Zeitschrift: IABSE proceedings = Mémoires AIPC = IVBH Abhandlungen
Band: 6 (1982)
Heft: P-52: Statical model tests of suspension bridges under construction

Artikel: Statical model tests of suspension bridges under construction
Autor: Iwaki, Y. / Ueda, K. / Kimura, K.
DOI: https://doi.org/10.5169/seals-36658

Statical Model Tests of Suspension Bridge under Construction

Essais statiques sur modèle d'un pont suspendu en construction

Statische Modellversuche einer im Bau befindlichen Hängebrücke

Y. IWAKI
Chief Engineer
Mitsui Engineering & Shipbuilding Co., Ltd.
Tokyo, Japan

K. KIMURA
Engineer
Mitsui Engineering & Shipbuilding Co., Ltd.
Tokyo, Japan

K. UEDA
Manager
Matsuo Kyoryo Co., Ltd.
Osaka, Japan

Y. SHIMODA
Engineer
Matsuo Kyoryo Co., Ltd.
Osaka, Japan

SUMMARY

This paper presents the results of the statical test with a large three dimensional model, which simulates a suspension bridge with 3-span continuous truss stiffened girder having no hanger in the side spans. The purpose of this test is to observe the statical characteristics of the bridge model, and confirm the validity of the erection method and effect of wind bracing method, which is presented here. The accuracy of the analytical method is also certified.

RÉSUMÉ

L'exposé traite des résultats obtenus à partir d'essais statiques menés avec un grand modèle tridimensionnel qui simule un pont suspendu avec poutre de rigidité continue sur trois travées et ne possédant aucune suspension dans les travées latérales. L'objet de ces essais est d'analyser les caractéristiques statiques de ce modèle de pont. La validité de la méthode de montage et les effets de la méthode de contreventement, qui font l'objet du présent exposé, sont entièrement confirmés. La précision de la méthode analytique est également prouvée.

ZUSAMMENFASSUNG

Dieser Aufsatz legt die Ergebnisse eines statischen Versuches an einem grossen dreidimensionalen Modell vor, das eine Hängebrücke mit einem über drei Felder durchlaufenden, fachwerkartigen Versteifungsträger ohne Hänger in den Seitenfeldern simuliert. Zweck dieser Untersuchungen ist, die statischen Kennwerte des Brückenmodells festzustellen. Zudem wurden die Eignung des vorgesehenen Montageverfahrens und die Wirksamkeit des gewählten Windverbandsystems überprüft. Die Genauigkeit des analytischen Verfahrens wird ebenfalls bestätigt.

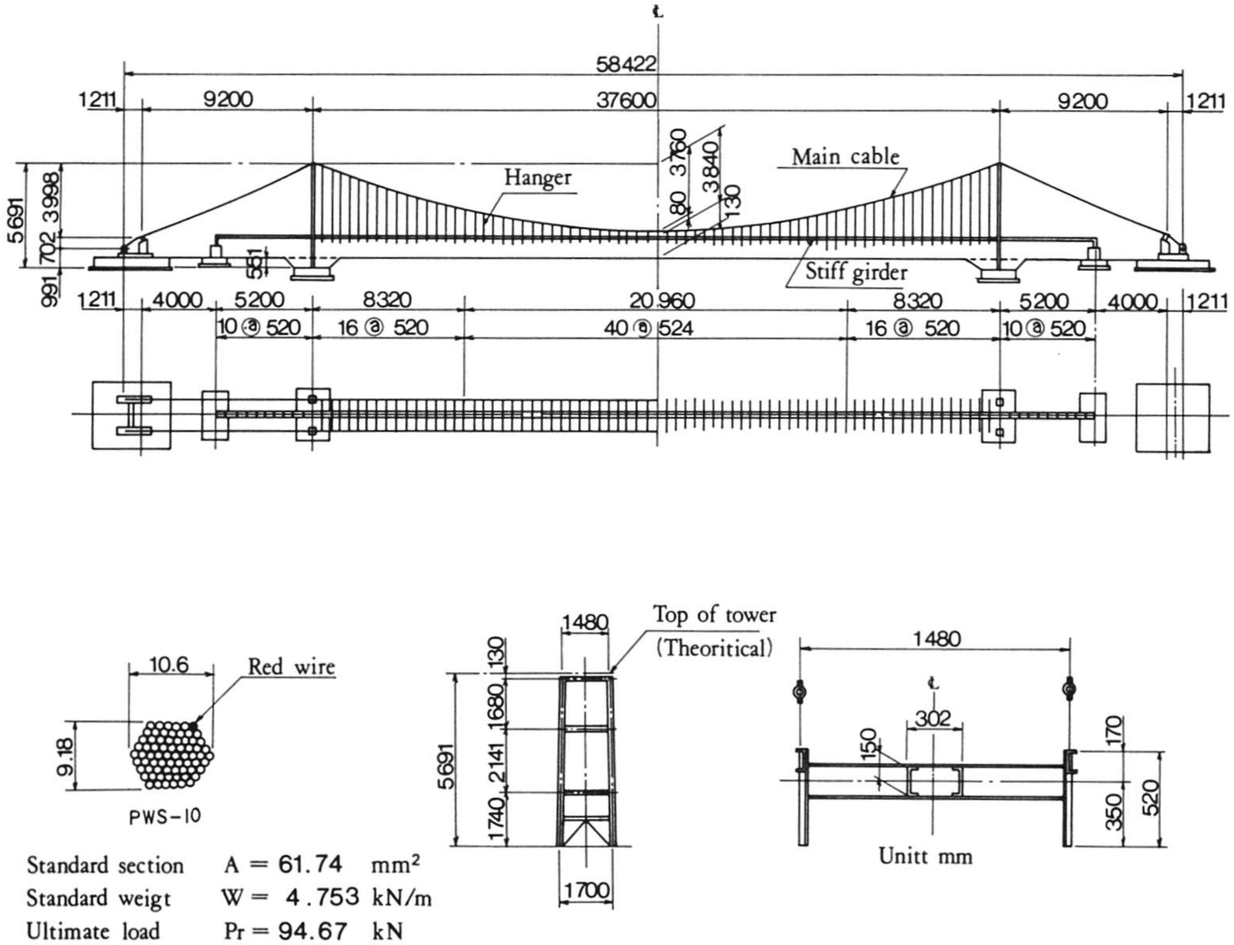
1. INTRODUCTION

Recently a number of long span suspension bridges such as the Honshu-Shikoku bridges and others are either under construction or being planned to be constructed in Japan.

