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VI

# Wind Resistant Design of a Cable-Stayed Girder Bridge

Le calcul de la résistance au vent pour le pont à haubans

Über den Windwiderstand der seilverspannten Brücke


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## 1. Introduction

Because of the development of structural analysis methods, computation measures, available materials and construction technique, dimensions and flexibility of recent bridges have been increased and the damping capacity of them has been decreased. As a result of these, recent bridges are liable to be subjected to not only static wind effects but dynamic ones. Cable-stayed girder bridges are one of the examples of them.

The authors consider that structures have to be designed against wind effects shown in Table-1. When a structure is rigid enough, only the aerodynamic wind forces shall be considered; but for a flexible structure, dynamic effects together with the static instability phenomena shall be considered. In the case of cable-stayed girder bridges, aerodynamic wind forces, aeolian vibration, galloping and/or torsional flutter among the wind effects listed in Table-1 will have a prime importance.

Table - 1 Wind Effects on Structures

| | | | |
|---|---|---|---|
| Wind effects | Static effects | Aerodynamic wind forces | Drag, lift, pitching moment |
| | | Static instability problem | Divergence |
| | | | Lateral buckling of girder |
| | Dynamic effects | Forced vibration | Random vibration |
| | | | Aeolian vibration |
| | | Self-excited vibration | Galloping |
| | | | Torsional flutter |
| | | | Coupled flutter |

As pointed out by J.F. Borges in his introductory report on the subtheme "Dynamic Loads", the estimation of wind velocity is a fundamental problem for the wind-resistant design of structures. The life time and height of structure, the local condition of structure site and the turbulence in wind mainly govern the estimation of wind velocity.

Among these factors wind turbulence will have two aspects in its influence on the estimation of wind velocity. The first is the spatial distribution of turbulence and the second is the structural response caused by turbulence. By considering the spatial distribution of turbulence, wind velocity on shorter bridges shall be greater than those on longer ones. The relation between wind velocity and the dimension of structures has already been derived in the tentative design criteria against wind effects for proposed Honshu-Shikoku bridges (1). On the other hand, effects of random vibrations excited by wind turbulence may be substituted by increasing the wind velocity so as to represent the expected maximum stress conditions in the structure, but, so far as the authors know, the quantitative modification of wind velocity has not yet been obtained.

In this contribution, the authors describe the wind-resistant design process of the Onomichi Bridge, including the estimation of wind velocity, results of the wind tunnel model tests and the vibration tests on the completed bridge. It is already shown that cable-stayed girder bridges not always possess a satisfactory stability against dynamic wind actions (2) (3). The wind tunnel model tests for the original design of Onomichi Bridge showed an unsatisfactory aerodynamic stability, too. However, fabrication of the bridge had been simultaneously progressed during the model tests and only a limited change in the sectional shape of girder was possible. Among several alternatives, a plan to install a lane of open grating at the center of bridge floor showed an improvement in increasing the critical wind velocity and was adopted.

Vibration tests on the dynamic characteristics of the completed bridge such as natural frequencies, vibration modes and structural damping were conducted. The measured structural damping was comparatively low which showed the possibility of wind excited vibration.

In the conclusion, the authors emphasize the necessity of the thorough investigation by wind tunnel model tests in the designing process of cable-stayed girder bridges.

## 2. Outline of the Onomichi Bridge

The Onomichi Bridge, which is located in the Seto-Inland Sea and spans a sound of about 200 meter wide between Onomichi City and Mukaijima, is a cable-stayed continuous girder bridge of 215 meters center span and 85 meters two side spans. The continuous girder of the bridge consists of two plate girders 3.2 meter high and steel plate deck 10.4 meter wide.

