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Cable-Stayed Bridge with New Vierendeel-Type Girder

Pont haubanné avec un nouveau type de poutre Vierendeel

Schrägseilbrücke mit einem neuen Typ von Vierendeel-Träger

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SUMMARY

To accommodate increases in traffic, a double-deck type cable-stayed girder will be used in future. With the double deck, however, a very stiff girder will be required. If the girder is of a truss type, the flexural rigidity of the truss will be substantially above the optimum value, and then the solidity ratio of the truss will increase together with the worse aerodynamic stability. In addition, drivers' visibility on the lower deck will be poor, and painting or other maintenance will be difficult. The present paper demonstrates the effectiveness of new Vierendeel type of stiffening girder to solve the above-mentioned problems.

RÉSUMÉ

Afin de tenir compte de l'accroissement de la circulation, un tablier à deux étages sera de plus en plus utilisé dans l'avenir. Cet ouvrage à deux étages nécessite, toutefois, une poutre de rigidité très massive. Dans le cas d'une poutre à treillis, la rigidité latérale peut être substantiellement supérieure à la valeur optimale et sa solidité augmente en remettant en cause la stabilité aérodynamique. De plus, le champ de vision du conducteur est très limité. Les travaux de peinture, et certains autres travaux d'entretien, sont difficiles à exécuter. Le présent article montre les avantages de la poutre de rigidité Vierendeel de conception nouvelle.

ZUSAMMENFASSUNG

Um das zunehmende Verkehrsvolumen zu bewältigen, wird ein abgespannter Träger mit zwei übereinander liegenden Verkehrsebenen verwendet. Dieser Träger muss sehr steif sein. Wenn er als Fachwerkträger ausgebildet ist, so wird seine Biegesteifigkeit wesentlich grösser sein als der optimale Wert, und sein Festigkeitsverhältnis wird zusammen mit der schlechteren, aerodynamischen Stabilität steigen. Ausserdem wird ein Fahrer auf der unteren Decke schlechte Sichtverhältnisse haben, und Malerarbeiten sowie andere Instandhaltungsarbeiten werden schwierig sein. Die vorliegende Abhandlung zeigt die Wirksamkeit des neuen Vierendeel-Verstärkungsträgers, der das obenerwähnte Problem beseitigt.

1. INTRODUCTION

Cable-stayed bridges have great structural and economic advantages and can be constructed in a modern shape. With these features, cable-stayed bridges are expected to be built in large numbers in the future as bridges applicable to span lengths between those to be covered by girders or long-span suspensions. It is likely that some of them will reach a long span of 400-500m.

Also, an increase of heavy traffic volume will require the use of double deck that permits efficient handling of the traffic.

A truss is usually used for a stiffening girder of a cable-stayed bridge of the double deck type. This method, however, has several problems.

- The girder height of the stiffening girder in question inevitably becomes large with the flexural rigidity larger than an optimum value. This makes bending moments in the stiffening girder larger than economically advantageous values, thus leading to uneconomical bridges.

- As shown in Fig. 1, the truss height of the stiffening girder or the center-to-center distance of the upper and lower chord members, H_t, will be designed to be at least 7-8m (or 12.0m in a different case), to cover 4-4.7m overhead clearance, H_r, for vehicles running on the lower deck of the double deck structure, plus the space α for signals and signboards, and dimensions required for the floor beam and other structures.

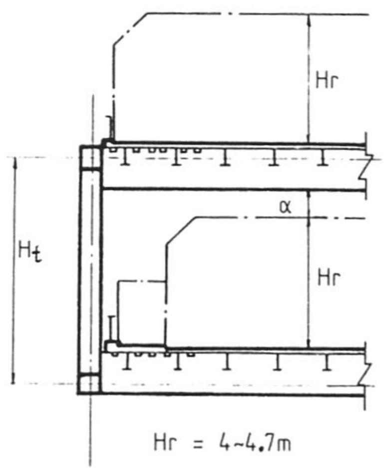


Fig. 1 Construction gauge of double deck bridge

- Thick and heavy structural members (diagonal and vertical members, etc.) cannot be avoided in the truss at a double-deck type bridge. Besides, gusset plates on the truss panel points, anchorage of stay cables, and complex, deep floor systems of the upper and lower decks, require a large exposed area of the truss in the horizontal direction, causing unfavorable aerodynamic stability of the cable-stayed bridge.

- Many unsightly structural members of complex shapes attached onto the back of the upper deck over drivers of vehicles running on the lower deck, and vertical and diagonal members of the truss obstructing the side view, may give an unpleasant impression to the drivers.

- A double-deck bridge having two-storied road surfaces gives a more complex structure than a single deck bridges. This makes a coated surface for repainting extremely large, thus causing a high maintenance cost. In addition, when the back of the upper deck is painted, the painting work must be minimized so as not to cause inconvenience for traffic on the lower deck.

The authors will give some suggestions as measures to solve these problems. This paper presents a new idea concerning cable-stayed bridges, namely, the use of a Vierendeel-type stiffening girder for cable-stayed bridges of the double deck type. Features of the new type of a cable-stayed bridge will be pointed out and then will be discussed one by one.

2. FEATURES OF NEW TYPE OF CABLE STAYED BRIDGE

First a proposed design of a new type of cable-stayed bridge is shown. Fig. 2 shows the general view, Fig. 3 the overall cross section, and Fig. 4 the cross section of the stiffening girder. Photo 1 shows a partial model of the stiffening girder. Next, features of this cable stayed bridge are listed below:

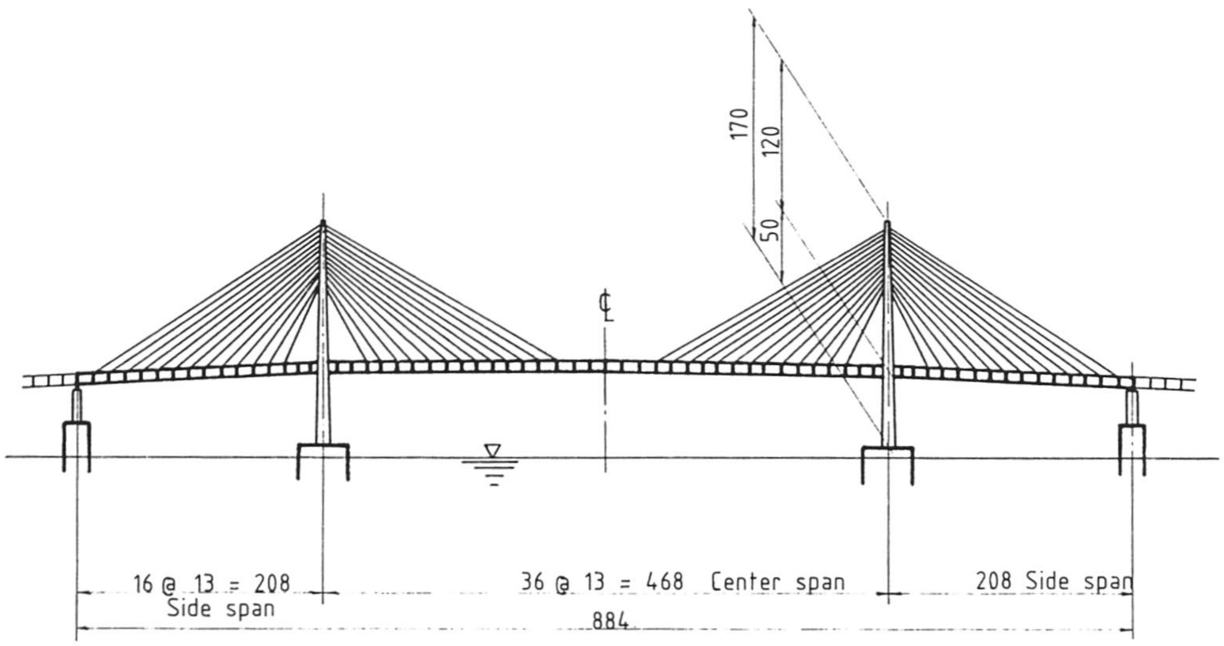


Fig. 2 General view at proposed design (unit in m)

