The long way from Teddington (1963) to Cambridge (2013) through 50 years of bridge aerodynamics

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Abstract

This paper presents a review of bridge aerodynamics from the first international wind engineering conference in Teddington (1963) to the present EACWE in Cambridge (2013). A certain attention is also devoted to the research achievements before 1963. The phenomena of interest in bridge aerodynamics are discussed through the milestone papers presented during the various conferences. A few examples of large-amplitude vibrations observed in real structures are also recalled. Moreover, an overview of bridge engineering development in the last 50 years is proposed, highlighting the boost given by the scientific progress in wind engineering. Finally, the major wind tunnel testing facilities and expertise in bridge aerodynamics are revisited.

1 Introduction

With the celebration of 50 years of wind engineering since the very first meeting in Teddington, 1963, the 6th EACWE in Cambridge gives to the community of wind engineers a unique opportunity to track the tremendous developments which this field has undergone in almost all its different specialities and branches of study and technological advances.

This paper is targeted to one of the most exciting sectors, whose progresses in the last half-century were and are beyond any imagination, even amongst the closest experts: wind effects on bridges, today more frequently shortened to "bridge aerodynamics". The Authors have imagined to go back in parallel along some of the main tracks which have characterized bridge aerodynamics developments all over those 50 years. In particular, on the one hand, the phenomena of concern in bridge aerodynamics will be revisited mainly on the basis of the scientific milestones set in the various wind engineering conferences. On the other hand, the outstanding technological achievements in bridge engineering will be highlighted, with particular attention to the key role played by the wind engineering laboratories around the world.

Moreover, in the paper the "past" (i.e., before 1963) will be shortly recalled through a summarized historical review. Therein, the fundamental contributions by Strouhal, von Kármán, Wagner, Theodorsen, Küssner, Sears, Den Hartog, Farquharson, etc. will be revisited in order to better guide the Reader along the developments described for each phenomenon of relevance in bridge aerodynamics. The decisive impact on bridge aerodynamics of the tragic event in 1940 of the collapse of the first Tacoma Narrows Bridge, Washington, US, will be recalled but attention will be paid also to several other cases of large-amplitude vibrations observed in real structures.

2 The phenomena of major concern in bridge aerodynamics

The issues the long-span bridge designer should care for and the inherent uncertainties in the "wind loading chain" was brilliantly overviewed 15 years ago by Davenport (1998). In particular, along with the static wind load, several aeroelastic phenomena have to be accounted for to guarantee the safety and the serviceability of flexible bridges. In the following a short description of these phenomena is reported along with the main milestones of research fixed at the various conferences on wind engineering.

2.1 Vortex-induced vibration

Vortex-induced vibration (VIV) is a phenomenon triggered by the resonance between the Kármán vortex shedding forces and one of the natural frequencies of the structure, which indeed presents strong nonlinearities and where a major role is played by self-excitation. Large-amplitude but usually non-catastrophic oscillations can occur in a limited range of wind speed, in which the spanwise correlation of the fluctuating pressures on the body surface is much higher. Also, the frequency of vortex shedding is not governed by the Strouhal law but is synchronized or "locked" to the vibration frequency of the structure ("lock-in"). Both the maximum amplitude of oscillation and the extension of the range of synchronization are dependent on a mass-damping parameter, the so-called Scruton number. For bridges this phenomenon can concern the cables, the towers or the deck and can represent either an issue of resistance (fatigue) or serviceability (discomfort), given that it may occur at relatively low wind speed. Moreover, for bridge decks vortex-induced vibrations in a torsional mode are often possible. Turbulence is known to have a strong effect on the synchronization mechanism, reducing the oscillation amplitudes and in some instances it is even able to completely inhibit the phenomenon.

The state-of-the-art knowledge of VIV before 1963 was quite limited. Strouhal (1878), studying the tones produced by a wire immersed in a wind stream, had observed that they were directly proportional to the flow speed and inversely proportional to the wire diameter. He was also aware that the sound intensity was increasing when the tones were coinciding with the wire natural frequency. The theory of vortex street for bluff bodies had been formulated by von Kármán (1911). Scruton (1955) had investigated the VIV instability for stacks of circular cross section. Furthermore, during the four months of active life of the original Tacoma Narrows Bridge, before the collapse due to the torsional vibrations, vertical oscillations in several modes due to lock-in had been observed (Farquharson *et al.*, 1949-1954). Also, in the '40s Dunn and von Kármán had investigated the synchronized oscillations of an H-shaped section in the plunging and in two rotational modes (Fung, 1993).

At the 1963 conference in Teddington Scruton (1965b) proposed a model for VIV, defining, in analogy to the airfoil theory, the self-excited lift force as the sum of components in phase and in quadrature with the motion with frequency- and amplitude-dependent coefficients, but he remarked that the variation of the structural frequency during VIV was negligible. At a given reduced wind speed the instability, if any, is then expected at the amplitude for which the total damping (structural + aerodynamic) of a vibration mode vanishes. Scruton also presented the tests performed on the full aeroelastic model of a tower of the Severn Bridge, UK, which seemed to be prone to VIV vibrations and proposed a sheeting between the top portion of the legs of the tower, working as a splitter-plate and preventing the formation of alternating vortices. A few years later, Parkinson & Modi (1967) investigated the VIV phenomenon for the circular, structural angle and Tacoma Bridge sections.

At the beginning of the '70s a couple of famous VIV models were published (Hartlen & Currie, 1970; Skop & Griffin, 1973), based on the coupling between a linear oscillator representing the structure and a nonlinear oscillator for the wake. Nevertheless, during the 1975 conference in Heathrow Scanlan (1977) presented a much simpler linear model, the main appeal of which was the possibility of identifying the needed parameters just with a decay-to-resonance test. The idea was to split the aerodynamic force in a self-excited term, which introduces negative aerodynamic damping, and a harmonic forcing term at the Strouhal frequency, which triggers the resonance of the structure. During the same conference Oey *et al.* (1977) investigated the hysteresis observed in the VIV response of lightly damped circular cylinders, while Miyata *et al.* (1977) investigated the relation between vibration amplitude and reduced velocity and damping for a box-girder section of a cable-stayed bridge considering the nonlinear dependence of unsteady aerodynamic forces on the amplitude of oscillation. Okubo *et al.* (1977) gave some design guidelines for improving vortex-related stability of cable-stayed bridge decks with particular attention to non-structural details such as handrails, curbs, flaps and fairings. Similar studies were published in the following decades and a noticeable example is the experimental campaign for the proposed Messina Bridge, Italy (Diana *et al.*, 2007b). In particular, a famous work on the effect of fairings of different geometry had been presented by Wardlaw (1971) during the 1971 conference in Tokyo.

At the end of the '70s Tamura & Matsui (1979) proposed a two-degree-of-freedom VIV model for a circular cylinder, where the wake oscillator is of the Birkhoff's type, that is the wake effect on the cylinder is assumed to be similar to that of an oscillating lamina. The analogy to the Magnus effect for a rotating circular cylinder was employed to determine the relation between lift coefficient and wake angular displacement. The model resulted to be able to capture several complex features of the phenomenon and satisfactory agreement was found with several series of experimental data for various Scruton numbers. Later, Tamura & Amano (1984) extended the model to continuous structures with variable mean velocity along the axis.

In the 1983 conference in Australia-New Zealand Shiraishi & Matsumoto (1984) reviewed and classified vortex-induced oscillations with special attention to bridge structures. Many bluff body geometries were tested for different angles of attack and three excitation mechanisms were individuated based on reduced critical wind speed, response amplitude patterns, flow and pressure characteristics. General clues for the suppression of VIV by geometric shaping of bridge sections were provided. During the following ICWE conference Kobayashi *et al.* (1988) investigated the instability in the bending mode, highlighting the presence of two mechanisms: the resonance with the Kármán vortices, which triggers the oscillations, and the appearance of motion-induced vortices, which amplifies the vibration amplitude.

At the beginning of the '90s Ehsan & Scanlan (1990) and Goswami *et al.* (1992) presented a single-degree-of-freedom nonlinear model based on a van-der-Pol-type equation, the parameters of which could be determined through a simple decay-to-resonance test. Experimental results were presented for two bridge sections (Tacoma Narrows Bridge and Deer Isle Bridge), a 4:1 rectangular cylinder and a circular cylinder. Larsen (1993) proposed a similar model but with a generalization of the van-der-Pol-type equation, requiring an additional parameter, to better follow the relation between response amplitude and Scruton number. Recently, Marra *et al.* (2011b) discussed the Ehsan-Scanlan model in details and applied it to the case of a 4:1 rectangular cylinder. An alternative method of identification of the aeroelastic parameters was also proposed (Marra *et al.*, 2011a).

An open issue concerning VIV is the effect of yaw angle (horizontal wind direction): according to many Authors, synchronization can occur only for winds nearly perpendicular to the bridge axis (Larsen *et al.*, 1999; Macdonald *et al.*, 2002; Fujino & Yoshida, 2002); nevertheless, Hosomi *et al.* (1995) in the case of a bridge girder schematized by a beam of constant or variable depth with a rectangular cross section (with a width-to-depth ratio of 2 at midspan) observed vortex-induced vibration for very large yaw angles, up to 77.5°. This issue was also investigated by Diana *et al.* (2003) for the proposed Messina Strait Bridge, Italy, observing the tendency to excitation also at yaw angles of 45°, though reduced in magnitude.

