State-of-the-Art on Long-Span Bridge Aerodynamics in China

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Summary

The State-of-the-Art on long-span bridge aerodynamics in China is presented in this paper with an emphasis on three aspects: contributions in the development of aerodynamic theories, probability-based assessment method in evaluating bridge aerodynamics, and aerodynamic vibration control. The theoretical development discussed in this paper comprises the full-mode flutter analysis method, some new findings in flutter mechanism and stabilization as well as Computational Fluid Dynamics techniques and application. A series of probabilistic assessment approaches is then proposed and applied in wind induced vibration investigation including flutter instability, buffeting response and vortex-induced vibration. The final aspect focuses on the aerodynamic vibration control of long span bridges experienced in China.

Introduction

With the beginning of new reform and open policy some 25 years ago, the development of bridge engineering in China entered a golden period. More than two hundred long-span bridges have been built across China's main rivers or bays. Till date, China keeps the world record span length for both steel arch bridges namely, the Lupu Bridge with main span of 550 m, and the reinforced concrete arch bridge, Wanxian Bridge with a 420 m long center span. With the opening of Runyang Bridge in May 2005, the span length of suspension bridge in China has been raised up to 1490 m. The longest cable-stayed bridge in China is the second Nanjing Bridge having a main span of 628 m.

With the rapid increase of bridge span length, bridge structures are becoming lighter and more flexible, which have resulted in an increased importance of aerodynamic study and design related to wind action, including aerodynamic instability, stochastic buffeting, and vortex induced vibration. The traditional theory of bridge aerodynamics is based on linear and modal superposition methods for flutter and buffeting prediction. For super long-span bridges, however, the influence of basic hypothesis in the traditional theory might not be negligible factors in aerodynamic or aeroelastic performance. Furthermore, the current practice of treating wind action on a long-span bridge usually adopts a deterministic approach based on direct wind tunnel tests or theoretical calculation with experimentally obtained parameters. Since many of these testing results and parameters involved in the method are physically uncertain variables and sometimes subjectively assumed values due to a lack of complete knowledge, it would make investigation more objective by taking a probabilistic assessment approach to determine the probability of occurrence of an unexpected limit state event during the structure's service life. Finally, if a long-span bridge design is predicted through a deterministic or stochastic method to have deficiencies in the aspect of aerodynamic performance, especially of aerodynamic instability, some countermeasures should be adopted to improve aerodynamic performance in order to meet with the appropriate wind resistance requirements.

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Theoretical Contributions

The first aspect is related to the recent theoretical contributions to bridge aerodynamics, including the full-mode-participation flutter analysis method, some new findings about flutter patterns and stabilization mechanism, as well as the development and application of computational fluid dynamics to deal with load coefficients, flutter instability and vortex induced vibration.

Full-Mode Flutter Analysis

With the application of flutter derivatives and three-dimensional structure models, a great deal of analytical investigations related to flutter problems of long-span bridges have been made so far, but almost all three-dimensional flutter analysis are carried out in the frequency domain and based on the idea of mode superposition, which is usually called multi-mode method [1]. The basic assumption of this method is that a dynamic coupling between natural modes takes place through self-excited aerodynamic forces. Although the multi-mode method can obtain quite precise results, theoretically, ideal flutter analysis should include the influence of all natural vibration modes, and not the selected ones [2][3]. The full-mode flutter analysis method, based on the whole system including a structure and flow around it instead of natural vibration modes, has currently been developed for long-span cable supported bridges [4][5].

For a bridge discreted as a n degree-of-freedom (DOF) structure whose deflections are described in terms of a deflection vector, {δ}, the equations of motion can be generally expressed as:

$$[M_s]\{\ddot{\delta}\}+[C_s]\{\dot{\delta}\}+[K_s]\{\delta\}=\{F\} \qquad (1)$$

where, $[M_s]$ is structural mass matrix; $[K_s]$ is structural stiffness matrix; $[C_s]$ is structural damping matrix; and $\{F\}$ is externally applied force vector, and particularly the self-excited force induced by flow in flutter analysis,

$$\{F\}=\{F_d\}+\{F_s\}=[A_d]\{\dot{\delta}\}+[A_s]\{\delta\} \qquad (2)$$

where, $\{F_d\}$ and $\{F_s\}$ are aerodynamic damping and stiffness forces, respectively; and $[A_d]$ and $[A_s]$ are the corresponding damping and stiffness matrices, respectively, and are represented by flutter derivatives.

Substituting Equation (2) into Equation (1) yields a new form of structural equation of motion, called system flutter equation:

$$[M]\{\ddot{\delta}\}+[C]\{\dot{\delta}\}+[K]\{\delta\}=\{0\} \tag{3}$$

where, $[M]$ is system mass matrix, $[M] = [M_s]$; $[K]$ is system stiffness matrix, $[K] = [K_s] - [A_s]$; and $[C]$ is system damping matrix, $[C] = [C_s] - [A_d]$.