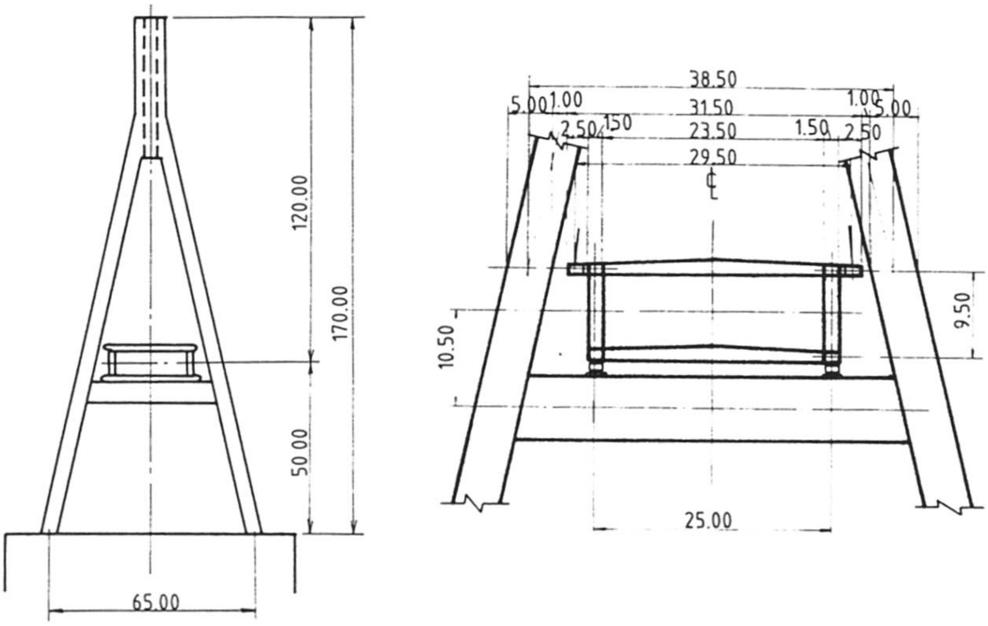


Fig. 3 Cross section at proposed design (unit in m)

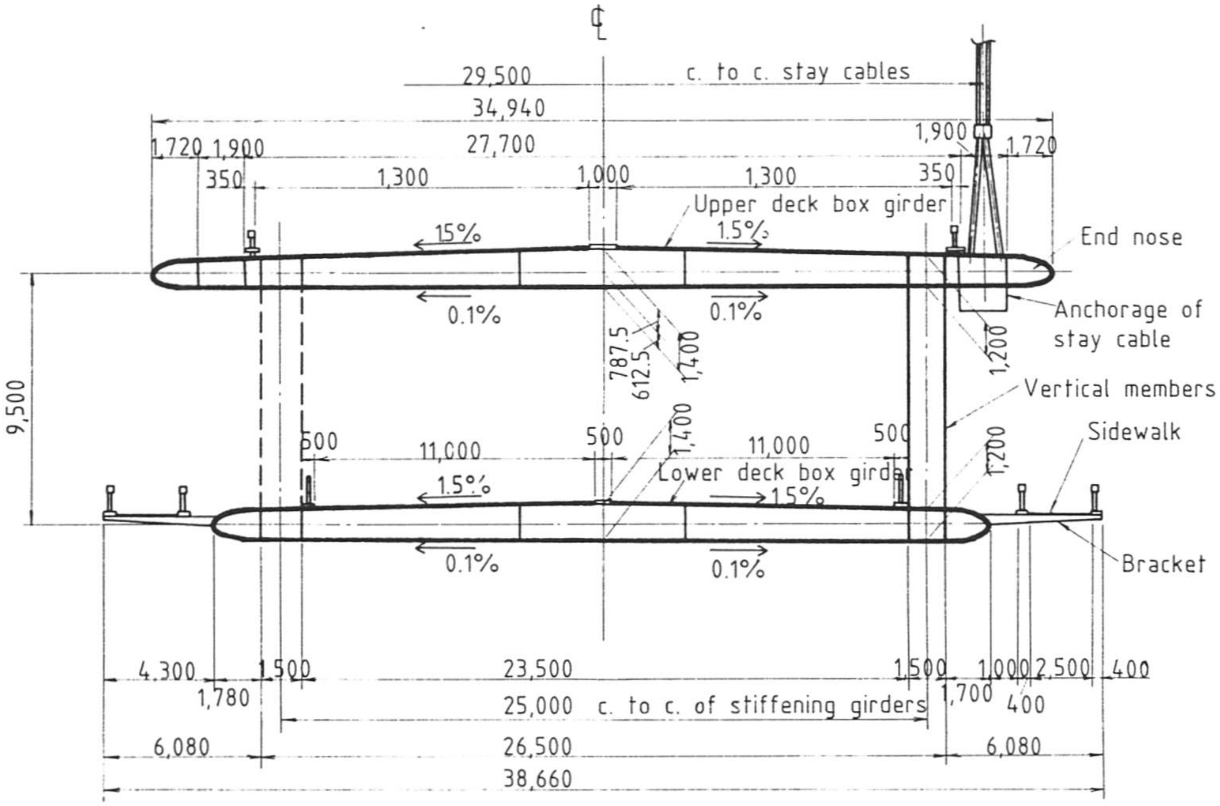


Fig. 4 Cross section of stiffening girder (unit in mm)

- As seen in the cross section of the stiffening girder shown in Fig. 4, a kind of composite construction is used, in which the upper and lower decks are respectively formed by a shallow box girder having a streamline section, and these two independent box girders, upper and lower, are connected with vertical column members so that they can form a Vierendeel-type with the upper and lower chord members.

- A simple construction, with various structural members installed in the upper and lower box girders, contributes to a smaller bridge-body surface area without projections or recesses on these upper and lower box girders. This is advantageous for maintenance, particularly for repainting.

- Anchoring of stay cables can be achieved using a space inside the upper deck box girder. This will make them inconspicuous as well as advantageous in terms of aerodynamic stability. (Fig. 5).

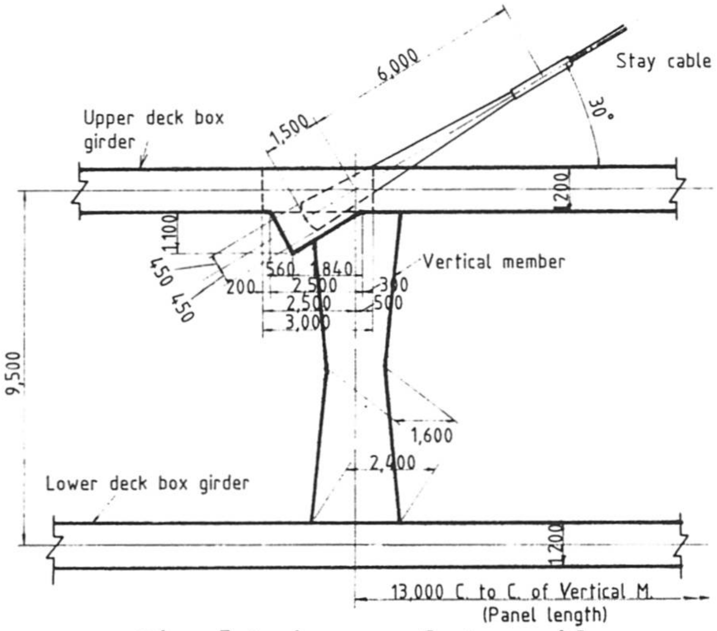


Fig. 5 Anchorage of stay cable (unit in mm)

- In a double-deck type stiffening girder, a distance between the upper and lower chord members, namely, a truss depth, inevitably has to be large enough to ensure a sufficient overhead clearance for vehicles on the lower deck. Limitation of slenderness ratio, however, makes it impossible to use very slender truss members. Because of this, the truss as a stiffening girder inevitably has to have a larger overall flexural rigidity than the optimum value. Therefore, free choice becomes impossible and an optimum flexural rigidity cannot be provided with. With the Vierendeel girder, however, the use of a more slender vertical columns leads to a considerable drop in the stiffness of the vertical member, without changing the girder depth. It is therefore possible to decrease the flexural rigidity of the Vierendeel-type of stiffening girder, to any desired level. That is, the flexural rigidity of the stiffening girder can freely be made extremely small, ensuring the possibility of a free choice in the reduction of the flexural rigidity of the stiffening girder with an optimum flexural rigidity. Thus, an economical cable-stayed bridge to satisfy a particular purpose of use could be designed.

- Unlike ordinary truss-type stiffening girders, because it uses vertical members only and not diagonal or other protruding members, the interval between vertical members is in comparison large. Such a structural feature will contribute to a decrease in the aerodynamic solidity ratio of the truss, ensuring higher aerodynamic stability.