In the last decades many cases of vortex-induced vibration of bridge structures have been registered. To the knowledge of the Writers the first case of VIV for a bridge tower is that of the Forth Suspension Bridge, UK, which was under construction at the time of the 1963 conference in Teddington. Vibrations with amplitude of more than 1 m in the fundamental bending mode at erection and freestanding phases were probably due to its reduced mass as compared to similar constructions and required the installation of a peculiar temporary damping system, consisting of an external mass of about 15 tons connected to the top of the tower (Scruton, 1965a,b). Permanent and temporary damping devices (tuned-mass and active-mass dampers) were also installed in the towers of the Akashi Kaikyo Bridge, Japan, to restrain the oscillations predicted by wind tunnel tests (Honda *et al.*, 1995).

As for the deck oscillation, after the first Tacoma Narrows Bridge, the most famous case of vortex-induced vibration was that of the Great Belt East Bridge, Denmark, which during the final phases of deck erection showed low-frequency vertical oscillations of the main girder, that were considered unacceptable with respect to the comfort of the bridge users (Larsen *et al.*, 1999). After wind tunnel tests guide vanes were devised and installed in correspondence of the lower edges of the deck to mitigate the excitation. Later, Terrés-Nicoli *et al.* (2003) experimentally studied VIV for the deck of the Great Belt East Bridge in the torsional mode with particular attention to the fluid-structure interaction mechanism responsible for the onset and the actual magnitude of the oscillations. Significant hysteresis was observed for the 1:70 scale section model tested.

The vortex-induced vibration of the Great Belt East Bridge had a great resonance in the scientific community but many other bridges suffered from this type of oscillations. A few examples are recalled hereafter. The Deer Isle Bridge, Maine, US, a suspension bridge with a main span of 329 m and an H-type cross section, opened for traffic in 1939 and stiffened in several stages after the collapse of the Tacoma Narrows Bridge, showed significant harmonic vertical bending vibrations at a wind speed around 9 m/s, much larger than those observed at higher wind velocities (Kumarasena *et al.*, 1991). Very alarming vertical oscillations had appeared also on December 2nd, 1942, as reported by Vincent (1965), although the nature of the wind-excitation had not been clarified. The Long's Creek Bridge, Canada, a cable-stayed bridge with a main span of 217 m and a steel deck stiffened by two longitudinal beams, was found to vibrate with amplitudes of about 20 cm. The oscillations were provisionally mitigated with ten open structure tote boxes filled with rocks suspended to the deck and immersed in the water and later suppressed by installing fairings (Wardlaw, 1971). Owen *et al.* (1996) reported of the significant vibrations observed for the Kessock Bridge, UK, a cable-stayed structure with a main span of 240 m and a steel deck with an open cross section. In several occasions amplitudes of oscillation larger than 11 cm were observed in the vertical modes for wind speeds in the range 23-25 m/s and in one case the bridge showed also large torsional vibrations. Tuned-mass damper devices had been installed in the structure before the monitoring campaign but they did not succeed in suppressing the wind excitation. Interestingly, lock-in response appeared for both Eastern and Western winds although in the second case, due to the higher turbulence intensity the vibrations were sustained for shorter periods. Another significant example is the Second Severn Crossing Bridge, UK, a cable-stayed bridge with a main span of 456 m and a deck composed by two longitudinal steel girders, transverse trusses and a reinforced concrete slab. As discussed by Macdonald *et al.* (2002), the prototype bridge suffered from frequent vertical vibrations, which were not expected from wind tunnel tests, as a result of lower structural damping and different turbulence characteristics. The excitation was suppressed by the installation of baffle plates under the deck.

VIV vibrations were observed not only for cable-supported bridges, as demonstrated by the case of the Trans-Tokyo Bay Crossing Bridge, Japan (Fujino & Yoshida, 2002). This is a straight ten-span continuous steel box-girder bridge with a main span of 240 m and variable cross-section height. The structure exhibited vibrations exceeding 50 cm, occurring for the first vertical mode in the wind-speed range 13-18 m/s. To reduce the oscillation amplitude several tuned-mass dampers were installed together with 49 cm-high vertical plates fastened to the post of crash barriers over the deck. Another very recent example is the Volgograd Bridge, Russia, whose vertical vibrations in May 2010 with a maximum amplitude of 40 cm (Weber *et al.*, 2013) became an attraction on the web. The bridge presents a main span of 155 m with a slender steel deck. Semi-active tuned-mass dampers were installed to suppress the vibrations in the first three vertical modes.

2.2 Galloping

Galloping is a single-degree-of-freedom dynamic instability typical of slender structures with non-circular bluff sections, due to cross-flow motion-induced forces which produce negative aerodynamic damping. It is characterized by large oscillations at one of the transversal structural modal frequencies (usually the lowest) with amplitude indefinitely growing with the wind speed beyond a certain critical value. Galloping can also arise in torsion (Glauert, 1919) but, due to the intrinsic unsteady nature of the phenomenon and its relevance for non-streamlined bridge decks, this phenomenon is often called "torsional flutter" and it will be discussed in the following section.

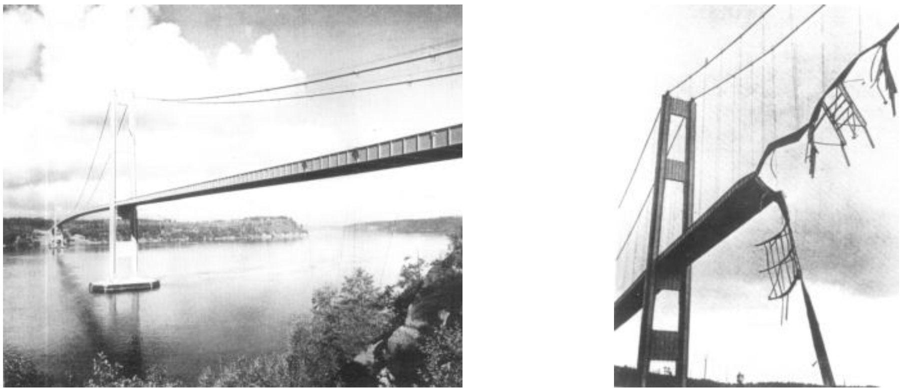
Galloping in the plunging degree-of-freedom is generally not a problem for bridge decks, as it may occur only in case of very bluff girders, which are usually too stiff to give rise to this type of instability. Nevertheless, it is an important phenomenon for bridge towers and noncircular hangers. For instance, Dyrbye & Hansen (1997) recalled that the hexagonal pylon of the Lodemann Bridge, Germany, a 68 m main span cable-stayed footbridge, in November 1972 was ruined due to galloping.

Before the 1963 conference in Teddington, the mechanism of galloping excitation of transmission conductor lines had been explained by Den Hartog (1932, 1956), who elaborated the homonymous criterion for the critical wind speed on the basis of the slope of the lift coefficient and magnitude of the drag coefficient. Later on, Scruton (1960) employed the quasi-steady theory to estimate the steady vibration amplitude of galloping structures.

Nevertheless, a significant boost to the research on this phenomenon was given by Parkinson and coworkers with the papers presented at the first international conferences on wind engineering (Parkinson, 1965; Parkinson & Modi, 1967), where the nonlinear quasi-steady theory to galloping was formulated and applied to the case of square and rectangular two-dimensional cylinders, showing successful qualitative and quantitative agreement with experiments. The influence of isotropic turbulence on the galloping behaviour of rectangular cylinders was investigated by Laneville & Parkinson (1971), showing that the quasi-steady theory is still applicable provided that the static aerodynamic coefficients are measured in a turbulent flow with the same turbulence intensity. Also, turbulence can either suppress galloping in case of sufficiently elongated rectangular cylinders (such as 2:1) or foster the instability in case of cylinders with short afterbody (such as 1:2). During the same conference Novak (1971) clearly distinguished between soft and hard galloping and extended Parkinson's theory to tower-like structures in turbulent shear flows. He also dealt with the problem of interaction between galloping and vortex-induced vibration. Otsuki *et al.* (1971) reported a wide range of experimental results on prismatic bars of rectangular cross section with side ratios varying from 1 to 4. They considered the influence of angle of attack and their study included torsional instability. Examples of interference between VIV and galloping can be conjectured from their results. By contrast, this issue was clearly dealt with for rectangular cylinders by Wawzonek & Parkinson (1980). In a paper presented at the last ICWE conference in Amsterdam Matsumoto & Laneville (2011) investigated the complicated problem of the relation between galloping and Kármán vortex shedding, highlighting that the former is able to appear when the latter is mitigated or suppressed. Recently, Mannini *et al.* (2013) studied the combined VIV-galloping instability with experiments on a 3:2 rectangular cylinder.

