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Aerodynamic Behaviour of Very Long Cable-Stayed Bridges during Construction

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Abstract

Stonecutters and Sutong Bridge have pushed the world record for main span length of cable-stayed bridges to over 1000m. The design of these bridges, both located in typhoon prone regions, is strongly influenced by wind effects during their erection. Rigorous wind tunnel test programmes have been devised and executed to determine the aerodynamic behaviour of the structures in the most critical erection conditions. Testing was augmented by analytical and numerical analyses to verify the safety of the structures throughout construction and to ensure that no serviceability problems would affect the erection process. This paper outlines the wind properties assumed for the bridge sites, the experimental test programme with some of its results, the dynamic properties of the bridges during free cantilevering erection and the assessment of their aerodynamic performance. Along the way, it discusses the similarities and some revealing differences between the two bridges in terms of their dynamic response to wind action.

Keywords: Cable-stayed bridges; wind engineering; wind tunnel testing; construction.

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1. INTRODUCTION

Two new cable-stayed bridges in China have surpassed the current world record in main span length: Sutong Bridge with 1088m and Stonecutters Bridge with 1018m main span, cf. Fig. 1.

The bridges both feature steel superstructures in the main span, with Stonecutters Bridge adopting a novel twin-box section, cf. Fig. 2.

Stonecutters Bridge has short all-concrete backspans whereas Sutong Bridge exhibits a more conventional arrangement with steel superstructure. The tower configurations are shown in Fig. 3.

Sutong Bridge is the key element of a large river crossing, whereas Stonecutters Bridge is located in the urban area of Hong Kong.

This paper reports on the investigations undertaken to study the aerodynamic behaviour of these bridges in the erection condition. These studies included extensive wind tunnel testing as well as analyses of the aeroelastic excitation phenomena in different stages of construction.