Based on the assumption that the amplitude of vibration is small at the onset of flutter, it is sufficient to analyze a vibration problem with exponential time dependence function in solving the system flutter equation, as follows:

$$\{\delta\}=\{\phi\}e^{\lambda t} \tag{4}$$

Substitution for $\{\delta\}$ and its derivatives from Equation (4) into Equation (3) gives

$$\left(\lambda^2[M]+\lambda[C]+[K]\right)\{\phi\}=\{0\} \tag{5}$$

where, l is eigenvalue of the flutter system and {f} is the corresponding eigenvector of the system. Flutter critical condition is indicated by a real part of a complex eigenvalue becoming zero at the lowest wind speed. Equation (5) can be transformed to a linearized form or normalized eigenvalue equation in the direct form and the inverse form, as follows:

$$[D]\{x\}=\lambda\{x\};\ [E]\{x\}=\gamma\{x\} \tag{6a, b}$$

$$\{x\}=\begin{Bmatrix}\lambda\{\phi\}\\ \{\phi\}\end{Bmatrix} \tag{7}$$

where λ and γ are, respectively, direct eigenvalue and inverse eigenvalue, $\gamma=1/\lambda$; $\{\phi\}$ is the corresponding flutter vector; and $[D]$ and $[E]$ are dynamic matrix and inverse dynamic matrix, respectively.

$$[D]=\begin{bmatrix}-[M]^{-1}[C] & -[M]^{-1}[K]\\ [I] & [0]\end{bmatrix} \tag{8}$$

$$[E]=\begin{bmatrix}[0] & [I]\\ -[K]^{-1}[M] & -[K]^{-1}[C]\end{bmatrix} \tag{9}$$

The direct form of a normalized eigenvalue Equation (6a) can be used for multi-mode flutter analysis, and the full-mode flutter analysis should employ the inverse form, Equation (6b), which can be used to solve for partial eigenvalues by the Inverse Vector Iteration with QR Transformation developed by the authors [4]. Several typical examples of multi-mode and full-mode flutter analysis can be found in [5], [6], [7].

$$\xi_\alpha=\frac{\xi_{\alpha s}\omega_{\alpha s}}{\omega_\alpha}-\frac{1}{2}\frac{\rho B^4}{I}A_2^*-\frac{1}{2}\frac{\rho B^4}{I}\frac{\rho B^2}{m_h}\Omega_{h\alpha}F_{h\alpha}-\frac{1}{2}\frac{\rho B^4}{I}\frac{\rho B^2}{m_p}\Omega_{p\alpha}F_{p\alpha} \tag{10}$$

$$\xi_h=\frac{\xi_{hs}\omega_{hs}}{\omega_h}-\frac{1}{2}\frac{\rho B^2}{m_h}H_1^*-\frac{1}{2}\frac{\rho B^4}{I}\frac{\rho B^2}{m_h}\Omega_{\alpha h}F_{\alpha h}-\frac{1}{2}\frac{\rho B^2}{m_h}\frac{\rho B^2}{m_p}\Omega_{\alpha p}F_{\alpha p} \tag{11}$$

$$\xi_p=\frac{\xi_{ps}\omega_{ps}}{\omega_p}-\frac{1}{2}\frac{\rho B^2}{m_p}P_5^*-\frac{1}{2}\frac{\rho B^4}{I}\frac{\rho B^2}{m_p}\Omega_{\alpha p}F_{\alpha p}-\frac{1}{2}\frac{\rho B^2}{m_h}\frac{\rho B^2}{m_p}\Omega_{hp}F_{hp} \tag{12}$$

$$\Omega_{ij}=\frac{\omega_i^2}{\sqrt{\left(\omega_j^2-\omega_i^2\right)^2+4\left(\xi_i\omega_i\omega_j\right)^2}} \tag{13}$$

$$(i, j=\alpha, h, p)$$

$$V_\alpha=\left(\frac{1}{C_\alpha},\frac{\rho B^2\Omega_{h\alpha}\sqrt{H_2^{*2}+H_3^{*2}}}{m_hC_\alpha},\frac{\rho B^2\Omega_{p\alpha}\sqrt{P_2^{*2}+P_3^{*2}}}{m_pC_\alpha}\right) \tag{14}$$

$$V_h=\left(\frac{\rho B^4\Omega_{\alpha h}\sqrt{A_1^{*2}+A_4^{*2}}}{J_mC_h},\frac{1}{C_h},\frac{\rho B^2\Omega_{ph}\sqrt{P_1^{*2}+P_4^{*2}}}{m_pC_h}\right) \tag{15}$$

$$V_p=\left(\frac{\rho B^4\Omega_{\alpha p}\sqrt{A_5^{*2}+A_6^{*2}}}{J_mC_p},\frac{\rho B^2\Omega_{hp}\sqrt{H_5^{*2}+H_6^{*2}}}{m_hC_p},\frac{1}{C_p}\right) \tag{16}$$

where

$$C_\alpha=\sqrt{1+\left(\frac{\rho B^2}{m_h}\Omega_{\alpha h}\sqrt{H_2^{*2}+H_3^{*2}}\right)^2+\left(\frac{\rho B^2}{m_p}\Omega_{\alpha p}\sqrt{P_2^{*2}+P_3^{*2}}\right)^2} \tag{17}$$

$$C_h=\sqrt{\left(\frac{\rho B^4}{J_m}\Omega_{h\alpha}\sqrt{A_1^{*2}+A_4^{*2}}\right)^2+1+\left(\frac{\rho B^2}{m_p}\Omega_{hp}\sqrt{P_1^{*2}+P_4^{*2}}\right)^2} \tag{18}$$

$$C_p=\sqrt{\left(\frac{\rho B^4}{J_m}\Omega_{p\alpha}\sqrt{A_5^{*2}+A_6^{*2}}\right)^2+\left(\frac{\rho B^2}{m_h}\Omega_{ph}\sqrt{H_5^{*2}+H_6^{*2}}\right)^2+1} \tag{19}$$

