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Recent topics in the wind engineering

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Abstract

Wind resistant design is of great importance for large scale projects of bridges as well as buildings. However comparing the seismic effect, it is apt not to understand it well, because the contribution in the structural design increase rapidly as its scale and most of the wind-action include various vibration such as the self-excited vibration and the random vibration. In this paper importance of the wind resistant design is demonstrated referring history of the modern long span bridges and method of the wind resistant design is also illustrated.

Keywords: wind resistant design, long span bridges

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1. Introduction

The wind action on structures is very usual phenomena in daily life, which everyone can experience. However it is also true that it is not only one of severe natural disasters but also it is hard to understand how large its effect can be. It is well known that there is a long list of structural damages due to the wind action and history of development of the suspension bridges coincides with history of the wind accident. According to these lessons, main key issues in the structural responses against the wind actions are extracted and illustrated in Figure 1. Shortly those actions can be explained as a chain ling of structural dynamics, aerodynamics and atmospheric exposures. It means that the wind action on structures and its wind resistant design should be discussed from those combined viewpoints. In this paper, examples of the wind induced vibrations of bridges is reviewed and some new trends in the wind resistant design are introduced.

2. Significance of the wind action

A part of bridge damages, which are publicly reported, are listed in Table 1. Referring the fact the modern suspension bridge was proposed around 1800 after the industrial revolution, reports of structural damage of bridges increased simultaneously after 19th

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century. Besides it can be found that main counter majors were to give additional stiffness to the structure by adding some members, such as stay-cables and truss reinforcement. From a viewpoint of modern wind resistant design, stiffening the whole structure is one of ways to suppress the vibration but it is neither easy nor efficient in comparison with aerodynamic improvement, because it must dissipate the kinetic energy of the self-induced vibration.

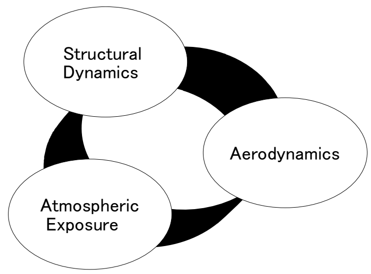


Fig. 1. Link of wind action on structures

Table 1. List of monumental failure of long-span bridges

Due to Wind force

1879 Tay Bridge / short wind force and derailment

Due to Aero elastic vibration

1823	Brighton Chain Pier Bridge(UK) / torsional vibration
1826	Menai Strait Bridge(UK) / reparation of deck system
1830	Nassau Bridge(D) / broken chain
1850	Niagara-Lewiston Bridge(US) / collapsed
1932	George Washington Bridge(US) / bending vibration, truss reinforcement
1937	Golden Gate Bridge(US) / reinforcement of the trussed deck, change from open truss to closed truss
1938	Thousand Island Bridge(US) / bending vibration, installation of stay-cables
1939	Deer Isle Bridge(US) / bending vibration, installation of stay-cables
1939	Bronx Whitestone Bridge(US) / truss reinforcement
1940	Tacoma Narrows Bridge(US) / collapsed due to torsional vibration

2.1 Tay Bridge, UK

The Tay Bridge, a 85 span trussed high girder bridge of 3,160m, is well known as its collapse on December 28, 1879, as in Figures 2 and 3. Reason of this collapse has been widely understood as lack of the lateral stiffness of the truss girder due to too small wind load specification. It can be proved by the fact that the steam locomotive was found in the truss frame in the water and salvaged. However according to many reports on it, tight schedules of big bridge projects, change of the structure redesign due to lack of foundation support capacity and maintenance process seem to have been importance keys in this collapse. In the wind load its designer Thomas Bouch decided 12lb/ft^2 (~5MPa) for the sever storm as the wind load referring an advice of the Astronomer Royal. After this accident, the wind load specification was changed to 56 lb/ft^2 (~20MPa) for the Force Railway Bridge. On the other hand Japanese Road design code specifies 30MPa for the deck girder.

