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Autor: Ito, Manabu / Nakamura, Yasuharu
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Aerodynamic Stability of Structures in Wind

Stabilité aérodynamique des structures

Aerodynamische Stabilität von Bauwerken

Manabu ITO
Professor
University of Tokyo
Tokyo, Japan

Yasuharu NAKAMURA
Professor
Kyushu University
Fukuoka, Japan

Manabu Ito, graduated in 1953, and received his Dr. Eng. degree in 1959, from the University of Tokyo. He has been engaged in teaching and research on bridge engineering and structural dynamics at the same university, and is now Professor of Civil Engineering. He has been also involved in technical committees on various bridge projects in Japan.

Yasuharu Nakamura graduated from the Department of Applied Physics, University of Tokyo in 1955, and received his Dr. Eng. degree from the University of Tokyo in 1967. He is now Professor at the Research Institute for Applied Mechanics, Kyushu University, Fukuoka. His major fields of activity are aeroelasticity and wind effects on structures.

SUMMARY

Dynamic instabilities induced by wind in flexible structures are reviewed under the classification into galloping, torsional flutter, coupled flutter and vortex excitation as well as low-speed flutter. Design practices to cope with these phenomena are also described. Finally mention is made of the experimental techniques in a wind tunnel.

RÉSUMÉ

Cette revue traite de l'instabilité aérodynamique des structures flexibles selon la classification suivante: oscillations de flexion, de torsion, de flexion-torsion et oscillations dues à la formation de tourbillons. Pour éviter ces phénomènes, des mesures pratiques sont présentées. L'étude mentionne encore brièvement les techniques expérimentales utilisées lors des essais en soufflerie.

ZUSAMMENFASSUNG

Durch Wind an verformbaren Bauwerken verursachte dynamische Instabilitäten werden besprochen, wobei zwischen Biege-, Torsions-, gekoppelter Biege- und Torsionsschwingungen (Flatterschwingungen) und durch Wirbelablösungen verursachte Schwingungen unterschieden wird. Entwurfsmassnahmen, um diesen Erscheinungen zu begegnen, werden dargestellt. Zum Schluss werden noch Versuchstechniken bei Windkanaluntersuchungen angeführt.

1. INTRODUCTION

Wind effects on structures are quite old and yet new problem in engineering. A distinctive feature of wind-induced phenomena in structures is their variety as seen in Table 1; there is a possibility of multiple failure or unserviceability modes for a specific structure at different wind speeds, and some of the phenomena can occur concurrently.

The most tragic event in the history of wind engineering was the collapse of the Tay Bridge in 1879. It stimulated the need for adequate assessment of wind loading on structures. In fact a number of structures, particularly cable-suspended bridges, suffered severe damage due to wind, going back more than a half century earlier. Nevertheless, it was since the original Tacoma Narrows Bridge fell in 1940 that the aerodynamic stability of non-aeronautical structures has attracted the attention of engineers and researchers.

Today the problems seem to be under control. With modern trend in design and construction, however, considerations of dynamic effects of wind on structures have become increasingly important. They should be extended to a wide variety of structures as well as to cable-suspended bridges. There are still frequent reports of actual or predicted oscillations of tall stacks, tower-like structures, box girder bridges and slender structural members.

Under these circumstances, the object of the present survey will be focussed on the aerodynamic instabilities which cause structures to oscillate in wind and may lead to structural or functional failure. As shown in Table 1, various types of dynamic phenomena including the so-called vortex-excitation, galloping and flutter come within this category. It must be noticed here that the terminology in structural aerodynamics is not necessarily unified yet, so that the consensus is needed at the earliest opportunity.

Although several informative states-of-the-art on structural aerodynamics have already been available (e.g. [1] ∿ [3]), in what follows the phenomenological mechanisms of the aerodynamic instability of structures and the control measures against these phenomena will be briefly described on the basis of the recent research works and the practical experiences.

Table 1 Classification of wind effects on structure

<table>
<tr><td rowspan="3">static</td><td colspan="4">effect of time-averaged wind pressure, wind force</td></tr>
<tr><td rowspan="2">static instability</td><td colspan="3">divergence</td></tr>
<tr><td colspan="3">lateral buckling</td></tr>
<tr><td rowspan="5">dynamic</td><td rowspan="4">dynamic instability</td><td>galloping</td><td rowspan="2">single-degree-of freedom</td><td rowspan="3">divergent-amplitude response</td></tr>
<tr><td>torsional flutter</td></tr>
<tr><td colspan="2">coupled flutter</td></tr>
<tr><td colspan="2">vortex excitation, low-speed flutter</td><td rowspan="2">limited-amplitude response</td></tr>
<tr><td colspan="3">turbulence response (gust response, buffeting)</td></tr>
</table>

2. AERODYNAMIC INSTABILITIES OF STRUCTURES

Most dynamic instabilities of structures under wind action fall within the category of aeroelastic flutter. Fig. 1 shows the block diagram illustrating a closed loop formed by a structure in the wind and the incremental aerodynamic force due to the structural motion. Flutter is an oscillatory aeroelastic instability that occurs in this closed loop. Most structures exposed to the wind are aerodynamically bluff so that the flow around structures is separated. Flutter of structures in the wind is accordingly characterized by separated-flow flutter for which no exact theory has been established. In what follows, attention will be concentrated to flutter of two-dimensional bluff structures with sharp corners. Unless otherwise stated, a bluff structure simply means a two-dimensional bluff structure with sharp corners.

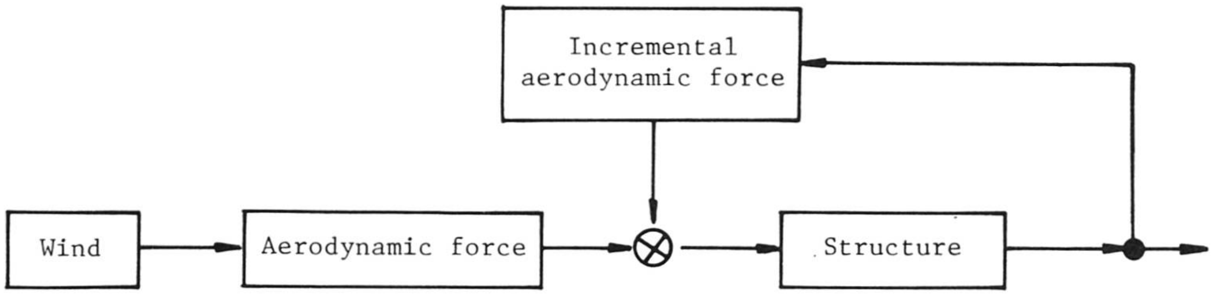


Fig. 1 Block diagram illustrating the onset of flutter

On a stationary bluff structure, the two separated shear layers, free to interact, are basically unstable and roll up to form a staggered array of discrete vortices, known as Karman vortex street. In the separated base cavity, namely, in the near wake, the pressure is low so that a bluff structure experiences a high pressure drag. In addition, periodic vortex shedding causes oscillatory aerodynamic forces and moment to act on a structure even if it is held stationary. Strouhal number S is defined by $S = f_V h/V$, where f_V is the frequency of vortex shedding, h is the characteristic length of a bluff section, and V is the speed of the incident flow. In general, S is dependent not only on the geometry of the cross section of a bluff structure but also on Reynolds number. However, if the separation points of a bluff structure are fixed at its sharp corners, S is almost independent of Reynolds number so that the frequency of vortex shedding increases in proportion to the flow speed. The values of S usually ranges from 0.1 to 0.3 [4].

The pressure loading that causes flutter of a bluff structure is principally on its afterbody surface, the part of the cross section downstream of the separation points. Therefore, the most important physical parameter relevant to flutter of a bluff structure is the size and the shape of its afterbody. Rectangular prisms with different depth-to-height ratios are the most convenient shapes to investigate flutter of bluff structures.

The incremental aerodynamic forces and moment acting on an oscillating bluff structure have two main frequency components (see, for example, [5]). The aerodynamic forces and moment having a frequency f same as that of the oscillating structure are called the frequency response components, while those with a frequency equal to f_V, the frequency of vortex shedding for a stationary structure, are called the Strouhal components. In the following sections describing the aerodynamic mechanisms of flutter, the frequency response components will be particularly considered. Although the full phenomenon of any separated flow is highly nonlinear, it may be justified to assume linearity in the frequency response components at small amplitudes.

There are many forms of flutter of bluff structures, each depending on a different aerodynamic mechanism of excitation. Several of them will be described in this review. These include galloping, torsional flutter, coupled flutter and low-speed flutter together with vortex excitation. Other important classes of flutter, not mentioned in this review, include the subspan oscillation of bundled power line conductors [6], the ovalling of steel stacks [7] and the fluid-induced vibration of tube bundles [8] which is not really wind-induced but often the source of troubles in heat exchangers of power plant systems.

As shown in Fig.2, a resonant oscillation of a bluff structure can be excited in a narrow range of wind speed centered the one in which the frequency of vortex shedding f_v coincides with a natural frequency of the structure. This is called vortex excitation. In vortex excitation, there is a complex interplay between vortex shedding and the structural motion. Therefore, it may be justified that vortex excitation is not a forced oscillation in a conventional sense, but rather categorized as one of the aeroelastic flutter phenomena. A reduced wind speed is defined as $\bar{V} = V/(fh)$. When $\bar{V}$ is large, the oscillation is relatively slow, and vice versa. The reduced resonant wind speed for vortex excitation $\bar{V}_{cr}$ is defined as the reciprocal of the Strouhal number S.