Flutter Mechanism and Stabilization

Based on the concept of full-degree coupling analysis, a two-dimensional three DOF flutter analysis method was proposed to reveal the mechanism of flutter oscillation [8], and was applied in the theoretical analysis of stabilizing mechanism on a series of typical bridge cross sections [9].

For a two-dimensional bridge section model with three DOF including heaving, swaying and torsion, the equations of motion can be expressed in terms of eighteen flutter derivatives. The oscillation frequencies and damping ratios of a three DOF section model can be derived and represented by the combination of flutter derivatives and phase lags between motions. Through double iterations of wind speed and oscillation frequency, the total damping ratios can be written by the formulas (10), (11), (12) [10], where m_h, m_p and J_m are mass and mass moment of inertia in the corresponding degrees of freedom; ξ_{hs}, ξ_{ps} and ξ_{as} are structural damping ratios; ω_{hs}, ω_{ps} and ω_{as} are natural circular frequencies; ρ is air mass density; B is bridge deck width; ω_h, ω_p and ω_a are iterative circular frequencies; H_i^*, P_i^* and A_i^* (i = 1, 2, …, 6) are dimensionless flutter derivatives; F_{ij} ($i, j = a, h, p$) is dimensionless parameters defined by flutter derivatives and phase lags; and Ω_{ij} is dimensionless equivalent frequency between two motions and is defined in Eq. (13).

Based on the above full-degree coupling formulations, the two-dimensional three DOF flutter analysis can be performed to simultaneously investigate the relationship between systematic oscillation parameters and aerodynamic derivatives, and one of the most important result of this analysis is the coupling effect of the various DOF in flutter oscillation. The participation level of motion in each DOF at the flutter onset can be described by three flutter modality vectors with the endpoints as in Eqs. (14), (15), (16), (17), (18) and (19) [10].

With the full-degree-participation method, five main groups comprising of thirteen cross sections (*Fig. 1*) have been systematically investigated by employing a simplified way with only two DOF including heaving and torsion [9]. For the first two groups of

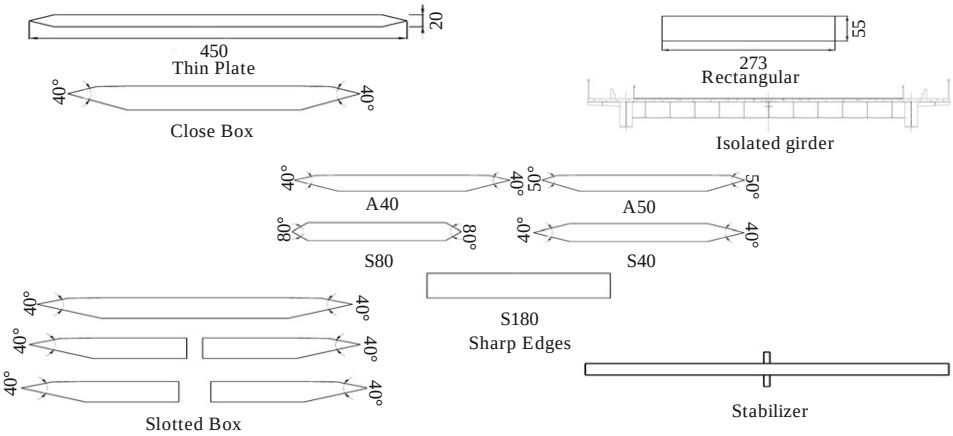


Fig. 1: Configurations of various cross sections

cross sections comprising of streamlined thin plate, close box, bluff rectangular and isolated girders, the flutter patterns and corresponding critical speeds can be calculated by the method with the same properties of structural dynamics. It was found that the more streamlined cross section is,

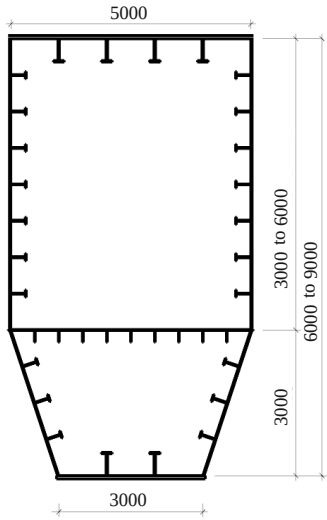


Fig. 2: Rib section of Lupu Bridge

the more the heaving mode will participate at the flutter onset and the higher flutter speed can be obtained. The third group of cross sections varies in the flutter patterns and critical speeds vary with the sharpness of edges. With increase in the edge sharpness, participation of the heaving DOF and the flutter speed increase accordingly. In order to reduce the aerodynamic forces based on the configuration improvement of cross section of bridge decks, there are two effective solutions, slotted box and the deck with central stabilizer. The flutter stabilization of both devices can be illustrated by flutter patterns described by flutter modality vectors. It seems that the participation of heaving DOF helps to steadily increase flutter critical speed for both slotted box and the cross section with central stabilizer.

