Historical view of long-span bridge aerodynamics

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Abstract

Some features of bridge aerodynamics of wind engineering are reviewed at this time, in view of the collapse of the Tacoma Narrows Bridge and also to celebrate Professor Alan G. Davenport’s 40 years of contribution to the field. Primary highlighted topics are: description of motion-dependent forces for flutter instability, presentation of gust responses caused by turbulent winds and suppression problems of wind-induced vibrations.

Keywords: Long-span bridge aerodynamics; Suspension bridge; Cable-stayed bridge; Flutter instability; Gust response; Wind-induced vibration; Vibration control; Historical view

1. Introduction

Going for the record in stretching the spans of bridges much farther, wind effects, particularly coupled flutter instability on those flexible structures provided with specified properties and configurations must be intensively investigated. Contemporary procedures for the wind-resistant design of long-span bridges started around late 1950s and early 1960s, keeping step with expanded economical trend and project demands to construct many of infrastructures in developed countries after the 2nd world war. There should also be another factor of technical preparations to push the trend. There are, first of all, a couple of research works associated with the specified phenomenon of torsional flutter which arose after the collapse of the Tacoma Narrows Br. Furthermore, there are some attempts at including sophisticated ideas to model the effects of natural wind fluctuations on gust responses and also attempt at formulating motion-dependent forces applicable to flutter onset.

Some historical views of long-span bridge aerodynamics will be presented comprehensively in this contribution, from early stage studies after the Tacoma
Narrows Br. to the recent constructions of long-span suspension bridges and cable-stayed bridges. The highlighted topics are flutter instability, buffeting or gust responses and vibration control.

2. Flutter instability

2.1. Early stage of wind effects study and design

In 19th and early 20th centuries, a considerable number of medium-span suspension bridges were constructed. Many of these displayed instability in the wind, and there were collapses also. The causes, as well as physical backgrounds, are not disclosed in the historical records. At any rate, it can safely be said that further development of suspension bridge structures to cross longer spans was achieved by using parallel wire and its air spinning method. The introduction of a design procedure based on the deflection theory was also quite essential. However, installation of thin plate–girder deck resulted in the collapse of the Tacoma Narrows Br., and the reason for this is a lack of knowledge on necessary stiffness for the deck and instability in the wind. The fact that had to wait for the future discoveries in this point was that the mechanism of torsional flutter lead to the collapse, and this was caused by dynamic vortex separation from the windward edges of the stiffening girder and this descended along the floor web, always synchronized with torsional motion, as shown in Fig. 1(b) [1].

Since this accident, the importance of wind-resistant design for long-span suspension bridges has been highly recognized and has led to many research works and investigations on bridge aerodynamics. Farquharson et al. [2] conducted a series of wind tunnel experiments on the Tacoma Narrows Br. Bleich [3] made analytical

![Fig. 1. The collapse of the Tacoma Narrows Bridge, (a) torsional motion led to collapse, (b) separated vortex descending by flow visualization [1](a)](image)

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studies on the flutter problem. In them they developed the instability analysis using motion-dependent forces on a thin airfoil given by Theodorsen [4] in the field of aeronautics. Theodorsen’s formulas are written for the form of harmonic motion as

\[
L = -2\pi \rho b V^2 \left\{ f_1 \left( \phi + \frac{b}{V} \eta \right) + f_2 \frac{b}{2V} \phi \right\},
\]

\[
M = \pi \rho b^2 V^2 \left\{ f_1 \left( \phi + \frac{b}{V} \eta \right) - f_3 \frac{b}{2V} \phi \right\},
\]

where \( f_1-f_3 \) are theoretically derived functions of reduced frequency \( k = \omega b / V \), and \( \eta, \phi \) are non-dimensional vertical heaving and torsion, respectively. In this connection, Pugsley [5] suggested a model, that might provide a framework for the introduction of measured aerodynamic forces into the bridge flutter problem.