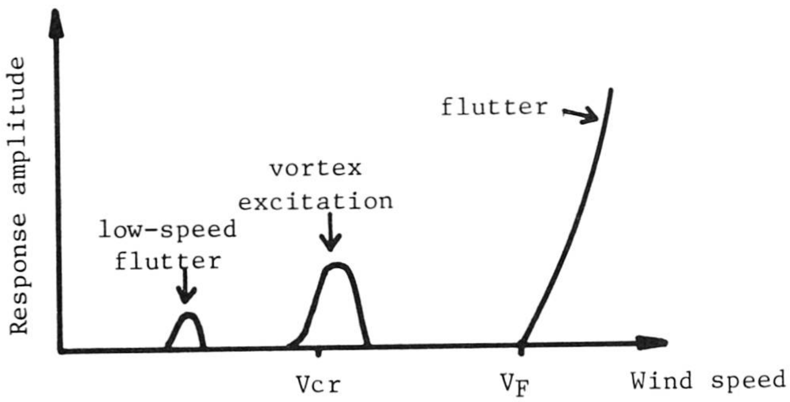


Fig. 2 Typical response of bluff structures in smooth wind

2.1 Galloping

A term, galloping, is referred to as flutter of a bluff structure in a single degree of freedom in transverse translation. Typically, some of rectangular prisms including a square are susceptible to galloping. Consider a bluff structure placed in a uniform flow with a constant speed V, as shown in Fig.3. If the structure moves transversely with a constant speed $\dot{y}$, a steady aerodynamic lateral force $F[\dot{y}]$ acting on the structure per unit span is given by

$$F[\dot{y}] = \frac{1}{2}\rho V^2_{rel} h C_{Fy}(\alpha), \tag{1}$$

where ρ is the air density, V_{rel} is the relative flow speed, $C_{Fy}(\alpha)$ is the coefficient of the steady aerodynamic lateral force, and α is the incidence given by $\alpha = \tan^{-1}(\dot{y}/V)$. Although no existing theory is yet capable of predicting $C_{Fy}(\alpha)$, it is obtained experimentally by steady force measurements as a function of α. Next, consider a bluff structure which is oscillating transversely with a speed $\dot{y}$. If the oscillation is slow enough, it may be assumed that the structure, at an instant of oscillation cycle, experiences the same force as given by eq. 1. For simplicity, it may be assumed further that α is so small that all relevant quantities are linearized as,

$$\alpha = \frac{\dot{y}}{V}, \quad C_{Fy}(\alpha) = \frac{dC_{Fy}}{d\alpha}\alpha, \quad \text{and} \quad V_{rel} = V. \tag{2}$$

The linearized equation of motion for a spring-mounted structure is given by

$$m\ddot{y} + c\dot{y} + m(2\pi f_n)^2 y = \frac{1}{2}\rho V h \frac{dC_{Fy}}{d\alpha}\dot{y}, \tag{3}$$

where m is the mass of the structure per unit span, c is the damping coefficient, and f_n is the still-air frequency of the structure. If

$$\frac{dC_{Fy}}{d\alpha} > 0, \tag{4}$$

is satisfied, the aerodynamic force on the right hand side of eq. 3 can amplify the structural motion. It follows that galloping occurs spontaneously from rest provided that the aerodynamic force exceeds the structural damping. In galloping, there is little change in frequency because of large mass ratio $m/(\rho h^2)$. Eq. 4 is sometimes referred to as Den Hartog's criterion [9].

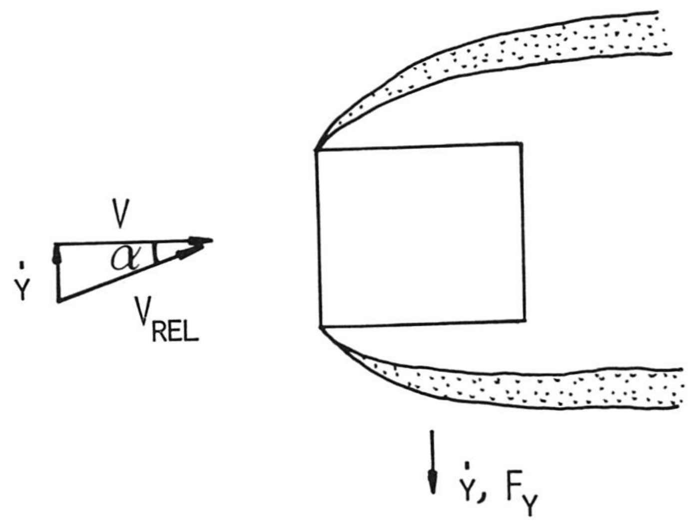


Fig. 3 Section and flow geometry for the quasi-steady analysis of galloping

Parkinson and his associates established a nonlinear quasi-steady aerodynamic theory of galloping by approximating the experimental $C_{Fy}(\alpha)$ curve with a polynomial of $\dot{y}/V$ [10]. Introducing such a polynomial into eq. 3, they obtained a weakly nonlinear autonomous differential equation capable of solution by the approximation method of Krylov and Bogoliubov. Fig. 4 shows the experimental $C_{Fy}(\alpha)$ for a square prism which indicates that $dC_{Fy}/d\alpha$ is positive for $0 \leq \alpha \leq 12$ deg, approximately. Fig. 5 shows the variation of the steady-state amplitudes of galloping for a square prism with the reduced wind speed. It indicates good agreement between theory and experiment; in the theory, a polynomial to the seventh power of $\dot{y}/V$ was used to approximate the $C_{Fy}(\alpha)$ curve. A variety of nonlinear galloping responses are obtained depending on functional forms of $C_{Fy}(\alpha)$ [11]. For example, soft galloping, a galloping instability starting spontaneously from rest with a stable limit cycle, is associated with $dC_{Fy}/d\alpha$ varying from positive to negative with α. On the other hand, hard galloping, a galloping instability requiring an initial threshold amplitude, is associated with $dC_{Fy}/d\alpha$ varying from negative to positive with α. The theory was also successfully applied to galloping of three-dimensional slender structures [12] and structures in turbulent flow [13, 14].

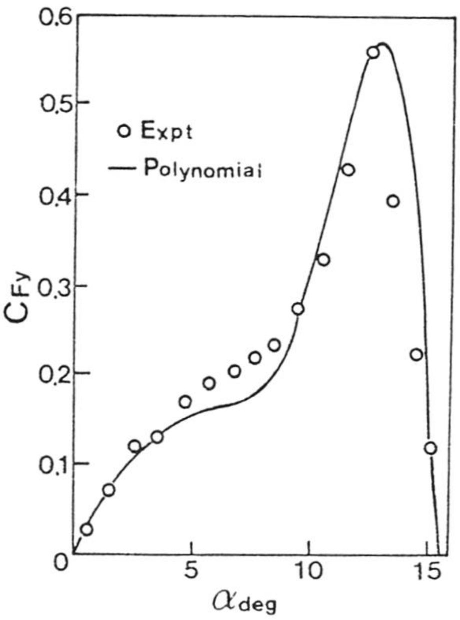


Fig. 4 Lateral force vs incidence for square prism [10]

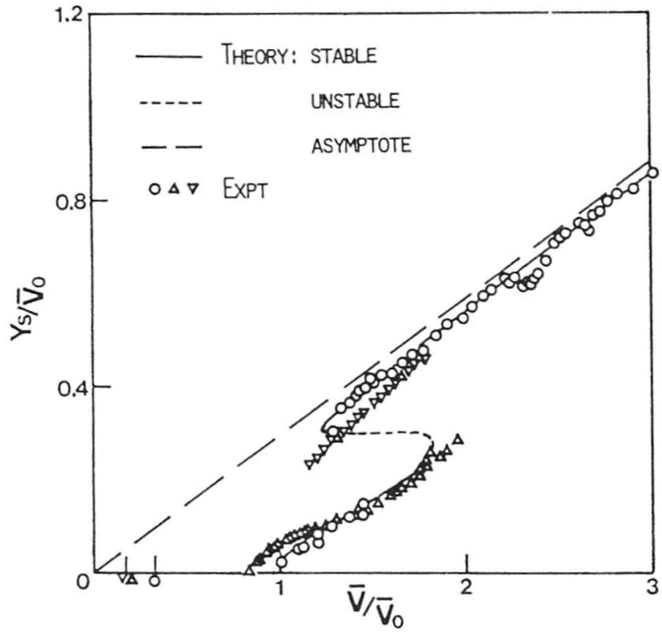


Fig. 5 Amplitude-velocity characteristics for galloping square prism [16]

The quasi-steady aerodynamic theory of galloping ignores the effect of the undulation of the wake due to the structural motion, herein referred to briefly as the fluid memory effect, together with the effect of the periodic vortex shedding behind the structure. Therefore, the validity of the quasi-steady aerodynamic theory of galloping is limited to a range of high reduced wind speeds. When the reduced wind speed is lowered, the above-mentioned two effects seem to become increasingly significant, thus invalidating the quasi-steady aerodynamic theory. There is no theory available for predicting low-speed galloping. Fortuitously, however, the two effects are often cancelled together so that the applicability of the quasi-steady aerodynamic theory is extended to considerably low reduced wind speeds, say, 2 to 3 times $\bar{V}cr$ [15].