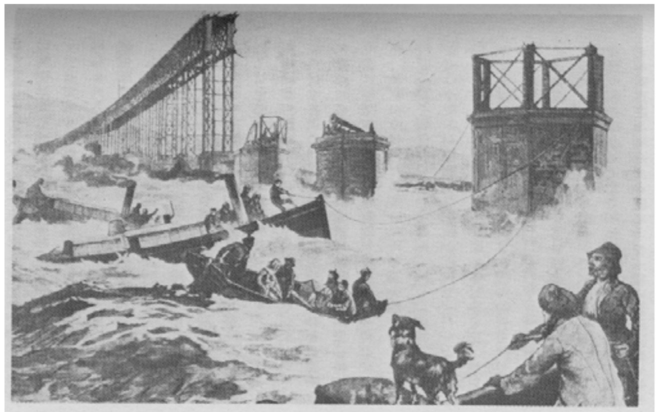


Fig. 2. Artist drawing of the collapse

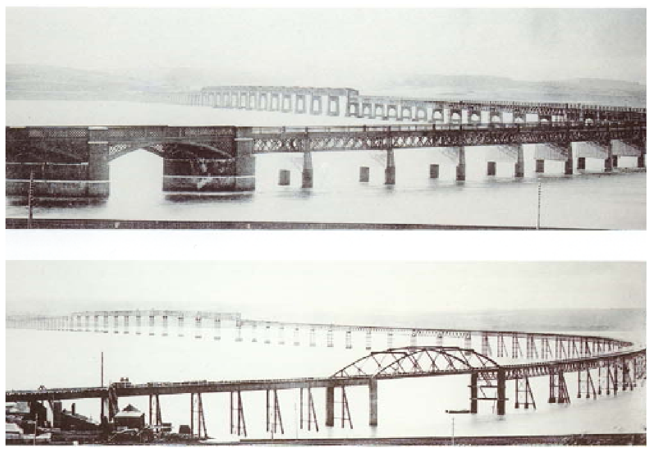


Fig. 3. Old and new Tay bridge

2.2 Tacoma Narrows Bridges, USA

The Old Tacoma Narrows bridge was one of technically advanced bridges at the completion, which is designed basing on the flexure theory. It is also very famous that its collapse occurred in only few months after its opening ceremony, as in Figure 4. It is understood its reason was lack of knowledge in bridge aerodynamics and poor aerodynamic performance of the stiffening deck. According to the flexure theory, the main cable system can support all of loads and the deck stiffness can be decreased to the minimum. The extreme shape of the deck under this idea is the plate-girder as illustration in Figure 5. Only demerit of this type was aerodynamics.

After the collapse, many investigations were made to clarify the reason. Among them a wind tunnel for a full bridge model was constructed and full bridge wind model testings were conducted at University of Washington, USA by Prof. Farquharson[1]. In Figure 6, one of their results is illustrated and the observation is also plotted. Important findings of this testings were as follows

1. Responses occur as order of the natural modes from the lowest.
2. Possibility of destructive vibration in the torsional mode was found.

