stays in the main span. These cables were separated by large distances, normally varying between 30 and 80 *m*, that required a large

4. Vibration Phenomena Directly Induced by Wind and Rain

Wind is one of the most conditioning factors in the design of cable-stayed bridges, particularly of stay cables. These structural elements suffer both from direct excitation along the surface and from indirect action on the deck and towers, which result in oscillation at the supports.

The study of wind effects is normally conducted by separating the static component, associated with a mean flow velocity of air, and the dynamic component, related with vortex shedding and atmospheric turbulence. The current text presents a brief description of wind loads applied to stay cables, concentrating on the effects of the corresponding dynamic component. Particular vibrating phenomena are introduced, namely buffeting, vortex-shedding, galloping, aerodynamic interference, rain-wind induced vibration and a set of vibration phenomena that have been identified more recently, like the dry galloping and drag crisis phenomena. The mechanisms that govern these phenomena are explained according to current theories, as well as possible techniques for the suppression or attenuation of the vibrations.

4.1 Wind Loads on Stay Cables

When related to structural engineering problems, the wind is a flow generally characterised by three time-dependent velocity components $U(t)$, $V(t)$ and $W(t)$, along three mutually orthogonal directions. These components depend on a mean wind velocity U, which is horizontal and has the dominant wind direction, and on the fluctuating components $u(t)$, $v(t)$ and $w(t)$, according to

$$\begin{aligned} U(t) &= U + u(t) \\ V(t) &= v(t) \\ W(t) &= w(t) \end{aligned} \tag{4.1}$$

By immersing a body (a stay cable) in this flow, surface pressures are generated. If the body is fixed, the developed pressures depend on the characteristics of the flow and on the geometry of the body. If the body is free to oscillate, as happens with stay cables, the developed pressures are modified by vibration. An interaction excitation results, which is designated as aeroelastic excitation.

Although in practice these wind effects are related, they are frequently analysed separately for the sake of simplicity.

Considering the stay cable represented by a non-circular cylinder of infinite length immersed in a two-dimensional flow, the incident forces per unit length are calculated from integration of the generated surface pressures under the following conditions: (a) fixed cylinder immersed in a smooth flow; (b) fixed cylinder immersed in a turbulent flow; and (c) moving cylinder immersed in a turbulent flow. These independent conditions allow the identification and

5. Indirect Excitation

5.1 General

The vibration of the deck and towers caused by wind, traffic and earthquakes, produces an *indirect excitation* of the cables through the motion of their anchorages (*Fig. 5.1*). In certain circumstances, the induced cable vibrations attain very high amplitudes. Two phenomena can be identified under these circumstances, described here as *external* and *parametric* excitation. The *external excitation* corresponds to an amplification of motion applied at some anchorage perpendicular to the cable chord, while the *parametric excitation* corresponds to oscillations in the direction of the chord. The phenomena of *external* and *parametric excitation* have been observed in several bridges in the past (some examples are the Brotonne bridge in France, the Ben-Ahin and Wandre bridges in Belgium, and the Annacis bridge in Canada). The literature on the topic of *indirect excitation* concentrates essentially on studies on the *parametric excitation* phenomenon, which is considered more important. The usual approach to the study of *parametric excitation* consists of an evaluation of the cable resonance condition from the dynamic equilibrium equations of a single cable under harmonic motion of the supports. A modified Mathieu-Hill type equation is obtained [75–78], which is characterised by a set of secondary resonances, i.e. the response to a harmonic of frequency ω is not increased exclusively at resonance (when the natural frequency of the cable system coincides with ω), but also at specific ratios between the exciting frequency and the system natural frequency: 1/2, 1/3, 2 and 3. It is possible then to define instability regions, i.e. intervals of frequency oscillation of the supports where high amplitudes of vibration occur and to characterise both the threshold amplitude for the occurrence of instability, and the maximum amplitude of oscillation inside the instability regions.

The various phenomena of cable vibration have been most frequently studied by considering the stay cables isolated from the bridge structure. It is important however to understand the type of interaction that these elements exert with the deck/towers system when integrated in the bridge structure, as well as the interaction between the various stay cables on a bridge.