In planning and constructing such bridges, special considerations must be given to the extremely servere climatic and other conditions, including typhoons and earthquakes, to which the bridges are exposed from time to time.

The experiment explained in this paper aims:-
(a) To observe statical characteristics of deflection and stress in a long span suspension bridge under construction.
(b) To cinfirm the validity of the erection method preposed in this paper, thereby certifying the accuracy of analytical calculation method developed by the authors [1] [2]. In the experiment, a large three dimensional model is used, which simulates "a 3-span suspension bridge with a continuous truss stiffened girder, that has no hanger in the side spans", (Fig. 1).

The experiment is required by the reasons that there are only a few experimental reports on the behaviours of the suspension bridge with rigidly connected stiffening girder, during erection [3], and these testing models are too small for the detailed and reliable measurements and observations of those behaviours, in case of the erection method proposed in this paper.



Standard section	A = 61.74 mm²
Standard weigt	W = 4.753 kN/m
Ultimate load	Pr = 94.67 kN
Young's modulus	E = 197.0 kN/mm²

Fig. 1. General view of model

The proposed erection method is as follows:

1) Erection of blocks of the truss stiffened girder is carried out by connecting them together successively from the tower side to span center, balancing the forces and deflections of center span and the side span blocks, for stability against wind turbulence.
2) Hinges for erection are provided in the truss stiffened girder of the center span, for reduction of hanger stresses.
3) Erection blocks are composed of a complete truss stiffened girder block, for shortening of construction time and manhours.
4) The truss stiffened girder for the side span is erected by first assembling the blocks from the tower side towards the span center, untill it overhanges to the temporary support near the abutment. It is then set down on the support shoe at the abutment by jacking. This method reduces the cost of construction.
5) Setting of the last block in the middle of the center span is carried out by the surcharge method for reduction of cost.
6) All erection members are brought to the site by sea, and blocks of the center span are hoisted into position by the lifting crane and assembled by the block erection method.

The following matters are investigated in the experiment.

1) Measurement of deflections and forces under construction as well as in the complete bridge state, and comparing them with the calculated values.
2) For the erection method.
   a) Relations between deformation of stiffening girder and tension force of the hanger by the pull in loading.
   b) Observation of clearance of stiffening girder at the hinge point for erection.
   c) Observation of surcharge method of last block setting in the middle of center span.
   d) Relations between deformation of stiffening girder and force of jack-up loading at the site temporary support in the side span.
3) Observation of deflection of stiffening girder and cable by lateral loading equivalent to wind load.
4) Observation of effects of wind bracing proposed in this paper.

2. THEORITICAL FORMULA FOR ANALYTICAL METHOD

Fundamental formula for the axial forced member showin in Fig. 2 based on the finite displacement theory is as follows [2].

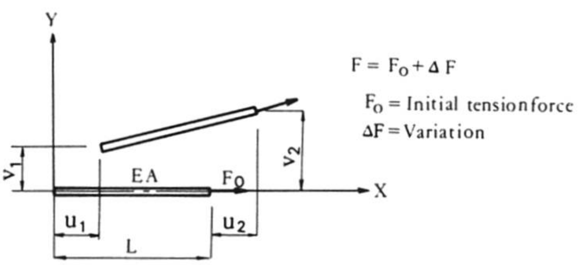


Fig. 2. Axial forced member

Strain:

$$\varepsilon_x = \left[\left(1 + \frac{\partial u}{\partial u}\right)^2 + \left(\frac{\partial v}{\partial x}\right)^4 \right]^{1/2} - 1$$

$$\fallingdotseq \frac{\partial u}{\partial x} + 1/2\left(\frac{\partial v}{\partial x}\right)^2$$

here,

$$\frac{\partial u}{\partial x} = \frac{u_2 - u_1}{L} = \frac{DU}{L}$$

$$\frac{\partial v}{\partial x} = \frac{v_2 - v_1}{L} = \frac{DV}{L}$$

Strain energy:

$$U_x = \frac{1}{2} EA\int_O^L \varepsilon x^2 \cdot dx$$
$$= \frac{EA}{L}\left[\frac{1}{2}(U_2 - U_1)^2 + \frac{1}{2}\frac{(u2 - u1)\ (v2 - v1)^2}{L} + \frac{1}{8}\frac{(v2 - v1)^2}{L^2}\right]$$

Axial forces:

$$fx_1 = \frac{\partial U_x}{\partial u_1} = \frac{EA}{L}\left[(u_2 - u_1) + \frac{(-v_1 + v_2)}{2L}(v_1 - v_2)\right]$$
$$fy_1 = \frac{U_x}{v_1} = \frac{EA}{L}\left[\frac{N}{EA}(v_1 - v_2) + \frac{(-v_1 + v_2)^2}{2L^2}(v_1 - v_1)\right]$$
$$fy_2 = \frac{\partial U_x}{\partial u_2} = -fx_1\ , \quad fy_2 = \frac{\partial U_x}{\partial V_2} = -fy_1$$

here, $N = \frac{EA}{L}(-u_1 + u_2) + Fo$

Indication by Matrix:

$$\mathbf{F} = \mathbf{KU}$$
$$\mathbf{K} = \mathbf{K}o + \mathbf{K}_1 + \mathbf{K}_1$$

$$\mathbf{K}o = \frac{EA}{L}\begin{bmatrix} 1 & 0 & -1 & 0 \\ 0 & 0 & 0 & 0 \\ -1 & 0 & -1 & 0 \\ 0 & 0 & 0 & 0 \end{bmatrix}, \quad \mathbf{K}_1 = \frac{EA}{L}\begin{bmatrix} 0 & 0 & 0 & 0 \\ 0 & \alpha & 0 & -\alpha \\ 0 & 0 & 0 & 0 \\ 0 & -\alpha & 0 & \alpha \end{bmatrix}$$

$$\mathbf{K}_2 = \frac{EA}{L}\begin{bmatrix} 0 & \beta & 0 & -\beta \\ 0 & \gamma & 0 & -\gamma \\ 0 & -\beta & 0 & \beta \\ 0 & -\gamma & 0 & \gamma \end{bmatrix}$$

here, $\alpha = \frac{Fo}{EA} + \frac{DU}{L}$, $\beta = \frac{DV}{2L}$, $\gamma = \frac{DV^2}{2L^2}$