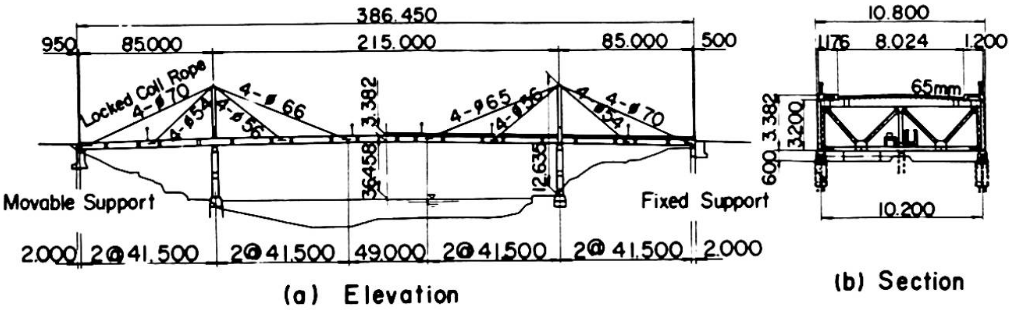
As shown in Figure-1, the girder is stayed at both sides by locked coil ropes in a fan shape. The ropes are supported by two towers of 72.6 meter high and are fixed at the girder ends and tower ends.

Natural frequencies of the bridge are approximately calculated as shown in Table-2. The ratio of fundamental natural frequencies in torsional mode and flexural mode is about 2.93.

Table - 2 Natural Frequencies of the Onomichi Bridge

(c/s)

| Mode | Symmetric Mode | | | Asymmetric Mode | | |
|---|---|---|---|---|---|---|
| Order | 1st. | 2nd. | 3rd. | 1st. | 2nd. | 3rd. |
| Vertical flexural vibration | 0.581 | 1.385 | 1.795 | 0.914 | 1.562 | 2.249 |
| Torsional vibration | 1.706 | 3.942 | 4.536 | 3.055 | 3.978 | 5.543 |




(a) Elevation (b) Section

**Figure-1 Elevation and Section of Onomichi Bridge**

## 3. Estimation of expected wind velocity

By reason of the importance and the life time of the Onomichi Bridge, 100 years period was chosen as the return period for estimating wind velocity.

About 5 kilometers apart from the bridge site, a meteorological observatory station exists and observed wind velocity data of 10 minutes duration after 1942 are available. Assuming the double exponential distribution of probability density of the annual maximum wind velocity, the return values in period of 50 and 100 years at the station are estimated as 21.5 and 22.8 m/s, respectively. The values should be modified by considering the difference of topographical condition between the bridge site and the station.

On the other hand, in connection with the meteorological survey for the proposed Honshu-Shikoku bridges, multi-regression analysis upon return values of wind velocity in the area of Seto Inland Sea were conducted. In the analyses, the influences of local topographical conditions such as the openness and undulation of topography, rate of sea area and others were taken into account. As the estimated values at the bridge site, 29.6 and 31.8 m/s for 50 and 100 years return periods were obtained by this method. However, because the multi-regression analyses were conducted for applying the wide area of Seto Inland Sea and it was not so sufficient to apply for the estimation of wind velocity in the local area, the values obtained by this method were ignored and 22.8 m/s was chosen as the fundamental value for estimating wind velocity at the bridge site.

By taking into account of the effect of convergence of wind in a narrow channel and other topographical conditions, wind velocity at the bridge site was estimated 1.2 times of those at the station, that was 27.4 m/s. It was considered that the comparatively low value of wind velocity was resulted from the greater roughness of ground surface around the bridge site. Therefore, 1/4 was assumed as an exponent of the power law for the vertical profile of wind velocity, which resulted in the modification factor of 1.377 at the altitude of the bridge girder of 36 meters.

As described in the introduction, wind velocity should vary according with the dimension of structure. The modification factor for the span length of 215 meters is 1.208 according to the design criteria for the proposed Honshu-Shikoku bridges.

As the result of modification of wind velocity mentioned above, 45.5 m/s was obtained. The aerodynamic stability of the bridge was judged by this value.

## 4. Wind loads and calculated critical wind velocities

In design of the Onomichi Bridge, horizontal wind loads of 1680 kg per linear meter of the girder and of 300 kg per unit area ($m^2$)of towers were taken into account according to the "Design Specifications for Steel Highway Bridges". As shown later, the wind tunnel model tests showed smaller value of drag acting on the girder than the above mentioned value.

The critical wind velocity for the lateral buckling was calculated by a formula derived by Hirai and Okauchi (4). In the calculation, values of drag and lift coefficients obtained by the wind tunnel model tests were used. The calculated value was 148.5 m/s and was far beyond the above mentioned wind velocity of 45.5 m/s.