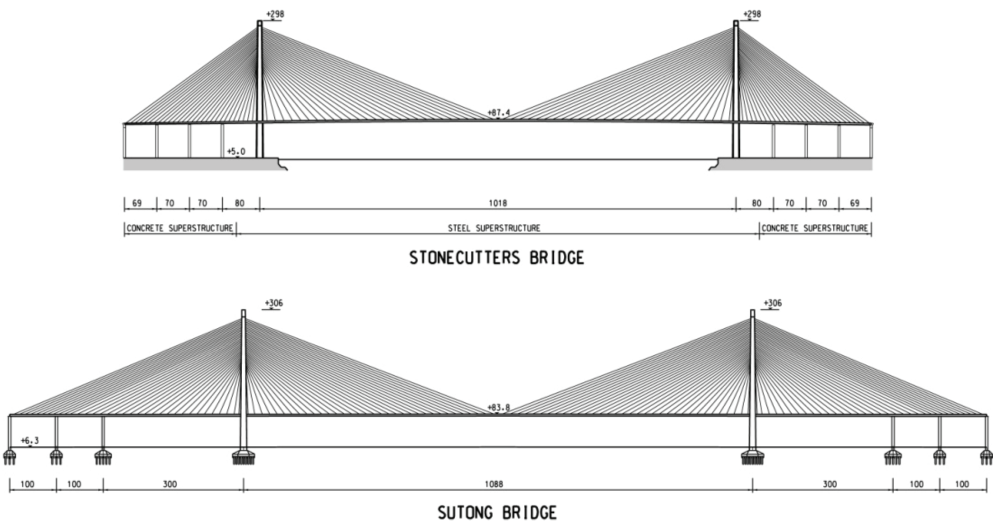

Fig. 1: General arrangement of bridges

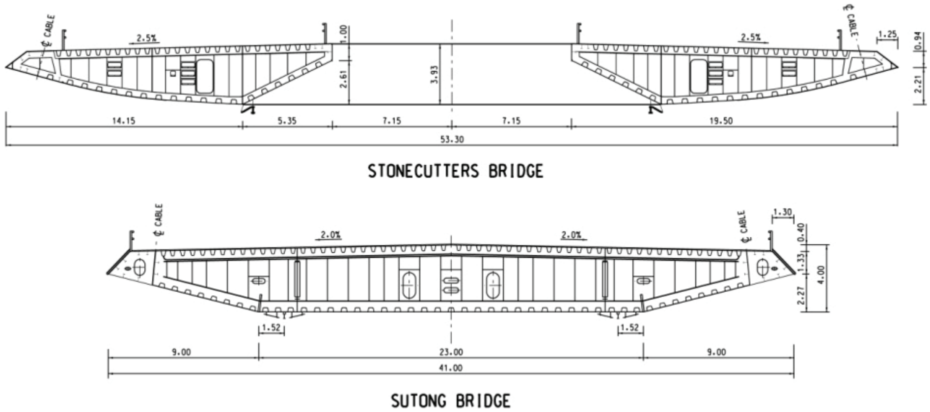

Fig. 2: Main span cross sections

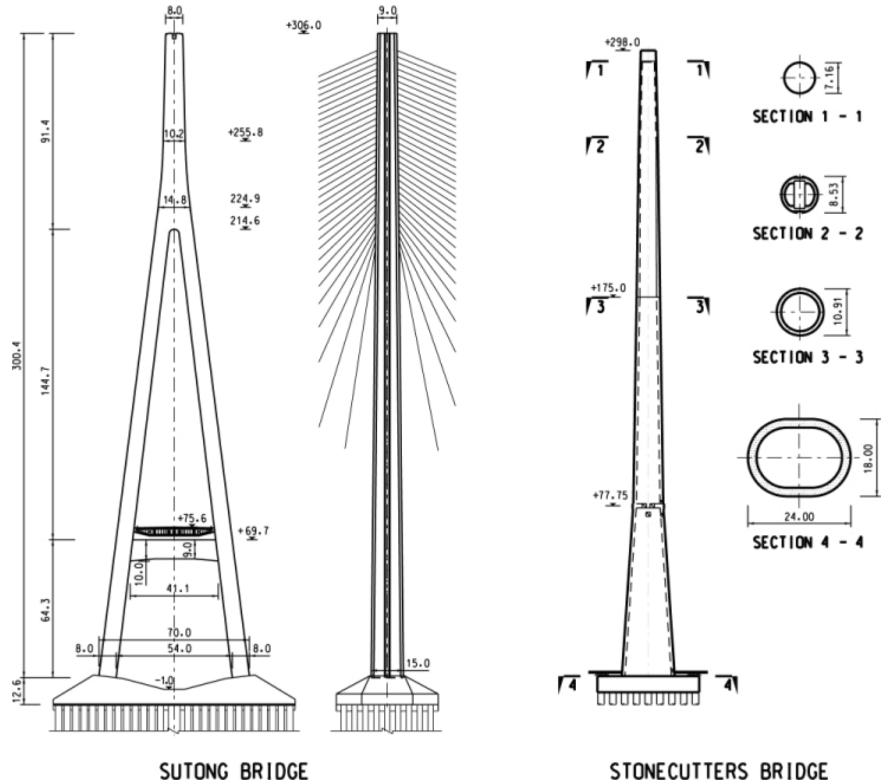

Fig. 3: Configuration of bridge towers

2. DESIGN SPECIFICATIONS ON WIND CHARACTERISTICS

Suitable design criteria need to be established as the basis for an analysis of the adequacy of the partially erected bridge under wind loads. Design wind specifications are to reflect the wind conditions on site in a probabilistic sense. Due to the shorter exposure period the erection condition is generally designed for smaller wind loads than the in-service condition.

The wind characteristics adopted for Sutong Bridge were taken from the Chinese Code [1] and adjusted according to results of detailed analyses of the local wind climate. For Stonecutters Bridge a 50m high mast was set up at the bridge location and continuous wind readings were taken over a period of over one year to understand the site-specific wind characteristics.

The Sutong Bridge project, whose design is based on allowable stresses, specified a 100 year return period wind for the in-service condition and a 30 year return period wind for the construction stage. The design specifications for Stonecutters Bridge however only specify one reference wind with a 120 return period, following the methodology of British Standard BS 5400 for bridge design. In this Limit State Design approach the difference between in-service and erection condition is accounted for by different load safety factors. Whilst BS 5400 uses safety factors of 1.4 and 1.1 respectively, the design basis of Stonecutters Bridge specifies safety factors of 1.9 and 1.2 in recognition of the different extreme value distribution of wind events in typhoon regions such as Hong Kong. These safety factors are adopted from the Structures Design Manual of the Highways Department of Hong Kong.

Figure 4 compares the adopted profiles of wind speed and turbulence intensity. For Stonecutters Bridge (SCB), two different sets wind properties were devised, namely a land and an ocean fetch scenario corresponding to winds approaching over the mountains and from the sea, respectively. The properties for Sutong Bridge (STB) are for open sea conditions.