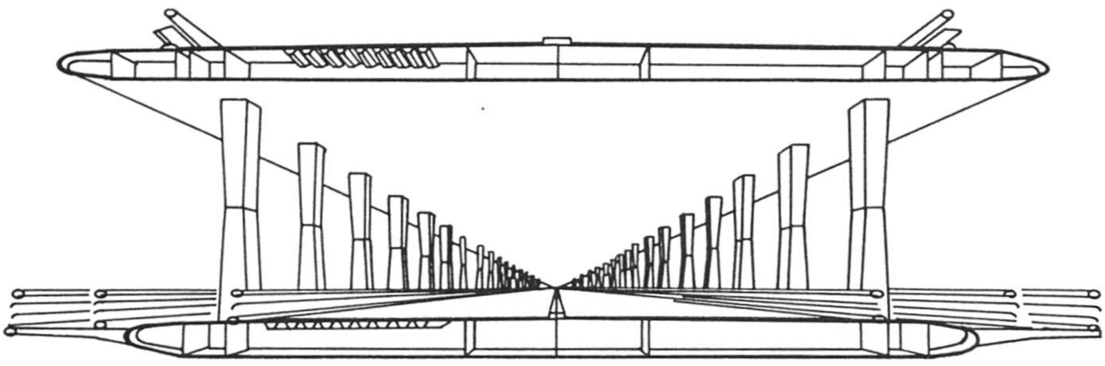


Fig. 6 Front view

Vehicles running on the lower deck are assured of good side view. Diagonal members tend to obstruct the view far more than only vertical members which are widely spaced and pass through the view instantaneously. The Vierendeel-type without diagonal members is free from such defects. This design also provides a smooth back surface of the upper deck over vehicles on the lower deck, which is free from unsightly structures seen with ordinary double-deck type cable-stayed bridges (Fig. 6).

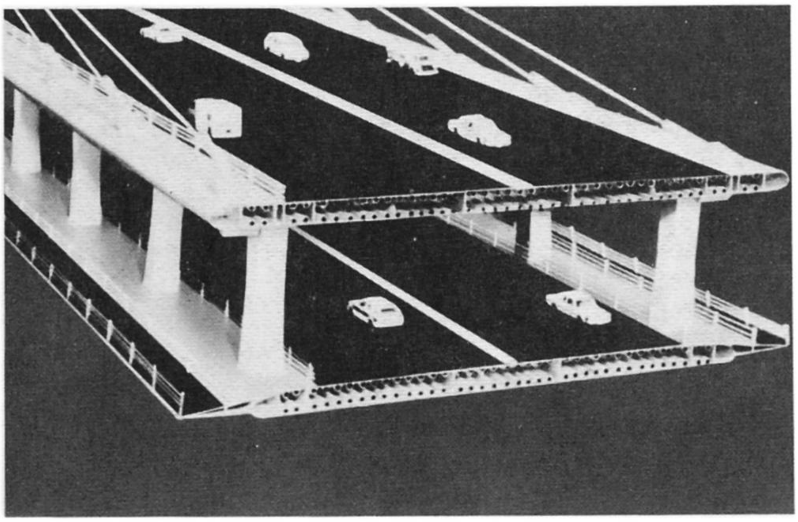


Photo 1 Partial model of stiffening girder of proposed design.

- In the composite construction, orthotropic plate steel decks are used fully as a part of the upper and lower chord members to provide the bridge body with increased lateral and torsional rigidities, and to give a simplified floor system for a lighter steel weight.

For these features, first of all the new construction mentioned in Chapter 2 will be outlined, and then, it will be compared with ordinary truss-type stiffening girders in terms of aerodynamic characteristics including the solidity ratio. Furthermore, a comparative study will be done on changes of the characteristics of a cable-stayed bridge due to the selection of the flexural rigidity of the stiffening truss. The paper will also give the result of a study on an effect which control of the stiffness of vertical members, would have on the flexural rigidity of the Vierendeel-type. The result of a proposed design for a cable-stayed bridge model of the new type also will be discussed.

3. CONSTRUCTION OF VIERENDEEL-TYPE GIRDER

Although the girder appears to be of simple construction, a very complicated structure is built inside the upper and lower deck box girders. The following description refers to the half part of the construction which is fully symmetrical with respect to the center line of the bridge.

- Fig. 7 shows a perspective view of the configuration, in which Scheibes a serving as two rows of longitudinal diaphragms at the upper and lower end connecting positions of vertical members V are provided along the whole length of the bridge. The Scheibes a and two web plates b to be composed of the vertical members, are structurally connected with the box-girder flange plate held between a and b. Actually, slits are cut in the flange plates to allow complete connection of Scheibes a and web plates b. They completely transmit to the upper and lower-deck box girders a bending moment that is transmitted from the vertical member and acts in the truss plane.

  In the upper-deck box girder two more Scheibes c are provided further outside Scheibes a. They take cable stresses from the anchoring girder g for the stay cable f.

  Another major longitudinal member installed in the box girder is a row of longitudinal diaphragms d to resist deformation of the cross section of the box girder itself. There are also small ribs to reinforce the flange plates i, and longitudinal and transverse ribs to constitute an orthotropic plate steel deck on which vehicles run.

- In Fig. 8, which shows a longitudinal section, two rows of diaphragms e in the transverse direction are provided at connection positions of two flange plates h of the vertical member. They form a floor beam F like a box girder together with the flange plate i of the deck box girder. The floor beams F are subjected to loads acting on the deck with the longitudinal and transverse ribs constituting the orthotropic plate steel deck. Besides, the floor beams F in the crossing section built inside the upper and lower box girders serve as inner floor beams and form, with the vertical members V on the right and left sides of the bridge, a rectangular rigid frame, thus resisting deformation of the cross section of the whole stiffening girder.

- A vertical member with a rectangular section is illustrated in Fig. 4-8, but does not necessarily have to be rectangular and may be circular in the cross section. In the latter case, connection of the member with box girders can be achieved by its penetrating panel zone formed by the intersection of the longitudinal diaphragms a with the transverse diaphragms e, in which case the bending moments in the direction of the two orthogonal axes transmitted from the vertical member respectively transmitted to the inner box girders consisting of longitudinal diaphragms a and to those consisting of transverse diaphragms e. (See Fig. 9)

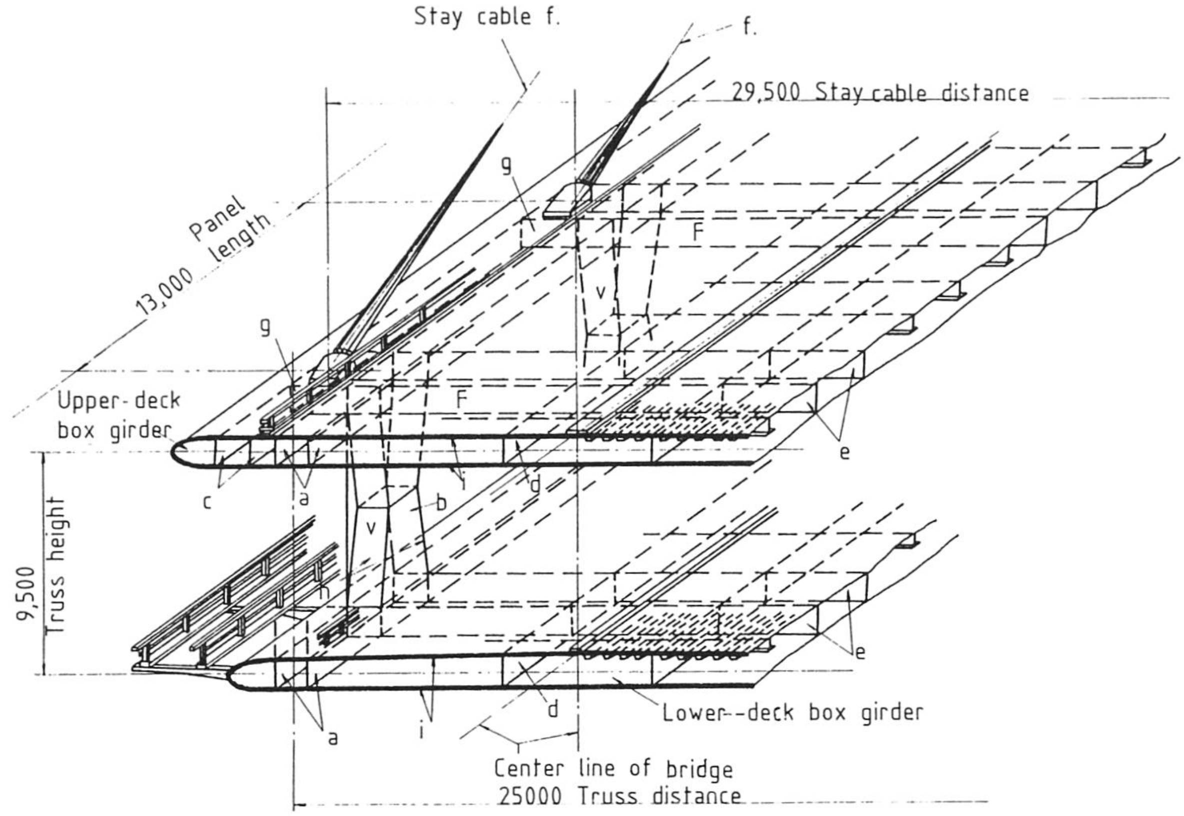


Fig. 7 Construction of Vierendeel-type girder (unit in mm)