2.3 Flutter

Flutter is a dynamic instability basically involving a torsional and a vertical bending vibration mode of the deck with similar shapes. The mechanism through which energy is extracted from the flow and feeds the structural oscillations is related to the variation of modal frequencies, that tend to approach due to fluid-structure interaction, and to the phase between the two degrees of freedom. In case of bluff cross sections the instability may involve only a torsional mode (torsional flutter), similarly to the case of airfoils at stall conditions. However, the presence of a vertical bending mode with similar



(a) (b)

Figure 1: First Tacoma Narrows Bridge, Washington, US, before (a) and after (b) the collapse on November 7, 1940 [from Farquharson *et al.* (1949-1954)].

shape and close frequency may cause a reduction of the critical wind speed. Torsional flutter was the cause of the collapse of the Tacoma Narrows Bridge (Figure 1) and, a hundred years before, of the Brighton Chain Pier Bridge, UK (Figure 2). Flutter gives rise to limit-cycle oscillations of large amplitude, increasing with the wind speed, that can easily lead to the collapse of the structure. For bridges this instability often appears as a torsional rotation about a fixed point upstream of the cross section centerline (Selberg, 1965; Wardlaw, 1971), therefore differing significantly from airfoil classical flutter, where a phase shift between vertical and torsional motion is always present. This type of instability mode was hypothesized to have been responsible for the severe damage of the Menai Strait Bridge, UK, in 1839 (Vincent, 1965). The flutter phenomenon is analytically approached by assuming that so-called "self-excited" forces stem from the motion of the body.

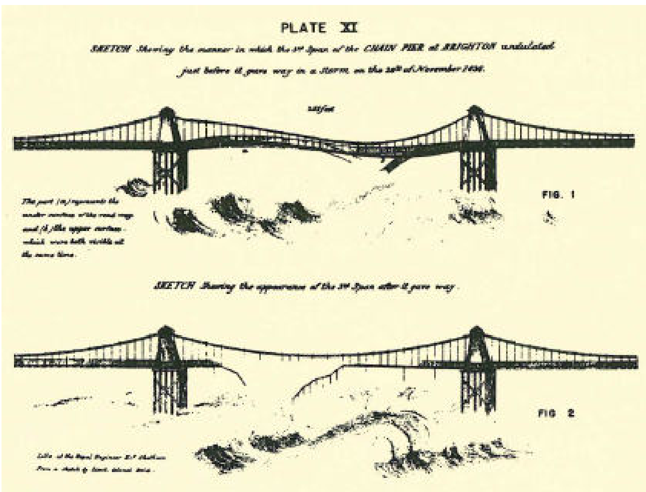
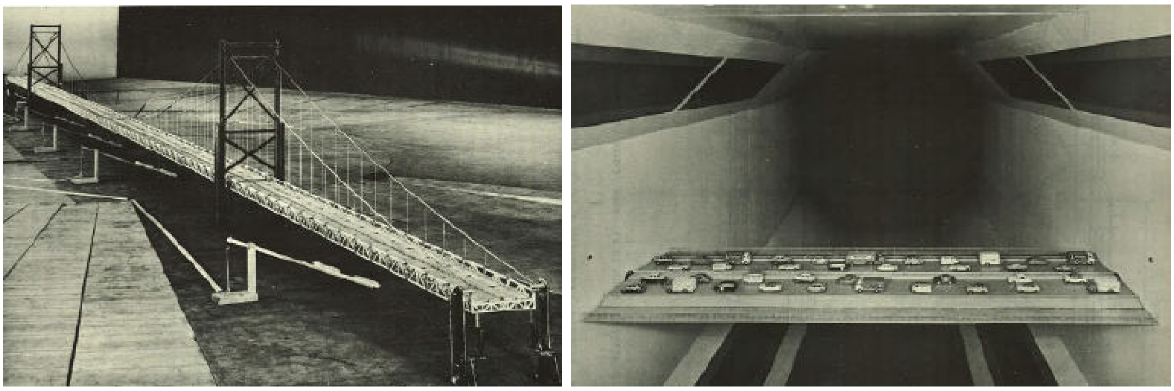


Figure 2: Catastrophic oscillation in the first skew-symmetric torsional mode of the Brighton Chain Pier Bridge, UK, in 1836 [from Scruton (1965a)].

At the time of the first international conference on wind engineering the theory of flutter for airfoils was already consolidated. Based on potential flow theory and Kutta condition, Theodorsen (1934) had



(a) (b)

Figure 3: Section model (a) and full aeroelastic model (b) of two different deck solutions for the Severn Bridge, tested at the National Physical Laboratory, UK [from Walshe (1965)].

obtained in closed form the aerodynamic forces on a harmonically oscillating airfoil or airfoil-aileron combination and calculated the critical flow speed. Some years before, Wagner (1925) had already obtained the expression of indicial lift for an airfoil undergoing an instantaneous step change of the angle of attack. Halfman (1952) had tested a symmetric airfoil forced to vibrate harmonically in the heaving and pitching modes in a two-dimensional flow: the results corroborated the prediction of the previously mentioned vortex-sheet theory, unless the Reynolds number was too low. Bleich (1948) had first applied Theodorsen's analytical airfoil theory for bridge decks while Steinman (1950) had proposed a quasi-steady formulation for self-excited forces. Farquharson *et al.* (1949-1954) had reported all the analytical, numerical and experimental investigations after the collapse of the Tacoma Narrows Bridge. In particular, they extensively described the methods of model testing developed at the University of Washington. Similar techniques were described by Frazer & Scruton (1952) in their report on the wind tunnel campaign on the Severn Bridge, UK (Figure 3).

The importance of flutter for long-span bridges was already in the minds of the pioneering delegates of the 1963 meeting in Teddington, as evident from the work presented by Rocard (1965), who discussed the mechanism of instability for suspension bridges with an eye to the classical theory for airplane wings, intervening also on the issue of the relationship between vortex shedding and flutter. Selberg (1965) proposed an empirical formula for the critical wind speed of a flat plate dynamically equivalent to the bridge deck considered and introduced an experimental coefficient to quantify the actual aerodynamic performance of the deck cross section with respect to the theoretical case. A similar approach was followed later also by Frandsen (1966). While presenting the experimental activity on bridge aerodynamics in the United States, Vincent (1965) reported of the torsional flutter instability in the first skew-symmetric mode of the Golden Gate Bridge, California, US, registered during the storm of December 1, 1951, which appeared at a wind speed around 20 m/s and reached an amplitude of 6.5° at about 25 m/s (Figure 4). Some years later Hirai *et al.* (1967) considered the effect of several parameters (flow angle of attack, turbulence, solidity ratio, frequency ratio and damping) on the flutter critical wind speed for a few cross-sectional geometries (flat plate, truss, H-section). Ukeguchi *et al.* (1966) first published some results on the experimental measurements of self-excited forces for bridge decks by recording the loads on a sinusoidally driven model. The nondimensional coefficients that relate the force components in phase and in quadrature with the structural motion to the displacements and velocities of displacement are the well-known flutter derivatives.

A milestone in the history of bridge aerodynamics was set during the 1971 conference in Tokyo,

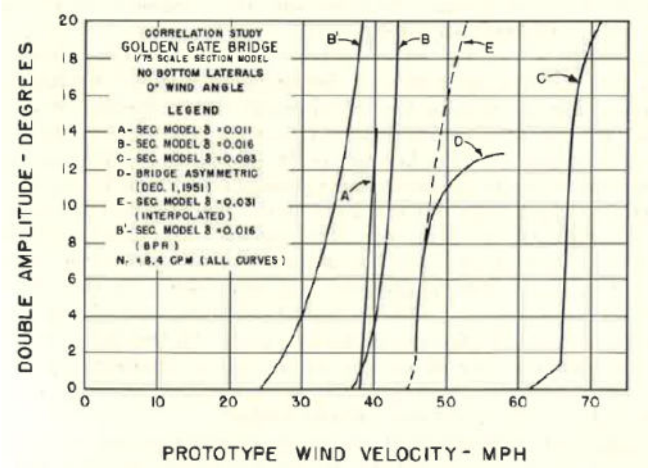


Figure 4: Wind-tunnel predictions and full-scale recording of flutter for the Golden Gate Bridge, California, US [from Vincent (1965)].

where Scanlan (1971) reported flutter derivative results for a wide range of bridge deck geometries obtained by recording the free-decaying motion of elastically suspended section models from imposed initial conditions with and without wind. This method, known as free-vibration method, requires a simple experimental apparatus and at that time also a simple identification algorithm was used; more advanced identification techiques have been used later on (e.g. Scanlan (1977); Jiming (1988); Sarkar *et al.* (1992); Diana *et al.* (1992); Jakobsen & Hjorth-Hansen (1993); Singh *et al.* (1995); Costa & Borri (2007); Bartoli *et al.* (2009)). He also presented some attempts to measure the time evolution of the aerodynamic moment due to a step change in angle of attack (indicial function) for a few deck girders. Finally, in that paper Scanlan discussed the role of turbulence, assuming that the effect of buffeting forces can be superimposed upon that of self-excited forces measured in turbulent flow. He proposed a method to determine flutter derivatives as the ratio of response spectra to buffeting force spectra. Examples of measurements of flutter derivatives in turbulent flow for an H-shaped deck section, with and without fairings, were presented some years later (Huston *et al.*, 1988). Sabzevari (1971) determined by inverse Laplace transformation the bending and torsional indicial functions for the first Bosporus Bridge, Turkey.