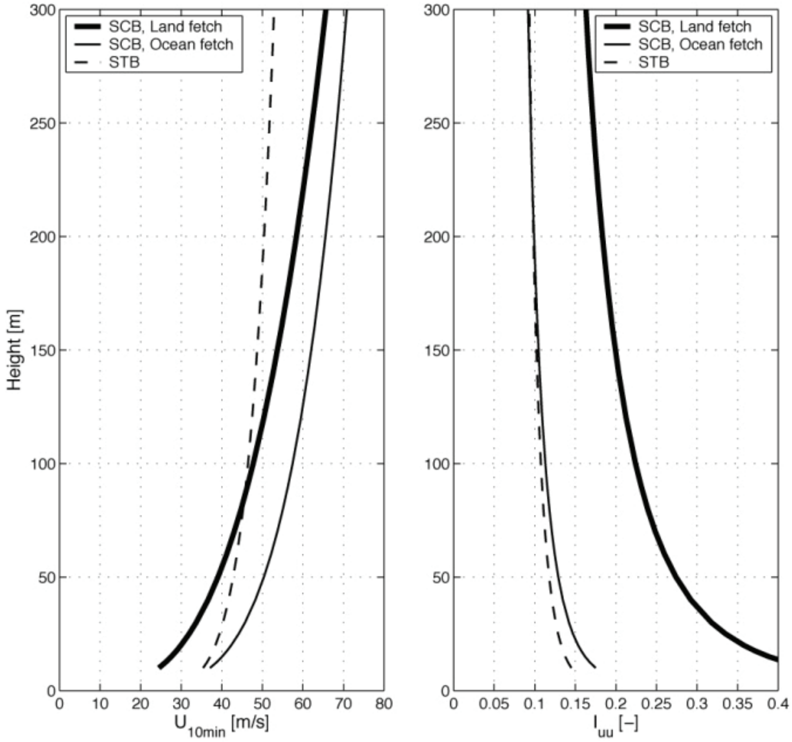

Fig. 4: Design wind characteristics: mean wind speed and longitudinal turbulence profiles

3. AERODYNAMIC SECTION PROPERTIES

Extensive section model tests were performed for both projects. Firstly the aerodynamic forces on the static cross section were determined.

The section model tests for Stonecutters Bridge included extensive testing of a number of different configurations. It was realised that the drag of the girder was very sensitive to the temporary hand rail articulation, specifically that of the toe board. A solution was sought where drag loads were acceptable whilst maintaining the required safety of workers. A mesh-type toe board was developed which reduces obstruction of the airflow but still provides a barrier at foot height. The overall drag of the deck with 200mm high mesh toe board was 13% lower than that with a solid toe board of the same height. The wire mesh toe board has been adopted and is used herein.

The static aerodynamic force coefficients of the basic erection cross sections are shown in Fig. 5. Herein, a positive angle of incidence corresponds to a deck section which is rotated clockwise in a horizontally approaching wind. The results are for Reynolds numbers of approximately $5x10^5$ and smooth flow for both cases. The tests were repeated with incoming turbulent flow. The results consistently showed an increase in drag due to the turbulence.