The aerodynamic mechanism for the onset of soft galloping, that is, a positive $dC_{Fy}/d\alpha$ at $\alpha = 0$, presents one of the challenging problems of bluff body flow. It should be mentioned that the well-known free-streamline model is not applicable to galloping because it assumes a uniform pressure inside the wake. It is the secondary flow in the wake that drives a bluff structure into galloping. A convenient shape for a systematic study on this problem is again a rectangular prism. A positive $dC_{Fy}/d\alpha$ at $\alpha = 0$ for a rectangular prism is obtained over a range of d/h extending from about 0.75 to 3.0, where d is the section depth and h is the section height [16, 17]. At d/h = 3.0, the separated shear layers of a rectangular prism just reattach to the downstream corners of the prism. Therefore, it is obvious that the vanishing of galloping is associated with flow reattachment. The aerodynamic condition for the initiation of soft galloping at d/h = 0.75 is more complicated, but it is again associated with flow reattachment for the reason suggested recently by Nakamura and Tomonari [18].

2.2 Torsional Flutter

Torsional flutter is an oscillatory aeroelastic instability of a bluff structure driven essentially in a single degree of freedom in torsion. It is sometimes called torsional galloping. In contrast to transverse galloping, torsional flutter can occur for bluff structures centered the one for which the separated shear layers just reattach to the downstream corners. Again, some of rectangular prisms are susceptible to torsional flutter. Soft torsional flutter at high speeds, that is, torsional flutter at high speeds starting spontaneously from rest can occur for rectangular prisms, placed at $\alpha = 0$ with an axis of torsion at the center of the cross section, with d/h ranging from about 2.5 to 5.5 [19, 20]. At d/h = 2.5, the flow separates throughout but with the shear layers being in close proximity to the downstream corners, while at d/h = 5.5, shear layers reattach the surfaces and only short separation bubbles remain near the upstream corners. Rectangular prisms with d/h smaller than 2.5 are stable at rest but become unstable at large amplitudes; namely, they exhibit hard flutter characteristics.

In what follows, the axis of torsion is assumed to be near the center of the cross section; if the axis is far upstream or downstream of the structure, torsional flutter is reduced to transverse galloping. While the quasi-steady aerodynamic theory neglecting the fluid memory effect is valid to transverse galloping, the fluid memory effect is responsible for the onset of torsional flutter [21]. As pointed out by Scanlan et al. [22] and Yoshimura and Nakamura [23], it is sometimes more convenient to analyse the problem in the time domain rather than in the frequency domain to see the vital role played by the fluid memory effect in the onset of torsional flutter.

Consider a bluff structure subjected to a forced sinusoidal torsional motion about an axis in a uniform flow. Assuming the aerodynamic response system under consideration to be linear, the non-dimensional aerodynamic moment response $C_M[\theta]$ acting on the structure is expressed in a complex-number notation by

$$C_M[\theta] = \mathrm{Re}[Z_\theta(i\bar{V})\theta], \tag{5}$$

where θ is the torsional displacement, $Z_\theta(i\bar{V})$ is the frequency response of the aerodynamic moment, and i is equal to $(-1)^{1/2}$. It is also convenient to rewrite eq. 5 in a form

$$Z_\theta(i\bar{V})\theta = C_{M\theta}(\bar{V})\theta + C_{M\dot{\theta}}(\bar{V})\dot{\theta}, \tag{6}$$

where $-C_{M\theta}(\bar{V})$ and $-C_{M\dot{\theta}}(\bar{V})$ are called the aerodynamic stiffness and damping coefficients, respectively, both being functions of the reduced wind speed. The condition for the onset of torsional flutter is given by

$$C_{M\dot{\theta}}(\bar{V}) > 0. \tag{7}$$

On the other hand, the indicial aerodynamic moment response $\Phi_{M\theta}(\tau)$ is defined by the aerodynamic moment response due to a unit step torsional displacement of a bluff structure, where τ is the non-dimensional time equal to Vt/h. It should be mentioned that owing to the fluid memory effect, it takes some time for $\Phi_{M\theta}(\tau)$ to approach asymptotically $\Phi_{M\theta}(\infty)$ which is identical with the steady aerodynamic coefficient $C_{M\theta}(\infty)$.

In view of the linear aerodynamic response system, the frequency response $Z_\theta(i\bar{V})$ is given by a Fourier transform of the impulsive response $d\Phi_{M\theta}/d\tau$ as

$$Z_\theta(i\bar{V}) = \int_0^\infty \frac{d\Phi_{M\theta}}{d\tau} \exp[-i2\pi\tau/\bar{V}]\, d\tau. \tag{8}$$

It follows that

$$C_{M\dot{\theta}}(\bar{V}) = \int_0^\infty \{\Phi_{M\theta}(\tau) - \Phi_{M\theta}(\infty)\} \cos(2\pi\tau/\bar{V})d\tau, \tag{9}$$

and in particular

$$C_{M\dot{\theta}}(\infty) = \lim_{\bar{V}\to\infty} C_{M\dot{\theta}}(\bar{V}) = \int_0^\infty \{\Phi_{M\theta}(\tau) - \Phi_{M\theta}(\infty)\} \, d\tau, \tag{10}$$

for slow oscillation.

Eq. 10 represents a simple relation between the onset of torsional flutter and the form of the indicial aerodynamic moment response. Namely, high-speed torsional flutter can occur provided that the area enclosed by $\Phi_{M\theta}(\tau)$ and its asymptote is positive, while the oscillation is stable provided that the area is negative. The validity of eqs. 9 and 10 was demonstrated by an experiment using a towing water tank on models of bluff structures of simple shapes [23]. As illustrated in Fig. 6, the type of indicial moment response for torsionally unstable bluff structures was characterized by an initial abrupt overshoot followed by a rather slow settling down to the steady-state condition. Obviously, it satisfies the condition for the onset of torsional flutter as given by eq. 7. The theoretical prediction was verified by good agreement in a comparison between the Fourier-transformed $C_{M\dot{\theta}}(\bar{V})$ and the directly measured one. In the case of the indicial motion, it was straightforward to identify the flow structure that produces the characteristic moment response such as shown in Fig. 6. That is, in response to the indicial change in incidence, vortices were formed and discharged at the sharp upstream corners of a bluff structure, and they travelled downstream past the structure as time went on. In short, the fluid memory effect is responsible for the onset of torsional flutter of bluff structures.

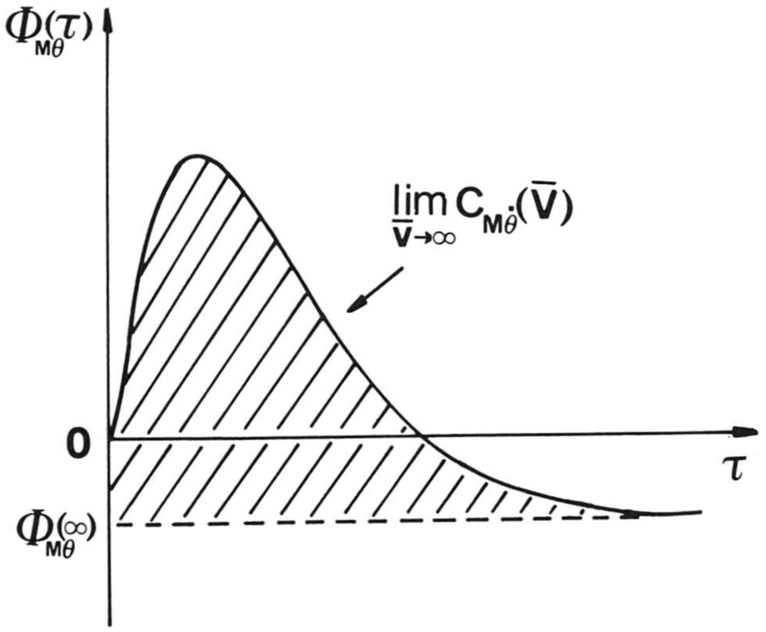


Fig. 6 Indicial aerodynamic moment response and the onset of torsional flutter

2.3 Coupled Flutter

Consider a two-dimensional aeroelastic system of a bluff structure with two degrees of freedom, transverse translation and torsion, where both the elastic axis and the center of gravity lie at the center of the cross section. For simplicity, the structural dampings are assumed to be zero. The linearized equation of motion for this coupled aeroealstic system is given in a matrix form by

$$\begin{bmatrix} m & 0 \\ 0 & I \end{bmatrix}\begin{Bmatrix} \ddot{y} \\ \ddot{\theta} \end{Bmatrix} + \begin{bmatrix} m(2\pi f_n)^2 & 0 \\ 0 & I(2\pi f_\theta)^2 \end{bmatrix}\begin{Bmatrix} y \\ \theta \end{Bmatrix} = \begin{bmatrix} L_{\dot{y}} & L_{\dot{\theta}} \\ M_{\dot{y}} & M_{\dot{\theta}} \end{bmatrix}\begin{Bmatrix} \dot{y} \\ \dot{\theta} \end{Bmatrix} + \begin{bmatrix} L_y & L_\theta \\ M_y & M_\theta \end{bmatrix}\begin{Bmatrix} y \\ \theta \end{Bmatrix} \quad , (11)$$

where the two terms on the right hand side represent the frequency response components of the incremental aerodynamic forces and moments which, if the sign reversed, are the aerodynamic damping and stiffness terms, respectively.