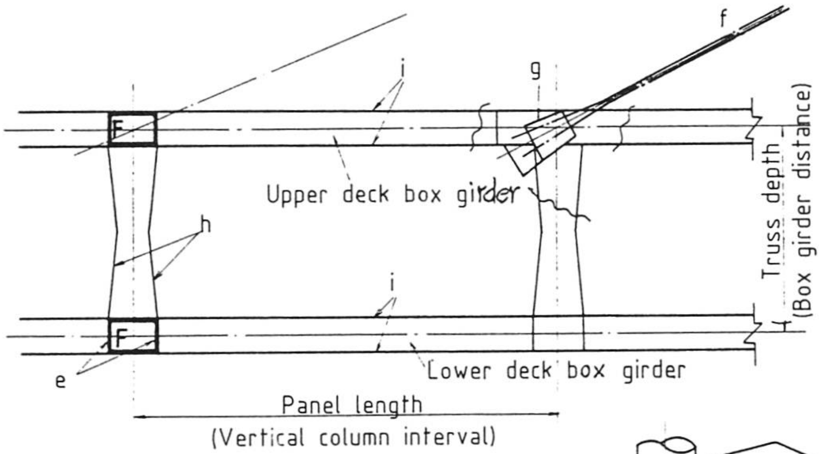


Fig. 8

Longitudinal section of stiffening girder

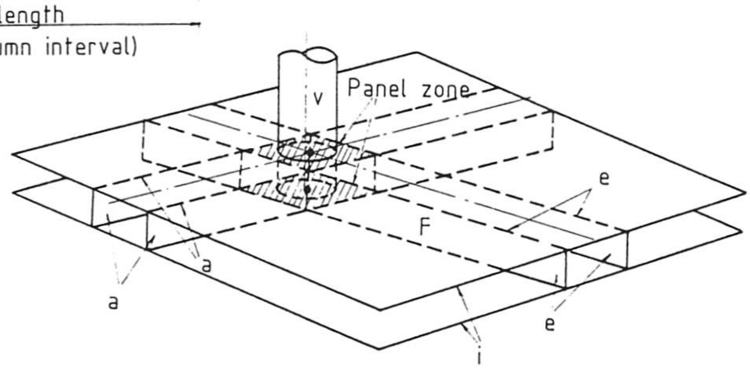


Fig. 9

Vertical column with circular section

In place of a column with a circular section a vertical member with an elliptical section can be employed when there are aerodynamic requirements.

- As seen in Fig. 7, no strength member is used outside the longitudinal diaphragm c of the upper-deck box girder and the longitudinal diaphragm a of the lower deck box girder, but wind noses are provided to improve aerodynamic characteristics. Furthermore, a sidewalk bracket can be projected from the nose.

4. WIND TUNNEL TEST OF SECTION MODELS

4.1 Introduction

For a long-span cable-stayed bridge, aerodynamic characteristics of its stiffening girder are naturally very important. Diagonal and vertical members used for a web member of a conventional truss-type stiffening girder, and gusset plates and a cable connecting device at a panel point of the truss, produce a considerably large value of the solidity ratio of the exposed area of the stiffening girder's web. Because of this, a self-excited oscillation due to a negative damping effect, which often causes trouble with a truss-type stiffening girder, easily occurs. Besides, due to the large solidity ratio and the double deck design, the stiffening girder assumes a shape like a box girder with a full-web. The stiffening girder is also disadvantageous regarding vortex excited oscillation. In contrast with this, the new type does not use any diagonal member or gusset plate, and the cable connecting devices, etc., are not so exposed. This makes possible a small solidity ratio contributing to improved aerodynamic performance.

A wind tunnel test was carried out using two-dimensional section models for two types of stiffening girders, namely, a conventional truss-type stiffening girder and a new Vierendeel-type stiffening girder. A discussion centered on a comparison of aerodynamic properties of these two types of design follows here.[1)]

4.2 Test cases

Test cases are listed in Table 1.

Table 1 Test cases

Measured angle of attack for Strouhal No.	Angle of attack in spring support test	Measured wind speed in 3 component forces (m/sec)
	-5°	
0°	-2.5°	7
	0°	
5°*	2.5°	14
	5°	

* For the Vierendeel type only.

4.3 Test models and dimensions of the model

The model is shown; Vierendeel-type in Figs. 10 and 11, truss-type in Figs. 12 and 13, and Vierendeel-type only in Photo 2. The model dimensions are listed in Table 2.

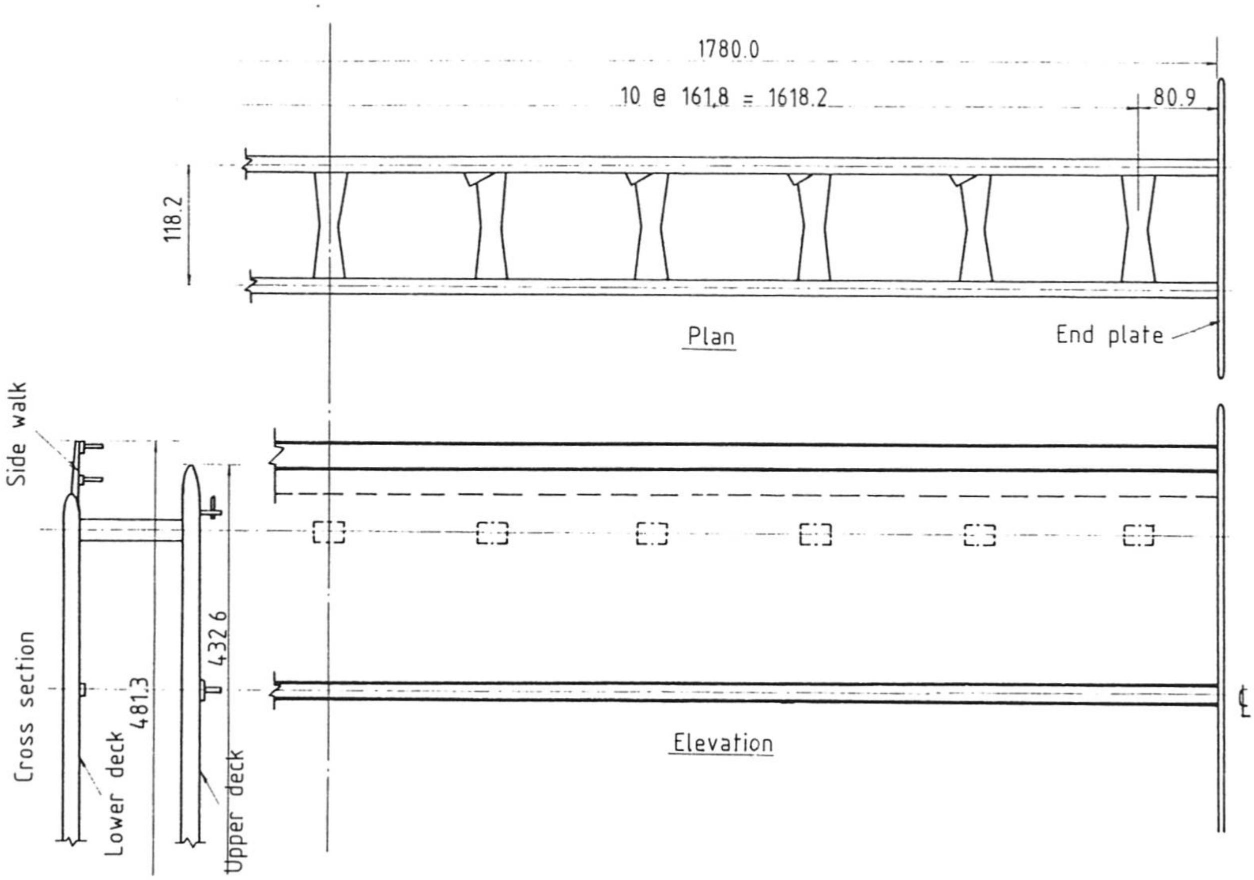
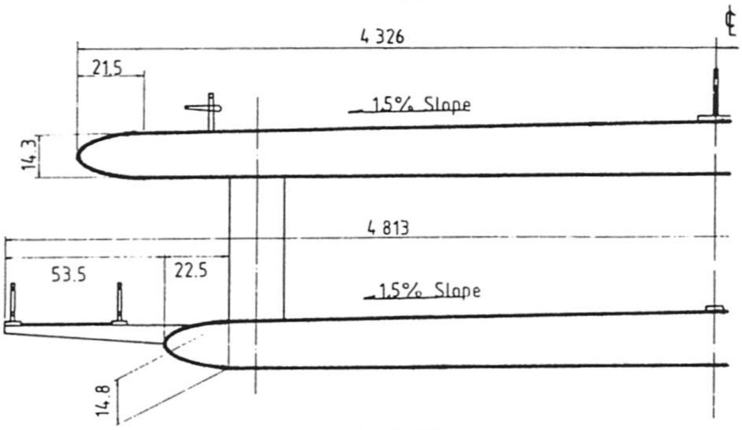