From an engineering point of view, the goal is to quantify the amplification of the deck/tower motion and define the amount of damping necessary to prevent the cable system from undergoing large vibrations. This Chapter analyses the problem of harmonic oscillation of the supports considering the *external* and *parametric excitations* separately, and presenting formulae for predicting the occurrence of instability and for the estimation of maximum amplitudes of vibration. The issue of cable-structure interaction is also discussed.

5.2 External Excitation

The horizontal cable represented in *Fig. 5.2* is subjected to a harmonic oscillation $z_B(t)$ of the anchorage B, given by

$$z_B(t) = z_B \cdot \sin \omega t \tag{5.1}$$

6. Control of Vibrations in Cable-Stayed Bridges

6.1 General

Once the mechanisms that generate cable vibration have been identified, appropriate countermeasures to reduce or suppress large oscillations should be taken. These countermeasures can be implemented at the source, in such a way that the generation of vibrations is limited, or at the structure, through the installation of control systems that have the effect of limiting the response, either by de-tuning the cables or by increasing their damping. In general, these measures can be classified into three types, namely *aerodynamic, structural* and *mechanical*, and belong to a category of so-called *passive techniques of control*. Although extensively used around the world with proven effectiveness in structural performance, these techniques have inherent limitations [86]. In recent years, however, some attempts have been made to implement *active* and *semiactive control* techniques, using some technological transfer from the area of Control Engineering to Structural Engineering. *Active* and *semiactive* control are, in effect, extensions of *passive control techniques* where the structural motion is controlled or modified by an actuator commanded by a control system through external supply of energy. These systems may actually combine the three types of measures described above with *passive control* devices. Practical applications of these devices are however at the research stage and will not be considered in the current publication.

The following Sections present a brief description of possible systems for control of vibration, focusing in particular on the installation and design of dampers and tuned-mass dampers in cables, the most commonly used approaches.

6.2 Vibration Control Systems

The occurrence of cable vibrations can be prevented or limited by specific operation conditions and maintenance procedures. For the case of indirect vibrations, i.e. vibrations generated by oscillation of the anchorages, possible implementation measures include limiting the number, speed and mass of crossing vehicles and maintenance of the road surface. Deeper interventions may attempt to modify the shape (*aerodynamic control)*, the mass and stiffness (*structural control*), or the damping (*mechanical control*). For indirect oscillations, these measures can be introduced both at the deck and towers, while direct oscillations require the introduction of control systems on the cables.

6.2.1 Aerodynamic control of vibrations

The *aerodynamic control of* vibrations is performed by modifying the cross sectional configuration. For the bridge deck, some appendages are attached, as represented in *Fig. 6.1*

7. Case Reports

The complex phenomena associated with cable vibrations and control have been individually described and analysed so far. On the basis of the contributions from designers, consultants, researchers and manufacturers, this chapter describes a number of practical situations of cable vibration design studies/occurrences which impelled the investigation and evaluation of control design/retrofit measures. Various situations have been covered in terms of occurrences and extension of measures undertaken. The contributors to this chapter, are fully acknowledged for sharing their experiences and providing extremely valuable information.

Cable-stayed bridges constructed in the 1980s and the early 1990s were not investigated for possible cable vibrations. Therefore no special measures were undertaken at the design stage to prevent oscillations, especially the installation of dampers. In the late 1980s and early 1990s, the new technique of individual protection of the strands motivated in several cases the release of the bundle of strands from protective pipes and, consequently, new problems of vibration occurred, related to galloping and interference galloping between strands of the same bundle, eventually increased by particular resonance conditions [84]. Yet, many bridges constructed in this period have not exhibited noticeable vibrations, especially those with medium or low span, as the intrinsic damping of a cable clearly increases with the decrease of its length. At the same time, some of these and other more recent bridges have suffered from vibrations on very few occasions, caused by particular wind and resonance conditions. Such was the case of the Skarnsundet bridge, described in this chapter. The limited number of occurrences and the evidence of no damage have motivated a non-retrofit. Other bridges were not expected to experience certain weather conditions like rain, as was the case of the Puente real bridge, located in Badajoz, Spain, where rain-wind vibration occurred. In this case, additional resonance effects were observed that motivated the installation of dampers close to the anchorages.