There are two different ways in which energy can be extracted from the wind stream in a coupled aeroelastic system of a bluff structure [24]. Doubtlessly, an instability which is similar to either transverse galloping or torsional flutter in a single degree of freedom can occur in a coupled aeroelastic system under the action of the aerodynamic damping terms. In contrast, the other type of instability, herein referred to as classical type flutter, can occur under the action of the aerodynamic stiffness terms with an important difference of phase between the two degrees of freedom. For example, if the phase difference ϕ is equal to $\pi/2$, the dominant cross stiffness force $L_\theta\theta$ is in phase with $\dot{y}$, thus being capable of controlling the transverse motion. Given a wind speed, the phase difference may be dependent not only on the aerodynamic forces and moments, but also, in a complicated manner, on other parameters such as the frequency and mass ratios, and in general on the positions of the elastic axis and the center of gravity. It is the central problem in flutter analysis to identify the phase characteristics.

In the case of an airfoil, flutter is usually of the classical type since neither galloping nor separated-flow torsional flutter can occur. In the case of a bluff structure, however, both of the two types of flutter, the classical and the single-degree-of-freedom type, can occur according to situations. In the latter case, very simple ralations [25] were developed to predict the flutter characteristics because of the absence of inertial and elastic couplings in eq. 11.

Although classical type flutter is best exemplified by an airfoil, it is not of rare occurrence for some of bluff structures. Two such examples will be mentioned. The first is concerned with bridge deck sections. A bridge deck section is basically plate-like so that it is in itself susceptible to classical type flutter. An addition of stiffening members to the deck increases the gross bluffness, thus converting the instability from the classical type, occurring at relatively high wind speeds, to the single-degree-of-freedom type, occurring at relatively low wind speeds. The second is concerend with ice-accreted bundled power line conductors [26]. Because ice-accreted conductors are very bluff, they are often subjected to transverse galloping. However, the frequencies of vertical translation and torsion are very close together in the case of bundled conductors, hence causing the aerodynamic coupling to become exceptionally strong. This may sometimes lead to the onset of violent classical type flutter [27].

2.4 Vortex Excitation

Vortex excitation of a bluff structure can occur in a narrow range of wind speed centered the one in which the frequency of vortex shedding f_v coincides with one of the natural frequencies of the structure f_n. It can be either in transverse translation or in torsion. In-line vortex excitation can occur when the in-line structural frequency is close to twice f_v. It is relatively weak and rarely serious in air flow, although it may be serious in water flow [28].

At reduced wind speeds smaller than $\bar{V}_{cr}$, the reduced resonant wind speed, the response of a bluff structure is small and narrow-band random with a dominant frequency equal to f_v (see, for example, [29]). As $\bar{V}_{cr}$ is approached, another dominant frequency f_c, close to f_n, manifests itself; that is, a beat modulation is observed. As $\bar{V}_{cr}$ is exceeded, the response builds up rapidly until a maximal, fairly steady oscillation with a single dominant frequency equal to f_c is reached. During this process the two frequencies abruptly lock in to a common value f_c. This is called the frequency locking-in which characterizes vortex excitation as a typically nonlinear oscillation. With further increase in wind speed, the opposite process follows. The oscillation at high $\bar{V}$ is again relatively small and narrow-band random with a dominant frequency equal to f_v.

In the range of the frequency locking-in the vortex frequency in the wake also locks in to f_c and remains almost constant throughout apparently in violation of the Strouhal relation. An increase in amplitude in vortex excitation improves, particularly in the range of the frequency locking-in, the spanwise correlation in vortex shedding which is not completely coherent along the span even for a bluff structure with fixed separation points. Clearly, the improvement in the spanwise correlation in vortex shedding further amplifies vortex excitation.

Notwithstanding the recent development of the numerical methods such as the direct computation of the Navier-Stokes equations and the discrete vortex model, the structure of the near wake and the mechanism of vortex shedding behind a bluff structure at high Reynolds numbers have not yet been amenable to a detailed theoretical analysis [30]. The nonlinear fluid-structure interaction in vortex excitation projects an order of magnitude more complex problems.

Several mathematical models [31] were proposed in an attempt to simulate periodic shedding of vortices behind a bluff structure. They are broadly categorized as fluid oscillators. The concept of a fluid oscillator is helpful to understand vortex excitation, although it has its roots not in the equations of fluid mechanics but rather in other fields such as mechanics and electricity. As far as vortex excitation in transverse translation is concerned, it is a single-degree-of-freedom, nonlinear lift oscillator. It is given by an differential equation

$$\ddot{C}_L - f(\dot{C}_L)\dot{C}_L + (2\pi f_v)^2 C_L = F[\dot{y}], \tag{12}$$

where the nonlinear damping term, $-f(\dot{C}_L)\dot{C}_L$, and the influence of the structural motion on the lift oscillator, $F[\dot{y}]$, have to be specified empirically. If eq. 12 is coupled with the equation of motion of a bluff structure,

$$m\ddot{y} + c\dot{y} + m(2\pi f_n)^2 y = \frac{1}{2}\rho V^2 h C_L, \tag{13}$$

vortex excitation may be determined by solving these two equations.

In view of eqs. 12 and 13, it is evident that vortex excitation is not a simple forced oscillation but rather a form of aeroelastic flutter. More precisely, it is coupled flutter occurring in a fluid-structure system where the natural frequencies of the two subsystems are f_v and f_n, respectively. Fig. 7 illustrates the response of a bluff structure in the presence of vortex shedding.

While the upper loop in the figure is identical to that of Fig. 2, the lower loop indicates the mechanism of vortex excitation as coupled flutter.

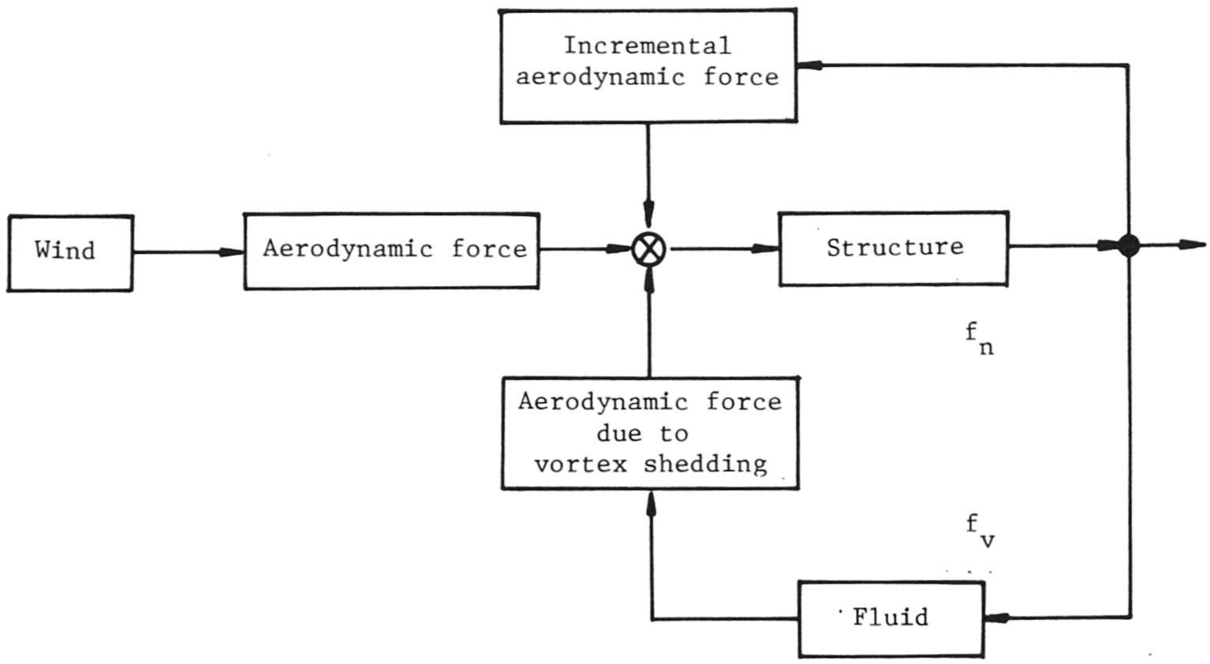


Fig. 7 Block diagram illustrating the response of a bluff structure in the presence of vortex shedding

The essential feature of the fluid-structure interaction as an aeroelastic system is that the natural frequency f_v of the fluid subsystem is proportional to the wind speed. Accordingly, as the wind speed approaches $\bar{V}_{cr}$, the two natural frequencies become close together so that an instability can emerge as flutter which is strongly coupled with the resonance characteristics. To proceed further, it is more convenient to consider a bluff structure oscillating inexorably rather than freely in the wind. Eq. 12 suggests that as $\bar{V}_{cr}$ is approached, there is an abrupt phase shift, due to resonance, in the frequency response component of the unsteady lift force relative to the structural motion. Eq. 13 suggests in turn that during the abrupt phase shift a large nagative damping lift force can be generated so that the structural motion is amplified. The abrupt phase shift in vortex excitation has been demonstrated in the unsteady force measurements on rectangular prisms [5, 15]. It should be mentioned that the mechanism of onset for vortex excitation has much in common with that for flutter of bundled conductors mentioned earlier. Also, since the onset of vortex excitation is associated with linear resonance, it may not be directly related to the nonlinear frequency locking-in phenomenon. Finally, as Fig. 7 indicates, transverse galloping (or torsional flutter) can occur concurrently with vortex excitation. There can be strong or weak interaction between the two according to situations [32].