Fig. 4. Vibration of the Tacoma Narrows bridge

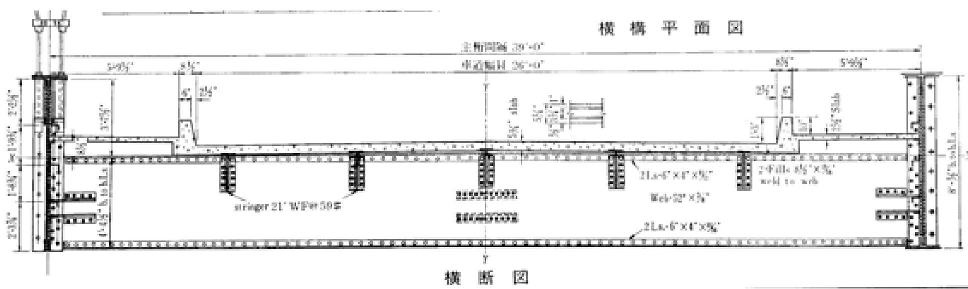


Fig. 5 Cross section of the Old Tacoma Narrows bridge

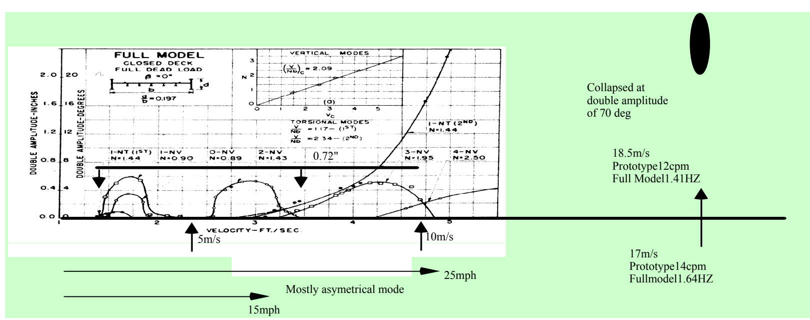


Fig. 6 Full bridge model test and observation

Comparing the observation (horizontal solid line and black eclipse in the figure), the response at low wind speed range shows reasonable agreement but in the flutter occurrence only qualitative coincide was found More than 50 years have passed, but this is very rare example to make a comparison between wind tunnel testings and the field observation even from the modern viewpoint.

2.3 Kessock bridge, UK

Kessock bridge with 240m in main span and 1056m in total length, completed in 1982, is a cable stayed bridge near Inverness in Scotland UK (in Figure 8). This bridge is one of bridges with TMDs (in Figure 7) to suppress the vortex-induced vibration. Vibration of 90-200mm in the amplitude due to vortex shedding was observed in wind only from Moray Firth[2]. To suppress this vibration of the vertical fundamental mode eight TMDs were installed (as in Figure 7),

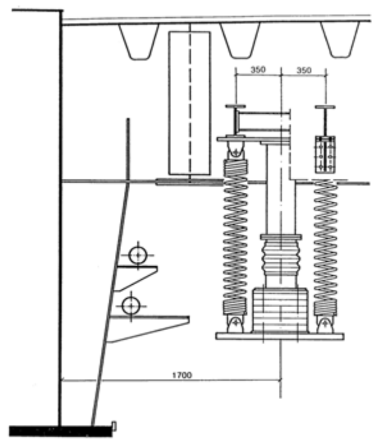


Fig. 7. TMD of Kessock bridge

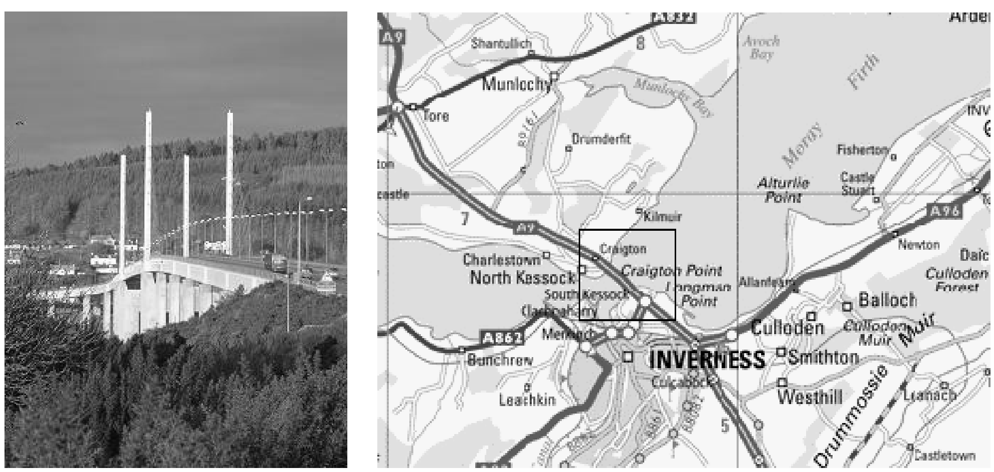


Fig. 8. The Kessock bridge

2.4 Trans-Tokyo Bay highway bridge, Japan [3]

Trans-Tokyo Bay Highway in Figure 9 is a 10-span continuous steel box girder bridge, completed in 1995 ,that is 1,630 m in total length including 240 m spans in maximum length. In the wind tunnel examination, it was reported that two or more vertical bending vibration modes would occur due to vortex-induced oscillation Before the opening, vibration with the amplitude of±50 cm was observed at a wind of 15 to 16 m/s. It is also reported that the logarithmic decrement of 0.028 - 0.044 were observed by the field dynamic testing. Comparison between the wind tunnel testings and observation is illustrated in Figure 10.

It can be found that the response of this bridge is almost same with the wind tunnel test. Looking at this fact some TMDs were installed to suppress vortex-induced vibration for some vibration modes.