Those research works led thereafter to the development of long-span suspension bridge projects, for instance, the Forth Br. (truss deck; completed in 1964) and the Severn Br. (shallow box deck; 1966) in UK on which many wind tunnel tests have been carried out by Scruton et al. [6]. An earlier project to construct a long-span suspension bridge, before the Honshu–Shikoku bridges in Japan, had just started around the same time. With special reference to their first case of the Wakato Br. (truss deck; 1962), Hirai and Okauchi [7] developed empirical formulation to describe the motion-dependent moment force on such actual stiffening decks as various trusses and a shallow box (Fig. 2).

Fig. 2. Earlier wind tunnel measurement of the motion-dependent force [7], (a) shallow truss girder deck, (b) shallow box girder deck for the Wakato Bridge.
The moment force comes from the torsional motion of the sectional model in the wind tunnel tests, as follows:

\[ M = \frac{\pi}{4} \rho Ab V^2 (\alpha + i\beta) \phi, \]  

where \( \alpha \) and \( \beta \) are functions of reduced frequency \( k = \omega b / V \), \( \phi \) is harmonic torsional motion and \( i = \sqrt{-1} \). Inapplicability of airfoil aerodynamics for the actual bridge decks has been recognized and discussed since such earlier stage. Selberg [8] introduced an empirical formulation to describe the actual aeroelastic flutter behavior of suspension bridges by simplifying airfoil aerodynamic forces.

Actual prototype for stiffening decks took a variety of discussions and investigations from structural as well as aerodynamic viewpoints to get better performance in each project. Looking at long-span suspension bridges over 1000 m span, the traditional truss stiffening deck was developed from earlier rigid designs of the George Washington Br. (non-stiffening deck completed in 1931 and original double truss deck in 1962), the Golden Gate Br. (1937) and the Verrazano Narrows Br. (1964) to the lightweight designs of the Tacoma Narrows Br., the Mackinac Straits Br. (1958) in USA, the Forth Br. in UK and the Tagus River Br. (1966) in Portugal. The progress to use a shallow box deck is recognized in the proposal by Leonhardt [9] of a mono-cable suspension bridge, with inclined hanger for the design competition (1960) of the Tagus River Br. The investigation for the Severn Br. design in NPL [10] was aimed at developing from even light truss deck an innovative streamlined box deck with thin fin-shaped wings for sidewalk at both sides.

2.2. Development of motion-dependent force formulation

The description of those motion-dependent forces, used even at present, has been formulated around the middle of 1960s. Ukeguchi et al. [11] proposed, after Hirai’s formulation, an aeronautical type of description of both lift and moment so as to extend to vertical heaving and torsion coupled flutter. Their aerodynamic force coefficients are obtained from the experimental measurement on sectional bridge decks in the forced harmonic motion. At the same time, Scanlan and Sabzevari [12] gave another expression of motion-dependent lift and moment forces, named aerodynamic derivatives later [13]:

\[ \frac{L}{m} = H_1 \dot{h} + H_2 \dot{x} + H_3 x, \]
\[ \frac{M}{mr^2} = A_1 \dot{h} + A_2 \dot{x} + A_3 x, \]  

where \( h \) and \( x \) are heaving and torsional motions. This expression has been utilized afterwards throughout many occasions of research works. Tanaka [14] also discussed...
similar formulation in aeronautical type, different from the above Scanlan’s notation as

\[
L = \pi \rho b^2 \omega^2 \left( L_{yR} \dot{y} + L_{yl} \frac{\dot{y}}{\omega} + L_{\phi R} \dot{\phi} + L_{\phi l} \frac{\dot{\phi}}{\omega} \right),
\]

\[
M = \pi \rho b^4 \omega^2 \left( M_{yR} \dot{y} + M_{yl} \frac{\dot{y}}{\omega} + M_{\phi R} \dot{\phi} + M_{\phi l} \frac{\dot{\phi}}{\omega} \right),
\]

(4)

where \(y\) and \(\phi\) are heaving and torsional harmonic motions. This formulation has taken an active role in the flutter verification for the Honshu–Shikoku Bridges.