3. EXPERIMENT

3.1 Model

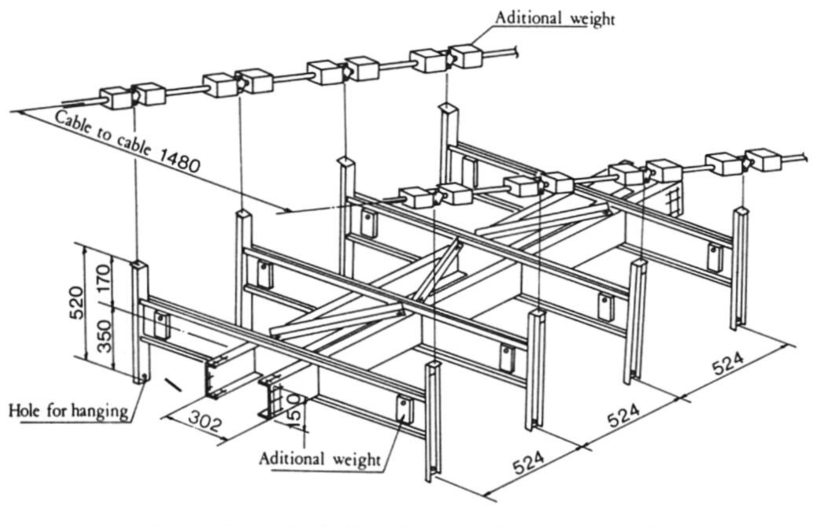


Fig. 3. Model of stiffening girder

The model of suspension bridge used in the experiment is shown in Fig. 1, and a stiffening girder composed of steel channel beams simulates the truss stiffened girder as shown in Fig. 3. The center span of the prototype is 940 m long. Scale of length is 1/25, and weight is 1/6, 36. Conditions of of similarity are obtained by Buckingham's Π theorem. Main cable is composed of a pararell wire strand (PWS-70, 70 piano wires with 1.06 mm dia.), and hanger is made of the same single piano wire.

3.2 Experiment

The flow of experiment work is as follows:

- Statical test
  - Vertical loading test (Photo 1)
    - Complete bridge test
    - Backward test
    - Forward test
  - Lateral loading test (Fig. 6, 7, Photo 2)
    - Complete bridge test
    - Backward test

The test process of experiment is shown in Fig. 4, The items and points of measurement are described in Fig. 5.