Fig. 9. Trans-Tokyo Bay bridge

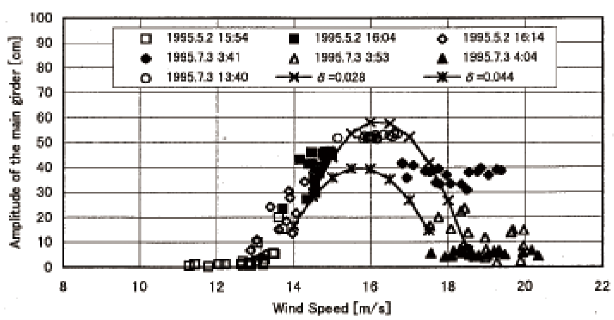


Fig. 10. Observation vs. response in wind tunnel test[3]

3. Aerodynamics and structural dynamics

3.1 Wind tunnel testing

For long time the wind tunnel test has been only tool to investigate aerodynamic characteristics of structures. Although CFD is improved quickly, the wind tunnel testing under well adjusted similitude in the modeling is believed as the most reliable approach for the verification of stability of structures against the wind action.

Roughly wind tunnel test can be classified into the full bridge model test and the section model test. Literally in the full bridge model test, the model should be scaled down at specified scale in detail. From a viewpoint of similarity law, the full bridge model is optimum and it will be easy to understand the result of the testing. However when the scaling ratio is not small enough to ensure the detail modeling, size of the model must be huge. In case of the Akashi Kaikyo bridge, length of its 1/100 scale full bridge model is 40m. It required the world largest wind tunnel, which accommodate the whole model (in Figure 11). The section model has scaled section but it does not have the mode shape(in Figure 12).

Fig. 11. Full bridge model for the Akashi Kaikyo Bridge and wind tunnel for it

Fig. 12. A section model and test section of a wind tunnel

Strictly speaking, this is not a model, which satisfies the similarity law, and it cannot show behavior of the prototype directly. To understand output of the testing, interpretation must be required basing on some assumptions. Although there is this limitation, this model is very convenient to know aerodynamic characteristics of the section of the deck, such as aerodynamic derivatives, aerodynamic forces, pressure distributions and so on. Besides large wind tunnel facility will not be required, because the model is just a section. This is a reason why this testing method is widely used for slender structure with similar section shape from one end to another, such as suspension bridges and airfoils.

3.2 One example of similarity for elastic/locking partial model

One example which is out of usual similitude is safety verification of a tower of suspension bridge in completion. It is usually required to investigate wind induced vibration of a tower of a suspension bridge not only during construction but also after completion (possible vibration mode shapes in Figure 13). In case of a free standing tower it will be very easy to model it as elastic mode. However the vibration after completion is a part of a global vibration of the whole system. When only the tower model is used because of limitation of the size of a wind tunnel, the whole structure and the modeled tower must be equivalent. This discussion is similar to equivalent modeling of the section model.   In the section model following formulations is applied.

Looking at one natural mode and assuming only velocity component of the wind force the equation of motion can be as follows;

$$m\ddot{u} + c\dot{u} + ku = \frac{1}{2}\rho U^2 C^* A\left(\frac{\dot{u}}{\omega L}\right) \tag{1}$$

When longitudinally uniform deck under wind action can be assumed and one dominant vibration mode is looked at, equation (1) can be rewritten as equation (2).

$$\int_s m\phi^2 dx\ddot{q} + \int_s c\phi^2 dx\dot{q} + \int_s k\phi^2 dxq = \frac{1}{2}\rho U^2 C^* A\left(\frac{\dot{q}}{\omega L}\right)\int_{wind}\phi^2 dx$$

$$\frac{\int_s m\phi^2 dx}{\int_{wind}\phi^2 dx}\ddot{q} + \frac{\int_s c\phi^2 dx}{\int_{wind}\phi^2 dx}\dot{q} + \frac{\int_s k\phi^2 dx}{\int_{wind}\phi^2 dx}q = \frac{1}{2}\rho U^2 C^* A\left(\frac{\dot{q}}{\omega L}\right) \tag{2}$$

where suffixes of integration, “a” and “wind”, mean integral areas of the whole structure and the exposed part of the structure to the wind respectively.

Mass parameter $\frac{\int_s m\phi^2 dx}{\int_{wind}\phi^2 dx}$ in equation (2) is called as the “equivalent mass”, and similarity of this equivalent mass can make two different system equivalent in the equation of motion with the wind induced force.   This is an essential idea to realize the section model testing.