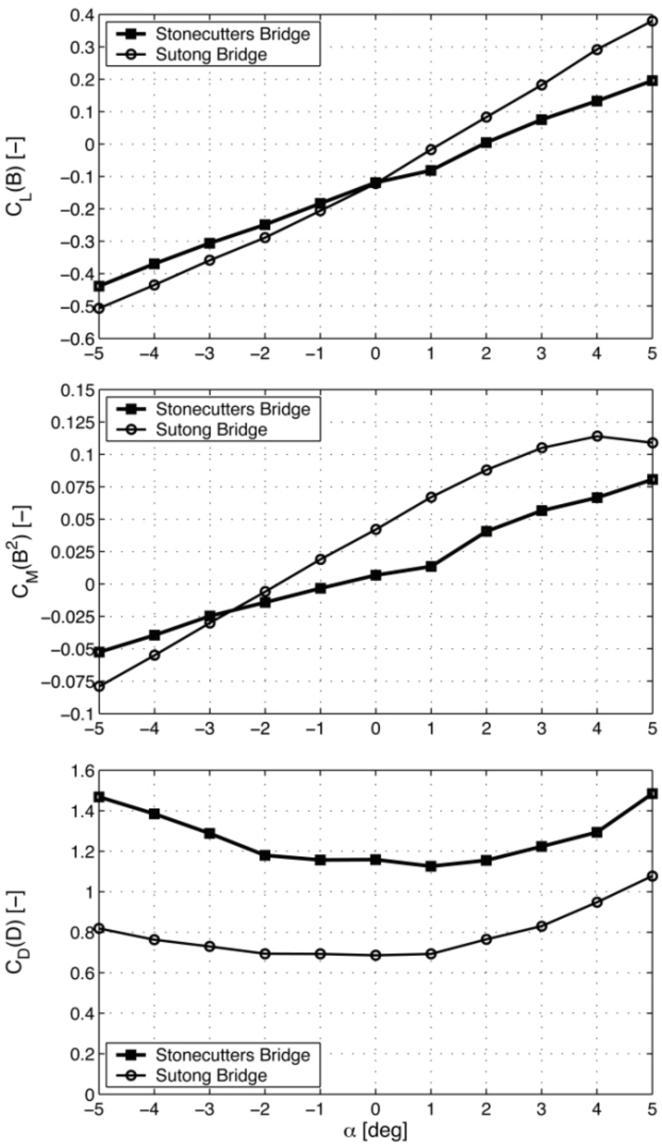

Fig. 5: Section model test results (smooth flow): static aerodynamic force coefficients of deck girder in construction condition

4. DYNAMIC PROPERTIES OF THE STRUCTURES

The dynamic characteristics of a structure, important for the response to wind input, are best expressed by its modal properties. These change continuously as the bridge erection in free cantilevering proceeds. Fundamentally the two bridges exhibit similar behaviour. In fact, the first vertical and lateral frequencies are close throughout and almost identical at the maximum length of the Stonecutters cantilever. However, the second vertical mode is considerably higher for the Sutong deck, which is due to an approximately 50% higher bending stiffness of the box girder. For the second bending mode the flexure of the girder plays a dominant role over the stretching of the cable stay system. The difference between the two structures is even higher in the fundamental torsional frequency with the Sutong deck twisting at about double the Stonecutters torsional frequency. This is caused on one hand by a somewhat higher torsional stiffness of the Sutong girder over that of the Stonecutters grillage deck whilst at the same time the torsional inertia is much higher for the Stonecutters deck due to the concentration of the self-weight away from the bridge centreline. On the other hand, the Sutong A-shaped pylons have a 70% higher torsional stiffness than the single shaft poles of Stonecutters Bridge.

5. AERODYNAMIC DECK STABILITY

Aerodynamic instabilities arise from a complex dynamic interaction between the changes in aerodynamic forces and the dynamic response of the structure. Aerodynamic instability of the structure can be thought of as negative damping, where the energy input per cycle of oscillation is larger than that absorbed by structural damping, thus leading to a divergent amplitude response. Instabilities are characterised by a critical wind speed, above which the structure is unstable.

The instability boundary of the Stonecutters Bridge deck was established by performing aeroelastic testing in the wind tunnel. The section model was spring-mounted on a rig, stiffnesses and masses adjusted to adhere to the scaling laws. For classical flutter, a coupled response of vertical and torsional modes, the critical wind speed reduces the closer the two frequencies f_T (torsion) and f_V (vertical bending) are. A range of frequency ratios f_T/f_V was therefore tested.

For Sutong Bridge no aeroelastic section model test was conducted. The instability boundary was determined analytically from the results of forced-oscillation tests on the section model. The section was set into sinusoidal motion in vertical (heave) and rotational (pitch) motions and the aerodynamic forces measured. The motion-induced forces were then expressed in terms of aerodynamic derivatives, which allow the computation of the instability wind speed by numerical means. Derivatives were determined for three angles of incidence: -3°, 0 and 3°.

All tests were done in smooth flow. This is the most critical condition as incoming turbulence is commonly understood to inhibit the excitation. The required flutter boundaries for the construction stage are between 50m/s (at 5° incidence of the mean flow) and 95m/s (at horizontal angle) for Stonecutters Bridge and a constant 64m/s for Sutong Bridge. These required flutter wind speeds were achieved throughout. Testing was only undertaken up to the equivalent full-scale wind speed required under the relevant design specifications, i.e. the actual flutter wind speed was not established.