Also, the critical wind velocity for the coupled flutter was calculated by introducing aerodynamic forces on the flat plate derived by Theodorsen, for the purpose of reference, though the air flow around the bridge girder usually separated from the surface of structure and the theory based on the potential flow could not be applied. The calculated value was 78 m/s and exceeded the above mentioned value of 45.5 m/s.

## 5. Wind tunnel model tests

Measurements of three components of aerodynamic forces and instability tests were conducted on section models of 1/25.6 scale. A wind tunnel of Göttingen type was used for the tests, which had the test section of 3.0 meter high and 1.8 meter wide. The maximum wind velocity of the tunnel was 23 m/s. A detailed description of the tunnel is shown in the reference (5).

In the measurements of aerodynamic forces, an electrical beam balance was used. In the instability tests, the model was mounted horizontally on a spring system with its spanwise axis normal to the wind flow. The model was allowed vertical and/or pitching motions separately or in coupled motion.

The polar moment of inertia and the mass of model per unit span were simulated to those of the prototype. No reliable value of structural damping of the actual bridge was available for the authors, those of the prototype in flexural and torsional motion were assumed to be 0 06 and 0.05, respectively. Damping of the model were kept as low as possible and, for the model of modified final design, an additional damping was given by a set of electro-magnetic dampers. The ratio of torsional frequency to the flexural ones of the prototype in the fundamental mode was about 2.93, but because of the installation mounting the model, the ratio of the model was about 2.

The similitude on the reduced velocity was used in the conversion of wind velocity from model to prototype. In other words, the value of reduced velocity V/NB, in which V, N and B were the wind velocity, the frequency of vibration and the representative linear dimension of model and prototype, was assumed same for model and prototype.

As the wind tunnel model test on the original design progressed, it was revealed that a negative slope of lift coefficient curve was found in the measurement of aerodynamic forces and galloping vibration started in comparatively low wind velocity in the instability test; so, changes in external shape of the bridge girder were required. However, at that time, fabrication of the girder was simultaneously progressed and only a slight change was possible.

Several alternative plans were proposed and tested in the wind tunnel and finally a plan to install a lane of open grating at the center of girder was adopted. Table-3 shows the required and actual values of models for the original and modified final design. For the brevity, results of the model tests only for the original and modified final design are shown in this contribution.

Aerodynamic coefficients of the girder sections are shown in Figure-2. The negative slope of lift coefficient appeared in the original design could not be diminished even in the modified final design. However, as seen in Figure-3, the dynamic behavior of model was improved. Figure-3 (a) shows relations between amplitude and wind velocity in flexural vibration when models were subjected to horizontal wind. Conditions of models were different, so wind velocity converted to the prototype is shown in the figure.

Table - 3 Values of Model

| Model | Weight | | Polar moment of inertia | | Structural damping | |
|---|---|---|---|---|---|---|
| | required | actual | required | actual | flexural $\delta_h$ | tortional $\delta_\alpha$ |
| | gr. | gr. | gr-cm-s$^2$ | gr-cm-s$^2$ | | |
| Original | 7667 | 7648 | 3620 | 3430 | 0.060 | 0.022 |
| Modified final | 7667 | 7679 | 3620 | 3650 | (0.029<br>0.065* | (0.008<br>0.050* |

| Frequency | | Frequency ratio | |
|---|---|---|---|
| flexural Nh | torsional $N\alpha$ | required | actual |
| c/s | c/s | | |
| 1.55 | 3.48 | 2.93 | 2.25 |
| 1.80 | 3.90 | 2.93 | 2.17 |

* With additional damping by electro-magnetic damper units.

In the case of original design, vertical vibration of restricted amplitude set on at 9.6 m/s wind velocity and held out to the wind velocity more than 30 m/s. Beyond 35 m/s, the amplitude grew rapidly and the vibration became catastrophic. The predicted critical wind velocity of the prototype was about 38 m/s The tests on modified final design were conducted with two different damping values as shown in Table-3. The value of logarithmic decrement in vertical mode with additional damping were almost same with the test on original design. However, in the modified design, a restricted vibration set on at the wind velocity of about 13 m/s and lasted until 16 m/s. Beyond the wind velocity of 48 m/s, a vertical flexural vibration occurred again and the vibration became catastrophic with the increase of wind velocity. Thus, the critical wind velocity of 48 m/s was predicted for the modified design. The restricted vibration in the modified design seems to be an aeolian vibration and the catastrophic one a galloping. In the case of original design, it can be considered that overlaping of aeolian vibration and galloping have occurred.