This equivalent mass can be a solution for the similarity of the tower model as a part of the global structural system. When spans of a cable stayed bridge are not symmetrical (one example in Figure 14), similar discussion can be applied to design the wind tunnel model of its tower.

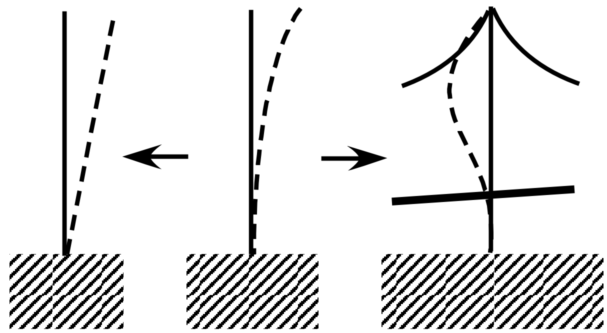


Fig. 13 Images of vibrations of towers

Fig. 14 Katsushika Harp Bridge, Japan

3.3 Aerodynamic countermeasures

To suppress the wind induced vibrations, aerodynamic devices to control the air flow are sometimes applied. Double flaps are illustrated in Figure 15 as one example of aerodynamic countermeasures. On the other hand to control stay-cable's vibration installation of dampers or/and helical strakes are very common, like in Figure 16.

Fig. 15. Tozaki Viaduct of the Honshu-shikoku bridges,Japan

Fig. 16. Cable dampers and helical strakes of Brotonne Bridge, France

4. Aero-elastic investigation

4.1 Natural frequency analysis

Structural dynamic analysis of a carefully prepared structural model is very essential as the first step in the wind resistant design. Especially it is always required to evaluate the natural frequencies, the natural mode shapes and the equivalent masses, which are calculated using detailed structure models (in Figure 17). For examples, sometimes mass contribution of structurally coupled vibration becomes great. This happens in coupled vibration of a deck in lateral and torsional directions, due to discrepancy in location of the gravity center and the stiffness center (in Figure 18).

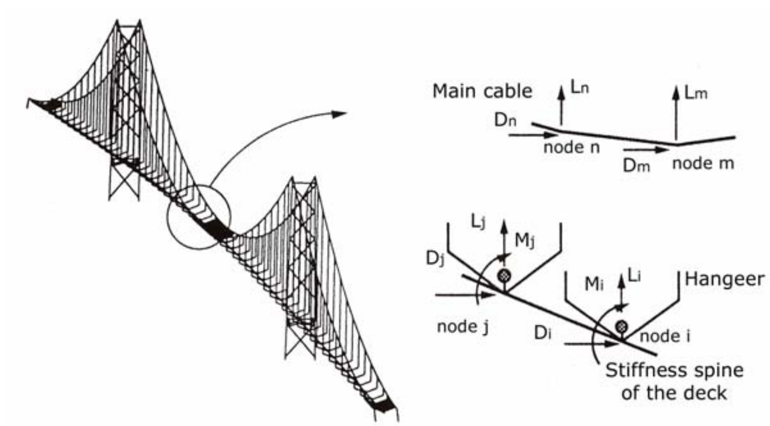


Fig. 17 Structural frame model for the Akashi Kaikyo bridge

4.2 Numerical response analysis

When unsteady aerodynamic forces are extracted in the wind tunnel experiments and applied to FEM structural model as in Figure 17, solutions of those equations of motion as Equation (3) give aero-elastic behavior of the structure due to the wind action.