For Stonecutters Bridge the twin-box (vented) arrangement of the deck is a major contributor to keeping the deck stable even at small frequency separations whereas for Sutong stability is mostly achieved by sufficiently high torsional frequencies. Single-degree of freedom instabilities were also not observed within the critical range of wind speeds.

6. BUFFETING RESPONSE

Buffeting is a pseudo-random dynamic response of the structure caused by fluctuating aerodynamic forces which arise from the velocity changes of the gusty wind. These gusts are due to the turbulent nature of the atmospheric boundary layer. Buffeting response of the towers was not found to be a problem. Amplitudes at low (operational) wind speeds are low and at high wind speeds they still do not pose a threat to the structural integrity. For deck and cables, however, a careful analysis of the magnitude of buffeting response is critical.

The current study relied on a combined approach of wind tunnel testing and numerical buffeting simulations. The aeroelastic tests of tower and deck erection stages were conducted in a boundary layer wind tunnel to reproduce the desired turbulent wind field. Erection equipment was modelled to accurately represent the situation during erection. Particularly the lifting gantry located at the tip of the cantilever adds substantial drag and hence increases the lateral response.

The mean and peak dynamic buffeting responses were measured and used to validate a numerical buffeting simulation model. The numerical model is based on a three-dimensional Finite Element model of the bridge. This approach [2] allows for a detailed modelling of all structural features, including the aerodynamic properties of the different members. The model is effectively exposed to a theoretically derived wind event, which satisfies the stochastics of the prescribed turbulent wind field data. The transient response of the structure is monitored and analysed to determine design forces and displacements. Herein the results of these numerical analyses are reported.

For Sutong Bridge the maximum balanced cantilever condition was critical to some items, namely the rotational restraint provided for the deck at the tower. The bridge is built in balanced cantilevering until reaching closure to the backspan deck, which occurred at 157m long cantilevers. This situation, however, is similar to more standard medium span bridges and therefore not discussed further herein.

The lengths of the bridge cantilevers before installation of the closing segments were 506m and 541m for Stonecutters and Sutong Bridge, respectively.

6.1. Vertical response

The critical condition for vertical buffeting response during cantilevering erection is the stage where the new segment has been installed but the corresponding set of mainspan stay cables has not yet been stressed. In this situation the dead load of the cantilevering new segment induces large hogging in the bridge girder. Further, the front set of stay cables to the previously installed segment is heavily stressed. When the vertical buffeting effects act in the downward direction, these forces add to the dead load forces and lead to a critical situation in girder and stay cables.

For the analyses a structural damping of 0.5% of critical was used. Figure 6 shows the distribution of peak bending moments in the girder and the respective cable forces. Clearly, the critical condition for Stonecutters Bridge is the high-turbulence land fetch wind. Sutong Bridge, exposed to sea fetch type wind of low turbulence, shows a higher demand than the corresponding (sea fetch) Stonecutters wind. This is due to the higher susceptibility of the cross section to vertical buffeting, as manifested in the lift and moment coefficient slope, see Section 3. Note that the deck of Sutong Bridge is vertically supported by the tower whereas the Stonecutters deck is vertically only supported by the stay cables, which explains the higher cable forces near the tower.

The peak cable forces of Stonecutters Bridge between ocean and land fetch scenarios are relatively close, which is due to the static wind component contributing more to the cable forces than to the bending moments in the girder. The mean wind speed is higher for the ocean fetch, see Section 2.

For Stonecutters Bridge, the peak hogging moments with full buffeting lead to an overstress of the girder. It was decided to place temporary ballast at a position roughly 100m behind the construction front to counteract this effect and to ensure the safety of the structure in the critical erection conditions, Fig. 6.

6.2. Lateral response

Whilst the basic characteristics of the lateral response are relatively similar between the two bridges, the way the forces are carried to the foundations is very different. Firstly, the twin-deck arrangement of Stonecutters Bridge leads to a Vierendeel-type transverse bending behaviour where the lateral bending moments are carried both by bending of the longitudinal and cross girders as well as by axial force difference between the longitudinal girders. The deck is also stiffer laterally than that of Sutong Bridge. Further, the tower only provides a restraint for lateral forces by a bearing. The Sutong Bridge girder remains almost axial force free due to lateral buffeting but is restrained by the tower not only laterally but also in its rotation about the vertical axis. This restraint is realised through a series of temporary fixing cables between deck and tower cross beam. The remaining lateral bending moment in the girder not taken by these restraints is carried to the backspans and resisted by transverse forces on the backspan piers. This effect is important to the lateral buffeting behaviour of Sutong Bridge particularly during cantilevering as the buffeting wind effects on the sidespan tend to counteract that of the mainspan through the modal coupling and hence reduce the dynamic buffeting response. This reducing effect is essentially absent in Stonecutters Bridge which therefore experiences a greater dynamic amplification as manifested in the ratio between peak and mean lateral deflections, see Table 1.