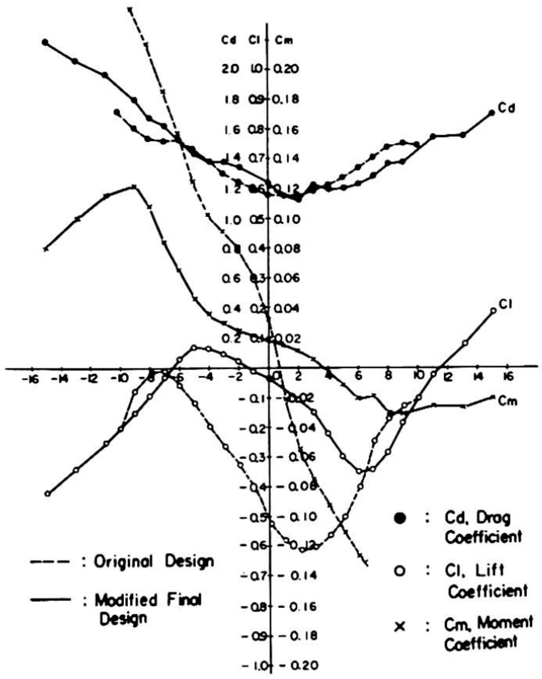
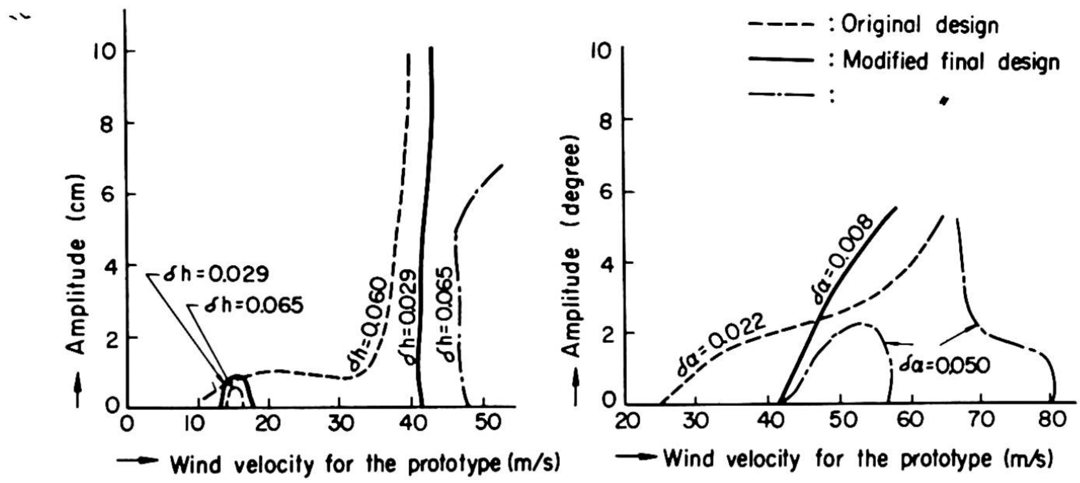