Fig. 10 General view of section model of Vierendeel-type (unit in mm)

Fig. 11 Cross section of section model of Vierendeel-type (unit in mm)



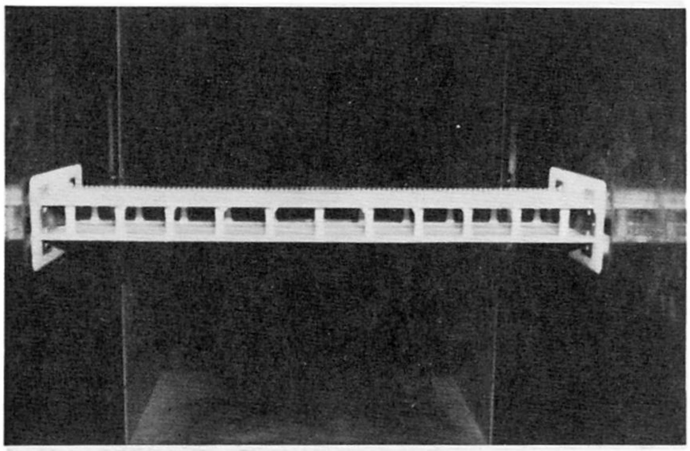


Photo 2 Two-dimensional section model in wind tunnel

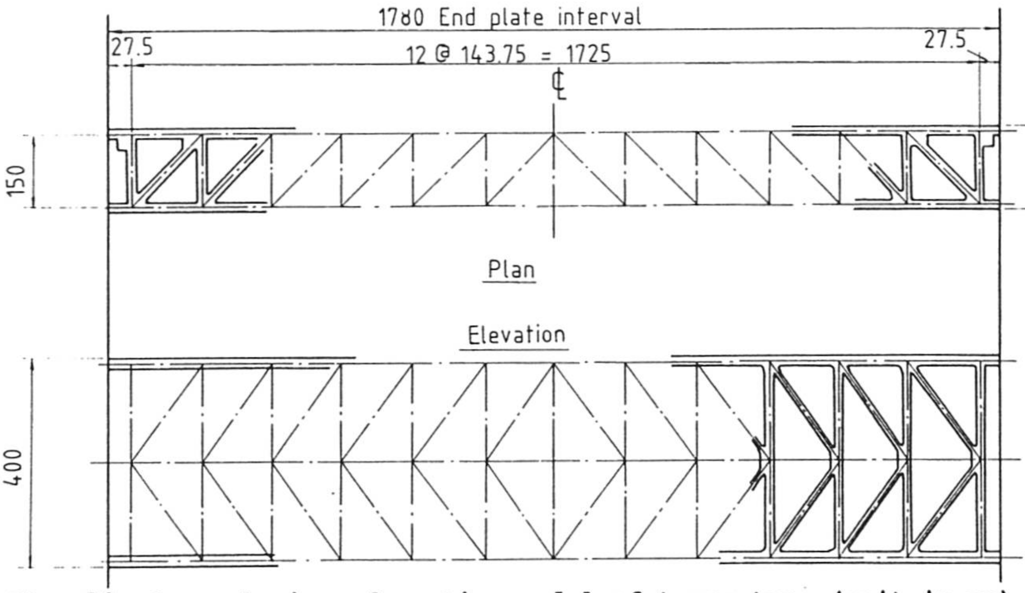


Fig. 12 General view of section model of truss-type (unit in mm)

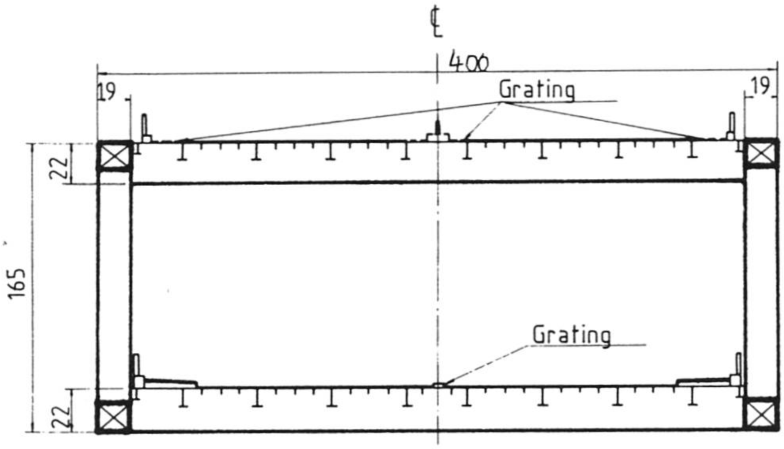


Fig. 13 Cross section of section model of truss-type (unit in mm)

4.4 Method of measurement and test apparatus

Only Photos 3 and 4 are shown, all others being omitted.

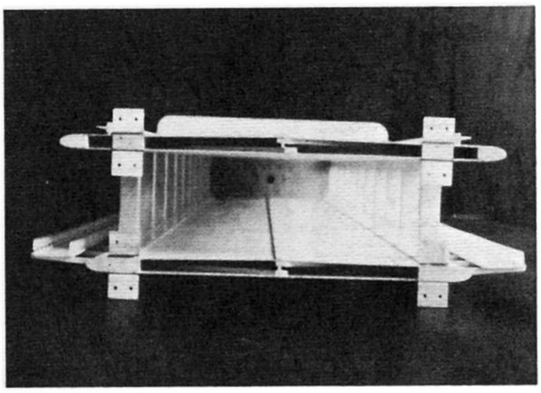


Photo 3 Cross section of section model of V.-type

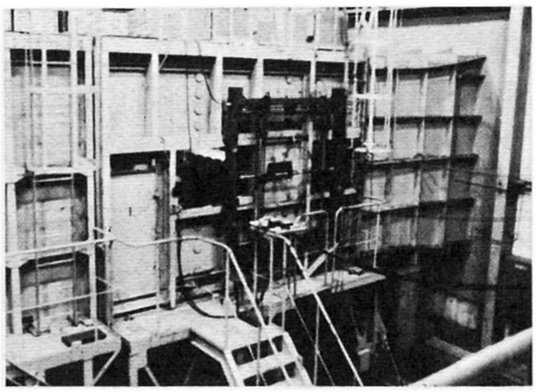


Photo 4 Overall view of spring support test

Table 2 Dimensions of Spring-Mounted Section Models

	In full-scale terms	Value required for model	Actual value (Reproduction of required value)
Reduced scale		1/80.34	1/80.34
		1/80	1/80
Weight	39.02 t/m	6.064 kg/m	6.669
	36.31	5.673	8.381
Polar inertia	502.2 $tm\ sec^2/m$	1205.5 $g.cm\ sec^2/m$	1504.5
	568.8	1388.7	1336.1
Frequency of vertical oscillation	0.380 Hz	(2.65) Hz	2.68
	0.514	(3.06)	3.356
Frequency of torsional oscillation	1.075 Hz	(7.50) Hz	6.92
	0.942	(5.61)	5.724
Frequency ratio	2.83	2.83	2.58
	1.832	1.832	1.71
Logarithmic decrement	—	0.03	Vertical 0.02 ~ 0.04 Torsional 0.03 ~ 0.035
	—	0.02 ~ 0.04	Vertical 0.054 Torsional 0.020
Spring support interval	—	(79.5) cm	97.5
	—	(60.8)	60.3
Ratio in terms of wind speed	—	11.52	Vertical 11.4 Torsional 12.5
	—	13.43	Vertical 12.25 Torsional 13.16

Note: The values in the upper column of each item are for the Vierendeel-type, and those in the lower column for the truss-type.