2.5 Low-speed Flutter

Some of bluff structures including rectangular prisms are susceptible to single-degree-of-freedom flutter either in transverse translation or in torsion at low reduced wind speeds, often considerably lower than the resonant wind speed $\bar{V}_{cr}$ (see Fig. 2). The excitation is weak and its onset is restricted to a narrow range of wind speed. There are often more than one ranges of wind speed where the instability occurs. A recent investigation [33] has shown that the onset of this instability is attributable to the motion-dependent leading edge vortices. In other words, the aerodynamic mechanism of low-speed flutter is identical to that of high-speed torsional flutter which has been described earlier. In some cases vortex excitation and low-speed flutter can occur together in the same range of wind speed (see Fig. 7). It is suggested that some of the instabilities observed on cable-stayed bridges can be related to low-speed flutter [34].

2.6. The Effect of Turbulence on Flutter of Bluff Structures

The natural wind to which bluff structures are exposed is highly turbulent. It is obvious that turbulence causes buffeting of bluff structures [35, 36]. However, turbulence also influences the steady-force and flutter characteristics of bluff structures in various ways [37]. It increases entrainment through enhanced mixing between the separated shear layers and the near wake, while it increases diffusion and cancellation of vorticity and reduces the spanwise correlation in vortex shedding.

As far as the steady-force characteristics of a two-dimensional bluff structure are concerned, the effect of increased entrainment among others is often most significant, and this leads to earlier reattachment of the shear layers to the afterbody surface. It was found [38] that there is a significant effect of turbulence intensity. Although the effect of turbulence scale seems, within the range tested, to be secondary [38], a recent measurement has indicated that it may be sometimes significant [39]. The argument on the effect of turbulence scale has not yet been settled.

Turbulence can significantly modify the flutter characteristics of bluff structures at high wind speeds. The effect of turbulence on high-speed galloping is understandable on the basis of the quasi-steady aerodynamic theory. That is, turbulence, by earlier reattachment of shear layers, causes hard galloping to become soft, and soft galloping to become weaker and eventually suppressed [13, 14, 17]. A recent measurement [32] has suggested that the effect of turbulence on high-speed torsional flutter is similar to that of high-speed galloping. As the reduced wind speed is lowered, the correlation between the onset of flutter and the steady-flow reattachment is lost both in galloping and in torsional flutter.

As far as vortex excitation is concerned, few systematic measurements have been made so far. According to [32], torsional vortex excitation is amplified by turbulence for rectangular prisms with small d/h, while it is weakened for rectangular prisms with large d/h. Parkinson and his associates [38] found that there is no significant effect of turbulence on transverse vortex excitation of a D-section prism with its flat face normal to the flow. Howell and Novak [40] found that turbulence can amplify transverse vortex excitation of a circular cylinder. Clearly, more investigations are needed to assess the precise effect of turbulence on vortex excitation.

Regarding low-speed flutter of bluff structures, it has been shown [32] that the effect of turbulence is to weaken and eventually suppress it. Finally, it should be remarked that any suppositions on the effect of turbulence on flutter of bluff structures need to be treated with some caution. This is because most of the experimental verifications have been carried out with grid-generated turbulence where the turbulence scale rarely exceeds that of a bluff structure.

3. DESIGN CONSIDERATIONS

3.1 Provisions in Design Code

Generally speaking, current design code for structures states only abstract provisions concerning aerodynamic stability; at most the values of the Strouhal number and the dynamic lift coefficient for a few typical cross-sections have been given. The first systematic wind-resistant design regulation for flexible structures known by the authors was that for the Honshu-Shikoku Bridges in Japan [41]. The flow of the design process is as shown in Fig. 8.

More detailed design rules for bridge aerodynamics were proposed recently in the United Kingdom [42], in which, on the basis of a series of wind tunnel tests with typical bridge cross-sections, the formulae to estimate the maximum amplitude of vortex-excitation respose and the critical wind speeds for various aerodynamic instabilities are given. The authors feel, however, that the dynamic response of bridge deck to wind is very much sensitive to slight modifications to its cross-sectional shape from their experiences, so that it seems difficult to evaluate it by the use of a limited number of structural parameters.

Except for the structures or structural members having rather large stiffness or having some typical and simple cross-sectional shapes, the aerodynamic characteristics of which are well understood from past experiences, the wind tunnel tests will be inevitable as the design aid.

3.2 Methods of Assuring Aerodynamic Stability

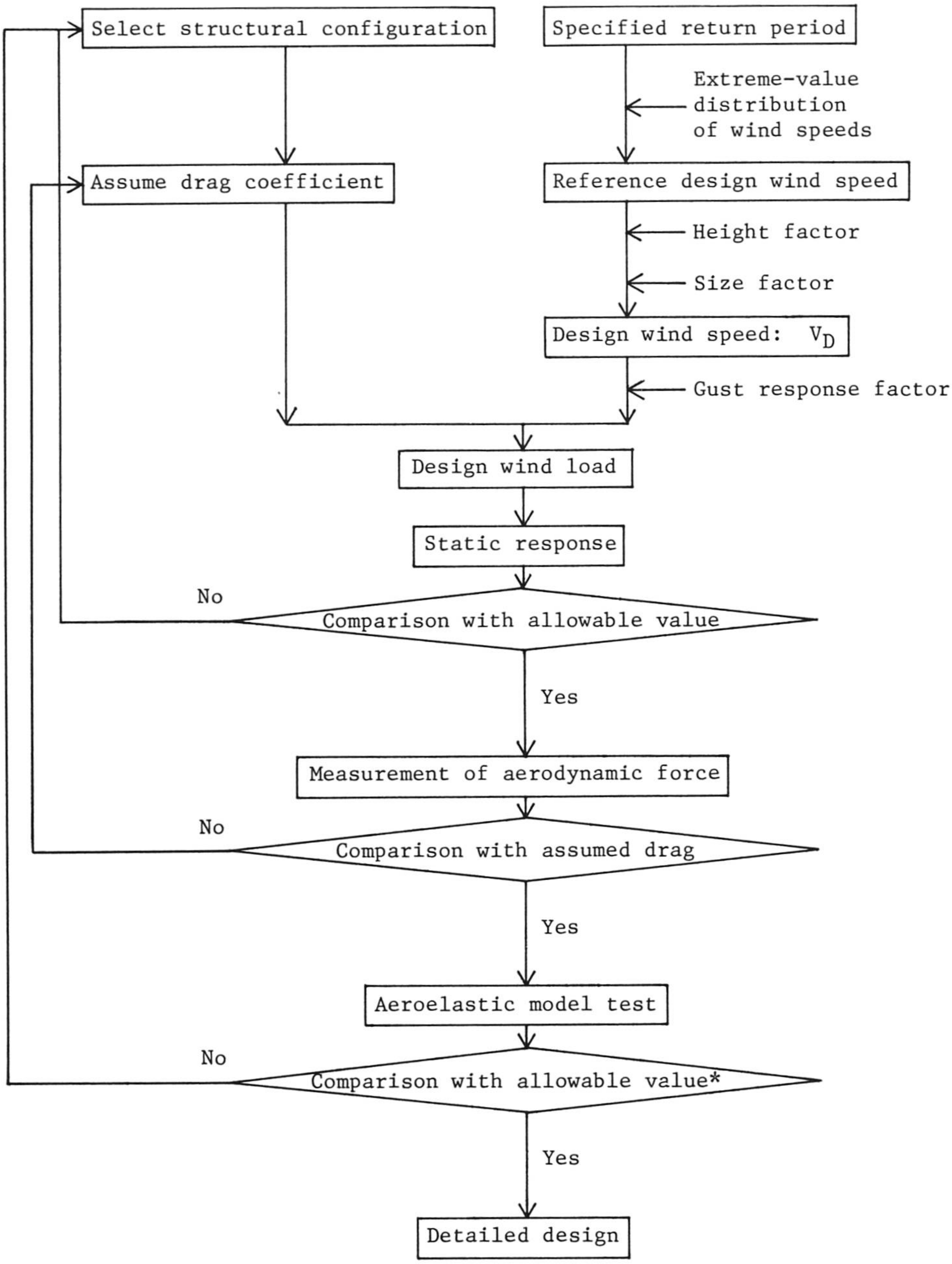
If the critical wind speeds at which the aerodynamic instabilities occur are well above the design wind speed, or the response amplitudes are so small enough as not to cause any structural or functional trouble, there is no problem. Otherwise appropriate countermeasures should be taken in design stage to prevent or suppress such phenomena. These countermeasures can be classified into structural means and aerodynamic means. In what follows, the review will be made by referring to the past excellent surveys [43, 44, 45] as well as the recent experiences in bridge decks [46]. Which method is used depends on the compatibility of the changes with other features, such as economy, function and appearance of the structure.