$$[M]\ddot{u}+[C]\dot{u}+[K]u=[F_A]\ddot{u}+[F_V]\dot{u}+[F_D]u \tag{3}$$

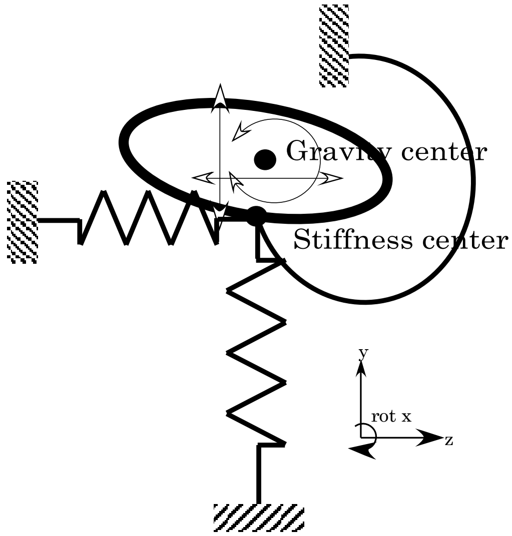


Fig. 18 Coupling mechanism of a deck

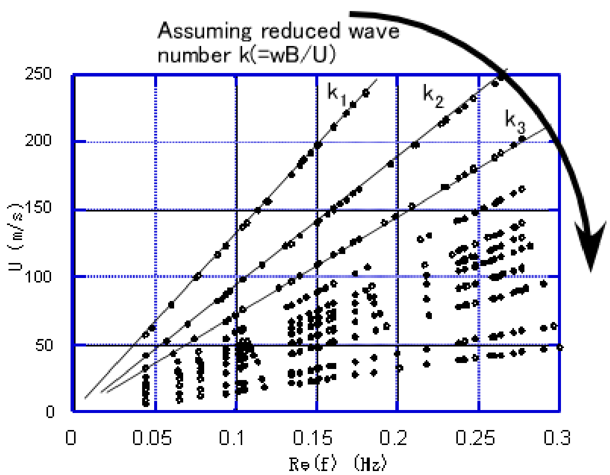


Fig. 19. Tracking aero-elastic roots

The unsteady aerodynamic forces at every nodes are formulated as in Figures (4,5) [4,5]

$$\begin{Bmatrix} L \\ M \\ D \end{Bmatrix} = \frac{1}{2}\rho U^2 \begin{bmatrix} B & 0 & 0 \\ 0 & B^2 & 0 \\ 0 & 0 & B \end{bmatrix} \left\{ \begin{bmatrix} kP_1^* & kP_2^* & kP_5^* \\ kH_1^* & kH_2^* & kH_5^* \\ kA_1^* & kA_2^* & kA_5^* \end{bmatrix} \begin{Bmatrix} \dot{y}/U \\ B\dot{\theta}/U \\ \dot{z}/U \end{Bmatrix} + \begin{bmatrix} k^2P_4^* & k^2P_3^* & k^2P_6^* \\ k^2H_4^* & k^2H_3^* & k^2H_6^* \\ k^2A_4^* & k^2A_3^* & k^2A_6^* \end{bmatrix} \begin{Bmatrix} y/B \\ \theta \\ z/B \end{Bmatrix} \right\} \tag{4}$$

$$\begin{Bmatrix} L \\ M \\ D \end{Bmatrix} = \pi\rho\omega^2 \begin{bmatrix} B^3 & & 0 \\ & B^4 & \\ 0 & & B^3 \end{bmatrix} \left\{ \begin{bmatrix} L_{yI} & L_{\theta I} & L_{zI} \\ M_{yI} & M_{\theta I} & M_{zI} \\ D_{yI} & D_{\theta I} & D_{zI} \end{bmatrix} \begin{Bmatrix} \dot{y}/\omega B \\ \dot{\theta}/\omega \\ \dot{z}/\omega B \end{Bmatrix} + \begin{bmatrix} L_{yR} & L_{\theta R} & L_{zR} \\ M_{yR} & M_{\theta R} & M_{zR} \\ D_{yR} & D_{\theta R} & D_{zR} \end{bmatrix} \begin{Bmatrix} y/B \\ \theta \\ z/B \end{Bmatrix} \right\} \tag{5}$$

where U, B, ω, ρ, L, M, D, y, θ, z, k are the wind speed, width of the model, circular frequency, density of the air, the lift force, the aerodynamic moment, the drag force, the vertical displacement , the twisting displacement the horizontal displacement respectively and the reduced wave number(= ωB/U). One of ways to get solution is to track eigenvalues as reduced wave number-damping plain and reduced wave number – frequency plain [5] as illustrated in Figure 19. Refereeing comparison among experiment and analysis in Figure 20, analytical results can explain the experiment but there still remains room for improvement.