The values in parenthesis are target values set for the test plan.

The weight values include cable weight.

4-5 Test results and discussion

The space limitation allows to show only typical results of the tests, but the description will be made in reference to the results of other tests not mentioned here. The Vierendeel-type model is stable against a torsional oscillation until a wind speed in full scale of about 100 m/sec is reached. Therefore, no V-A curve (wind speed - response amplitude curve) is shown here.

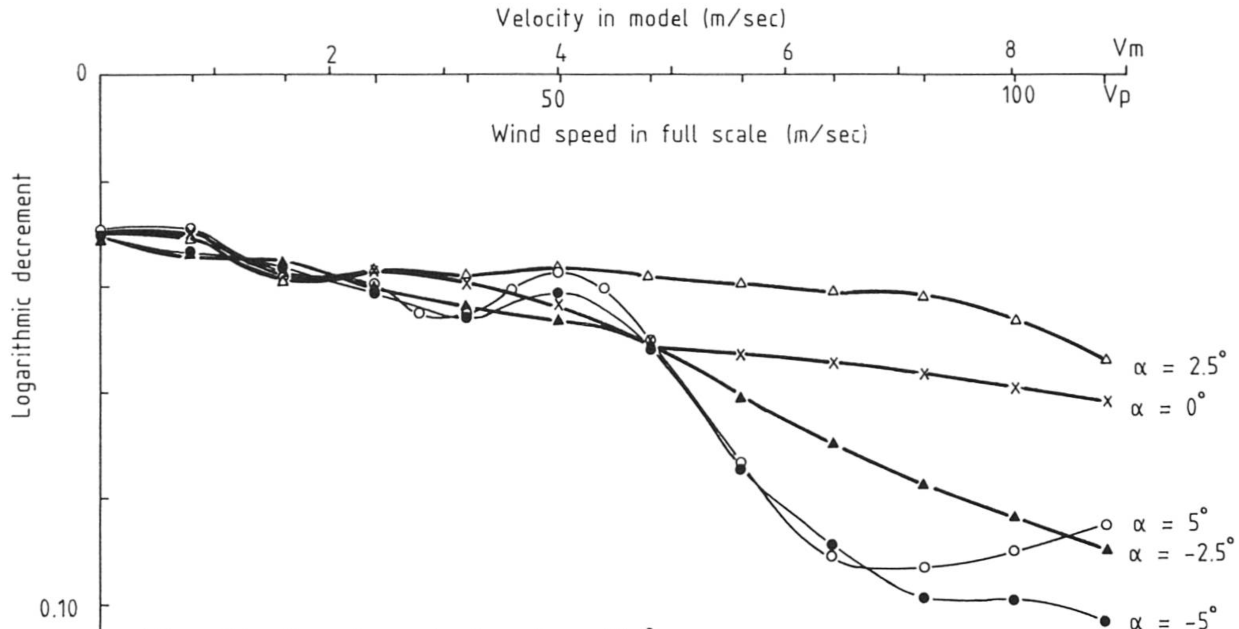


Fig. 14 Aerodynamic damping, V-δ curves
(at double amplitude of torsional oscillation $2\phi = 2°$)
α : angle of attack

Fig. 14 shows V-δ (wind speed - logarithmic decrement) curves at the torsional double amplitude of 2° for the angles of attack (α) of -5°, -25°, 0°, 2.5° and 5°. In the figures, V_m denotes the wind-tunnel wind speed, and V_p the wind speed in full scale. The decrement generally increases with a rise of the wind speed, but at α=±5° it temporarily falls in the low wind speed region, unlike other angles of attack. This appears to be attributable to the vortex, but the wind speed giving the lowest decrement (about 4 m/sec in terms of the wind-tunnel wind speed and about 50 m/sec in the full-scale wind speed) is slightly lower than the value of about 4.75 m/sec (the wind-tunnel wind speed), at which the frequency of a vortex oscillation comes into agreement with that of the

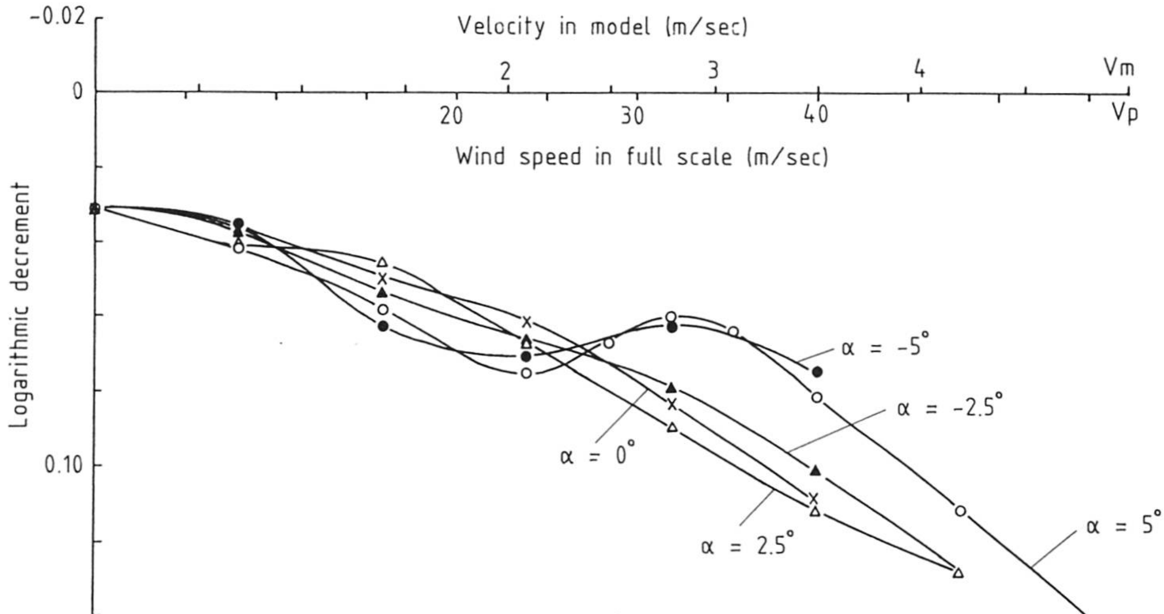


Fig. 15 Aerodynamic damping V-δ curves
(at double amplitude of vertical oscillation $2\eta \doteqdot 15mm$)
α : angle of attack