Large unexpected rain-wind vibrations were also observed in two different bridges located in Texas, USA. The need to understand these vibrations and develop retrofit measures led to extensive monitoring of the two bridges and to the test of damper prototypes before a definite retrofit measure was chosen, based on the installation of dampers at all stay cables. Many recently constructed bridges have dampers installed in cables from the beginning, as a consequence of the demands posed by more or less extensive studies on cable vibration phenomena conducted at the design stage, as in the case of the Öresund and Erasmus bridges, described here. The installation of cable dampers is not however a guarantee of protection against vibrations. The strong instabilities associated with rain/wind vibration and the deposition of snow under moderate wind (also not fully understood by the current state-of-art) that were observed on these bridges, causing damage to those devices and producing large vibrations, have necessitated a higher damping capacity than the original dampers provided for them.

One possible cause of the inadequacy of dampers to control vibrations might be that the efficiency was only a fraction of that estimated at the design stage.

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Appendix A—Deformational Characteristics of Suspended Cables

A.1 Objectives

The current Chapter discusses the most important formulae that are required for a description of the deformational characteristics of a suspended cable. These include the definition of cable profile under selfweight (e.g. sag and angle of deviation at the anchorages), the evaluation of undeformed and deformed cable length, and the estimation of the installed tension. Simplified formulae and numerical modelling are also discussed, as well as the levels of error attained for the approaches followed.

The intended application is the design and analysis of stays in cable-stayed bridges, although other types of structures whose cables fit the basic assumptions made here, such as suspension bridges and guyed masts, are also considered.

A.2 Static Behaviour

Suspended cables are structural elements characterised by a significant non-linear behaviour. The relatively low level of stress attained by these elements (determined by fatigue considerations) makes this non-linearity predominantly geometric in nature.

A precise description of a cable suspended between two fixed points (*Fig. A.1*) should include the bending and axial deformation, marked by the mechanical stiffnesses EI_0 and EA_0, respectively. It should also take into consideration the installed axial tension T_0 and selfweight (the latter normally constant along the cable length, as long as the cross section remains constant), and finally the end conditions. Given the large displacements caused by the low flexural stiffness, second order effects should also be included.

The evident complexity of the above stated problem is further compounded by the difficulty in a rigorous assessment of the degree of restraint of rotations at the anchorages.

Some simplifications, which enable a more accurate and simple determination of the cable profile $z(x)$ and tension $T(x)$ are, however, possible.

Appendix B—Fundamentals of Cable Dynamics

B.1 Objectives

The basic aspects associated with the linear theory of cable vibration developed by Irvine and Caughey [131] for shallow cables are presented. These include formulae for the quantification of natural frequencies and modal shapes both for horizontal and inclined cables that are valid for almost all of the cable-stayed bridge vibration problems. An improved formulation developed by Triantafyllou is also presented. Finally the inclusion of bending effects in the dynamic description of a stay cable is discussed and simplified formulae for the evaluation of natural frequencies are presented.

B.2 Linear Theory of Vibrations of Horizontal—Cables

B.2.1 Basic assumptions and equilibrium equations

The linear theory of vibrations was derived for a rigidly supported cable with a small sag-to-span ratio d/L $(0 \leq d/L \leq 1/8)$ under the assumption of a quasi-static elastic deformation, i.e., of a constant dynamic component of cable tension τ along the cable length.

Considering a horizontal cable of chord length L and mass per unit length m suspended between two supports at the same level (*Fig. B.1*), the application of a small displacement causes motion of a generic point P from dead load configuration position $\overline{P}(x, 0, z)$ to $\overline{P'}(x + u, v, z + w)$, where u, v and w represent the small components of motion along the horizontal (in-plane and out-of-plane) and vertical directions, respectively.