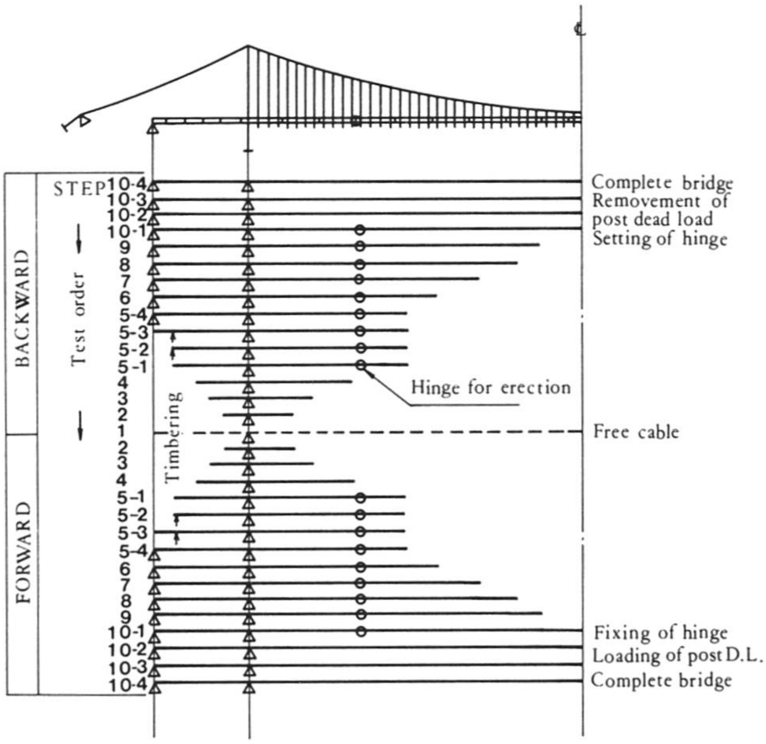


Fig. 4. Procedure of experiment

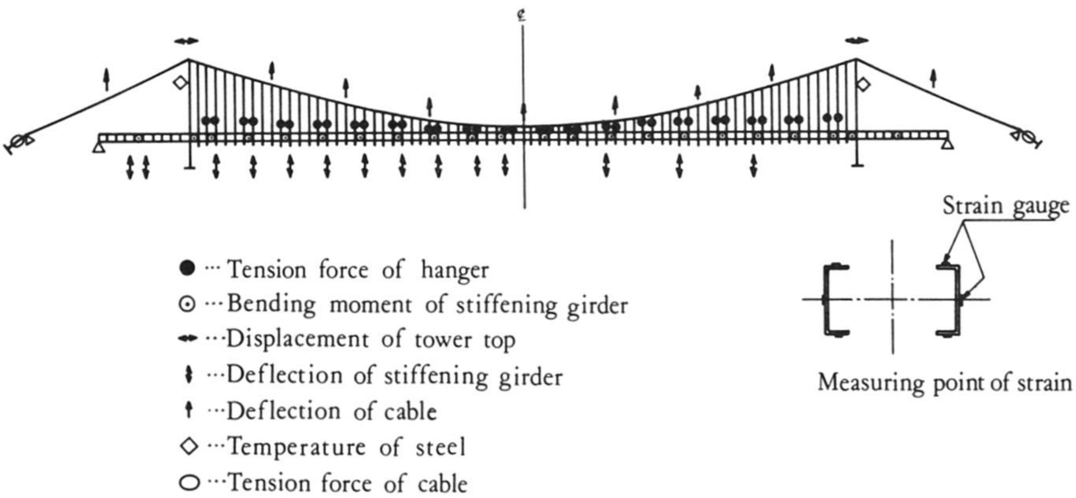


Fig. 5. Items of measurement

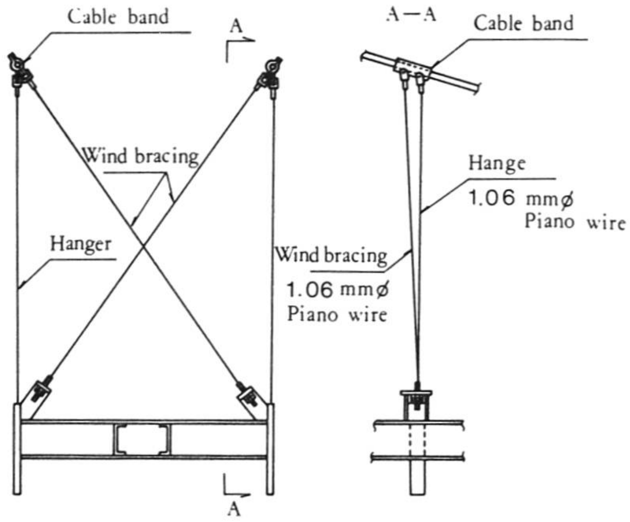


Fig. 6. Wind bracing

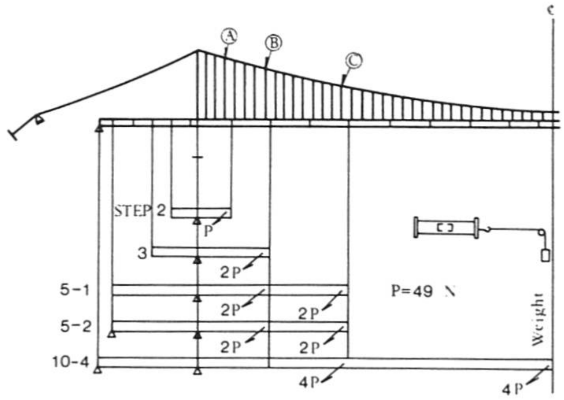


Fig. 7. Procedure of lateral loading test

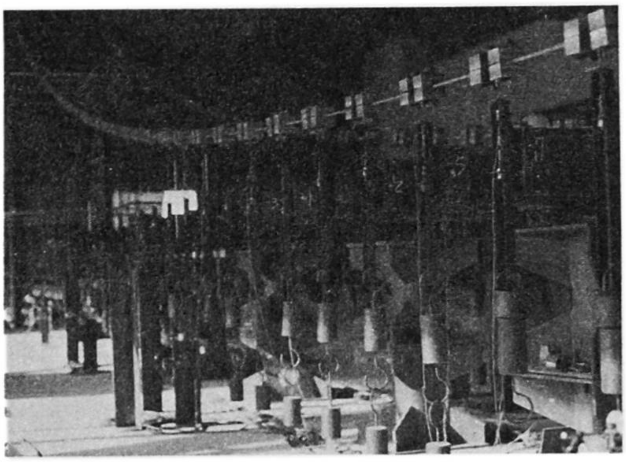


Photo 1 Vertical loading test

Photo 2 Lateral loading test

In this experiment, "backward" means process from the complete bridge state to the free cable state, and "forward" means from the free cable to the complete bridge state as shown in Fig. 4. The backward and forward tests are repeated twice each, and practically no difference was noted between them. The backward test result is adopted in this paper.

4. RESULTS OF EXPERIMENT

4.1 Vertical loading test

1) Complete bridge state

Before the test in the construction state, vertical loading test in the complete bridge state is carried out toconfirm the validity of the analytical calculation method as mentioned previously.
The results are shown in Fig. 8, 9.

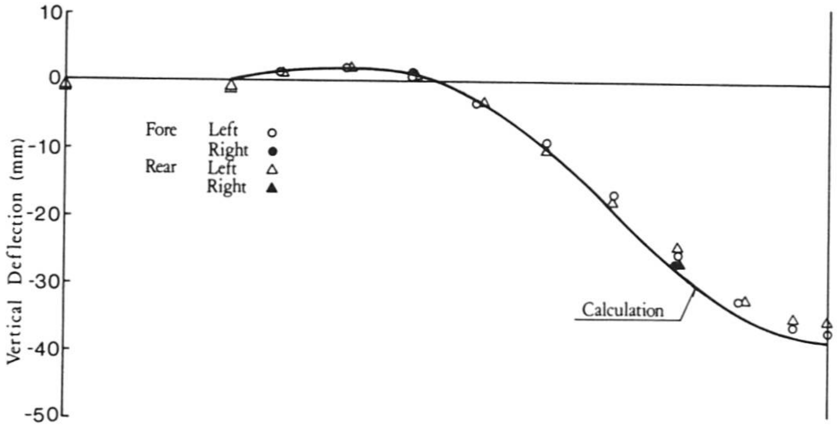


Fig. 8 Vertical deflections of stiffening girder (Sym., complete bridge)

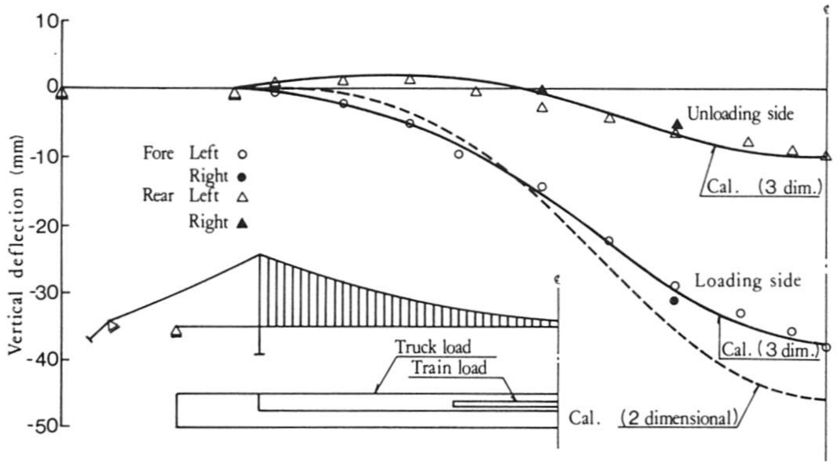


Fig. 9. Vertical deflections of stiffening girder (Ansym., complete bridge)

In a symmetrical loading case, the mean value of the measured vertical deflection of stiffening girder is 35.9 mm at the middle of the center span, while the analytically calculated value is 38.8 mm. Consequently, the difference between the measured and calculated values is 7.5 %. In an eccentrical loading case, the deflection on the loading side is 38.1 mm and that on the unloading side is 10.1 mm. Calculated values of the three dimensional analytical method are 37.1 mm and 9.8 mm respectively, which means that the differences are 2.7 % and 3.1 % respectively.