Davenport *et al.* (1971) compared the results obtained in section-model and full-aeroelastic-model tests and proposed an advantageous compromise between the two types of setups, the taut strip model. The aeroelastic behaviour of bridges during erection stages was then discussed. They also pointed out the role of turbulence on flutter instability, finding that it raises the critical wind speed for coupled vertical-torsional flutter of flat plate and truss-stiffened decks, whereas it only marginally postpones the onset of torsional instability for H-shaped plate-girder decks. On this issue slightly different results had been obtained four years before by Hirai *et al.* (1967), who observed a decrease of critical velocity in fluctuating wind for classical coupled flutter and no variation or small increase for torsional flutter. On this point analytical time-domain analyses were performed by Bucher & Lin (1988) employing stochastic averaging procedures. They added to the square of the mean wind velocity the contribution of a Gaussian random process and found that turbulence can play either a destabilizing role, if the critical mode is to act alone, or a stabilizing role, if it is coupled with other modes, as it fosters the vibrational energy transfer towards more stable modes. In the latter case the lack of spanwise correlation of the turbulent field was shown to be very important. Bartoli *et al.* (1995) adopted Itô's differential rule to account for the contribution of oncoming turbulence with a specified spectrum on the definition of the stability threshold for linear and nonlinear structural systems. A destabilizing effect of turbulence was observed by Diana *et al.* (1999) for the proposed Messina Bridge. In Aachen 1987 Scanlan (1988) discussed the important question of the gust duration which allows an oscillation build-up that can have catastrophic consequences for the structure.

In Scanlan's formulation of self-excited forces (Scanlan, 1971) lateral sway of the bridge and drag force were neglected as insignificant. Nevertheless, according to Davenport (1966) a third degree of freedom is sometimes necessary. Singh *et al.* (1995) presented for the first time experimental results wherein the full set of 18 flutter derivatives were measured. Few years later Katsuchi *et al.* (1998) showed the importance of the drag-force flutter derivatives for a high-drag deck section, such as that of the Akashi Kaikyo Bridge, Japan, while Chen *et al.* (1999) investigated the effect on the flutter mechanism of additional modes with respect to the basic critical pair.

Kimura & Tanaka (1992) showed that the cosine rule can be applied to calculate the aerodynamic damping in case of yawed winds. Diana *et al.* (2003) measured the flutter derivatives of the Messina Strait Bridge for various yaw angles, observing different absolute values but similar trends with the reduced wind speed.

After Scanlan (1971) and Sabzevari (1971) the issue of indicial aerodynamic moment response of a bridge deck was recalled eight years later by Yoshimura & Nakamura (1980) and this topic has remained in the head of the researchers until today (e.g. Borri & Costa (2003)), also thanks to the clear overview given ten years ago by Caracoglia & Jones (2003).

With respect to the linear model for self-excited forces proposed by Scanlan, Miyata *et al.* (1977) considered the influence of the amplitude of oscillation by measuring the unsteady forces with the forced-vibration technique. The nonlinear character of the aeroelastic forces was also studied by Diana *et al.* (1995), who proposed a time-domain model and an identification procedure based on the extended Kalman filter.

In the 1995 conference in New Delhi Matsumoto *et al.* (1995a) gave a deep insight into the flutter mechanism of rectangular and elliptical sections with the aim to come up with a strategy to stabilize it in the case of bridge decks. In this work the inter-dependence between some flutter derivatives was emphasized and the step-by-step analysis was proposed as an alternative to complex eigenvalue calculation of critical wind speed, allowing to understand the role played by each flutter derivative in the instability onset. Later, with the same purpose to shed some light on the flutter mechanism, simplified formulas to calculate flutter critical wind speed have been proposed (Bartoli & Mannini, 2005; Chen & Kareem, 2006; Mannini *et al.*, 2007). Mannini & Bartoli (2007) also investigated in the wind tunnel the probabilistic properties of flutter derivatives for a box-girder deck section and made a first attempt to evaluate the uncertainty propagation from the aeroelastic coefficients to the calculated flutter instability threshold, obtaining a probability distribution of the flutter critical wind speed.

Finally, a significant effort of researchers has always been devoted to investigate the way to delay flutter instability beyond the design wind speed for the bridge site by passive or active devices able to improve the aerodynamic performance of the deck cross section. In this context the pioneering experimental work of Scruton (1952) for the Severn Bridge is an awe-inspiring example. During the 1963 conference in Teddington Selberg (1965) expressed his doubts about the use of slots and grids on a bridge deck, especially if the grids may be closed by ice or snow. His results for a truss-stiffened girder deck showed that the configurations with open slots were better than the solid one only for particular ranges of angles of attack. On this point, more recently Matsumoto *et al.* (2001) studied the effect on flutter stability of grating on plate-like bridge decks for various angles of attack, opening ratios and locations of the grids, also in the presence of fairings. The grating did not appear very efficient apart from particular configurations and indeed strongly destabilizing effects were sometimes observed. In the 1971 conference in Tokyo Wardlaw (1971) discussed simple shape modifications able to significantly improve the flutter stability of a trapezoidal deck section. Later, Fujino *et al.* (1995)

used the rational function approximation for time-domain unsteady aerodynamic forces in order to apply the control theory to the bridge flutter problem. In particular, the effectiveness of two active control surfaces attached on the edges of the deck was investigated. Many other works would be worth mentioning but cannot be discussed here for the sake of brevity (e.g. Huston *et al.* (1988); Miyata *et al.* (1988); Larsen & Gimsing (1992); Larsen (1999); Xiang & Ge (2003); Diana *et al.* (2007b)).

2.4 Buffeting

In aeronautics buffeting denotes the dynamic excitation of the aircraft tail due to the interference with the wake of another part of the airplane, such as a stalled wing. By contrast, in wind engineering with buffeting one refers both to the excitation mechanism of a structure due to the wake of another structure (Scruton, 1965a) and to the effect of turbulence naturally present in the atmospheric boundary layer. With respect to the classical problem of response of a system to a multi-correlated random load, in the case of buffeting it is crucial to account for the effect of subcritical self-excited forces, which change the modal frequencies and the damping ratios of the structure and induce a certain degree of coupling between the modes (in particular between vertical bending and torsional modes with similar shapes).

At the time of the first Conference on Wind Effects on Building and Structures (1963) the main contributions on buffeting were from the aeronautical field, where Küssner (1936) had set up the theory of the response of an airfoil to a step change of the flow speed and Sears (1941) had done the same for a sinusoidal gust. The functions they obtained can be used for the buffeting analysis of wings respectively in the time and frequency domain. Later Liepmann (1952) had studied the lift force exerted upon a two-dimensional thin airfoil moving in an isotropic turbulence field. In the same work he also proposed the famous approximation to Sears' function. Few years before the Teddington meeting Davenport (1961) had published his theory of dynamic response of structures to the random excitation due to turbulence in the atmospheric boundary layer and then applied it to the particular case of a suspension bridge (Davenport, 1962). He had shown that the fluctuating wind has an effect as great or greater than the mean wind and that the vertical vibrations due to the vertical component of turbulence may be as significant as the horizontal action.

The response of bridges to turbulent wind is usually calculated by assuming the linear superposition of buffeting and self-excited forces and possibly using the flutter derivatives measured in turbulent flow (Scanlan, 1971). In fact, small-scale turbulence (say scales smaller than the deck dimensions) has mainly the effect of changing the aerodynamic behaviour of the deck cross section, while large-scale turbulence represents the most energetic source of random excitation. Tanaka (1971) pointed out the need of considering coupled oscillation of bending and torsion in order to obtain good agreement with the experimental results. He also noticed that in case of coupled vibrations the contribution of the imaginary part of the cross spectrum of fluctuating wind is non-negligible if the bending mode shape is different from the torsional one. Davenport *et al.* (1971) reported experimental results for the dynamic response to turbulent wind of full-aeroelastic and taut-strip models of various cross-section geometries (truss-stiffened, flat plate, H-shaped), in one case also during erection stages. Few years later Holmes (1977) compared the theoretical prediction of the buffeting response of a box-girder cable-stayed bridge based on the strip theory with full-aeroelastic model data. He observed that the use of correct aerodynamic damping had an important impact on the results and that the quasi-steady theory with an aerodynamic admittance of unity significantly overestimated the measured response. By contrast, this was underpredicted if the Liepmann approximation to the Sears function was adopted, whereas better agreement was found by employing a modified expression of the aerodynamic admittance (proposed by Vickery). Holmes' findings were confirmed some years later by the results of Walshe & Wyatt (1984), who measured the aerodynamic admittance function for a box-girder deck section, also supporting the concept of separate computation of chordwise and spanwise terms. For a slender box-girder bridge with inclined webs in the range of reduced velocity of interest they observed that the theoretical flat plate approximation could have been used to estimate the aerodynamic damping in the bending modes. Finally, they proposed a simple parameter to allow the bridge designer to evaluate whether detailed buffeting analyses are necessary or not. Grillaud *et al.* (1992) considered the interesting case of the Pont de l'Iroise, France, a 400 m main span cable-stayed bridge dynamically excited by the wake of an arch bridge located 100 m upstream. They estimated the aerodynamic admittance and the damping of the bridge deck and observed that the latter was more or less in agreement with the corresponding quasi-steady value depending on turbulence intensity and cross-section geometry. Larose *et al.* (1992) compared full-scale measurements, results for full-aeroelastic and section wind-tunnel models and the prediction of a theoretical model based on the quasi-steady aerodynamic theory, neglecting aeroelastic coupling between the modes. Three structures were considered: the Humber Bridge, UK, a 1410 m main span suspension bridge, the Farø Bridge, Denmark, and the Sunshine Skyway Bridge, Florida, US, two cable-stayed bridges with a main span respectively of 290 m and 365 m. Both the physical and the theoretical models showed good agreement with the real bridge data. An organic review of the buffeting phenomenon was presented in the 1995 conference in New Delhi by Xiang (1995), who also discussed the use of tuned-mass dampers to control the vertical vibrations. Xiang *et al.* (1995) proposed a time-domain analysis of bridge buffeting based on the simulation through the wave superposition method of the turbulent forces as multi-correlated random processes and expressed the self-excited forces as unit impulse functions. Jain *et al.* (1995) presented a complete multi-mode buffeting study and emphasized the effect of aeroelastic mode coupling in very-long-span bridges. A comparison with the results obtained through the SRSS single-mode approach was outlined. The effect of mode coupling on the response to turbulent wind of a suspension bridge with a center span of nearly 2000 m was also investigated by Chen *et al.* (1999). Cigada *et al.* (2003) measured the drag, lift and moment complex aerodynamic functions of two configurations of the deck proposed for the Messina Bridge by using an active turbulence generator able to produce an almost harmonic wind fluctuation. Borri & Costa (2005) applied indicial functions as time-domain counterpart of aerodynamic admittances.