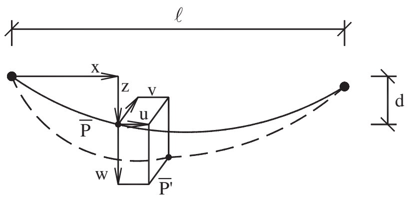

Fig. B.1: Components of displacement of a generic point P of a horizontal vibrating cable

The study of the dynamic equilibrium of a segment of length ds cut at point P (*Fig. B.2*) leads to the following linearised equilibrium equations

$$H \frac{\partial^2 v}{\partial x^2} = m \cdot \frac{\partial^2 v}{\partial t^2} \tag{B.1}$$

$$H \frac{\partial^2 w}{\partial x^2} + h_\tau \frac{d^2 z}{\mathrm{d}x^2} = m \cdot \frac{\partial^2 w}{\partial t^2} \tag{B.2}$$

$$\frac{h_\tau L_e}{EA_0} = \frac{mg}{H} \int_0^\ell w \, \mathrm{d}x \tag{B.3}$$

Appendix C—Assessment of Cable Force and Damping

C.1 General

The assessment of cable force during the construction or service life of a bridge can be done by several methods based on (i) the direct measurement of the stress in the tensioning jacks; (ii) the application of ring load cells or strain gauges in the strands; (iii) the measured elongation close to an anchorage; (iv) a topographic survey and (v) the indirect measurement of vibrations. The advantages and drawbacks of the various techniques are discussed in the following section, focusing in particular on the technique of tension estimation through the measurement of vibrations. The assessment of damping based on the cable response is also discussed.

C.2 Methods of Force Assessment

C.2.1 Direct measurement of stress in tensioning jacks

This method is employed during the construction of the bridge and provides a direct measure of the installed tension. Mars and Hardy [140] found errors of 10% to 15% in the application of this technique. It is therefore of utmost importance to ensure that hydraulic jacks are properly calibrated. This technique is not adequate to estimate the tension after construction of the bridge as the installation of the jacks is a long process and can produce damage in the anchorages.

C.2.2 Application of ring load cells or of strain gauges in strands

Ring load cells can be applied to either one or a few of the individual strands that form the cable, or to the set of strands. Therefore, either individual or average global measures of the installed tension are possible. Although the individual assessment of the strand tension would be of interest for the purpose of detection of cracks, the application to all strands of all cables would not be feasible. Therefore, the normal practice is to either select one or a few strands of a cable for installation of strain gauges or small load cells (see example in *Fig. C.1*), or to mount a ring load cell at one cable anchorage between two bearing plates. In any case the principle is the same, i.e., the measurement of deformation through strain gauge rosettes. The load ring cell is actually a spool of a heat-treated steel alloy with equally spaced bonded strain gauge rosettes. The number of rosettes is at least four, so that eccentricity effects are avoided. The rosettes are composed of two strain gauges, one for the axial and the other for tangential strain measurement. A strong aluminium housing filled with a high-density resin protects the strain gauges from moisture and impact damage. *Figure C.2* shows the installation of a ring load cell.

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Cable Vibrations in Cable-Stayed Bridges

The fifty years of experience of construction of cable-stayed bridges since their establishment as a new category among the classical types have brought an immense progress, ranging from design and conception to materials, analysis, construction, observation and retrofitting. The growing construction of cable-stayed bridges has also triggered researchers' and designers' attention to the problem of cable vibrations. Intensive research has been developed all over the world during the last two decades as a consequence of the numerous cases of cable vibrations exhibited by all types of cable-stayed bridges.

Despite the increased knowledge of the various vibration phenomena, most of the outcomes and research results have been published in journals and conference proceedings and scarce information is currently provided by the existing recommendations and codes.

The present book provides a comprehensive survey on the governing phenomena of cable vibration, both associated with direct action of wind and rain: buffeting, vortex-shedding, wake effects, rain-wind vibration; and resulting from the indirect excitation through anchorage oscillation: external and parametric excitation. Methodologies for assessment of the effects of those phenomena are presented and illustrated by practical examples. Control of cable vibrations is then discussed and state-of-art results on the design of passive control devices are presented.

The book is complemented with a series of case reports reflecting the practical approach shared by experienced designers and consultants: Yves Bournand (VSL International), Chris Geurts (TNO), Carl Hansvold (Johs. Holt), Allan Larsen (Cowi) and Randall Poston (WDP & Associates).