Computational Fluid Dynamics Techniques and Application

In general, there are two kinds of methods used in computational fluid dynamics, that is, the finite element-based method and discrete vortex-based method. The finite element method code FEM-FLUID was developed at Tongji University in 1999 [11], while the random vortex method was developed for the computer-based code RVM-FLUID at Tongji University in 2002 [12].

A typical application of CFD techniques refers to the vortex induced vibration simulation taking as an example, for Lupu Bridge in Shanghai, China. Lupu Bridge is a half-through arch bridge with the world record-breaking span length of 550 m. Two inclined steel arch ribs are 100 m high from the bottom to the crown, and each has modified rectangular box section with 5 m in width and a depth of 6 m at the crown and 9 m at the rib bases (*Fig. 2*).

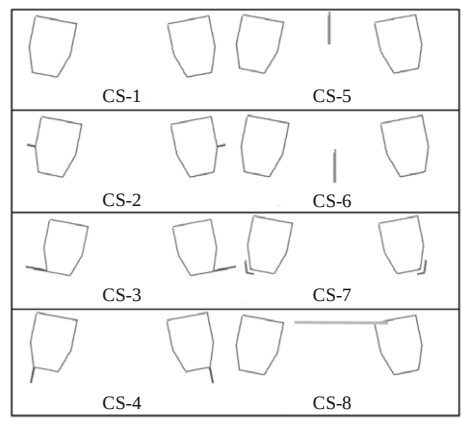


Fig. 3: Preventive means of ribs

The random vortex method code RVM-FLUID was performed on the two-dimensional model of a rib cross section with the average depth (H) of 7,5 m. It was found that severe vortex induced oscillation occurs at an amplitude of 0,028 H at the reduced frequency or Strouhal number S_t = 0,156. In order to improve vortex induced vibration of the bluff cross section of the ribs, several aerodynamic preventive means as shown in *Fig. 3* were numerically tested.

No.	Configuration	St.	z_{max}/H
CS-1	original	0,156	0,028
CS-2	2m plates	0,220	0,025
CS-3	2m plates (H)	0,137	0,034
CS-4	2m plates (V)	0,137	0,032
CS-5	4m stabilizer	0,137	0,032
CS-6	4m stabilizer	0,156	0,017
CS-7	4m deflectors	0,175	0,023
CS-8	cover plate	0,156	0,011

Table 1: Numerical results with CFD

The calculation results including Strouhal number and relative amplitude, z_{max}/H, are listed in *Table 1* [13]. There are four means including CS-2, CS-6, CS-7 and CS-8, which can reduce the amplitude of vortex oscillation. Among these four means, the best solution is the full cover plate, CS-8, which can minimize the amplitude to only about 40% of that in the original configuration.

Probability-Based Assessment

Besides the theoretical contributions, one of the most important issues of bridge aerodynamics in China is the development and application of probabilistic assessment or reliability analysis of wind induced vibration of long-span bridges [14]. Aerodynamic response of bridges subjected to wind loading is related to statistically random properties of atmospheric flow fields and physically uncertain variables of line-like structures. It is therefore more scientific and practicable to employ probability-based approaches to define the bridge failure due to flutter instability, buffeting response and vortex-induced oscillation with an occurrence probability for a given return period rather than providing a single safety factor [15].

Reliability Analysis of Flutter

Aerodynamic instability takes place when a bridge is exposed to wind speeds above a certain critical value which can be predicted by wind tunnel tests or theoretical calculation with experimentally identified parameters. Following this definition, the reliability analysis of the issue can be formulated as a limit state problem in which the critical flutter speed U_{cr} is exceeded by the extreme wind speed U_e for a given return period [16][17].

$$M = U_{cr} - U_e = C_w U_f - G_s U_b \tag{20}$$

where C_w is the conversion factor from a scaled model to the prototype structure; U_f is the experimentally determined basic flutter speed with some uncertainties of structural properties; G_s is the gust speed factor to account the influence of wind fluctuation and its horizontal correlation; and U_b is the annual maximum wind speed at the bridge deck location. The best-fitted distribution of the wind speeds in most meteorological stations in China has been confirmed to follow a Gumbel distribution expressed as

$$F(U_b) = \exp\left[-\exp\left(-\frac{U_b - b}{a}\right)\right] \tag{21}$$

in which the parameters $1/a$ and b are the dispersion and mode, respectively.