Figure –2 Aerodynamic Coefficients of Girder Section




(a) Flexural mode (b) Torsional mode

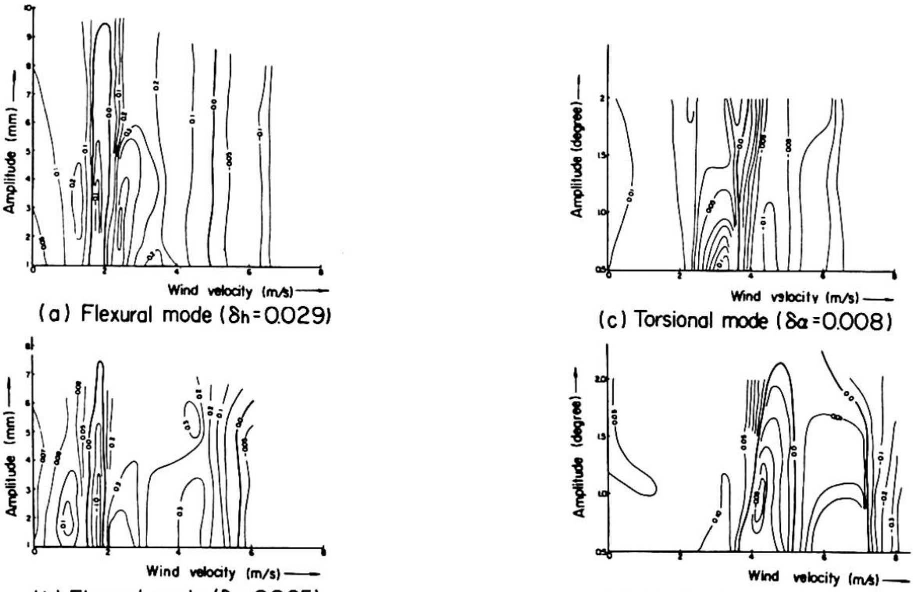
Figure – 3 Amplitude and Wind Velocity of Model Vibration

Figure-3 (b) shows relations of torsional amplitude and wind velocity. In this case, too, the manner of vibration was similar to those of flexural vibration. But the model of modified design with $\delta\alpha$ = 0.050 showed a noteworthy behavior in the range of wind velocity 67 to 80 m/s. In this range, when a small disturbance less than 2 degrees was given to the model, then the vibration died out, but when the initial disturbance exceeded 2 degrees, then the vibration diverged.

As seen in Figure-3, the value of structural damping influences the aerodynamic behavior of bridge. So the necessity of measuring the value of prototype was acutely felt and the vibration test of the Onomichi Bridge after its completion was scheduled during the model tests were progressed. For serving the prediction of aerodynamic behavior of the bridge after finding the value of structural damping of the completed bridge, a special analysis was applied to the records of wind tunnel model tests on the modified final design.

In general, the value of damping consists of structural damping and aerodynamic one and the aerodynamic damping varies according to wind velocity, amplitude of vibration, sectional shape of the structure and incidental angle of wind. From the diagrams showing the amplitude and numbers of vibration, we can obtain the values of damping corresponding to each amplitude and each velocity and can draw contour lines which show the relation among the values of damping, amplitude and wind velocity. Figure-4 shows such contour lines of flexural and torsional vibrations for the model of modified design with and without additional damping.

In Figure-4, (a) and (b) show contour lines in flexural motion without and with additional damping, respectively. The structural damping, which means the damping in still air, of the former is 0.029 and that of the latter is 0.065. The difference is about 0.035. If the superposition of structural damping was possible, zero contour in (b) must coincide with - 0.035 contour in (a). The comparison of (a) and (b) shows that this is correct qualitatively but not in the strictly quantitative meaning.




(a) Flexural mode ($\delta_h$=0.029) (c) Torsional mode ($\delta_\alpha$=0.008)

(b) Flexural mode ($\delta_h$=0.065) (d) Torsional mode ($\delta_\alpha$=0.050)

**Figure-4 Contour of Aerodynamic Damping**

Figure-4 (c) and (d) show contour lines in torsional motion. The structural damping of the former is 0.008 and that of the latter is 0.050 and the difference is about 0.04. In torsional vibration, too, zero contour line in (d) roughly coagree with - 0.04 contour line in (c). From the above facts, the authors consider that these contour lines offer an effective supplementary measures for predicting the aerodynamic behavior of prototype.

## 6. Vibration tests of the completed bridge

After the completion of the bridge, vibration tests for surveying mainly structural damping were conducted. In the tests, the bridge was vibrated by specially devised twin excitors which were able to generate reciprocating forces in phase or out of phase and thus able to excite the bridge in any of flexural and torsional motions. Figure-5 shows a plan of twin excitors. The exciting frequency is variable from 0.2 to 10 c/s. The maximum exciting force per each unit at 10 c/s is 15 tons. A remarkable feature of the excitors is that the position of unbalanced weights can be changed during the operation so as to keep constant exciting forces regardless of frequency. The other remarkable feature of them is that the exciting forces can be eliminated within short period by moving unbalance weights into zero output position. The former is useful for recording resonance curves and the latter for causing a damped free vibration.