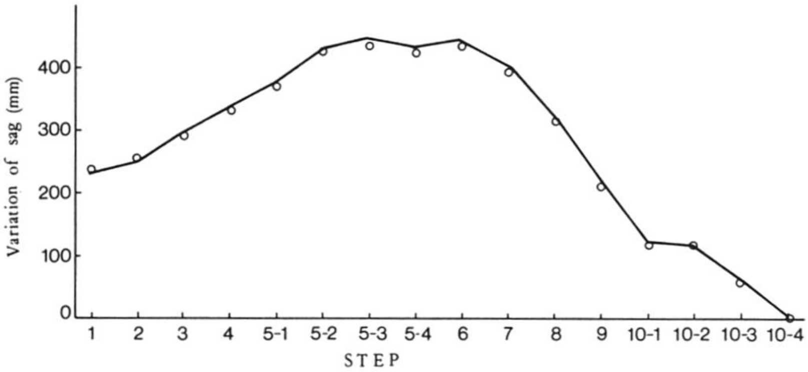


Fig. 10. Variations of sag in center span

2) Under construction state

Maximum variation of sag in the center span is 447 mm on STEP-6 as shown in Fig. 10, while the calculated value is 425 mm, showing a difference of 5 % between the measured and calculated values. On STEP-1, the measured value is 237 mm, while the calculated value is

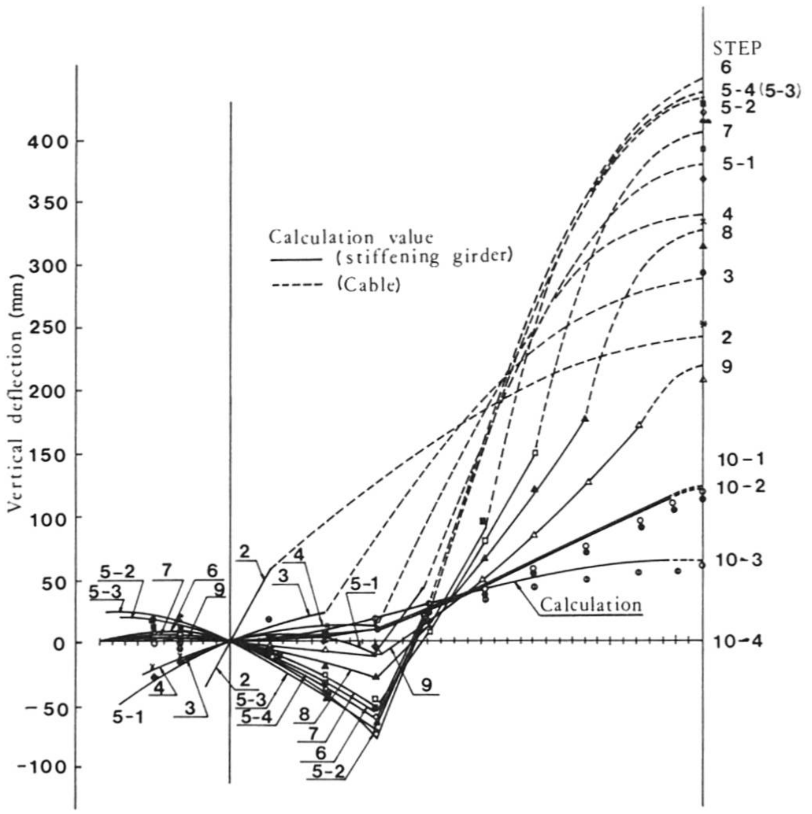
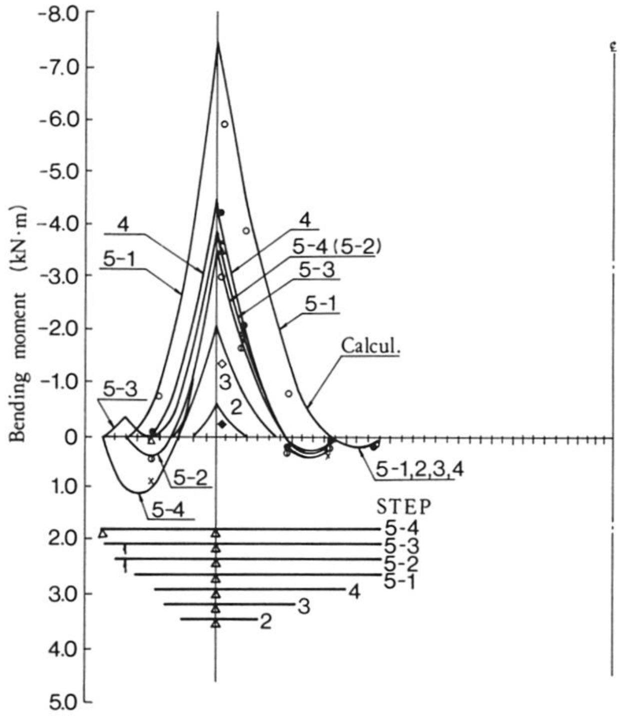


Fig. 11. Vertical deffections of stiffening girder

235 mm, showing a difference of 1 %.
Deflections of the stiffening girder are shown in Fig. 11. Max. deflection at the tip of the erecting girder in the centerspan is 177 mm on STEP-8, and 59.5 mm on STEP-5-2 at the erection hinge, while the calculated values are 187.2 mm and 63.1 respectively, with a difference of 5 ∿ 6 %.
Bending moments of the girder are shown in Fig. 12, 13. It is recognized that the maximum stress arises on the support of girder on the tower side on STEP-5-1. This stress is almost equal to the allowable limit of stress, and consequently, also the maximum overhanging state of stiffening girder in side span during construction. Variations of cable-tension force and horizontal displacement at the top of the tower on every step also coincide with the analytical calculated values as shown in Fig. 14, 15.



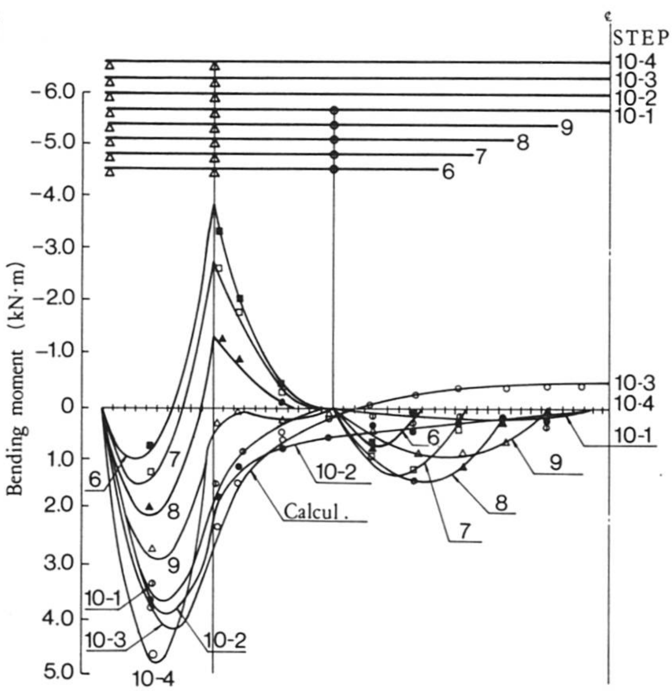


Fig. 12.
Bending moment of stiffening girder (1)