Afterwards considering the case for a full-span bridge, Scanlan [15] developed formulation of two degrees of heaving and torsion to extend to three degree’s including sway motion and along wind drag component

\[
L_{ae} = \frac{1}{2} \rho U^2 B \left[ 2K_h^* \frac{\dot{h}}{U} + k H^*_2 \frac{\dot{h}}{U} + H^*_3 \alpha + H^*_4 \frac{h}{B} \right],
\]

\[
M_{ae} = \frac{1}{2} \rho U^2 B^2 \left[ K_A^* \frac{\dot{h}}{U} + K_A^* \frac{\dot{h}}{U} + K_A^* \alpha + K_A^* \frac{h}{B} \right],
\]

\[
D_{ae} = \frac{1}{2} \rho U^2 B \left[ K_P^* \frac{\dot{h}}{U} + K_P^* \frac{\dot{h}}{U} + K_P^* \alpha + K_P^* \frac{h}{B} \right],
\]

(5)

where \(h\), \(\alpha\) and \(p\) are heaving, torsion and sway motions, respectively and \(L\), \(M\) and \(D\) are lift, pitching moment and drag forces, respectively. The above system of equations involving 12 aerodynamic derivatives was extended to one containing 18 aerodynamic derivatives [16]. Such an ideal type of complete expression relevant to three motion and force components has been firstly thought to have no appropriate example. However, it succeeded in explaining aerodynamic damping or exciting behavior (described later on) of the complicated coupled flutter instability as shown.

Fig. 3. 3D flutter onset in full-scale model of the Akashi Kaikyo Bridge [17]: (a) deformed by wind loads in \(\frac{1}{100}\) full-scaled aeroelastic model, (b) time history response at mid-point of center span (8.4 m/s).
in Fig. 3, which took place in the wind tunnel test [17] of a \(\frac{1}{100}\) scaled full aeroelastic bridge model for the Akashi Kaikyo Br. The figure shows a response record during coupled vibration of three degrees under large lateral deflection and windward-down rotation at the center by wind loading.

### 2.3. Three-dimensional analysis of coupled flutter

By using experimentally identified motion-dependent forces in a reasonable fashion, flutter instability can be developed in the form of complex eigenvalue analysis to get critical wind speed, aerodynamic damping or exciting and flutter mode shape and its frequency. Original analysis of flutter instability can be cited in Bliech’s works based on the simplified idea of modal analysis, as mentioned earlier. Following such procedures, Miyata and Yamada [18] started to establish a large size three-dimensional FEM model of the suspension bridge to combine all the structural and aerodynamic properties and variables and bring it to complex eigenvalue analysis. That was an appropriate tool to verify the aerodynamic stability of a long-span suspension bridge, the Akashi Kaikyo Br. just underway at that time.

Assuming a three-dimensional structural frame model of the entire bridge structure deformed under specified wind loads, the governing equation of motion can be described as follows:

\[
\mathbf{M}\ddot{\mathbf{u}} + \mathbf{C}\dot{\mathbf{u}} + \mathbf{Ku} = \mathbf{F}_v\dot{\mathbf{u}} + \mathbf{F}_d\mathbf{u}
\]

where \(\mathbf{u}\) is the deformed displacement vector, \(\mathbf{M}\) the deformed mass matrix, \(\mathbf{C}\) the deformed structural damping matrix, \(\mathbf{K}\) the deformed stiffness matrix, \(\mathbf{F}_v\) the motion-dependent aerodynamic force coefficient matrix proportional to velocity of motion and \(\mathbf{F}_d\) the proportional to displacement of motion.

Those aerodynamic coefficients \(\mathbf{F}_v\) and \(\mathbf{F}_d\) are defined as functions of reduced frequency \(k = \omega b/V\) and given in two degrees or three degrees according to specified case as follows [17]:

\[
\mathbf{F} = \begin{bmatrix}
L_s(\mathbf{u}) & L_s(\mathbf{f}) & L_s(\mathbf{v}) & 0 & 0 & 0 \\
M_s(\mathbf{u}) & M_s(\mathbf{f}) & M_s(\mathbf{v}) & 0 & 0 & 0 \\
D_s(\mathbf{u}) & D_s(\mathbf{f}) & D_s(\mathbf{v}) & 0 & 0 & 0 \\
0 & 0 & 0 & L_c(\mathbf{u}) & 0 & 0 \\
0 & 0 & 0 & 0 & D_c(\mathbf{v}) & 0 \\
0 & 0 & 0 & 0 & 0 & D_h(\mathbf{v})
\end{bmatrix}
\]