If the effect of turbulence is accounted for not only in the buffeting forces but also in the self-excited forces, the dynamical system becomes time-variant and therefore the classical spectral analysis of random vibrations is no longer applicable. For this reason Lin (1980) adopted a time-domain approach based on the Markov process theory and determined the first and second moments of bridge response quantities. Later, Bucher & Wall (1992) applied the method of stochastic averaging to obtain a description of the response in terms of probability density functions and threshold crossing rates, concentrating on a critical mode for which parametric excitation and stability effects are of importance.

An issue of particular interest in the historical evolution of buffeting analysis is the way to model self-excited forces in subcritical conditions. In particular, field measurements showed that the instantaneous wind angle of attack can be as large as $\pm 10°$ (Bocciolone *et al.*, 1992) and therefore it is important to account for its effect on the aeroelastic forces. A first attempt to do it was done by Xiang *et al.* (1995), who found a significant effect on the torsional response of the Shantou Bay Bridge, China. Later, Chen & Kareem (2000) proposed to filter the turbulent field in order to distinguish between a low-frequency contribution (lower than the first natural frequency of the bridge structure), which alters the self-excited forces through the angle of attack, and a high-frequency one, which possibly influences the deck aerodynamics. The resulting nonlinear time-domain approach consists of low-frequency force components modeled with the quasi-steady theory and high-frequency components based on the traditional unsteady theory but varying in space and time according to the effective low-frequency angle of attack. The nonlinear relation between self-excited and buffeting forces was also investigated by Diana *et al.* (1999). The same research group few years later proposed a time-

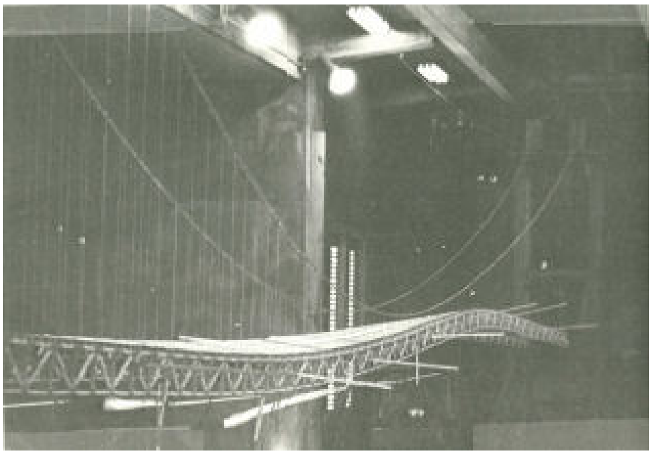


Figure 5: Lateral buckling instability for the full-aeroelastic model of a truss-girder suspension bridge tested in the large wind tunnel of the University of Tokyo [from Hirai *et al.* (1967)].

domain rheological model to account for the dependence of aeroelastic forces on reduced wind speed and angle of attack (Diana *et al.*, 2007a). Salvatori & Spinelli (2005) studied the effect of nonlinearities in the quasi-steady load model, including self-excited and turbulent contributions.

2.5 Static instabilities

Although generally less stringent than flutter, torsional divergence and lateral buckling may also represent an issue for very-long span bridges. The former is an aerostatic instability caused by the vanishing of the torsional stiffness of the structure due to the negative aerodynamic contribution related to a positive slope of the moment coefficient. The analytical formulation of the problem can be found in the paper presented by Scanlan (1977) during the 1975 conference in Heathrow. The actual importance of torsional divergence for bridges and the role played by turbulence was recently highlighted by Ge (2011), who claimed that the traditional static calculation in smooth flow may lead to unsafe results, as observed during the full-aeroelastic model tests of the Xihoumen Bridge, China, a 1650 m suspension bridge with a twin-box girder deck.

As observed by Hirai *et al.* (1967), lateral buckling may be of concern for bridge structures when the vertical bending and torsional stiffness are relatively low in comparison with the horizontal bending stiffness. These authors presented a formula to calculate the critical wind speed, compared its results with those observed from experiments on full-aeroelastic bridge models (Figure 5) and investigated the effects of wind fluctuation.

2.6 Cable vibration

Cable vibration often represents the major aerodynamic problem for cable-stayed bridges. Virlogeux (1998) reported of many cases of cable vibration, discussed the numerous types of excitation and reviewed the possible countermeasures. Another key paper on this topic is that presented by Richards (1965) during the 1963 conference in Teddington for the large-amplitude vibration of the Severn Crossing conductor due to yawed winds.

Rain-wind induced vibration was first consciously observed by Hikami & Shiraishi (1988) for the Meikonishi Bridge, Japan, a cable-stayed bridge with a main span of 405 m. Velocity-restricted oscillations with a maximum amplitude of 27.5 cm under 14 m/s were measured at full scale and then studied in the wind tunnel. The frequencies of oscillation were much lower than those for vortex-induced vibration, while the amplitudes were much higher. The instability was suppressed connecting the cables by wire ropes. The oscillations were attributed to the formation of a water rivulet on the upper windward surface of the cable, which renders aerodynamically unstable its apparent cross section. After this paper a large number of experimental, numerical and analytical works have been published on the subject (Matsumoto *et al.*, 1993; Flamand, 1993; Matsumoto *et al.*, 1995b; Main & Jones, 1999; Matsumoto *et al.*, 2003a,b; Zuo & Jones, 2010).

Another possible mechanism of cable excitation, along with conventional vortex-induced vibration, is due to intermittent two-dimensionalized Kármán vortex shedding at high reduced velocity (Matsumoto *et al.*, 1995b, 2003b). However, in some cases oscillations of inclined cables occurred without rain and could not be simply classified as Kármán vortex-induced vibrations, as observed for instance for the Fred Hartman Bridge, Texas, US, a cable-stayed structure with a main span of 381 m. The driving mechanism of "dry-cable" vibration has not been fully understood yet, nevertheless several important contributions were published during the wind engineering conferences (Matsumoto *et al.*, 2003a,b; Flamand & Boujard, 2009). Larose *et al.* (2003b) and Jakobsen *et al.* (2003) showed the possibility to observe non-symmetric mean pressure distributions on the cable surface in the critical Reynolds number range, depending on inclination and yaw angles; significant variations of lift and drag forces can then occur for small increments of wind speed, a situation that can easily lead to instability. The presence of axial flow cells as well as of low-frequency components in the fluctuating force spectra were also detected and it is argued they may play a role in the limited-amplitude aeroelastic excitation at high reduced velocities. Yeo *et al.* (2007) performed Detached-Eddy Simulations of flow around an yawed and inclined circular cylinder, showing the presence of alternately-developing swirling flow structures which move along the cylinder. These flow structures can produce low-frequency loadings on the cables and therefore they influence the force-generation and wind-excitation mechanism. Finally, Macdonald & Cammelli (2003) noted that the the wind load on the cables significantly influences the total damping of cable-stayed bridge vibrations in the lateral-torsional modes.

3 Evolution of long-span bridge structures

In recent years the development of both science and technology proceeded hand in hand with the need of creating structures of increasing size. The trend is quite clear in the field of bridge engineering, even if the evolution of long bridges has not followed a unique way. Several different solutions have been adopted for the arrangement of the cross section, according to different concepts and ways to face the major challenges due to wind actions.

From the analysis of the evolution of the major achievements of recent years, the cross sections used for the execution of bridge decks of large and very large spans can be subdivided into: 1) truss-girder sections, with high stiffness and poor aerodynamic performances; 2) box-girder sections with high aerodynamic efficiency, and 3) multi-cellular sections with very high aerodynamic performances.