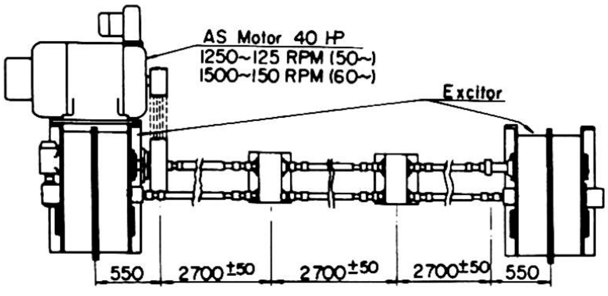
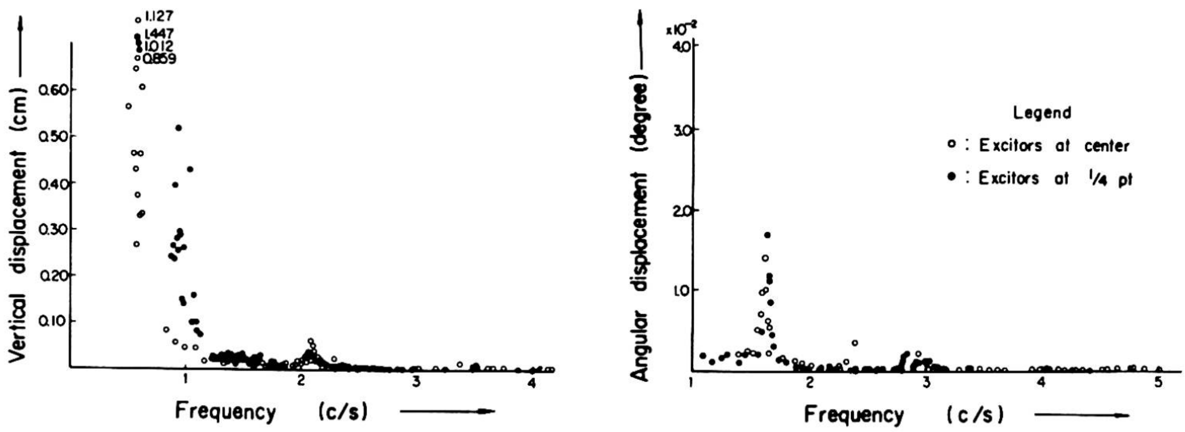



Figure-5 Plan of Twin Excitors




(a) Flexural Vibration (b) Torsional Vibration

Figure-6 Resonance Curve

Table - 4 Measured Natural Frequencies

(c/s)

| Mode | Symmetric mode | | Asymmetric mode | |
|---|---|---|---|---|
| | First | Second | First | Second |
| Vertical flexural | 0.58 | 1.38 | 0.92 | 1.62 |
| Torsional | 1.66 | - | 2.94 | - |

The excitors were installed at the center or at the quarter point of the center span. Motion of the bridge at every 1/8 point in the center span, every quarter point in the side spans and at the top of the tower were measured by using temporary installed accelerometers. Figure-6 (a) shows a resonance curve in vertical flexural vibrations and (b) shows those in torsional vibrations. Figure-7 shows modes of vibration in the fundamental symmetric and asymmetric modes of vertical flexural and torsional vibrations. Table-4 shows measured values of natural frequency. The comparison of Table-4 and 2 shows good coincidence of the calculated frequency and measured ones.