where \(L, M\) and \(D\) are lift, pitching moment and drag force coefficients, respectively, \(u, f\) and \(v\) are heaving, torsion and sway displacements, respectively, and the subscripts \(s, c, h\) mean stiffening girder, main cable and hanger cable, respectively. The aerodynamic forces on the stiffening deck are described in the form of 18 coefficient components.
Complex eigenvalue analysis was carried out by assuming the reduced frequency over an appropriate range concerned, making a convergence calculation to coincide the critical wind speed of flutter with a specified wind speed to give the displacements of the entire structure by wind loads. Fig. 4 [17] presents one of those measured results in a full aeroelastic model study for the Akashi Kaikyo Br. to compare with calculated ones by using the method above. Inclusion of the effects of sway motion and drag force is found out to be quite effective in describing measured aerodynamic behavior of damping or exciting motions.

Those realistic applications of three-dimensional flutter analysis method were effectively used for the better estimation of aerodynamic stability of the forthcoming projects of very long-span suspension bridge such as Messina Crossing Br. Many relevant factors and variables involved in the analysis procedures, of course, should be identified and prepared reasonably in advance.

3. Effects of wind fluctuation

3.1. Early stage of buffeting study

The sophisticated work in three series by Davenport [19–21] made wide and strong impact on researchers and engineers of those days who were interested in specified wind-resistant design and problems of long-span bridges as well as many other structures. The papers treated probabilistic behaviors of natural winds to include well the effects of wind fluctuation on gust responses of flexible structures. With those effects on long-span bridges, some of the gust responses have been partially recognized in earlier observations and measurements on full-scale bridges [2,22,23]. Buffeting, or random response, can be found in Liepman [24] and Scruton [22] before they were expanded in Davenport’s work [21].

Appropriate inclusion of gusty wind effects has succeeded in using statistical procedures and skillful ideas of random vibration, which have been completed in the
form of spectral analysis for stochastic process around late 1950s after pre- and post-war era. With the theoretical treatments on stochastic variables and stationary random series, intensive works by Rice [25], Blackman [26] and others can be cited. On the other hand, those papers and books by Eringen [27], Crandall [28], Robson [29] and others must be effective to describe and treat random vibrations in the form of spectral analysis. The understanding of the characteristics of natural winds and their effects on the structures has also been developed in a comparison of the mean pressures on small buildings in full-scale and in wind tunnel models undertaken by Jensen [30] and others.

The necessary starting point for those new understandings appear, afterwards, to be further full-scale studies. However, in the following decade, only a few studies of full-scale aeroelastic wind tunnel models have been carried out to investigate realistic gust responses inevitable to long-span suspension bridges. Hirai et al. [31] conducted a small grid turbulence test in their 8 m-wide wind tunnel. Davenport constructed a boundary layer wind tunnel in the middle of 1960s and investigated wind-induced responses on the Halifax Narrows Br. [32] in a well-simulated turbulent flow. Paying special attention to building the aeroelastic full-scale models of long-span bridges in a somewhat simplified manner, Davenport et al. [32] carried out a couple of full-scale aeroelastic studies in the boundary layer flows on the Tacoma Narrows Br., the Golden Gate Br. and an ideal flat-plate model. That was named a taut strip model, and could simulate appropriate set of mode shapes. Fig. 5 presents some comparisons of those experimental responses in different low and high turbulence levels for torsional flutter as well as buffeting. The success in processing such data in random response tests can be entirely attributed to the availability of digital computing procedures of on-line PDP-11 system and other systems.

It is quite well known that since 1970s many developments have occurred in connection with problems of Wind Effects on Buildings and Structures, which came to be called as Wind Engineering later. Works to put those results and understandings of fluctuating winds, gust responses and others together in the form of code practices also started, for instance, in the National Building Code of Canada.

![Fig. 5. Dynamic response of “taut strip” models in different flow regimes [32]. (a) “Tacoma” H-section model, (b) “Golden Gate” model.](image-url)
or even for the Wind-resistant Bridge Design Code and the Manual of the Honshu–Shikoku Bridges in Japan [34]. The effects of fluctuating winds on the Honshu–Shikoku bridges were principally introduced through correction coefficients and they added effects of gust responses to mean wind loads. Regarding the onset of flutter, incidentally, the effect of low-frequency part of fluctuation was quasi-statically taken into the correction factor for its critical wind speed, for instance, 1.08 for the Akashi Kaikyo Br. of 1991 m long center-span, under a 10% level of turbulence intensity.