Fig. 13.
Bending moment of stiffening girder (2)

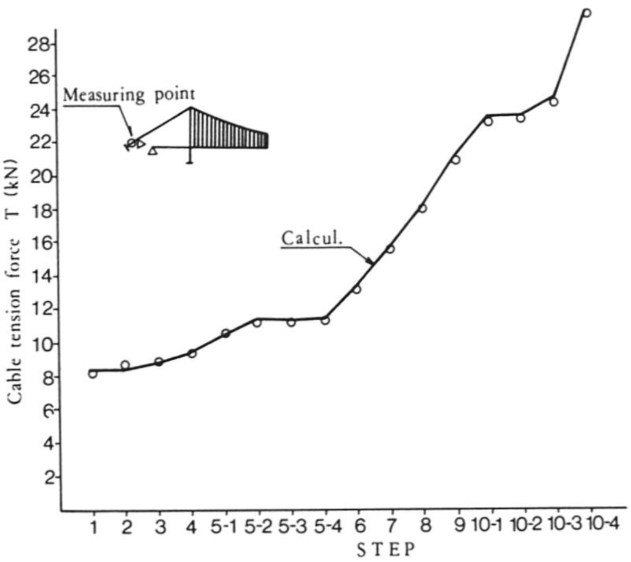


Fig. 14. Cable tension forces

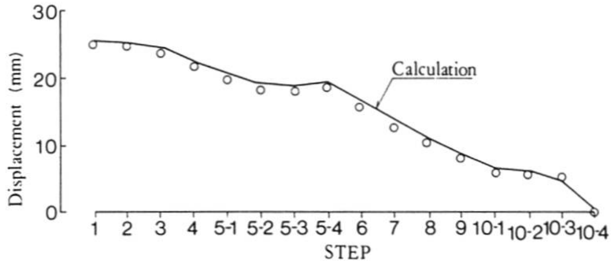


Fig. 15. Dispalcements of tower top

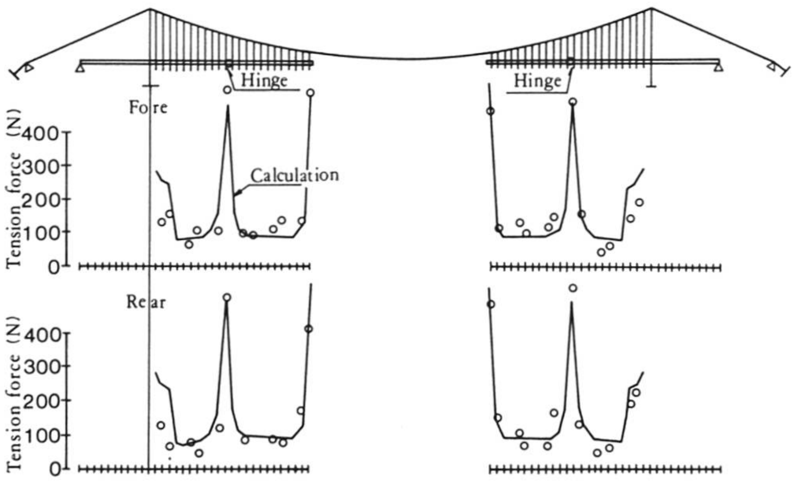


Fig. 16. Tension force of hanger STEP-7

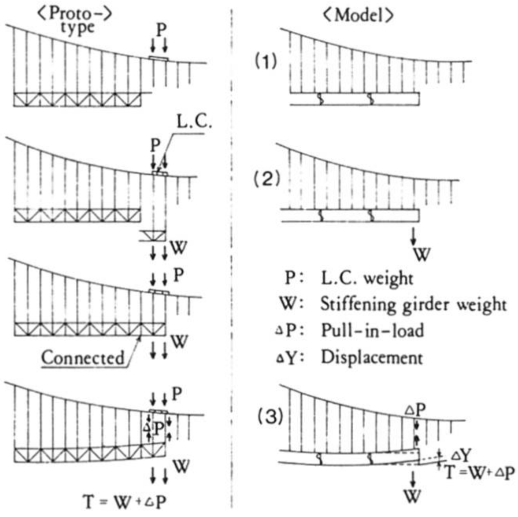
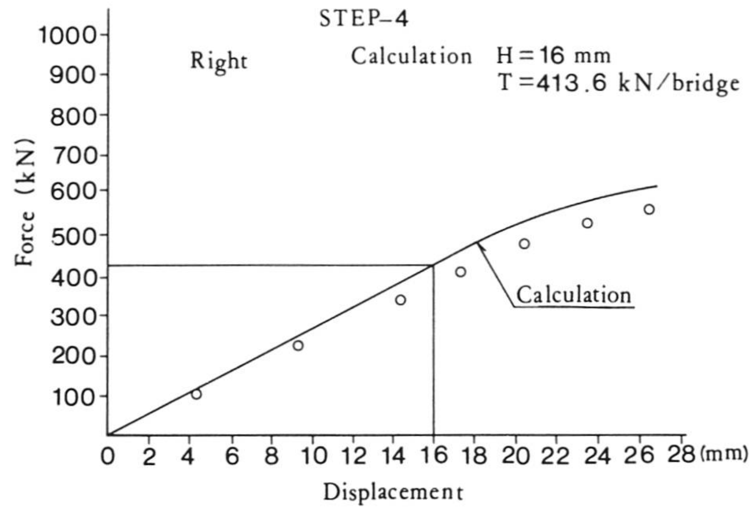


Fig. 17. Procedure of pull-in loading

Tension forces of the hanger represent the characteristics as shown in Fig. 16.
In view of the nearness of analytical results mentioned above to the experimental results, the analytical method presented here may well be considered so certified.

3) Investigations of the erection method.