3.2.1 Mechanical Means

It is sometimes possible to raise the critical wind speed out of the range of wind speeds likely to be encountered during the lifetime of the structure by sufficiently increasing structural rigidities. An usual method of increasing structural rigidity is to enlarge the cross-sectional dimensions of the structure. It brings on a double benefit in the case of vortex excitation; that is, the increase of resonant wind speeds is attained by the increase of both natural frequencies and cross-sectional dimension of the structure. The critical wind speed for vortex excitation is, however, often rather low, and therefore it may result in uneconomical and even unfeasible design to push out the resonant wind speed above the design wind speed by increasing the natural frequency and/or the cross-sectional dimension of the strucuture.

The increase of torsional stiffness is very effective to augment the critical wind speed of divergent-amplitude flutter which frequently governs the design of long span suspension bridges. When a stiffening truss with closed roadway deck is for example used, installation of bottom and upper lateral bracings to maintain closed box effect has been strongly recommended.

Suppression of the limited-amplitude response and increase of critical wind



*For divergent-amplitude response: $V_F \geqq 1.2\, V_D$ (e.g. [41])

Fig. 8 Flow of wind-resistant design for flexible bridge superstructures speed for certain types of flutter can be also accomplished by increasing mass and/or structural damping, but the increase of mass will in turn decreases the natural frequencies of the structure. Its effect depends on location and distribution of the additional mass.

As far as the structural damping is concerned, various kinds of artificial dampers have been proposed. Most of them are passive, that is they simlpy absorb energy. The examples are addition of viscous materials inside structural section or at joints and use of guy ropes attached by friction block for free-standing towers, impact chain dampers, tuned mass dampers etc. [43]. On the other hand, a few types of active control devices to produce mechanical force to oppose aerodynamic force have been developed. The examples in this category are application of hydraulic mechanisms [43, 47] or gyroscope [48]. Their effectiveness is, however, not yet fully recognized in the prototype structures.

3.2.2. Aerodynamic Means

The cross sectional shape plays an important role in the aerodynamic stability of a wind-sensitive structure. Usually the sectional shape and size are firstly determined in accordance with structural and functional requirements as well as on the basis of the past experiences.

Selecting aerodynamically stable cross section often brings about the most economical design. This is in particular the case with cable-suspended bridges. Use of a very bluff section is usually avoided except for those having a short span length or being located at the site where wind environment is not unfavourable, and instead a flow-smoothing or flow-dividing shape to cut down coherent vortex action in the wake and associated large pressure difference across the deck is adopted.

Streamlining is one of the solutions and, as in the example of the Severn Bridge (Fig. 9), approaches to a thin airfoil-like shape which is free from the separated-flow instabilities, including the vortex excitation, under horizontal wind. In this case, the classical type flutter theory may be applicable, and its critical wind speed is usually high. It must be borne in mind, however, that the shallow streamlined section is prone to stall under inclined wind and may have a possibility of increasing vertical gust response.

Fig. 9 Outward shape of the Severn Bridge

Another approach to attain wind-resistant design is use of the section with openings such as a latticed truss. Most of the suspension bridges constructed after the accident of the original Tacoma Narrows Bridge have been designed along this approach. Nevertheless, the truss-stiffened deck with solid handrails or thick curb stones on the closed roadway sometimes exhibits aerodynamic instability, the type of which is usually coupling of vertical bending and torsion. While this phenomenon is in appearance the coupled flutter, the torsional motion seems predominant in nature and hence different from the classical type flutter. Accordingly, it is intended that these deck-type stiffening trusses of long span suspension bridges are usually provided with openings on the roadway including open gratings as well as both upper and lower lateral bracings to constitute closed box effects structurally as seen in Fig. 10. Since some small obstacles to air flow may, as mentioned previously, cause unfavourable effects on overall stability of the bridge, stringers and handrails are recommended to be an open structure. However, a solid fence of appropriate height at the central reserve of roadway seems to imporve the aerodynamic characteristics of stiffening truss.

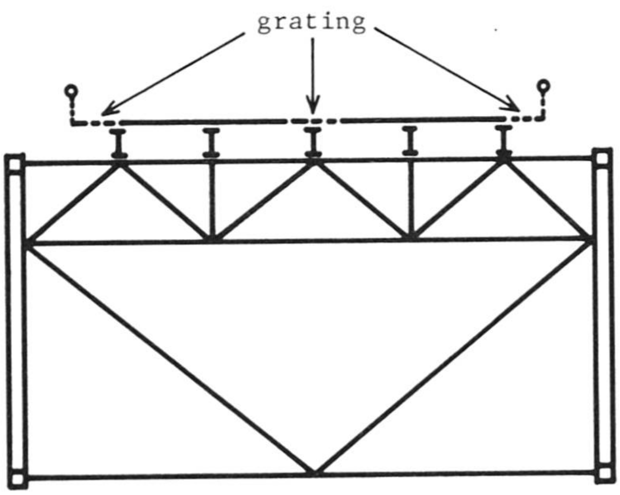


Fig. 10 Cross section of stiffening truss

In case that the substantial change of the basic cross-sectional type is not allowed for design reasons, some additional aerodynamic means are attached to the structure. These means have developed mainly to suppress the vortex excitation response of stacks with circular section, though many of them are also applicable to other sectional shapes. The object of these aerodynamic means is of course to reduce exciting force, that is to inhibit the regular vortex formation and decrease the spanwise correlation of the fluctuating surface pressures. The comprehensive surveys on the aerodynamic means for suppressing vortex-shedding excitation were made in the References [43] and [44]. In the latter, Zdravkovich classified these means into the following three categories in accordance with the phenomenological mechanisms:

i) surface protrusions, which affect separation lines and/or separated shear flows.
   e.g. helical strakes, wires, fins, studs or spheres, etc.
ii) shrouds, which affect the entrainment layers.
   e.g. perforated shroud, gauze, axial-rot, axial-slat, etc.
iii) nearwake stabilisers, which prevent interaction of entrainment layers.
   e.g. splitter plates, guiding vanes, base-bleed, slits cut across the cylinder, etc.

While most means in the first two categories are effective for all directions of wind, some means in the first and all means in the third category are unidirectional. Zdravkovich reviewed all the above means in detail, where it is commented that their effectiveness is reduced in the post-lock-in range, at high intensity of turbulence, and due to multi-cylinder interference.

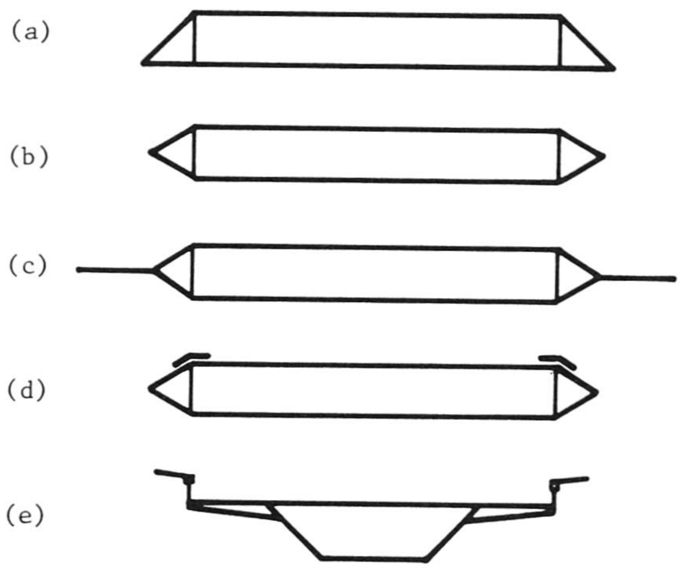


Fig. 11 Examples of stabilizers for bridge deck: (a,b) fairing, (c) fairing plus splitter plate, (d,e) flap

Similar aerodynamic means, in particular those in the third category cited above, can be adopted in bridge deck design. In case of cable-stayed bridges, solid web girders are preferred to trussed girders in view of structural, economical and aesthetical situations. Although calbe-stayed bridges are generally stiffer than suspension bridges, they are often prone to cause vortex shedding instability and low-speed flutter, or eventually galloping or torsional flutter. Under these circumstances, a large number of the cable-stayed bridges are provided with flaps, fairings or splitter plates as shown in Fig. 11. Of course, the girder of a cable-stayed bridge may be a truss, for design reasons such as double deck construction. These trusses will have relatively large solidity factor, and then the care for vortex excitation is necessitated.

4. AEROELASTIC WIND TUNNEL TESTS

Despite the better understanding of wind-induced effects on structures and the development of improved analytical means, wind tunnel tests are still unavoidable as a design aid; in particular for the assessment of aerodynamic stability of calbe-suspended bridge decks, the configuration of which is characterized by a great deal of complexity and variety. Several states-of-the-art on wind tunnel testing techniques are also available elsewhere (e.g. [49] ∿ [52]). Although wind tunnel testing is utilized in various fields, the short review focussing on the aeroelastic model tests will be made.

4.1 Similarity

Modelling of dynamic effects produced by wind introduces requirements for mechanical similitude as well as geometric similarity of aerodynamic shape. In the former, as well known, the equality of the five dimensionless parameters given in Table 2 is required between the model and the prototype.