Meanwhile, calculating the gust responses for a specified bridge by spectral analysis method proposed at that time, has been considered quite general and conventional approach to follow, for a long time it was used as explained below. Firstly, the structural information of natural frequencies and mode shapes have to be prepared in advance by an appropriate eigenvalue analysis for a specified single-degree freedom motion. Measuring those data of mean static force coefficients in wind tunnel tests, the factors and variables for fluctuating forces are to be given by quasi-steady approaches for determining the aerodynamic admittance as a transfer function and the aerodynamic damping. Assuming appropriate input data of natural winds in the spectral form of wind speed, the decay factor of spatial correlation, and the intensity of turbulence, finally, the root mean square values of response can be derived to estimate peak responses and other variables.

### 3.2. Three-dimensional analysis of gust response

Davenport’s seminal work is nearly 40 years passed, and the basic frame of calculation remains the same as of Davenport [21]. In this sense, the influence of his works is very essential in the construction of many prototype structures. However, as many concerned researchers agree, several basic problems still remain unsolved. For instance, more strict determination is required in connection with associated parameters like coherent characteristics of gusty wind, formulation of fluctuating forces caused by turbulent winds, sort of transfer function, or aerodynamic admittance between fluctuating wind and force. Formulation of the specified motion of the structural system concerned should also be contrived so as to estimate more realistic behaviors.

Observation in the $\frac{1}{100}$ full-scaled aeroelastic wind tunnel model for the Akashi Kaikyo Br. [17,35,36] is a good and instructive example and suggests that it is very likely to arrive at realistic behaviors from models of long-span suspension bridges under turbulent winds. With the rise of wind speed, the truss girder largely deflected, swayed and twisted in windward-down way, or negative to the wind at the center by wind loads. The twisted girder means the occurrence of an additional relative angle of the girder to the mean wind flow, which really varied along the bridge axis. Those gust responses observed under prescribed turbulent flows are quite random and coupled in a three-dimensional way, sometimes showing snake-like motions of a sort of wave propagation, under statically deflected and twisted displacements as mentioned. With comparison among a couple of gust responses observed in the tests, actually, horizontal responses were smaller than calculated by conventional spectral
analysis for individual motion, while vertical and torsional responses were larger [36]. The reason why those responses could not be estimated so correctly were that relatively better circumstances created in the wind tunnel test, may be caused by their complicated, rather three-dimensionally coupled fashion, and this suggests the need of a more sophisticated approach.

In the past conventional treatments of gust response analysis, as mentioned above, modal analysis techniques have been developed to use a set of mechanical modes determined without wind action. It is quite obvious that those mechanical modes are not the actual ones of the bridge structure vibrating in wind flows because of aeroelastic effects caused by motion-dependent forces. In this regard, the approach using motion-dependent forces for flutter analysis [18] can provide a better representation of wind-induced responses of long-span bridges in term of complex modes. Minh et al. [37] developed a complex-mode buffeting analysis method by integrating the structural system of the three-dimensional FEM model of a full bridge and all the three-dimensional set of static and motion-dependent forces in wind, where the assumption for aerodynamic damping is not needed to be specified. These complex modes can be herein named aeroelastic complex-modes to indicate the fact that all aeroelastic effects are inherently incorporated. The modal decomposition then can be made straightforward by a complex modal analysis scheme. A comparative consideration in the time domain formulation could result in a better fit with those observed data of the full-scale aeroelastic wind tunnel model tests for the Akashi Kaikyo Br., by assuming reasonable fitting for the spatial coherence of fluctuating wind speed measured in the wind tunnel. It can also follow a characteristic behavior of frequency evolution in each component among coupled motions with the change of mean wind speed.