Bridge	Name	P_F	K
Cable-stayed	Qingzhou	$5{,}0 \times 10^{-3}$	1,40
	2nd Nanjing	$1{,}9 \times 10^{-8}$	3,27
	Jingsha	$7{,}1 \times 10^{-8}$	3,06
	Haikou	$1{,}3 \times 10^{-5}$	2,31
	Nanpu	$5{,}1 \times 10^{-3}$	1,39
	Yangpu	$8{,}5 \times 10^{-4}$	1,68
Suspension	Yichang	$8{,}6 \times 10^{-6}$	2,38
	Jiangyin	$5{,}0 \times 10^{-4}$	1,76
	Humen	$1{,}4 \times 10^{-3}$	1,61
	Honguan	$9{,}5 \times 10^{-5}$	2,02

Table 2: Failure probability due to flutter

Although the failure probability from the safety margin, M, may be efficiently calculated by some approximate reliability methods or by applying simulation techniques such as Monte Carlo method, the first-order reliability method, in particular, the extended design point approach (EDP), is applied to calculate reliability index β and failure probability P_F [16], [17].

$$F_{PR}(B) = 1 - \exp\left\{-\frac{\pi T}{2}\sum_{k=1}^{n}\frac{\sigma_{\dot{x}k}}{\sigma_{xk}}\left[\exp\left(-\frac{b_{1k}^2}{2\sigma_{xk}^2}\right) + \exp\left(-\frac{b_{2k}^2}{2\sigma_{xk}^2}\right)\right]\Psi(b_{1k}, b_{2k})\right. \tag{22}$$

$$\Psi(b_{1k}, b_{2k}) = \prod_{i=1, i\neq k}^{n}\left[\Phi\left(\frac{b_{1i}}{\sigma_{xi}}\right) - \Phi\left(-\frac{b_{2i}}{\sigma_{xi}}\right)\right] \tag{23}$$

$$F_{MR}(B) = 1 - \exp\left\{-\frac{\pi T}{2}\sum_{k=1}^{n}\frac{\sigma_{\dot{x}k}}{\sigma_{xk}}\left[s_{1k}\exp\left(-\frac{b_{1k}^2}{2\sigma_{xk}^2}\right) + s_{2k}\exp\left(-\frac{b_{2k}^2}{2\sigma_{xk}^2}\right)\right]\prod_{i=1, i\neq k}^{n}\left[\Phi\left(\frac{b_{1i}}{\sigma_{xi}}\right) - \Phi\left(-\frac{b_{2i}}{\sigma_{xi}}\right)\right]\right\} \tag{24}$$

where
$$s_{ik} = \frac{1 - \exp\left(-\sqrt{\frac{\pi}{2}}q_k\frac{b_{ik}}{\sigma_{xk}}\right)}{1 - \exp\left(-\frac{b_{ik}^2}{2\sigma_{xk}^2}\right)} \quad (i = 1, 2) \tag{25}$$

$$q_k = \sqrt{1 - \frac{\sigma_{\dot{x}k}^2}{\sigma_{xk}\sigma_{\ddot{x}k}}} \tag{26}$$

$$F_{RD} = 1 - \prod_{i=1}^{m}\left[1 - \exp\left(-\frac{b_{1i}^2}{2\sigma_{xi}^2}\right)\right]^{N_i}\left[1 - \exp\left(-\frac{b_{2i}^2}{2\sigma_{xi}^2}\right)\right]^{N_i} \tag{27}$$

$$F_{GD} = 1 - \prod_{i=1}^{m}\left[\Phi\left(\frac{b_{1i}}{\sigma_{xi}}\right)\right]^{N_i}\left[\Phi\left(\frac{b_{2i}}{\sigma_{xi}}\right)\right]^{N_i} \tag{28}$$

where
$$N_i = \frac{\sigma_{\dot{x}i}T}{2\pi\sigma_{xi}} \tag{29}$$

The reliability analysis model formulated by four independent random variables was adopted to carry out the probabilistic reassessment and calibration for flutter instability of ten cable-supported bridges including six cable-stayed bridges and four suspension bridges in China. The wind tunnel tests of these bridges with aeroelastic models were performed at Tongji University recently. On the basis of the EDP approach, a computational program was developed to compute the reliability indices β and failure probabilities P_F, which are numerically listed in *Table 2* and compared to the traditional safety factors K [18].

First Passage Analysis of Buffeting

Since buffeting oscillation is a random dynamic response, a more reasonable approach is to estimate the highest level of response and ensure that the probability of its occurrence during the structure's service period is at an acceptable level. A general first passage model was suggested by the authors to deal with buffeting failure in which the maximum response reaches the lower bound level corresponding to the damage mode [19][20].

Perhaps the simplest approximation to the first passage probability is obtained by assuming that the threshold crossings in a stationary Gaussian response occur so rarely that they can be considered as statistically independent events. The first passage probability based on an assumption of a Poisson response process $X(t)$ in an interval of duration T can be calculated by Eqs. (22) and (23), where $\phi(x)$ is the standardized Gaussian distribution function; b_{1k} and $-b_{2k}$ are the upper and lower failure bounds of a n DOF structure, respectively; σ_x^2 and $\sigma_{\dot{x}}^2$ are the variances of $X(t)$ and $\dot{X}(t)$, respectively.