The '60s scenario could still see the Golden Gate Bridge, California, US (with a main span length of 1280 m, completed in 1937), as the longest suspension bridge in the world; at the same time cable-stayed bridges of maximum span of around 400 m were built. In 1964 the Verrazano-Narrows Bridge was completed, a double-decked suspension bridge in New York, which with its 1298 m span became, albeit slightly, the longest bridge in the world, and so remained until 1981, when the Humber Bridge, UK (1410 m), was opened to traffic. The latter plays a key role in the historical development of suspension bridges since it is characterized by an aerodynamic section. Nevertheless, the first example of long-span bridge with this type of section was the bridge over the River Severn, UK, characterized by a main span of 988 m, which had been completed in 1966. The same design concept was then followed

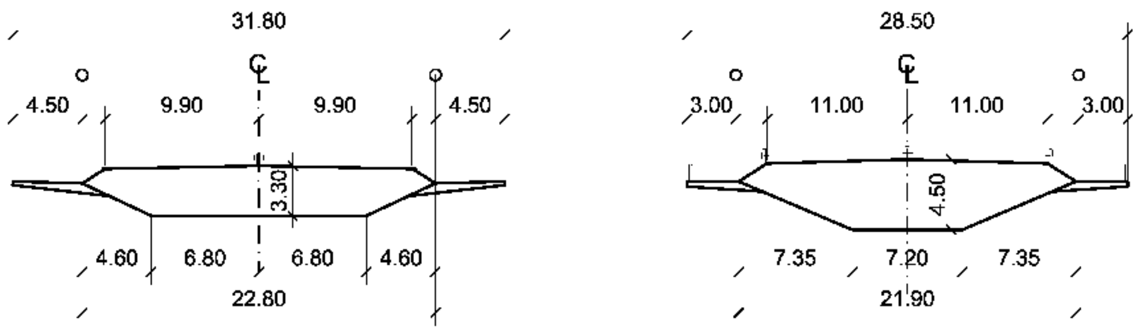


Figure 6: Cross sections of the Severn Bridge (left) and Humber Bridge (right), both in UK.

for the Great Belt East Bridge, Denmark (1624 m), opened in 1998. As reported by Walshe (1965) during the first wind engineering conference held at the National Physical Laboratory, Teddington, in 1963, the design of the Severn Bridge followed two different phases: at first, a truss-girder solution was adopted (and slightly modified during a successive re-design stage; see Figure 3(a)); at the final phase, a shallow steel-plated hollow box section was chosen, where the top surface of the deck formed the roadway (Figures 3(b) and left frame of Figure 6). Initially, for the design of the Humber Bridge the same section as that of the Severn Bridge was adopted; nevertheless, due to the too low torsional stiffness, when the structure was analyzed it was found necessary to increase the depth of the deck section by 50% from 3 m to 4.5 m in order to avoid flutter at the design wind speed, as shown in the right frame of Figure 6 (Walshe & Cowdrey, 1972).

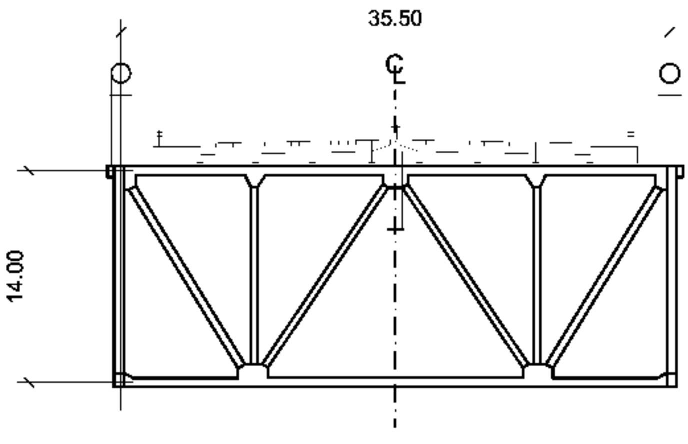


Figure 7: Cross section of the Akashi Kaikyo Bridge, Japan.

In the design of the Akashi Kaikyo Bridge, Japan (1991 m, opened in 1998 and still the world longest bridge), a more classical solution was chosen for the bridge deck: a high-stiffness truss-girder cross section with reduced aerodynamic performances (Figure 7). Already at the preliminary design stage, as reported by Miyata *et al.* (1988) during the 1987 conference in Aachen, it was stated that some truss-stiffened deck configurations guaranteed the required safety level for flutter instability while a box-girder deck solution, unless uneconomic designs were obtained due to excessive thickness of steel members, could be less effective with respect to flutter stability. According to the Authors' opinion,

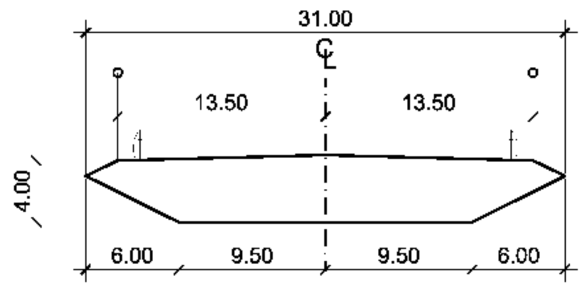


Figure 8: Cross section of the Great Belt East Bridge, Denmark.

in a configuration $L + 2L + L$ (side + central + side spans) the required performance level for flutter stability (critical wind speed higher than 78 m/s) could be reached for a closed-box deck solution only if the midspan was less than 1700 m. Other investigated solutions (such as perforated decks or laterally separated decks) were abandoned because of the too low torsional stiffness, leading either to possible static divergence instability or to a very low value of the frequency ratio, then anticipating coupled flutter. In order to make a 1:100 full-aeroelastic model test possible, a new large boundary layer wind tunnel was built at the Public Works Research Institute (PWRI) in Tsukuba, Japan (Miyata *et al.*, 1992) and the obtained results were shown during the 1995 conference in New Delhi (Miyata *et al.*, 1995), where it was observed that during flutter some modes of vibration were different with respect to the still-air ones.

For the Great Belt East Bridge, a streamlined section was chosen (Figure 8) since the tender project phase, because of lower construction and maintenance costs as well as for the possibility of optimized wind performances by introducing wedge-shaped edge fairings (Larsen & Gimsing, 1992); as many as 16 different trapezoidal box sections were tested in order to appreciate how modifications to the geometry could influence the aerodynamic stability. Several aspects were taken into account by wind tunnel tests (such as aerodynamic stability in service and during erection stages, dynamic wind loads, vortex shedding response of the girder) as reported by Larsen & Jacobsen (1992); the chosen section was also tested on a full-aeroelastic 1:200 scaled model in the 13.6 m wide wind tunnel built at the Danish Maritime Institute (Wagner Smitt & Brinch, 1992). As previously mentioned, due to vortex-shedding oscillations noticed during the welding of the girder, an improved design of the originally proposed guide vanes was tested in the wind tunnel and then installed before the official opening of the bridge. Subsequent inspections and measurements allowed to observe that the bridge no longer suffered from vortex-induced oscillations and that the adopted solution confirmed to be effective (Larsen *et al.*, 1999).

A totally new type of section was instead the one proposed for the Messina Strait Bridge, Italy, a tri-cellular cross section (Figure 9) stiffened by several transverse beams (Cheli *et al.*, 2007); such an innovative solution was also due to the need of very high aerodynamic and aeroelastic performances because of the exceptional span of the bridge (main span 3300 m long). The chosen solution could allow for a strong reduction of the drag force (and then of the static deformation in the horizontal plane), while the improved stability was checked with a vast wind tunnel test campaign.

At the same time, in the field of cable-stayed bridge structures there has been an increase in span lengths, reaching a thousand meters and more in the last three to four years: Sutong Bridge in China, 1088 m long, opened in 2008; Stonecutters Bridge in Hong Kong, China, 1018 m, opened in 2009; Russky Bridge in Russia, 1104 m, opened in 2012, presently the largest cable-stayed bridge in the world (Figure 10). The increase of the spans has been followed by the parallel evolution of

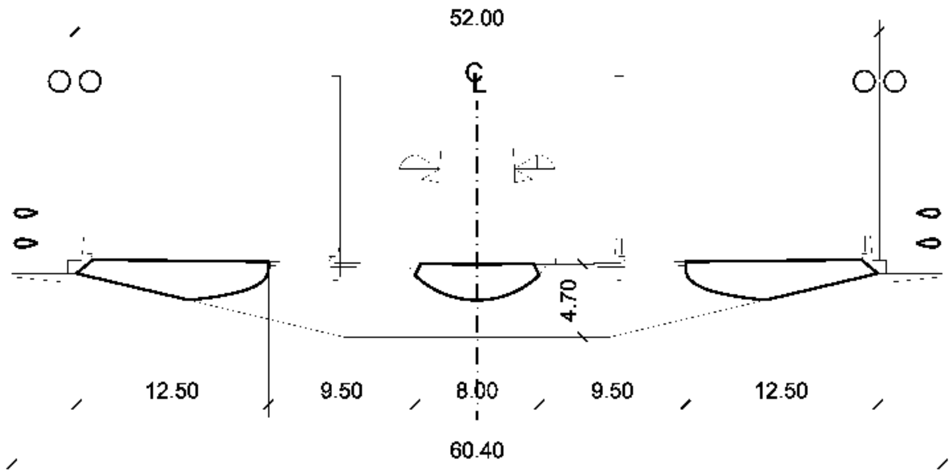


Figure 9: Cross section of the Messina Strait Bridge, Italy (1992 solution).