Since the completion of the Akashi Kaikyo Br. in the spring of 1998, some field measurements have been conducted to verify the structural and other performances expected for the actual bridge in a variety of responses, to earthquakes, strong winds, traffic vehicles and others. Fig. 6 presents one of those response records of the stiffening girder at the midpoint measured when a typhoon passed with the wind acting nearly perpendicular to the bridge axis for 10 or 20 min on an average. As for the sway displacement and motion, it is quite significant that a sort of long-term, rather than shorter averaging time, trend of fluctuation is accompanied with random vibration of the lowest frequency of sway motion. This fact may be caused by the change of wind loads due to wind speed specified from time to time. On the other hand, the vertical response seems relatively stable, with constant mean.

Accumulation of those data in full-scale measurements will be really effective to identify more accurate factors and variables associated with gust responses of long-span bridges. Comparative studies in the topics concerned is also necessary.

4. Suppression of wind-induced vibrations

In the cases of very long-span suspension bridges, flutter instability, even torsional flutter or coupled flutter in case of the Tacoma Narrows Br. should be investigated.
However, the vortex-induced vibrations on bluff box decks of medium-span bridges, the so-called rain–wind vibrations of stay cables of long-span cable-stayed bridges and others, are sometimes recognized to result in the need to be suppressed after the completion or at the final stage of all design works.

4.1. Configuration development

Keeping a step ahead with the development of long-span suspension bridges as mentioned previously, bluff plate–girder and box–girder deck bridges have been constructed for medium-span continuous bridges and cable-stayed bridges. In late 1960s, engineers and researchers’ attention has turned to the reports of couple of collapses and accidents on those girder bridges. A technical committee [38] was appointed in 1970 to go into the design criteria of box–girder bridges. The deliberations of the committee highlighted the deficiencies in the knowledge of aerodynamic behavior of bridges and initiated a series of wind tunnel tests to conduct a longer time parametric study on the aerodynamics of box–girder bridges, with the aim of producing comprehensive design rules.

Fig. 6. Time history observation in recent field measurement of wind and response during Typhoon Passage in the Akashi Kaikyo Bridge (courtesy of Honshu–Shikoku Bridge Authority).
Afterwards those investigations and actual designs and constructions in 1970s, it has been generally noticed [39] that those wind-induced vibrations are all quite dependent on configurations of decks and locations of additional elements, such as railings, parapets and others attached at the end of the deck. At the same time reduction effects of turbulence of winds on the bridge as well as structural damping attached to the structure, if possible, have also been recognized to be quite significant. Knowledge of the mechanisms of vortex-induced vibrations caused by vortex separation has been accumulated to produce a variety of countermeasures by which inconvenient wind-induced vibrations were reduced to allowable levels of motion. Such features, or developments of configuration, to change from original shapes of decks, are

(1) adoption of shallow sections,
(2) fitting a soffit plate or spoiler,
(3) use of tapered fairings to the side faces or inclining the web planes,
(4) use of low parapets and perforated railings,
(5) use of deflector flaps, single or double, or vanes.

Although these countermeasures are applied to many actual designs and constructions so far, the fact that has been disturbing bridge engineers is that the effectiveness on a specified case does not necessarily imply the same in another case in mitigating the vibration due to complicated phenomena of circumstances. One of solutions to such situation was to provide a couple of design rules and recommendations to treat specified responses on short- and medium-span bridges of bluff crosssections [40,41]. On the other hand, many ad hoc wind tunnel tests have been also carried out to get reasonable relevant results case by case.

4.2. Installation of passive/active damper

Let us go back to past to look for an illustrative example of an attempt to suppress a wind-induced vibration on actual bridge construction. One such is of Forth Br. The damping device was used for sliding block system for the main tower under erection [42]. This is purely a mechanical approach which attaches additional damping to required level of the actual structure. Installation of more sophisticated devices such as tuned mass damper might be effectively applied to limited area of the site and fabricated separately by specified design requirement. This happened in late 1970s, for example with the Citycorp Center and the John Hancock Tower buildings [43] and later the Bronx Whitestone Br. [44]. Tuned mass dampers were utilized afterwards in other box–girder deck bridges of medium span and pylons of cable-stayed bridges to reduce the vortex-induced vibrations of the completed ones and/or those being erected.