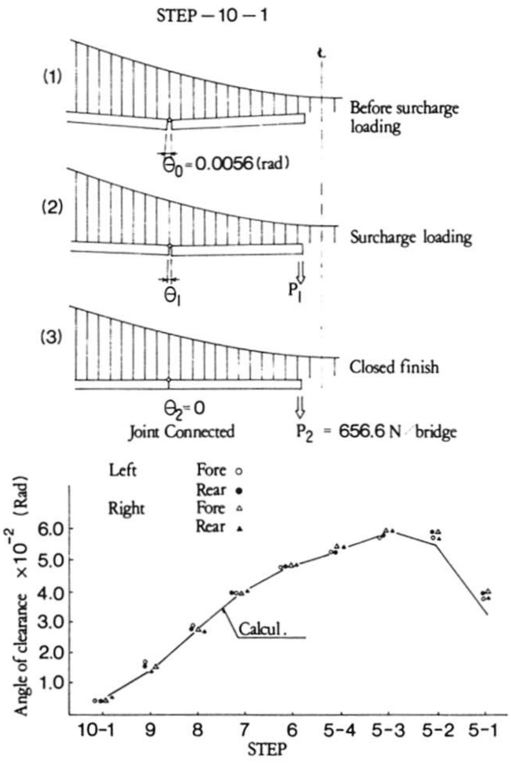
a) Relations between deformation of stiffening girder and tension force of hanger by the pull-in loading method.
Fig. 17 shows the main points of process for pull-in loading method, and it is made clear that the relation between the force applied and deformation is non-linear as shown in Fig. 18. The analytical method represents a nonliNear behavior with tolerable accuracy. For example, ΔP = 411.6 N/bridge is needed in case of ΔY = 16 mm



for model, where ΔP means 4,124 kN/bridge and ΔY means 400 mm for the prototype.

Fig. 18.
Displacements and forces by pull-in-loading

b) Clearance of the stiffening girder at the hinge point for erection. Fig. 19 shows the main points of the process by a closing method. Clearance varies with the progress of construction step as shown in Fig. 19. It is recognized that the minimum clearance occurs on STEP-10-1, and the angle of clearance θ = 0.0056 radian. Therefore, the surcharge load required is 656.6 N/bridge, which corresponds to ΔP = 117.6 kN perradian, where ΔP = 6,664 kN would be in prototype, for which the own weight of a lifting crane will be substituted in practice (see Fig. 20).



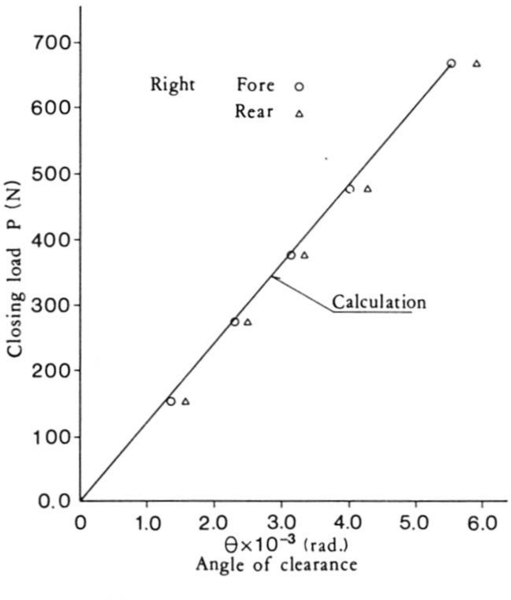


Fig. 19. Angles of clearance Fig. 20. Closing load of hinge

c) Surcharge method on last block setting in the middle of the center span. In order to close the block clerance of the stiffening girder, surcharge loading will also be needed, such as lifting crane (L.C.) as showin in Fig. 21, the load of which is P = 705.6 N/bridge in the model, i.e. P = 7,252 kN/bridge in the Prototype. Provided that L.C. is 294 N and block weight is 980 N in the model, $P = 705.6 - \frac{1}{2} \times 980 - 294 = -78.4$ N; which means P = 7,840 kN in the prototype. Hence a little or more surcharge load may be necessary. Fig. 22 shows this relation.

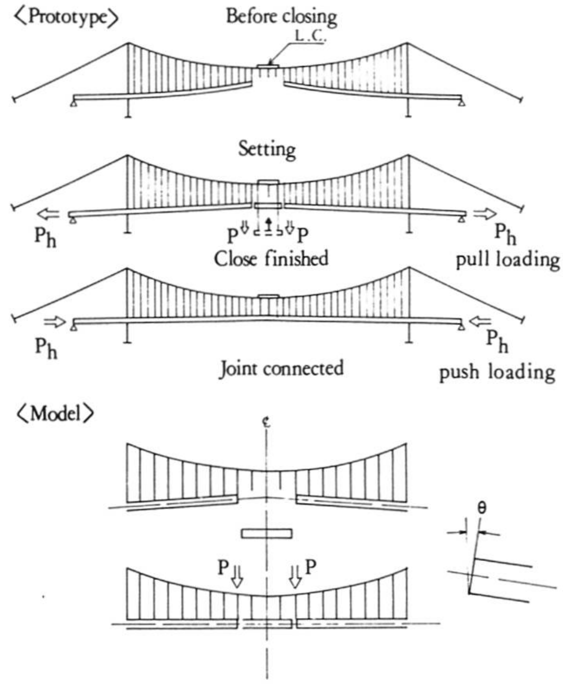


Fig. 21. Setting of final block

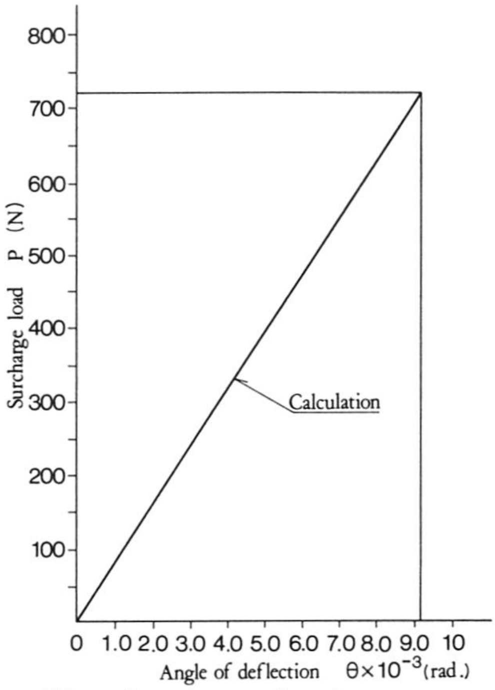
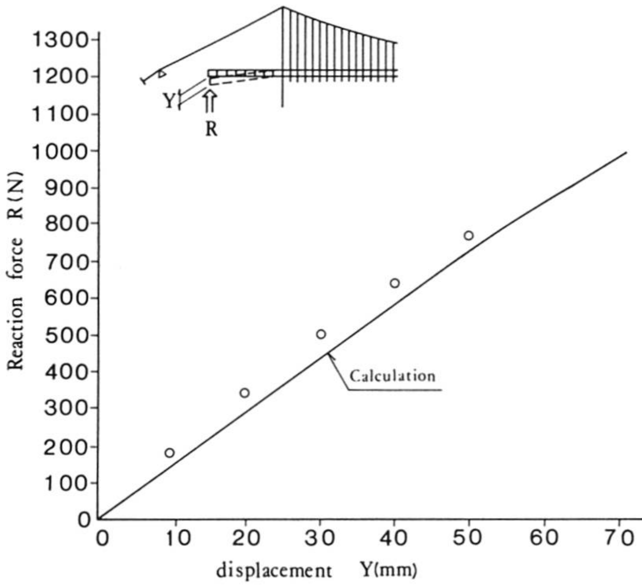