Introducing a bandwidth factor q, the first-passage probability based on the Markov assumption can be calculated by Eq. (24), (25) and (26).

Another approximate approach is based on the limit distributions of random response peaks. According to the extreme distribution theorem, any probability distribution function of peak responses must have two limited forms, the upper limit followed Rayleigh distribution and the lower limit followed Gaussian distribution. Assuming the threshold crossings at the upper barrier and at the lower barrier are statistically independent, the upper bound and the lower bound of the first passage probability can be expressed as in Eqs. (27), (28) and (29).

Based on the current buffeting theories and reliability assessment meth-

ods, a numerical simulation approach to the first passage probability, P_{Num}, for a long-span bridge under buffeting actions was also developed [21].

Jiangyin Suspension Bridge with the central span of 1385 m is taken as a numerical example of calculating the first passage probabilities of vertical, lateral and torsional displacements. The design wind velocity of 40 m/s is chosen for the bridge at the deck level. The first passage probabilities of the vertical (δ_V), lateral (δ_L) and torsional (δ_T) displacements induced by wind buffeting at the mid-span (L/2) are given in *Table 3* [22].

Items	δ_V (m)	δ_L (m)	δ_T (deg)
σ_x	0,2442	0,0162	0,0032
b_1	0,2442	0,0162	0,0032
b_2	–0,8360	–1,5250	–0,0175
F_{PR}	–0,8360	–1,5250	–0,0175
F_{MR}	–0,8360	–1,5250	–0,0175
F_{RD}	3,9E-1	–1,5250	–0,0175
F_{GD}	1,0E-1	<1E-10	5,1E-6
P_{Num}	1,0E-1	<1E-10	5,1E-6

Table 3: First passage probability of buffeting response in Jiangyin Bridge

Probabilistic Evaluation of Vortex Induced Vibrations

The first probabilistic evaluation case of vortex induced vibration (VIV) completed in China is that of the Lupu Bridge with a bluff arch rib section, a configuration for which servere VIV in vertical bending occurs in the construction stages including the maximum rib cantilever (MRC) and the completed arch rib (CAB) as well as the in-service stage completed bridge structure (CBS) [23]. The probabilistic evaluation began with the determination of vortex-shedding wind speeds, which were obtained through the aeroelastic full bridge model testing, in the antisymmetrical (a) and symmetrical (s) vertical bending modes, as shown in *Table 4* [13].

Speed	MRC	CAR	CBS
U_a(m)	10 to 20	20 to 36	14 to 25
U_s(m)	20 to 45	40 to 55	24 to 40
Both	10 to 45	20 to 55	14 to 40

Table 4: Vortex-shedding wind speeds of Lupu Bridge

The statistical analysis of the wind velocities at the bridge site was based on three main meteorological stations around Lupu Bridge, that is, Baoshan Station at the north, Chuansha Station at the east and Longhua Station at the southwest.

In order to obtain the longest accumulative period of possible vortex-induced oscillation, the statistical results at these three stations can be grouped into four velocity intervals: 0–10 m/s, 10–14 m/s, 14–20 m/s and 20–28 m/s. The accumulative period can be separately calculated using each station data in turn, then these three results can be averaged. The longest accumulative period, T_a, for given return years of N was finally obtained based on these three stations.

Based on the statistical analysis, the best-fitted distribution of maximum samples at each station was confirmed to be a Gumbel distribution described by Equation (21), which represents the probability value under $U < U_b$. The probability value P_u under $U_1 < U < U_2$ can be accordingly calculated as:

$$P_u\left(U_1 < U < U_2\right) = F\left(U_2\right) - F\left(U_1\right) \quad (30)$$

In considering different occurrence frequencies p_i at different wind directions i, the probability of the first occurrence of vortex-induced oscillation can be obtained by the following equation:

$$P = \sum_{i=1}^{16} P_{ui} p_i = \sum_{i=1}^{16} \left[F_i\left(U_2\right) - F_i\left(U_1\right)\right] p_i \quad (31)$$

Using all data of F_i (U) and p_i, the annual probability of the first occurrence, P_f, can be calculated for each station and then for the average of these three stations. The main results of probabilistic evaluation of vortex induced vibration of Lupu Bridge can be summarized in *Table 5* [13] [15].

Result	MRC	CAR	CBS
N(y)	10	10	100
T_a(day)	28	0,45	32
P_f(1/y)	0,163	$8{,}7 \times 10^{-5}$	0,072

Table 5: Probabilistic evaluation results of vortex-induced vibration of Lupu Bridge

Aerodynamic Vibration Control

For long span bridges with its intrinsic limit in aerodynamic performance, especially in destructive vibration, it is necessary to use countermeasures to control aerodynamic vibration to meet with the appropriate wind resistance requirements. The following section in this paper focuses on the engineering experience gained from several long-span bridges in aerodynamic vibration control and verification testing in China [24].