It is usually impossible, however, to satisfy all these requirements. For example, wind speed scale from the Froude number similarity requirement violates that from the Reynolds number similarity with a model of reduced linear scale tested in an atmospheric low speed wind tunnel. The Reynolds number similarity requirement is fortunately relaxed in sharp-edged structures. Equality of the structural damping in the model and the prototype is an important consideration when dynamic responses are pronounced. Adjustment of this parameter can be controlled with a sectional model but not necessarily with a full model of the structure.

Table 2 Dimensionless parameters

Parameter	Symbol*	Physical meaning
1. Elasticity	$\frac{E}{\rho V^2}$	$\frac{\text{Elastic force of the structure}}{\text{Inertia force of the air}}$
2. Inertia (Density ratio)	$\frac{\rho_s}{\rho}$	$\frac{\text{Inertia force of the structure}}{\text{Inertia force of the air}}$
3. Gravitational (Froude number)	$\frac{V^2}{gh}$	$\frac{\text{Inertia force of the air}}{\text{Gravitational force on the structure}}$
4. Viscousity (Reynolds number)	$\frac{hV}{\nu}$	$\frac{\text{Inertia force of the air}}{\text{Viscous force of the air}}$
5. Structural damping (Logarithmic decrement)	δ_s	$\frac{\text{Dissipated energy per cycle}}{\text{Total energy of oscillation}}$

*Notations appear at the end of the paper.

The requirements of Froude number and elastic parameter similarity result in the necessity for the Young's modulus ratio of the model and the prototype to equal the linear scale ratio. Because it is usually impractical to find such model materials, the alternative method is to scale the stiffness including the sectional properties of the structure. Similarly, instead of directly equating the mass density ρ_s of the model and the prototype, $m/(\rho h^2)$ or $I/(\rho h^4)$ is usually used as the inertia parameter. Furthermore, either of the elasticity parameter or the density parameter may be replaced by the reduced wind speed $\bar{V}$ defined in Chapter 2. When the frequency of oscillation is not affected by the wind, Scruton [53] showed that the inertia parameter and the damping parameter may be combined into the single parameter $m\delta_s/(\rho h^2)$ or $I\delta_s/(\rho h^4)$.

The extent of modelling of the gravitational and elastic parameters depends primarily on the stiffness properties of the structure under consideration. The modelling of the gravitational parameter (the Froude number) is important in cases where the action of gravity force plays an important role in structural stiffness, as in suspended structures. On the other hand, the scaling of elastic forces of the structure and inertia forces of the flow are important for the case where stiffness depends primarily on elastic forces, as in example for towers and stacks. In this case, design of the model is relatively easy because the elasticity scale and the wind speed scale can be taken independently. Some additional considerations are required for special structures. For example, Tryggvason [54] has emphasized the importance of scaling acoustic stiffness of volumes enclosed by pneumatic structures.

Most experimental investigations on the aerodynamic instability of structures have been conducted in wind tunnels with relatively short working sections, designed to produce uniform low-turbulence flow. Although their results are useful to understand the fundamental characteristics of the phenomena and testing in smooth flow is said to give conservative estimates of aerodynamic stability in the natural wind, it must be preferable to simulate the air flow in the wind tunnel to the natural wind. The properties of natural wind which

are of particular significance in the assessment of wind effects on structures are as follows:
1) vertical profile of the mean wind velocity
2) statistical descriptions of the fluctuating velocity components, in particular the intensity and scale of atmospheric turbulence as well as its frequency spectrum.

Various methods have been proposed in attaining the natural wind simulated in a wind tunnel. Of these, the most realistic simulations are at present obtained by turbulent boundary layer flows generated artificially or naturally by the action of surface shear over a long fetch of roughened tunnel floor. It goes without saying that such a boundary-layer wind tunnel necessitates an appropriate length of working section, even if the aerodynamic spires at the entrance of test section are used together. The most prevailing method to generate turbulence in the conventional type of wind tunnel is the use of grids installed at the entrance of working section as seen in the background in Fig. 12. The intensity of the turbulence can be controlled by the size of meshes and bars, but the scale of the turbulence is usually too small relative to the size of the structure model.

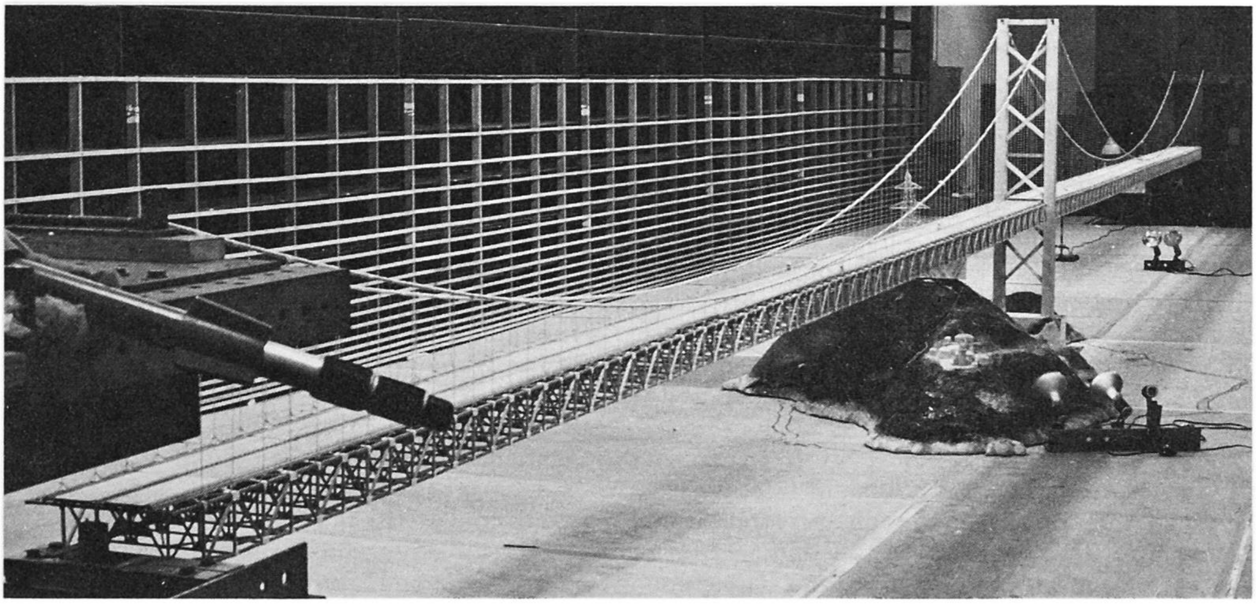


Fig. 12 Suspension bridge full model in wind tunnel

4.2 Aeroelastic Model of Structure

As far as the aerodynamic instability of structures is concerned, much effort has been exerted hitherto on the dynamic model testing of cable-suspended bridge decks, for which three methods of test have been used, namely the sectional, taut-strip [55] and full model test. Each of these three methods has the advantages and shortcomings, respectively, while it should be borne in mind that the selection of the type of wind tunnel model is made according to the purpose of experiment, as shown in Fig. 13. At the same time, the requirements of similitude conditions are different according to the model used and the nature of the experiment. For example, only the geometrical similitude and the viscous parameter requirement may be considered in the static rigid model, and on the other hand all the similarity requirements should in principle be satisfied in the full model aeroelastic test. In what follows the models to be used in dynamic aeroelastic test are reviewed.

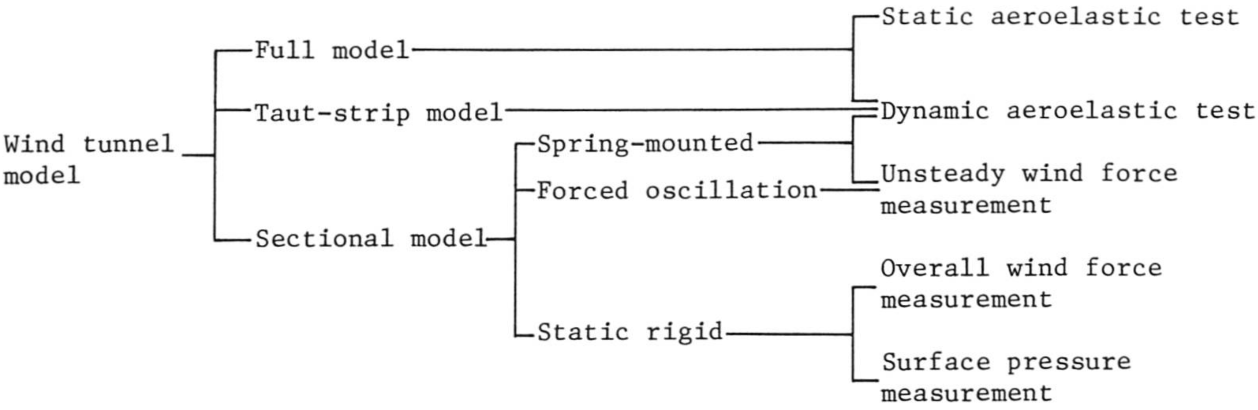


Fig. 13 Bridge models and their application in wind tunnel tests

(1) The Full Model

A full model is an aeroelastic model of the complete structure. As above mentioned, for dynamic studies it is necessary to model the mass distribution and the elastic characteristics as well as the geometric configuration of the prototype according to the scaling principles. The linear scale ratio for a large prototype may be very small, so that large reductions in size will introduce difficulties in model design or a very wide test section will be required. The wind tunnel of the University of Tokyo, shown in Fig. 12, has a 16 m x 1.9 m test section which was designed to adapt to long span bridge models. Vertically inclined winds can be obtained by adjusting the deflectors at the entrance of the test section. The National Aeronautical Establishment 30 ft x 30 ft wind tunnel in Canada seems to have the largest test section, together with the rather long fetch of tunnel floor. These wind tunnels are of course exceptional. In general, a full model test is costly and time consuming in the case of a very long structures having complicated configurations.