Fig. 22. Surcharge load for closing last block



d) Relation between deformation and reaction of the stiffening girder by jack-up loading on the site temporary support in the side span.
As shown in Fig. 23, relation between Y and R is linear, therefore on STEP-5-1, where $R/Y \fallingdotseq 14.7$ N/mm, $Y = 72$ mm would be required, and where $R = 14.7 \times 72 = 1{,}058.4$ N in the model, which means $Y = 1.8$ m, $R = 10{,}780$ kN/bridge in the prototype.

Fig. 23. Displacement and reaction force by jack up loading

4.2 Lateral loading test

1) Complete bridge state

Lateral deflections of the stiffening girder and cable are 12.0 mm and 15.0 mm shown in Fig. 24, while the calculated values are 12.0 mm and 14.3 mm. Differences are 0 % and 4.5 % respectively. Thus, good coincidence is obtained in spite of imperceptible loading.

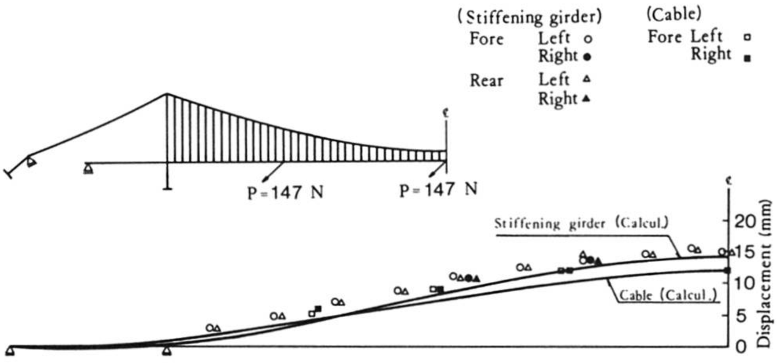


Fig. 24.
Laterial displacement (complete bridge)

2) Effect of wind bracing rope

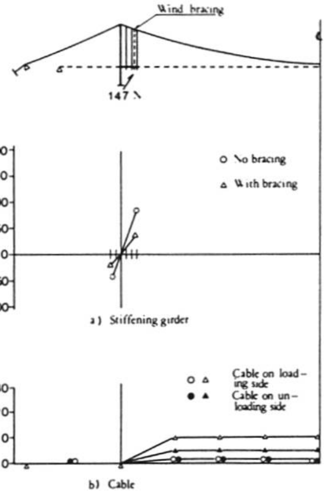


Fig. 25.
Lateral displacement
STEP-2

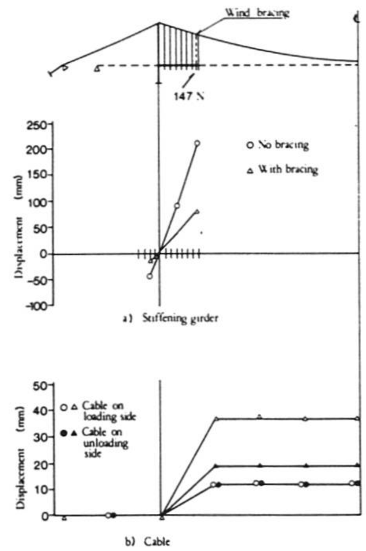


Fig. 26.
Lateral displacement
STEP-3

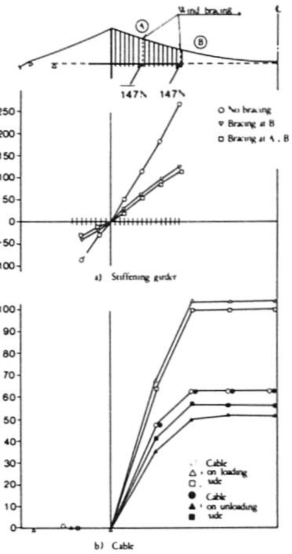


Fig. 27.
Lateral dispalcement
STEP-5-1

The deflections of the stiffening girder with wind bracing rope indicated in Fig. 6 are reduced to about 50 % of those without bracing as shown in Fig. 25 ∿ 27. But, those of the cable increase to 200 % without bracing. Difference between those with bracing at A, B and with bracing at B alone is only about 5 %, indicating that it is most effective at the tip of the construction girder.

5. CONCLUSION

1) The analytical calculation method was developed for statical deformations and stresses of the suspension bridge. The experimental results coincide with the theoretical results.
2) Erection method was presented. It is clarified that the erection method presented in this paper can be appleid with high accuracy.
3) Wind bracing method is effective in reducing lateral deflection of the stiffening girder during construction, contributing toward preventing large lateral deflections caused by lateral wind force against the damages of the tower link and end of the stiffening girder.

6. ACKNOWLEDGEMENT

The authors wish to express their gratitude to Dr. M. Itoh, professor, Tokyo University and Dr. S. Komatsu, professor, Osaka University in Japan for their valuable advice. The experiment was made possible with financial support from Mitsui Engineering & Shipbuilding Co., Ltd., and Matsuo Kyoryo Co., Ltd.

REFERENCES

1. IWAKI, Y., et al.: Analysis of suspension bridge by two dimensional finite displacement method, Mitsui Zōsen Technical Review No. 97, 1977.

2. IWAKI, Y., et al.: Structural analysis program of suspension bridge, Mitsui Zōsen Technical Manual, STAT III, 1978.

3. BERG, P., KOLLMEIER, H.: Beiträge von Hängebrückenbau, Veröffentlichung des Instituts für Statik und Stahlbau der Technische Hochschule Darmstadt, Heft 8, 1965.