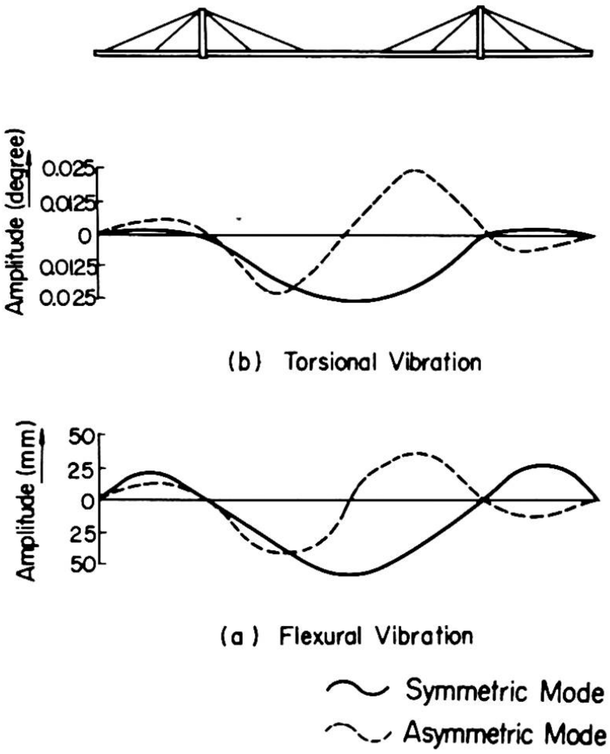



Figure-7 Modes of Vibration

Figure-8 shows diagrams of amplitude and number of vibration cycle in the damped free vibration of the bridge. The logarithmic decrements of the bridge were obtained by averaging the slopes in the diagram and were 0.05 for the vertical flexural motion and 0.035 for the torsional motion. The measured values are somewhat smaller than those assumed in the wind tunnel model tests.

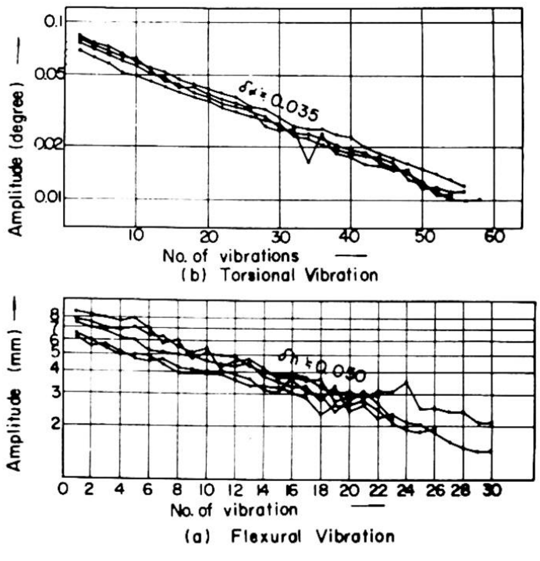



Figure-8 Decrement of Amplitude

## 7. Observation

As the results of wind tunnel model test show, the critical wind velocity of the Onomichi Bridge for the aerodynamic instability is not so high and possibly the restricted vibration occurs in low wind velocity. In fact, during the vibration tests, the bridge was subjected to wind velocity of about 13 m/s and a vertical flexural vibration of about 20 $cm/s^2$ acceleration, which was stationary, was caused by wind and was recorded. The observed frequency was almost equal to the natural frequency of vertical flexural vibration in the first symmetric mode.

For inspecting the dynamic behavior of the bridge under the wind action, two anemometers and ten accelerometers have been installed. One of the anemometers has been installed at the top of tower and the other at the center of main span. When wind velocity exceeds a certain amount, say 20 m/s, recording papers of these run fast and wind velocity of every two or three seconds can be obtained. Accelerometers are coupled with the anemometer at the center of main span, and when wind velocity exceeds the above amount, they start to record vibrations of the bridge. Vertical flexural, torsional and swaying vibrations can be observed.

Also, accelerometers can start to record vibrations caused by earthquake when ground acceleration exceeds a certain amount, say 5 cm/s².

8. Conclusion

In the introduction, the authors have classified wind effects on structures as shown in Table-1. The wind tunnel model tests on the Onomichi Bridge in steady wind have shown that aeolian vibration, galloping and torsional flutter of the bridge girder possibly occur and that cable-stayed girder bridges are liable to vibrate under wind actions as similar as other flexible structures.

When a structure is rigid enough, static wind loads such as drag, lift and pitching moment are enough to be taken into account in designing it. On the contrary, when a structure is flexible, not only static wind loads but dynamic wind effects on it should be considered. So it can be concluded that two major problems in the wind resistant design of structures are to estimate the design wind velocity and to consider dynamic wind effects on them.