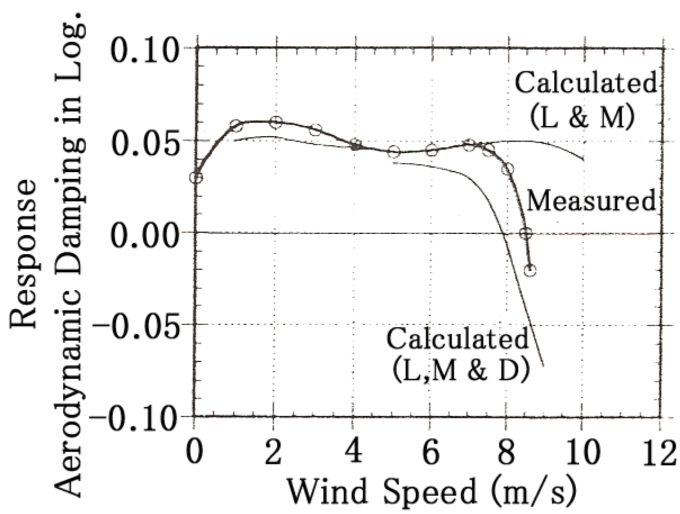


Fig. 20. Behavior of the Akashi Kaikyo bridge

5. Atmospheric exposure

5.1 Design wind speed

Atmospheric exposure will be classified into 3 categories for the structure-wind engineering as meso-scale exposure for motion of mostly low/high pressure system, micro-scale exposure for development atmospheric turbulence and local exposure for local topography's contribution. Although various contributions exist, primary interest for the wind resistant design is the design wind speed, which specifies the upper boundary of the wind action. In many examples of wind resistant design codes this design wind speed is given as a map after correction of the fundamental wind speed due to the topographical effect at the site and various safety factors, which is specifies on a map(one example in figure 21) as assumed common topography. The vertical wind speed profile is also a key for the wind speed correction, which is decided on appropriate ground roughness (in Figure 22). In the aeronautical engineering and the bridge wind engineering, the smooth flow has been used to the reference wind of the wind tunnel testing. Main reason of use of this smooth flow is that it can give the safe side evaluation and it will be possible to make wind tunnel experiments more equal condition than in some turbulent flow at various wind tunnel facilities. However it is clear that effect of the boundary layer turbulence play an essential role to interpret measurements in laboratories and observations of real bridges. Introduction of this effect in evaluation of the wind –induced response is understood to increase its importance.

Topographies of the roughness I,II,III,IV are on water, open terrain, suburb and down town/mountainous area respectively.

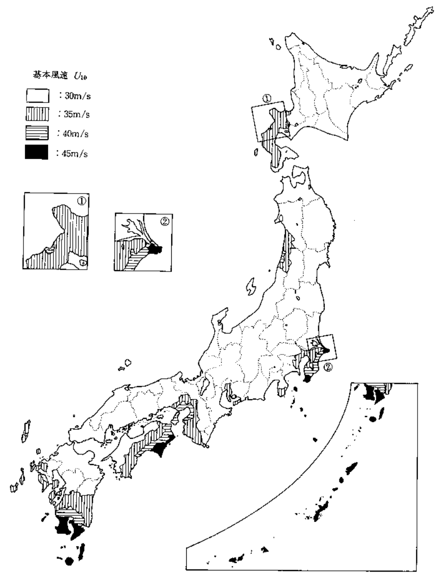
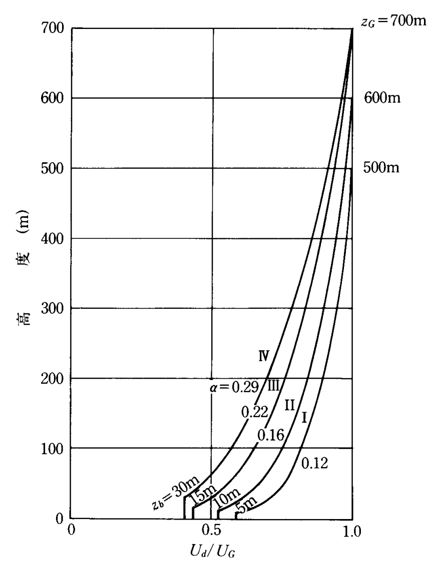


Fig. 21. Wind speed map in the wind resistant design manual, Japan[6]



Topographies of the roughness I,II,III,IV are on water, open terrain, suburb and down town/mountainous area respectively.