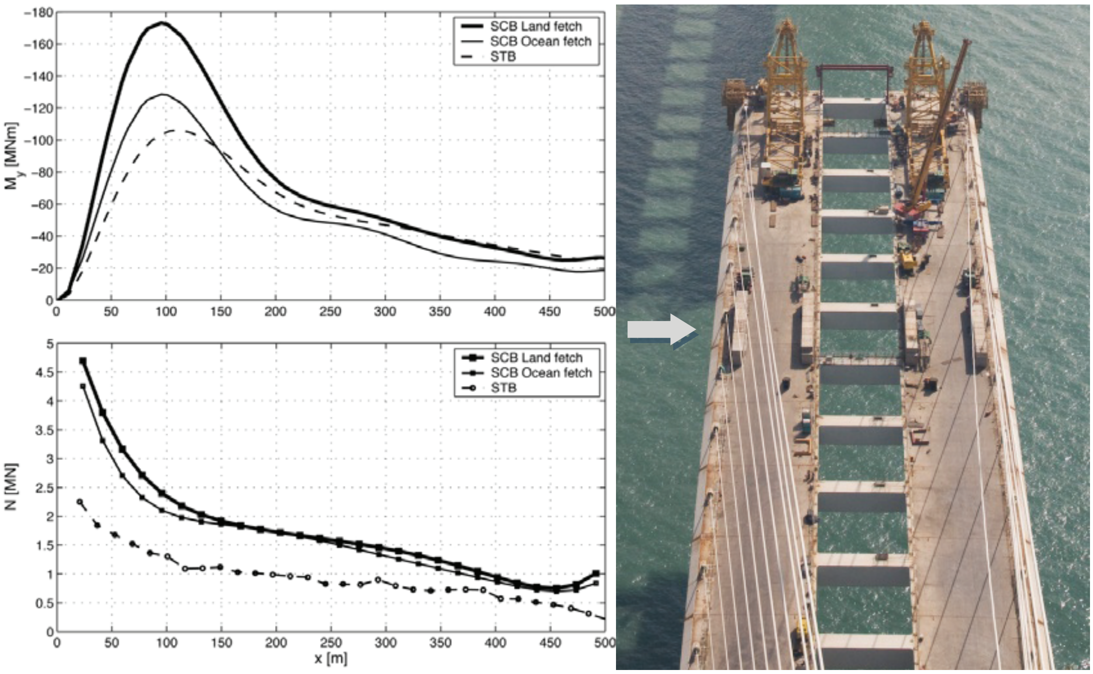

Fig. 6: Stonecutters Bridge. Left: Vertical buffeting response of critical erection condition: vertical bending moment in deck girder (top), cable forces (bottom); x is the distance from the tip of the cantilever. Right: Ballast placed on the deck.

7. CONCLUSIONS

A comparison of aerodynamic properties and aeroelastic behaviour of the two longest span cable-stayed bridges during erection has been presented. Although being similar in span length and construction methodology, the study highlights significant differences in many respects. These are mostly related to different wind characteristics on site, a strongly different deck cross section and a different configuration of the overall structure.

References

[1] Wind-resistant Design Specification for Highway Bridges, JTG/T D60-01-2004

[2] Morgenthal, G., Kovacs, I., Saul, R., 2005, "Analysis of Aeroelastic Bridge Deck Response to Natural Wind", Structural Engineering International, 2005, pp. 232-235

Table 1: Buffeting response: displacements of cantilever tip at maximum cantilever condition for design wind conditions. Values determined from buffeting analyses.

	Vertical Displacement [mm]		**Lateral Displacement [mm]**	
	Mean	**Peak**	**Mean**	**Peak**
Stonecutters Bridge, Land fetch	230	3210	970	2980
Stonecutters Bridge, Ocean fetch	350	2550	1370	3220
Sutong Bridge	240	1520	2440	4930