Fig. 4: Aeroelastic model of Yadagawa Bridge

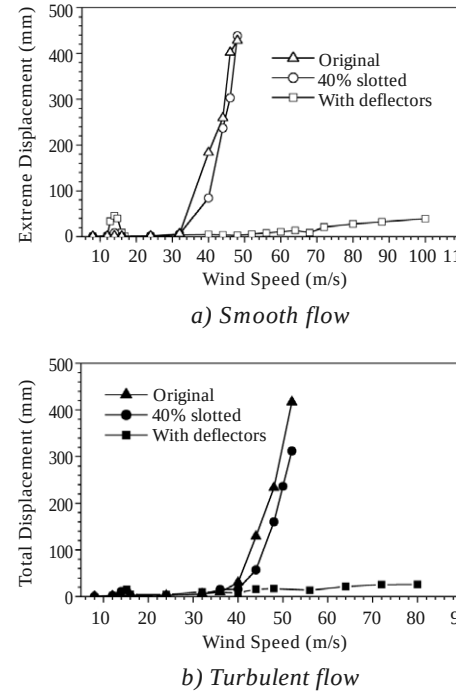


Fig. 5: Experimental results of Yadagawa Bridge

Galloping Control in Yadagawa Bridge

The object bridge, Yadagawa Bridge in Japan, is a steel-concrete composite bridge with a 84,2 m main span and two side spans of 67,1 m. The composite deck consists of two separated steel boxes and a reinforced concrete plate, with a width of 7,5 m. The depth of the steel girder is 3,2 m at both continuous supports and 2,2 m at the midpoint of the center span and at the two ends of the bridge, respectively. The very bluff cross section of the bridge may bring about the concern of galloping instability.

Since a substantial change of the basic cross section was not allowed in this investigation, two kinds of aerodynamic preventive means were tried, the slotted deck and the corner deflectors, which

Fig. 6: Aeroelastic model of Lupu Bridge

were selected based on the results of CFD analysis. The further experimental investigation of galloping-type instability through an aeroelastic full model was carried out in the TJ-3 (3rd Wind Tunnel at Tongji University) boundary layer wind tunnel with the working section of 15 m in width, 2 m in height, and 14 m in length shown in *Fig. 4*.

The 1:50 scale aeroelastic model respectively simulates three structural configurations including the original structure, the slotted structure and the structure with corner deflectors [25]. The results of the extreme deck deflections at the mid-span are graphically indicated and compared in *Fig. 5 a* under the smooth flow and *Fig. 5 b* under the turbulent flow [26].

Vortex Induced Vibration Reduction in Lupu Bridge

In order to confirm the numerical results with the code RVM-FLUID mentioned before, the aeroelastic testing of Lupu Bridge was carried out in the same wind tunnel. The 1:100 scale aeroelastic model of the full bridge simulates three situations including the maximum rib cantilever (MRC), the completed arch ribs (CAR) and the completion bridge structure (CBS)

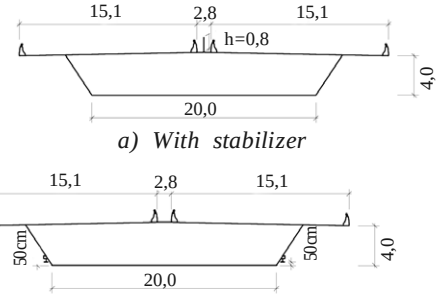
Fig. 7: Aeroelastic model of East Sea Bridge

as shown in *Fig. 6*. The wind tunnel testing was conducted including three situations and the bridge configurations; without or with preventive means A, the full cover plate, and means B, the cover plate with 30% air vent [27].

Table 6 lists the total experimental results including the maximum displacements of vertical bending oscillation of the arch ribs at the mid span (*L/2*) and the quarter span (*L/4*). It can be concluded that the preventive means A or B cases effectively reduce the amplitudes of vortex-induced oscillation [13].

Bridge Conf.	Prev. mean	U_{VIV} (m/s)	$z_{L/2}$ (m)	$z_{L/4}$ (m)
MRC	Orig.	16,3	0,813	0,216
	M-A	17,5	0,590	0,166
	M-B	16,3	0,249	0,069
CAR	Orig.	31,3	0,115	0,634
	M-A	33,8	0,066	0,358
	M-B	31,3	0,047	0,359
CBS	Orig.	17,5	0,040	0,164
	M-A	17,5	0,067	0,070
	M-B	17,5	0,067	0,023

Table 6: Experimental results of Lupu Bridge (U_{VIV} in m/s and z in m)



a) With stabilizer

b) Modified gantry place

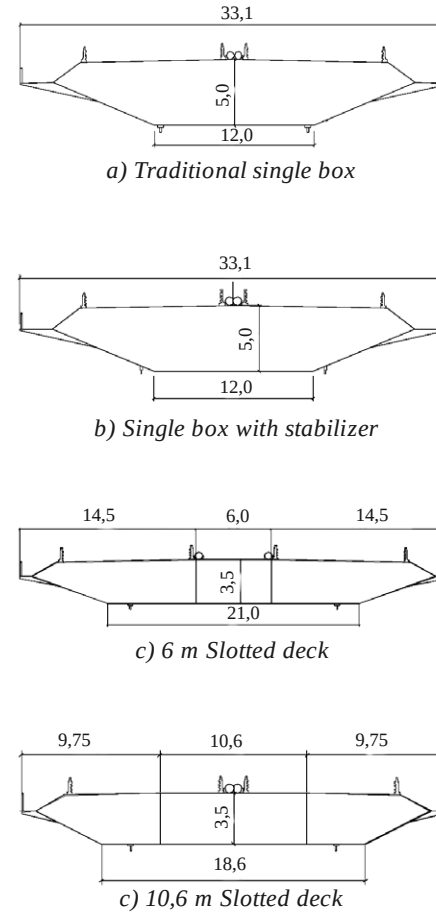
Fig. 8: Modified cross sections of East Sea Bridge