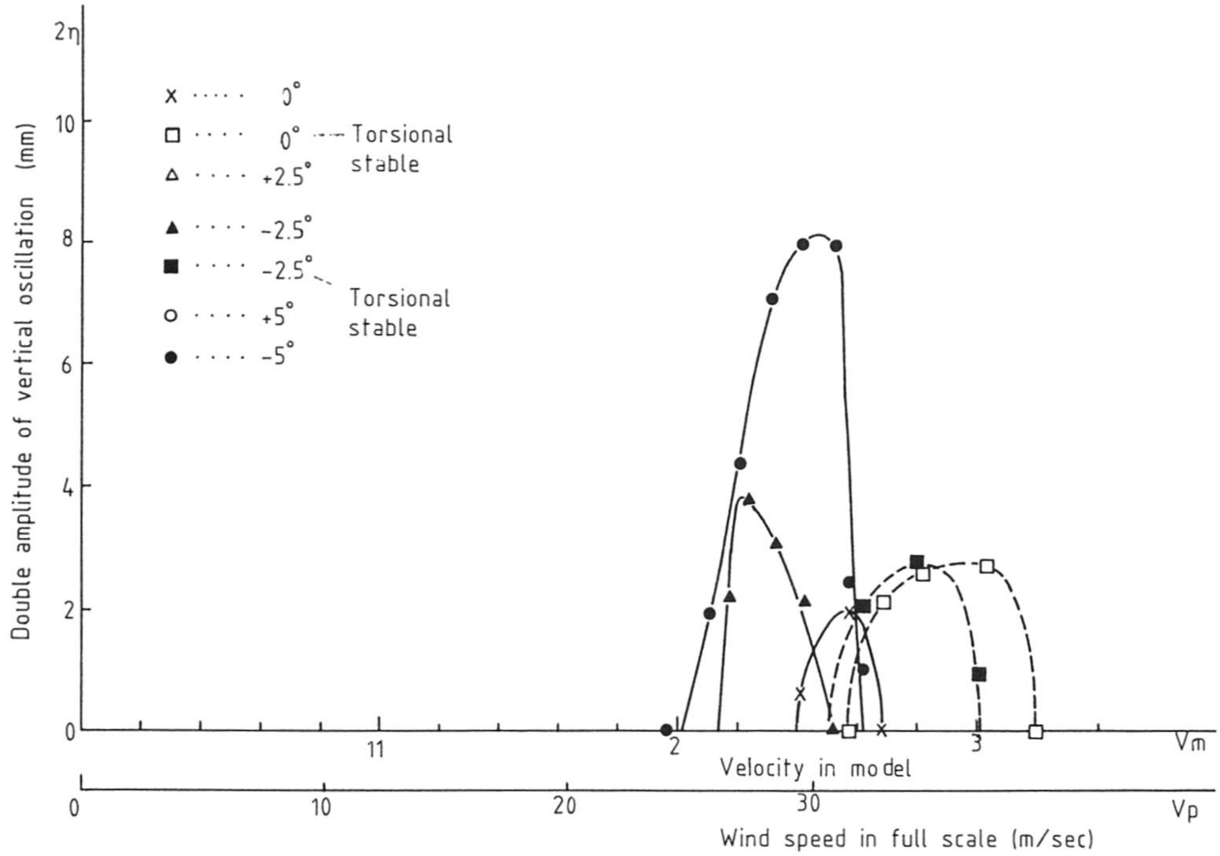


Fig. 16 Amplitude of vertical oscillation V-A curves

natural torsional oscillation, which was obtained in a back-flow vortex frequency experiment previously performed.

Fig. 15 shows V-δ curves for a vertical oscillation for the angles of attack (α) of -5°, -2.5°, 0°, 2.5° and 5°. It is noted that in the case of the torsional oscillation, at α=±5° and only at this angle of attack the decrement temporarily falls near the wind-tunnel wind speed slightly less than 3 m/sec (slightly more than 30 m/sec in the full scale wind speed). The decrement begins to fall at a wind speed, where the frequency of a vortex oscillation agrees with that of the natural vertical oscillation (slight less than 2 m/sec in terms of the wind-tunnel wind speed), and assumes the minimum value around the wind-tunnel wind speed slightly less than 3 m/sec, and then begins to rise in the high speed region. This is also considered to be attributed to a vortex, as seen in the case of torsional oscillation.

V-A curves for the truss-type model are shown in Fig. 16. The amplitude increases with an increase in an absolute value of the angle of attack in the region of negative angles of attack, but no oscillation occurs in the positive angle of attack region. In Fig. 17, almost no effect of the angle of attack at high wind speeds is observed with the V-δ curves for a vertical oscillation. Fig. 18 shows V-δ curves for a torsional oscillation.

These tests have proved that the Vierendeel-type has high stability against wind. That is:

- It does not permit generation of a flutter (a torsional or bending-torsional oscillation) until a wind speed in the full scale of about 110 m/sec is reached.

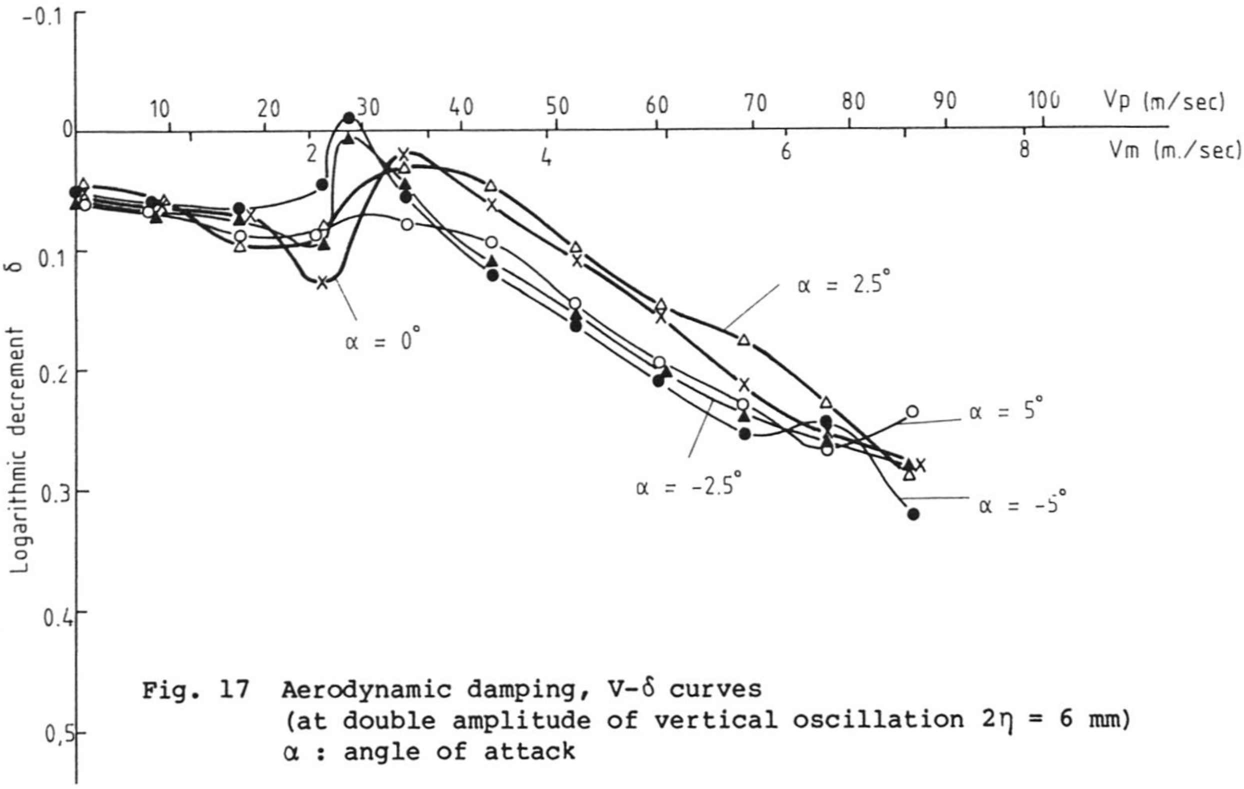


Fig. 17 Aerodynamic damping, V-δ curves
(at double amplitude of vertical oscillation $2\eta = 6$ mm)
α : angle of attack

The torsional decrement temporarily falls near a wind speed in the full scale of 50 m/sec when the angle of attack is ±5°, but no vortex excited oscillation is caused.

The vertical decrement temporarily falls near a wind speed in the full scale slightly more than 30 m/sec when the angle of attack is ±5°, but no vortex excited oscillation is caused.

The coefficient of drag at the angle of attack of 0° is 1.396, and the three components of force characteristics resemble those shown by a flat rectangular section.