The use of contemporary active damper devices started in the late 1980s to apply to high-rise main towers of suspension bridges under erection in particular. The application to free standing main tower of about 300 m high Akashi Kaikyo Br., shown in Fig. 7 [45,46], was really effective in reducing a variety of wind-induced
vibrations which were estimated in the wind tunnel studies in advance. These devices also worked as a sort of an exciter to carry out the field observations later. As for the main towers of this bridge, because of the huge structure possibly vortex-induced vibrations were still estimated persistently even after completion and have passive type of damping systems were to be installed inside the tower sections and between the stiffening girder and the towers.

As summarized above, those suppression procedures of wind-induced vibrations, independent of any configuration developments by changing the crosssection from original design or fitting supplemental elements, or installing appropriate damper devices, passive or active, have been studied so far and actualized in different ways. However, as for the adoption of contemporary damping devices, their design criteria are not satisfactorily investigated and compared with civil engineering structures. Whether maintenance free or not, as one of the design alternatives of conventional structural and/or aerodynamic design approaches, there will be a host of new challenges.

4.3. Rain–wind-induced vibration

When providing a new idea to create a specified structure or its element, or when developing a new technique to make a substantial progress, sometimes, major error may miss being noticed and failure may occur actually after the completion, as seen in case of the collapse of the Tacoma Narrows Br. The rain–wind-induced vibration of a long stay cable in the long-span cable-stayed bridges is the case. It was recognized over a period of years in early 1980s that a newly developed round shape and smooth surface coated with polyethylene (PE), has resulted in the onset of very
violent galloping-type vibrations for stay cables (Fig. 8(a)). Among a couple of actual measurements in prototype bridges, some higher peak amplitudes reached more than 10 times their cable diameters. Hikami [47] firstly noticed one of the primary reasons to excite the vibration because he knew the work on formulation of water rivulets caused by rainfall on the smooth surface of the cable under the axial flow in the rear wake. Such a violent onset on the stay cables has been clearly understood to be quite fatal for a while and the cable-stayed bridge system cannot survive.

Intensive efforts have been made to investigate the mechanisms of phenomena observed and to provide appropriate ideas to suppress those vibrations in two different ways. One is the aerodynamic means to develop surface configuration on round, smooth stay cables appropriately, and another is the mechanical approach to add structural damping of a certain level equipped by appropriate devices. Fig. 8(b) shows an example of surface improvements processed on the stay cables of 19 cm diameter and longest cable 460 m as in the Tatara Br. completed in the spring of 1999 [46,48,49]. Creating certain degree of roughness in concave holes, not uniformly, on the surface of PE coated cables was actually applied, and this succeeded in disturbing consistent formation of water rivulets. Another reason why not uniformly processed pattern-indentated cables were adopted on this bridge, is for keeping the same low level of wind loads as on the normal smooth surface cables, where the drag coefficient of 0.7 was prescribed at Reynolds number for the design wind speed range. Complete uniform roughness processed in concave pattern on the surface produced higher level at similar Reynolds number, 1.0–1.2 on normal cable
at lower Reynolds number. Actually, it was only reported that a couple of specified stay cables have experienced a small level, about a quarter of diameter, of damped vibration under wind speeds of 10–20 m/s in the rainfall, which was nearly the same as the predicted ones in the wind tunnel tests.

5. Concluding remarks of bridge aerodynamics

Some historical views on aerodynamic performances and procedures relevant to wind-resistant design of long-span suspension bridges and partly cable-stayed bridges were herein described comprehensively. On this occasion of celebrating 40 years of contribution to the field by Dr. Alan G. Davenport, I feel privileged to write and refer on the past significant breakthroughs, in ideas and practice.

Perspective developments may now be put forth from prevailing understanding and practice

(a) Complicated formulation is not necessarily effective in modelling specified phenomena. This is so without satisfactory data preparation and presentation of relevant variables and properties.
(b) Along with efforts of acquiring more accurate and reliable data in the wind tunnel tests, practical developments of numerical analyses should be encouraged more intensively to overcome deficiency between one and the other.
(c) Introduction of new ideas or technical concepts needs to be carried out carefully prior investigations. Aerodynamic behaviors associated with wind-induced responses concerned are quite susceptible to configurations, variables and properties of structures or their elements as well as specified turbulent winds at the site.

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