Fig. 22. Vertical wind speed profile in the wind resistant design manual, Japan

5.2 *Utilization of meteorological stations and statistical typhoon simulation*

The design wind speed will be decided referring wind speed observations at meteorological stations and extreme value statistical analysis of their annual maxima. However it will be easily understandable that those observations will already include effects of continuous change of their topography and difference of adopted anemometers. In Figure 23 and 24, annual maxima of the strong wind for these years are plotted.

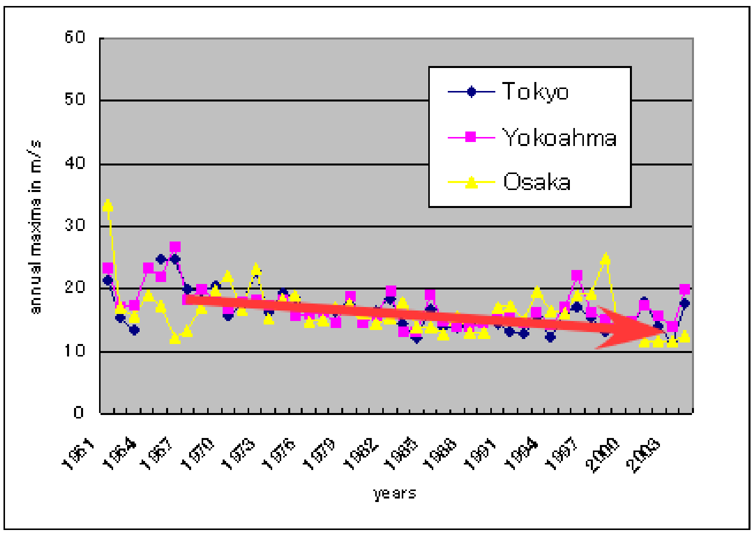


Fig. 23. Annual maxima of observed wind speed in urban areas

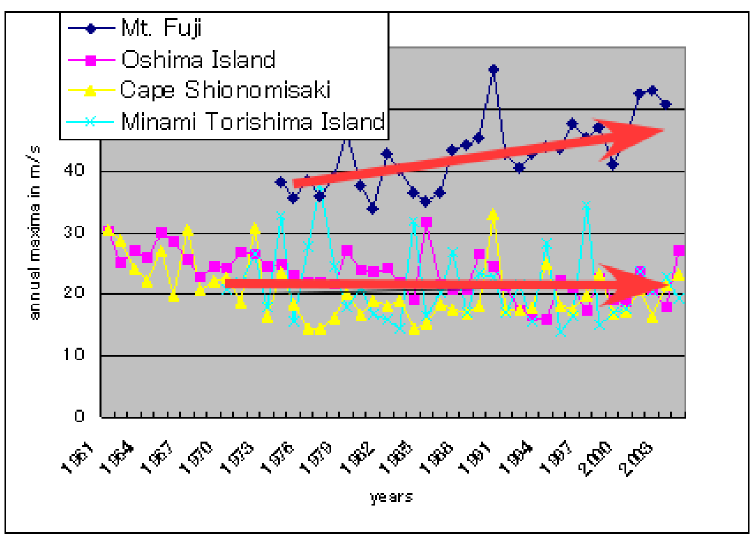


Fig. 24. Annual maxima of observed wind speed at cape, islands and Mt. Fuji

In Figures 23 and 24 it can be found that observations in urban are decrease slightly as years, although situations seems to be different at capes, islands and Mt. Fuji. Although it is not clear what this reason is, it is obvious that there exist some issues to disturb statistical homogeneity in the population of annual maxima of strong wind. Referring this fact, the statistical typhoon simulation is introduced as one of methods to realize many-year's homogeneous sampling. This is a method to simulate typhoon tracks basing on statistical information in every latitude and longitude meshes in occurrence, development or decay, direction of movement and speed of movement of virtual typhoons [7]. Although this is only one of many methods to get the design wind speed, both of typhoon simulation by CFD and this statistical approach will come to be widely applied.

6. Concluding remarks

Although there are many occasions to discuss structural aerodynamics at wind tunnels, the chain link of structural response in Figure 1 is very important point in the wind resistant design. To realize its balanced development, continuous investigations are carried out and some new approaches are introduced in this paper. To construct long-span bridges, it is very important to introduce careful wind resistant design backed up by reliable wind tunnel experiment, careful structural analysis and meteorological discussions. More reasonable the long-span structure design becomes, more frequently wind resistant design works will be required.

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