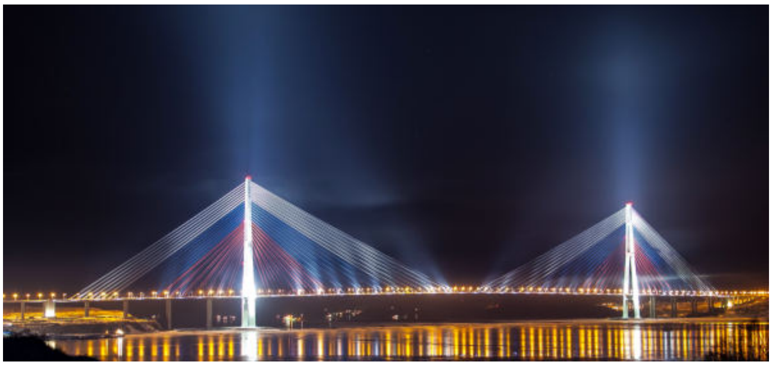


Figure 10: Russky Bridge, Vladivostok, Russia (source: http://en.wikipedia.org/wiki/Russky_Bridge).

the deck cross sections which are almost always optimized from an aerodynamic point of view; as an example, the cross section of the Sutong Bridge (Figure 11) was chosen after various wind tunnel tests aimed at deciding about its stability. In particular, in this case it was observed that both the lateral motion of the deck and cable planes participate to the instability mode (Chen *et al.*, 2005). Nevertheless, a completely different case is represented by the cross section of the Stonecutters Bridge, which presents a twin-girder aerodynamic deck (Figure 12). As a matter of fact, for this bridge a wide clear separation of 14.3 m between the two longitudinal box-girders was adopted and results indicate that the bridge was stable against flutter both during construction and in-service stages (critical 1-minute mean wind speed higher than the design wind speed of 95 m/s, see Larose *et al.* (2003a) and Hui *et al.* (2003)). A twin-girder deck solution was chosen also for the design of the 1545 m long suspended Gwangyang Bridge, South Korea (Kwon *et al.*, 2008), where the cross sectional shape of the main girder was optimized through section model wind tunnel tests at three different scales to maximize the aerodynamic stability and minimize the drag force.

However, in these last 50 years so many important and challenging bridges have been built which are relevant for some of their characteristics, both from the technical and the aesthetic points of view. Just to mention two cases among the others, a reference can be made to the Millau Viaduct, France,

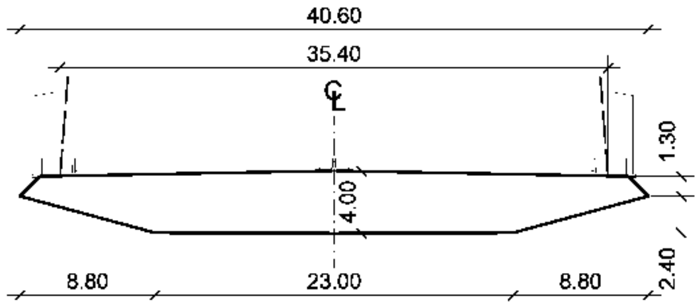


Figure 11: Cross section of the Sutong Bridge, China.

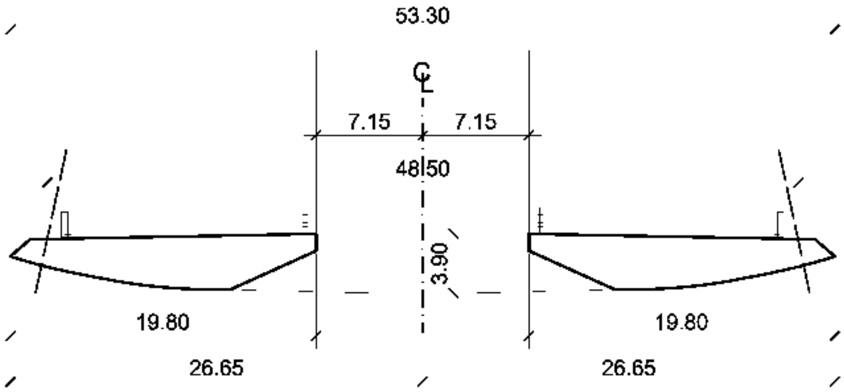


Figure 12: Cross section of the Stonecutters Bridge in Hong Kong, China.

and the Aizhai Bridge, China. The Millau Viaduct (Figure 13(a)) is the tallest bridge in the world, being the major masts as high as 340 m, that is more than the Eiffel Tower. Opened in 2004, it is a multi-span cable-stayed bridge, including six 342 m long central spans and two 204 m long lateral spans. The box-girder deck section was chosen after a vast experimental campaign on section models at CSTB, Nantes, France. A taut-strip model was also realized to determine the response of the bridge to turbulent wind. Nevertheless, it was of particular concern the behaviour of the structure during the construction phase and therefore a 1:300 full-aeroelastic model was tested and Barré *et al.* (1999) showed the reliability of this kind of tests even in complex orographic situations. The Aizhai Bridge (Figure 13(b)) is another spectacular example, built near Jishou, Hunan, China, and opened in 2012. The main span is 1146 m long and it is probably the highest suspension bridge of the world, being placed 336 m above the underneath valley.

4 Wind tunnel facilities for bridge aerodynamics

With increasing pressure to improve infrastructures throughout the world, bridge construction has constantly experienced engineering challenges over the last 50 years: technological developments and refinements in processes, materials and modeling allow bridges to be constructed longer and lighter than previously attempted. This has determined the increased need of bridge aerodynamics testing facilities through a plethora of wind tunnel laboratories. It is almost impossible to mention all of them

(a) (b)

Figure 13: (a) Millau Viaduct, France, (source: http://www.myconfinedspace.com) and (b) Aizhai Bridge, China.

in this brief chapter, since its number and features have grown amazingly over the last 20 years: one will necessarily refer here to a few ones in the following.

Modern methods of aerodynamic testing of bridge models were first set up by Farquharson and Vincent at the University of Washington Aeronautical Laboratory (UWAL), US, after the failure of the Tacoma Narrows Bridge. In 1941 the routinely military and aeronautical activity of the laboratory was interrupted to perform experiments to understand the reason of the bridge collapse occurred few months before. Later, a special new wind tunnel was built to test full-aeroelastic models of the original and the new Tacoma Bridge.

One of the first laboratories dealing with bridge aerodynamics was the National Physical Laboratory (NPL), in Teddington, UK, opened in 1900. Since the very beginning one of the areas of research of the laboratory was in the magnitude and distribution of wind forces on structures such as bridges and roofs. In 1919, under the superintendence of Sir Thomas Stanton, the Duplex wind tunnel was built, with a test section of 2 m by 4 m. In 1946 Frazer and Scruton started aerodynamic investigations primarily concerned with the design of the proposed Firth of Forth Road Bridge, UK, and Severn Bridge, UK (Figure 3). The latter, as remarked in the previous chapter, was a revolutionary structure in the history of bridge engineering and a temporary large wind tunnel was even built to perform tests on a 1:100 full-aeroelastic model (Figure 3(a)). Very important are also the sectional model tests for different designs of the Humber Bridge, UK. At NPL other famous structures were tested such as the Tamar Bridge and the first proposed design of the Runcorn-Widnes Crossing, both in UK.

Founded in 1965 by Alan G. Davenport, the Boundary Layer Wind Tunnel Laboratory (nowadays: the A. G. Davenport Wind Engineering Group) at the University of Western Ontario, London, Canada, belongs to longest tradition labs in bridge aerodynamics. The second generation wind tunnel was constructed in 1984 and includes three test sections. The low-speed test section is particularly well suited to Froude-number scaling. Application examples include a wide range of full-aeroelastic studies of long span bridges; it is also used for topographic studies and the wave tank can be em-

Figure 14: Alan G. Davenport (right) and J. Peter C. King (left) in the wind tunnel of the University of Western Ontario Laboratory, London, Canada, behind the 1:400 full-aeroelastic model of the Tsing Ma Bridge, Hong Kong, China (courtesy of the University of Western Ontario Laboratory).

ployed to investigate the interaction of wind and waves with offshore structures. Amongst the major projects dealt with by the University of Western Ontario laboratory are: Shenzhen Western Crossing, Tsing Ma (Figure 14) and Stonecutters Bridge (Hong Kong, China), Paso del Alamillo Bridge (Spain), Great Belt East Bridge (Denmark), Busan Geoje Grand Bridge and Kwang Ahn Grand Bridge (South Korea), Sungai Johor Bridge (Malaysia), Sunshine Skyway Bridge (Florida, US), Helgelands Bridge and Askoy Bridge (Norway), Confederation Bridge (Prince Edward Island, Canada).

CSTB Nantes, France, started in the '70s and developed later climatic wind tunnel facilities (named Jules Verne), designed to carry out also full-scale tests combining wind and other meteorological events (rain, sand, sun, temperature, snow, etc). CSTB Nantes has also two atmospheric boundary layer wind tunnels allowing reduced-scale reproduction of the wind established in the site of the test structure. Amongst major bridge aerodynamics projects dealt with at CSTB are worth mentioning: Millau Viaduct, Pont de Normandie, Iroise Bridge, Saint-Nazaire Bridge (France), or Rion Antirion Bridge (Greece), plus several footbridges (like the Simon-de-Beauvoir and the Léopold-Sédar-Senghor footbridges, Paris, France).

FORCE Technology, Division for Maritime Industry (formerly: Danish Maritime Institute, DMI), Hydro- and Aerodynamics Department, Lyngby, Denmark, has developed more than 30 years experience in aerodynamic investigations of bridges. Amongst the largest bridges in the world, the following structures were tested there: Stonecutters Bridge (Hong Kong, China), Messina Strait Crossing (Italy), Sutong Bridge (China), Pont de Normandie (France), Höga Kusten (Sweden), Great Belt East Bridge and Öresund Bridge (Denmark). As a concrete evidence of this large expertise, two dedicated conferences on Large Bridge Aerodynamics were organized in Copenhagen (1992 and 1998), the first one on the occasion of the inauguration of the new M. Jensen Wind Tunnel (and of the starting of the Great Belt crossing project).