In designing a full model with large reduction scale and complex shape, appropriate materials are not usually available and compromise procedures have to be adopted which yield necessary similitude conditions. In case of bridge models, for example, the road decks and girders are made up of short rigid segments of the correct external shape and interconnected to internal metal frames to provide the correct overall elastic stiffness. Moreover, the introduction of prescribed amount of structural damping presents certain fundamental difficulties in the full model test. Nevertheless, the three-dimensional full model test will be significant in such cases that the wind environment along the bridge axis varies due to geographical situations (e.g. Fig. 12), that the aerodynamic properties of the cross section along the bridge axis vary, or that the behaviour of the structure at various erection stages must be explored.

(2) The Sectional Model

Use of spring-mounted rigid models of typical lengths of the structure is an alternative but the most conventional practice to check the aerodynamic stability of long span bridges. Usually two pairs of spring unit support the model to allow heaving and pitching motions, so that the motions are confined to represent only one combination of vertical and torsional mode of vibration of the structure at a time. The end plates of appropriate size are attached to the model, as shown in Fig. 14, so that two-dimensional flow conditions are maintained over the entire length of the model. Under these circumstances, the sectional model test presupposes the validity of strip theory, in which the structure is divided into thin chordwise strips that are assumed to behave independently so that the overall structural performance can be calculated by spanwise integration. The air flow can be made turbulent, but it is not fundamentally possible to simulate all of the natural wind properties in the sectional model test.

In short, although low cost, relatively large scale and easy modification to the model are the advantages of the sectional model test, the limitations in view of two-dimensionality are unavoidable. Furthermore, when the ratio of the natural frequencies in torsion and vertical bending is large, it is sometimes difficult to attain satisfactory similitude to the model system.

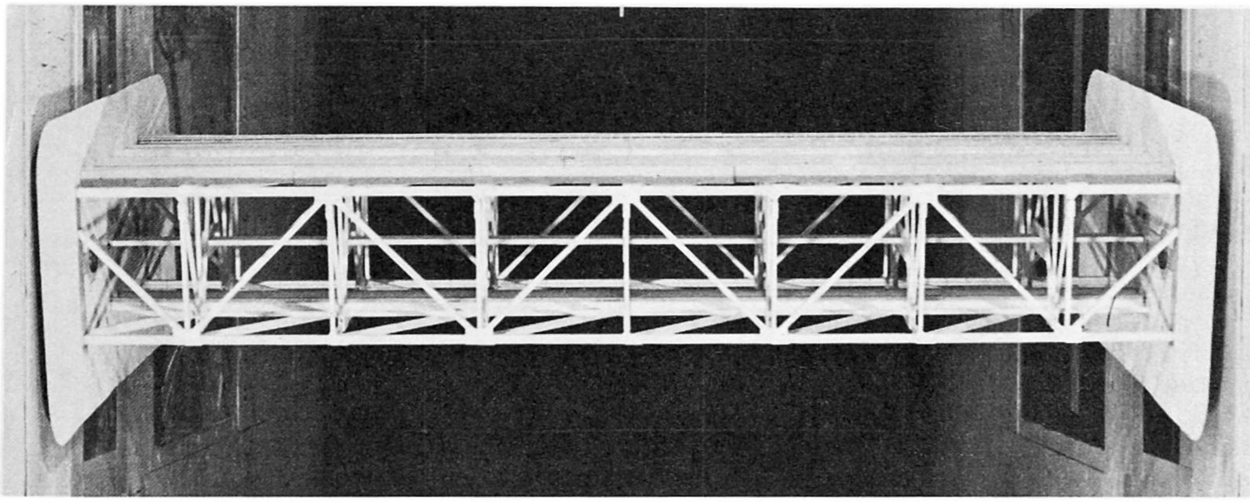


Fig. 14 Sectional model of bridge deck

(3) The Taut-Strip Model

The bridge deck model is attached to two parallel taut wires tensioned across the test section of a wind tunnel. These wires are designed to provide with the stiffness similarity of the model. The taut-strip model has the advantages over the sectional model tests in that it can be tested in appropriately simulated wind and the three-dimensionality of the model deformations are included, while it is simpler and less costly than the full model. However, the realization of the three-dimensionality of the structural behaviour in the taut-strip model falls short of the full model, while the experimental scheme will be not so simple as the sectional model test.

The discussion on the wind tunnel model testing given so far is equally applicable to the problem of reproducing the dynamic behaviour of towers, stacks and buildings. In these vertically standing structures, however, the influence of the gravitational parameter is usually insignificant. Only if details of the structural system can be easily reproduced, the full replica models are practicable. Cooling towers, chimneys or shells are examples of such structures, for which the special materials like a metalized epoxy should be sought to design the model satisfying the similitude requirements. In case of buildings and likewise structures, a few lower modes of vibration are dominant. Thus the simplified, equivalent aeroelastic models are often used. Their concepts are the same as the full model of long span bridges cited previously, but the number of degrees-of-freedom is usually finite. A common procedure is to mount the two-sway-mode rigid model on the spring-damper system fixed to the rotation table at the wind tunnel floor. When the cross section of the structure changes considerably along its elevation, models with more than two degrees-of-freedom may be employed to obtain more exact dynamic effects.

5. CONCLUSIONS

Some structural engineers may feel awkward in approaching to 'the problems of the wind effects on structures. One reason seems to come from that the problems are interdisciplinary stretching over meteorology, fluid dynamics and structural engineering, while another is the need of special experimental techniques in most cases. The wind-resistant design practice has been successfully improved in recent years, but structural engineers have to take note of pitfalls with the modern developments in design and construction.

In spite of apparent improvement in our knowledge of the phenomena and investigative procedures, there are still gaps to fill in this field. The most concerned for us is the correlation between the model test in the wind tunnel and the prototype behaviour in the field. While the full-scale measurements with large structures are costly and laborious, it is gratifying to find that they have recently been conducted in several countries. However, the modern structures are carefully designed against wind action and the attack of very strong wind at the site is infrequent, and therefore these full-scale measurements have been confined to the buffeting response or at best the limited-amplitude vortex excitation. Under these circumstances, the large scale bridge deck model test in the natural wind was conducted by the Honshu-Shikoku Bridge Authority, Japan, aiming at mediation between the wind tunnel model and the full-scale prototype, though it was two-dimensional test (Fig. 15).

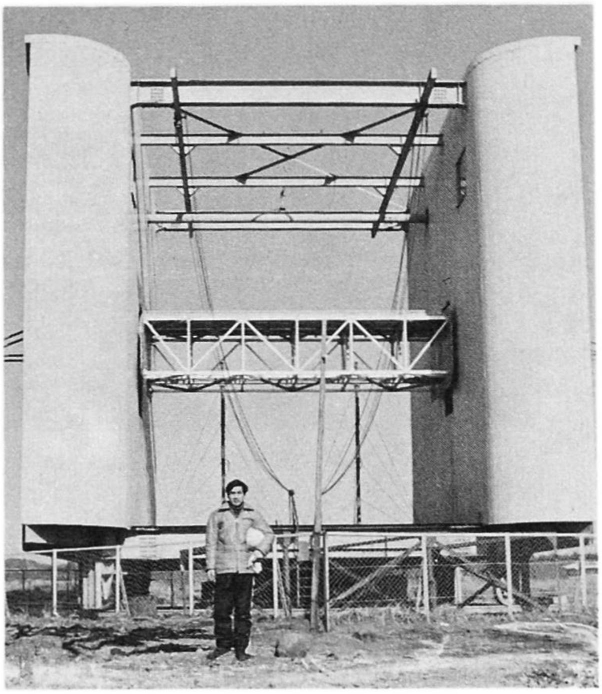


Fig. 15 Large scale sectional model in field

Finally, the items to be further pursued are

—— the effects of turbulence of the natural wind and three-dimensional behaviour of the structure on its aerodynamic instability,

—— the development of more realistic analytical procedure to predict the dynamic response of the structure in conjunction with the refinement of experimental techniques, and

—— the establishment of design rationale to ensure safety and serviceability of the structure in view of the aerodynamic stability.

NOTATIONS

c	damping coefficient	L	aerodynamic lift component	i	unit of imaginary number
d	section depth	M	aerodynamic moment component	I	polar moment of inertia
f	frequency	t	time	m	mass of a structure
F	aerodynamic force	$\bar{V}$	reduced wind speed	S	Strouhal number
C	aerodynamic force coefficient	$Z(.)$	frequency response	V	wind speed
E	Young's modulus	δ	logarithmic decrement	y	transverse displacement
$f(.)$	function	$\Phi(.)$	indicial aerodynamic response	α	angle of incidence
g	gravitational acceleration	θ	torsional displacement	ϕ	phase difference
h	characteristic length of a section; section height	ν	kinematic viscousity of air	ρ	mass density
				τ	non-dimensional time

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