Flutter Stabilization in East Sea Bidge

With the total length of 32 km, East Sea (ES) Bridge in Shanghai will become the longest sea crossing project in the world. The main bridge over the navigation channel is a cable-stayed bridge with a composite box girder of five spans, 73 m + 132 m + 420 m + 132 m + 73 m. Though the main span of 420 m is not long enough, the monoplane cable system is an issue of concern aerodynamically due to the low frequency ratio (torsion to bending) of 1,65.

With the emphasis on aerodynamic stability, the wind tunnel experiment with the 1:100 aeroelastic full bridge model was carried out in the wind tunnel (*Fig. 7*). It was found in the first phase of the testing that the original structure cannot meet the requirement of flutter speed of 84,6 m/s. Some preventive means had to be considered to stabilize the original structure.

After having compared several means, two countermeasures were experimentally proven to be effective in raising the flutter speed slightly, but enough. The first means involved the adoption of a central stabilizer with the height of 0,8 m above the deck (*Fig. 8a*), and the other is to purposely set two gantries for maintenance at the lower corner of the inclined webs of the box (*Fig. 8b*). These two measures can increase the minimum flutter speed up to 85,8 m/s and 90,2 m/s [28]. It is interesting to learn from this project that flutter stabilization can be realized by various ways, even by a slight change of small parts of a cross section.



a) Traditional single box

b) Single box with stabilizer

c) 6 m Slotted deck

c) 10,6 m Slotted deck

Fig. 9: Possible cross sections of Xihoumen Bridge

Deck Selection in Xihoumen Bridge

The main section of Zhoushan Island-Mainland Connection Project in Zhejiang Province, Xihoumen Bridge is proposed as a two-span continuous suspension bridge with the span arrangement of 578 m + 1650 m + 485 m, the main span of which is going to create a new record in box girder suspension bridges. Based on the experience gained from the 1624 m Great Belt Bridge, Denmark, with the flutter critical speed of 62 m/s and the 1490 m long Runyang Bridge with the flutter speed of 63 m/s, the span length of 1,600 m seems to be the intrinsic limit in the aspect of aerodynamic stability for classical suspension bridges with streamlined box deck, even with the more strict stability requirement of 80 m/s for Xihoumen Bridge.

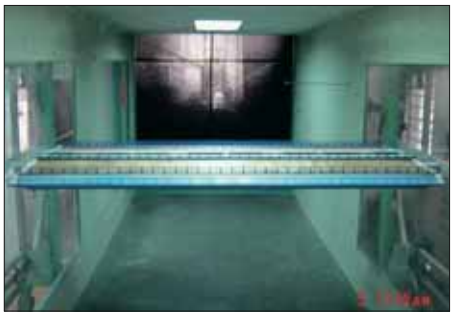


Fig. 10: Sectional model of Xihoumen Bridge

A traditional single box section and three other aerodynamically modified deck sections were selected for comparison. The wind tunnel testing of four sectional models in the scale of 1:80 as shown in *Fig. 9* were performed in the TJ-1 boundary layer wind tunnel with the working section of 1,8 m in width, 1,8 m in height and 15 m in length (*Fig. 10*).

Structure	$\alpha = -3°$	$\alpha = 0°$	$\alpha = +3°$
a) Box	68,2	45,8	47,5
b) Stabilizer	>90	>90	89,3
c) Slotted	88,4	>94	>94
d) Slotted	87,4	>94	>94

Table 7: Flutter speeds of Xihoumen Bridge (m/s) α is the angle of attack of the flow

The experimental results of flutter critical speeds are summarized in *Table 7*. Apart from the traditional single box section, the three remaining sections can meet the flutter stability requirement of 80 m/s, and the 6 m slotted deck configuration is finally selected as the scheme for further detailed design [29].

Conclusion

With the rapid progress of long span bridge construction in China, the state-of-the-art on long-span bridge aerodynamics has been briefly reviewed in respect of theoretical contributions, probability-based assessment and aerodynamic vibration control. The theoretical aspect mainly includes full-mode flutter analysis method, flutter mechanism and stabilization, as well as CFD techniques and application. The development and application of probability based assessment provides alternative approaches in undertaking probabilistic or reliability analysis in wind induced vibration including flutter instability, buffeting response and vortex-induced vibration. Finally, the recent engineering experience gained from four typical aerodynamic vibration control projects of long span bridges has been presented in this paper.

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