Another historical wind tunnel is that of the Department of Civil Engineering at the University of Tokyo, Japan, a large facility (16.0 m $\times$ 1.9 m) adequate to test full-aeroelastic models of suspension bridges (Figure 5).

The wind tunnel built in Tsukuba, Japan, by the Honshu-Shikoku Bridge Authority (HSBA) served as a huge facility for testing several bridges which were built in this area of Japan. The dimensions of

Figure 15: Wind tunnel in the Department of Civil and Earth Resources Engineering, Kyoto University, Japan (courtesy of Prof. Masaru Matsumoto and Prof. Hiromichi Shirato).

the wind tunnel (41 m $\times$ 4 m $\times$ 30 m, width $\times$ height $\times$ length) were defined after the wind-tunnel test specifications for the Akashi Kaikyo Bridge were issued by the HSBA in 1990. A linear scale of 1:100 or more was chosen as desirable for testing the new bridge, thus implying at least a 40 m wide wind tunnel; as no wind tunnel of such a large dimension was available in Japan at that time, it was decided to build a new one. The new laboratory was constructed in the campus of the Public Works Research Institute, the Ministry of Construction, as a collaborative project with the Honshu-Shikoku Bridge Authority. In the wind tunnel, not only the Akashi Kaikyo bridge but also other bridges have been tested: among these, the three Kurushima Kaikyo Bridges and the Tatara Bridge (at that time, with a main span of 890 m, the world's longest cable-stayed bridge) can be recalled (Fujino *et al.*, 2012).

The research on bridge aerodynamics was started in the Bridge Engineering Laboratory at Kyoto University, Japan, around the '60s under the direction of late Professor Ichiro Konishi. Theoretical studies on aerodynamics and wind tunnel tests for many long-span bridges have been conducted constantly in the laboratory, also under Professor Naruhito Shiraishi. Particular research interest not only on applications to practical subjects but also on the fundamentals of bluff body aerodynamics bloomed when Professor Masaru Matsumoto was leading the laboratory. Since 2008 Professor Hiromichi Shirato and Professor Tomomi Yagi are promoting the research activity.

The State Key Laboratory for Disaster Reduction in Civil Engineering at Tongji University, Shanghai, China, presents several wind tunnels of different dimensions. The largest one is 15 m $\times$ 2 m $\times$ 14 m (width $\times$ height $\times$ length). Many large Chinese bridges were tested there, such as the Yadagawa Bridge, the Lupu Bridge, the East Sea Bridge and the Xihoumen Bridge (Ge & Xiang, 2008).

The Rowan Williams Davies & Irwin Inc. (RWDI), managing four wind tunnels, two in Guelph, Ontario, Canada, one in London, UK, and one in Miami, Florida, US, is specialized in wind-structure interactions: from long-span vehicular to pedestrian bridges, from new structures to bridge renovations. Amongst recent projects in bridge aerodynamics the following can be mentioned: Metsovitikos Bridge (Greece), Orleans Island Suspension Bridge (Canada), Cooper River Bridge Charleston (South Carolina, US), Ironton-Russell Bridge (Kentucky, US) and Second Severn Crossing (UK).

The Institute for Aerospace Research at the National Research Council (NRC-IAR), Canada, dispose of six wind tunnels and in particular of a 9 m $\times$ 9 m facility. Several section and full-aeroelastic models of large bridges were tested there and in particular the existing and proposed Tacoma Narrows Bridges, studying also the potential wind interference effects between the two structures, and the Stonecutters Bridge. Important researches on full-scale models of stay cables were also performed.

The largest wind tunnel in the Southern hemisphere, very active in wind engineering problems, is that at Monash University, Australia, pioneered by Professor William H. Melbourne. Structures such as the West Gate Bridge and the Bolte Bridge, both in Australia, were tested there.

The CIRIVE (Inter-departmental Research Centre on Wind Engineering) at Politecnico di Milano, Italy, has opened in 2002 a large wind tunnel facility, with a test section 14 m wide and 4 m high, where several experimental campaigns were carried out for the Messina Strait Crossing, a project which presently seems to be canceled and will not be realized (at least in an immediate future). At present, this laboratory is involved in many projects of large bridges around the world, in particular with tests on full-aeroelastic models.

Although not as resounding as those previously listed, additional important contributions in bridge aerodynamics have come from several small laboratories, where significant experimental and theoretical work has been performed. Among these, the Authors cannot avoid to mention the CRIACIV (Inter-university Research Centre on Building Aerodynamics and Wind Engineering) Boundary Layer Wind Tunnel in Prato, Italy, which has been involved in bridge aerodynamics for 15 years.

5 Conclusions

The proposed review clearly shows that in the last 50 years bridge aerodynamics has been a very flourishing branch of wind engineering, which received a significant boost from the many challenging projects of long-span bridges around the world and gave important contributions also to basic research in bluff body aerodynamics and aeroelasticity. As noted by Scruton (1965a), if "the Tay Bridge [UK] disaster in 1879 is reputed to have prompted bridge engineers to make allowance for wind loading", "the present day interest in dynamic effects started in 1940 with the collapse of the suspension bridge over the Tacoma Narrows due to oscillations set up by wind". Scruton (1965a) also remarked that many bridge structures had been destroyed or damaged by wind and this may account for the unpopularity of long-span suspension bridges in Europe in the first half of last century. Nevertheless, the progress in the scientific knowledge due to research quickly allowed to overcome this reticence and then bridge aerodynamics itself has found nourishment in the studies for large futuristic structures, most of which have been realized in Europe. The large project to connect Honshu and Shikoku islands in Japan was initiated around the '60s and this produced an acceleration in the development of bridge aerodynamics, the connecting route consisting of many long-span bridges for which the design against wind loading and aeroelastic phenomena was one of the major concerns. However, since the beginning of the years 2000 China has taken the leadership in the realization of an impressive series of long-span bridges in less than ten years (Ge & Xiang, 2008). Also, a key role in the development of bridge aerodynamics has been played by large- and small-scale wind tunnel facilities which have appeared all over the world in the last decades. Nowadays the synergy between experiments and computational fluid dynamics simulations offers new interesting possibilities and likely will lead to significant scientific progress. Fundamental for the development of the state of the art have been the international and regional conferences that, thanks to the support of the International Association for Wind Engineering, have been periodically held after 1963 and wherein studies on bridge structures have always represented a centerpiece. A few symposia only devoted to bridge aerodynamics were also organized and particularly worth mentioning are those held in Copenhagen.

A few trends in the last 50-years research on wind effects on bridges can be highlighted. One might be the evolution from linear to nonlinear approaches to the main phenomena of interest. In fact, if the need to apply nonlinear models appeared very soon for problems such as galloping and vortex-induced vibration, later also for buffeting and flutter the researchers tried to account for effects such as the dependence on the mean angle of attack and the amplitude of vibration or for the nonlinear superposition of self-excited forces and loads due to oncoming wind turbulence. Another tendency seems to be the growing interest in the effect of Reynolds number on aeroelastic phenomena, in particular after some research works (e.g. Schewe & Larsen (1998)) have shown that this can play a significant role also in case of sharp-edged bluff body geometries. Since Reynolds number values easily larger than 10^8 (based on the deck width) can be of interest for large bridges, the mismatch with common wind tunnel tests is often of at least two orders of magnitude and this is nowadays of great concern for the experimenter. More and more efforts in the research have been devoted to the improvement of identification methods of aerodynamic/aeroelastic parameters and to the use of reliability-based approaches for the analysis of wind-excited bridges. Furthermore, a topic that has captured the attention of the researchers in the last 25 years and that has registered great progress is wind-related cable dynamics, mainly because the increase in the span of cable-stayed bridges and consequently in the length of inclined cables has made very urgent the problem of their possible vibration.

As for wind tunnel experimental techniques, it may be noted that section model tests are still the most common method of investigation, although a certain skepticism on their representativeness in the '60s and '70s may have let thinking they would have progressively been replaced by other types of experiments. By contrast, the fact they are simple, economic and of easy interpretation and that they offer the chance to obtain larger model scales and easily perform parametric studies, has determined their success hitherto, although for very important structures and critical cases the final validation with full-aeroelastic models is absolutely necessary.

Concerning the design of large bridges, it may be noted that after the Tacoma Narrows Bridge disaster the trend has been from truss-stiffened to quasi-streamlined box-girder decks but recently multiple-box cross section geometries clearly appeared to be the most efficient solution for very long span bridges. Indeed, the endeavour to realize the dream of overpassing the 3000 m threshold of main span through a multiple-box deck solution, although very close to succeed, has not been accomplished yet. This appears to be the immediate next challenge for the bridge engineering technology, which could be faced and solved very soon.

Finally, many important issues in bridge aerodynamics are still today open problems (more reliable models of vortex-induced vibration, a reformulation of self-excited forces for flutter and buffeting analyses or a better understanding of wind-excitation of inclined cables are needed, just to mention a few examples) and they are supposed to become more and more pressing due to the present increase in the dimension, flexibility and lightness of modern bridge structures, so that in the near future a strong research effort in this branch of wind engineering is still needed and breakthrough studies are expected.

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