There are several methods for estimating wind velocity, to which structures are subjected, but sometimes return values obtained by different methods differ from each other. The difference is considered to be mainly caused by the evaluation of influences of the local topographical condition at the structure site. On the local distribution of mean wind velocity, multi-regression analyses on return values, numerical calculation method based on the fluiddynamic equations, wind tunnel tests for topographical models and instrumental observation of the actual distribution are the evaluating methods. Studies for establishing an effective method of statistically estimating return values of maximum wind velocity taking into account the local topographical conditions of the structure site will be necessary.

At present, it is very difficult to represent dynamic wind effects on structures in terms of wind loads. In the near future, dynamic wind effect causing aeolian vibration on structures may be represented by a stationary external force acting on them and those causing random vibration (buffeting) may be represented by the equivalent increase of wind velocity. However, it would be essentially impossible to represent the dynamic wind effects causing self-excited vibrations in terms of wind loads, even in the case of soft flutter. Therefore from the view point of wind resistant design of structures, especially flexible ones, "wind effect" instead of "wind loads" shall be considered.

The value of critical wind velocity, which governs the dynamic instability of structures, should be investigated for self-excited vibrations. Only the wind tunnel model test is the measures for predicting the critical wind velocity for the prototype. Besides the critical value of wind velocity, dynamic responses of structures such as the amplitude and frequency of vibration can be revealed by the model test.

From a functional point of view, Selberg (6) proposed three kinds of critical wind velocity in the soft flutter problems according to their torsional amplitudes. In this case, prediction of vibratory amplitude of structures is indispensable for evaluating the critical wind velocity of them. Because the vibratory amplitudes in the soft flutter are governed by the value of structural damping, contour lines of aerodynamic damping related to amplitude and wind velocity as shown in Figure-4 offer an effective measures for predicting the critical wind velocity.

From the reasons mentioned above, the authors conclude that the design of flexible structures such as cable-stayed girder bridges or suspension bridges should be investigated by the wind tunnel model tests in the region located in zone of strong wind like our country.

In addition, measurements of structural damping, especially those in torsional mode, of completed bridges have an important meaning on the aerodynamic stability of structures and are desirable. Those values obtained in our test on the Onomichi Bridge were considerably low. The accumulation of values of structural damping measured on presenting structures is quite necessary.

Finally, the author emphasize that the observation on the dynamic behavior of structures under wind action contributes to the progress in wind resistant design method of them as same as it contributes to the inspection of structural safety.

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SUMMARY

This paper describes the design considerations of the Onomichi Bridge against wind effects such as the estimation method of design wind velocity, results of wind tunnel model test, vibration tests of the Bridge and installations observing the aerodynamic response of bridge. Basing on their classification of wind effects, the authors point out the possibility of causing a cable-stayed girder bridge aeolian vibration, galloping and torsional flutter and the necessity of considering dynamic wind effects besides wind loads in the design.

RÉSUMÉ

Cet article décrit les considérations de dimensionnement faites pour le pont Onomichi contre les effets du vent: Méthodes d'estimation de la vitesse du vent, résultats d'expériences faites sur modèle au tunnel aérodynamique, tests vibratoires sur le pont et installations observant le comportement aérodynamique du pont. Se basant sur leur classification des effets du vent, les auteurs relèvent la possibilité d'obtenir des vibrations sur un pont à haubans, des galoppades et des flottements tordants. Ils montrent la nécessité de considérer les effets dynamiques à côté des charges de vent dans le dimensionnement.

ZUSAMMENFASSUNG

Der Beitrag beschreibt die notwendigen Betrachtungen über den Windeinfluss, die bei den Studien der Onomichi-Brücke gemacht wurden, wie Schätzungsmethode für die in die Berechnung einzusetzende Windgeschwindigkeit, Modellversuche im Windkanal, Vibrationstests an der Brücke und Einbauten zum Beobachten des aerodynamischen Verhaltens der Brücke. Die Autoren stützen sich auf ihre Klassifizierung der Windeinflüsse, um die Möglichkeit von Schwingungen, Galoppieren und Torsionsschlingern an seilverspannten Brücken zu betonen. Sie weisen auf die Notwendigkeit hin, bei der Bemessung neben den ruhenden Windlasten auch die dynamischen Einflüsse zu berücksichtigen.