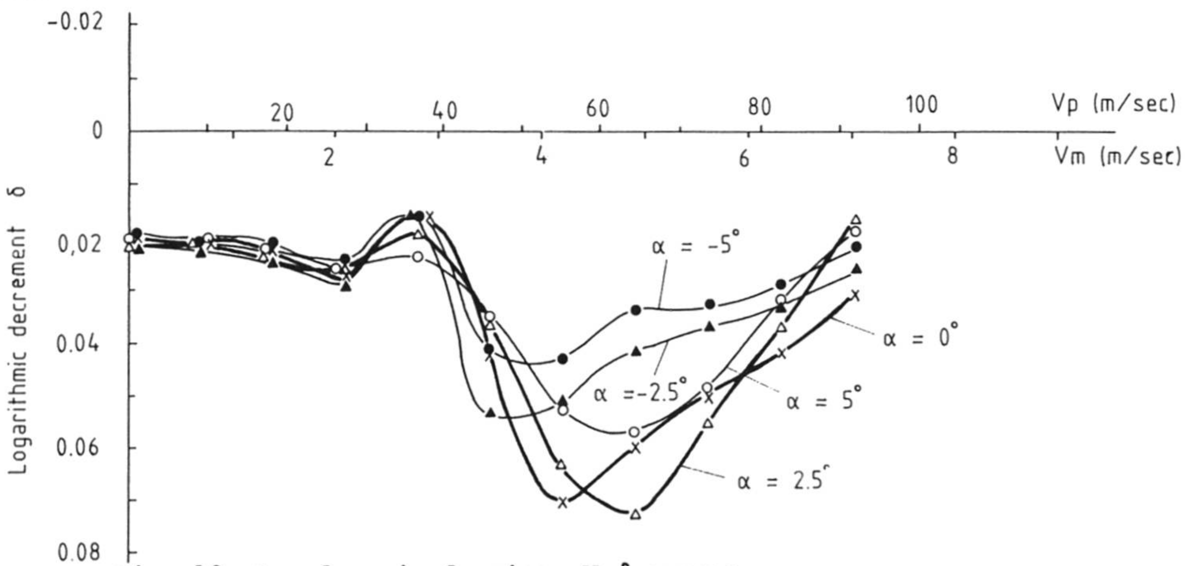


Fig. 18 Aerodynamic damping, V-δ curves
(at double amplitude of torsional oscillation $2\phi = 2°$)
α : angle of attack

In contrast to this, the following performances of the truss-type were observed, as the solidity ratio of the truss girder was larger:

- A vertical vortex excited oscillation is generated under a wind speed slightly more than 30 m/sec when the angle of attack is 0°. The amplitude increases when the angle of attack is negative, and no oscillation occurs when it is positive.

- In the experiments, a vortex excited oscillation of torsion occurred.

- In the high speed region, signs of occurrence of self-excited oscillation (stall flutter) with the degree of freedom of torsion of one are noted, but no oscillation occurs until a wind speed of about 90 m/sec is reached.

- Modified designs such as installation of a grating floor on the lower deck can prevent such oscillation.

- The coefficient of drag at the angle of attack of 0° is about 1.88.

Thus, in the truss-type oscillation prevention can be achieved by installing a grating floor on the lower deck, but drivers do not seem to like driving on the grating floor.

5. COMPARATIVE STUDY ON CHANGES OF CABLE-STAYED BRIDGE CHARACTERISTICS

It is needless to say that an optimum selection of the flexural rigidity of a cable-stayed bridge stiffening girder is important. The new type of stiffening girder will allow the free selection, and its detailed commentary will be given in this section.

First, let us consider characteristics generally encountered when the stiffness of a stiffening girder is changed.

The parameter concerned with rigidity of the member in question is defined as the ratio of the stiffening girder's flexural rigidity to the extensional rigidity of the cable, and rigidity of the main tower is assumed to be constant.

To express this, the following non-dimensional parameter, introduced in reference to a paper by Maeda, Hayashi, et al.[2] is used.

$$K = \frac{E_0 I_0}{\ell_0{}^2 E_C A_C}$$

where, $E_0 I_0$: The flexural rigidity of stiffening girder,
ℓ_0: The total girder length,
$E_C A_C$: The extensional rigidity of cable.

5.1 Member section and parameter K

Sections are assumed over the whole length of the members as follows:

- Stiffening girder

$A_0 = 0.240\ m^2.$

$I_0 = 8.5054\ m^4$

- Main tower

$A = 0.780\ m^2,$

$I = 7.250\ m^4$

- Cable

The sectional area of the cable is set to $A_C = 0.01893\ m^2$ (average).

- Parameter K

Several values of the parameter K concerning the flexural rigidity of a stiffening girder are set, and the moments of inertia of the stiffening girder corresponding to them are obtained. Changes centering around $I_0 = 8.5054\ m^4$ are provided as shown in Table 6.

Table 6

K	Stiffening girder moment of inertia $I(m^4)$
10^{-6}	0.014792
10^{-4}	1.4792
5.75×10^{-4}	8.5054
10^{-3}	11.4792
10^{-2}	114.792
1	1147.92

where, $\frac{E_0}{E_c} \doteqdot 1$, $\ell_0 = 884$ m, $A_C = 0.1893\ m^2$

$$I_0 = 0.01893 \times 884^2 \times K = 1.4792 \times 10^4 \times K.$$

5.2 Load

For loading, the following two cases are assumed. As the load, a unit uniform load w of 1.0 t/m is employed.

i) Loaded over the full span. ii) Loaded on the center span.

5.3 Basic dimensions of structural system (Fig. 19)

5.4 Results of calculation

The typical results of calculations are shown in Figs. 19 to 22.

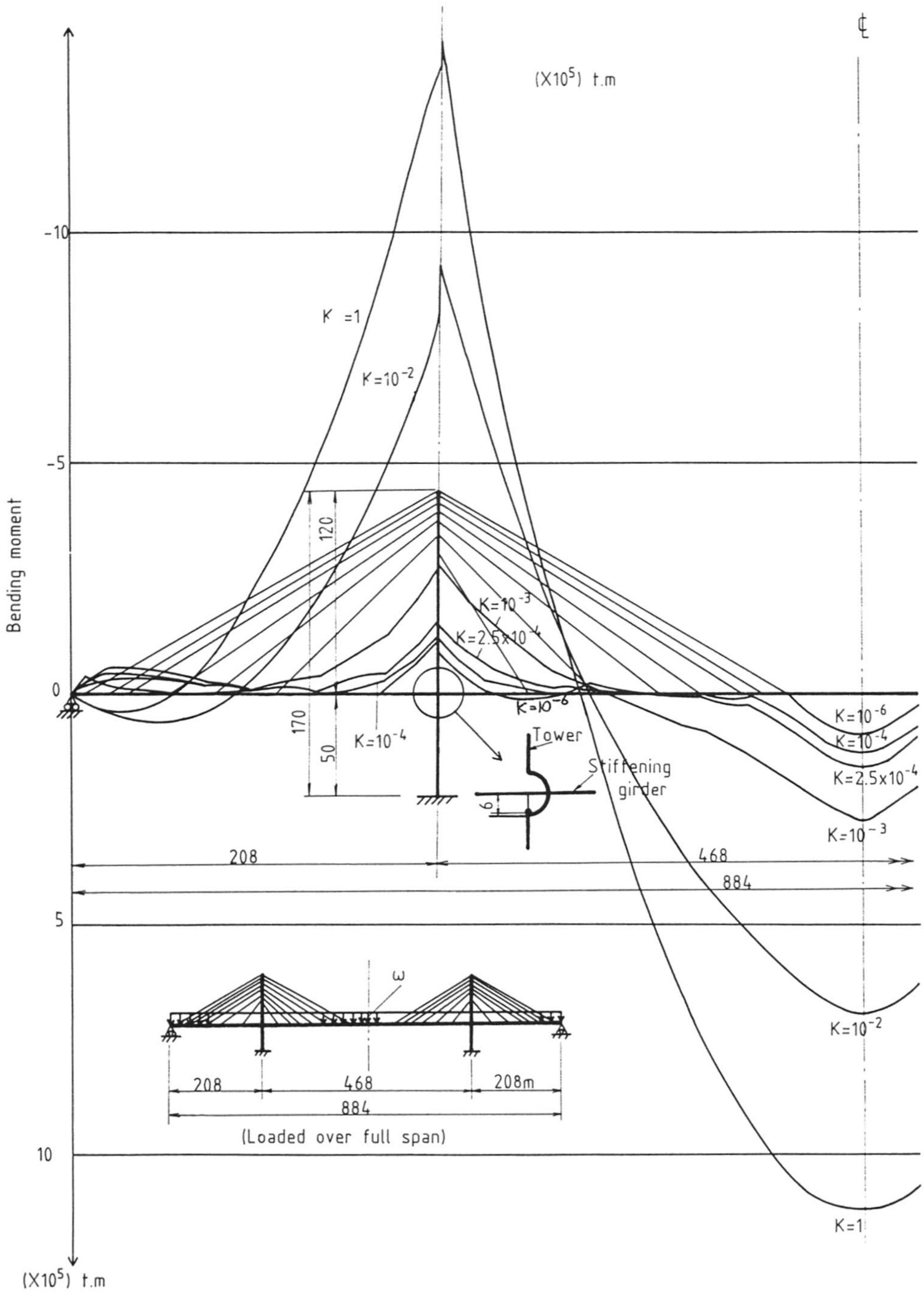


Fig. 19 Basic dimension of structural system & stiffening girder bending moment (unit in m)

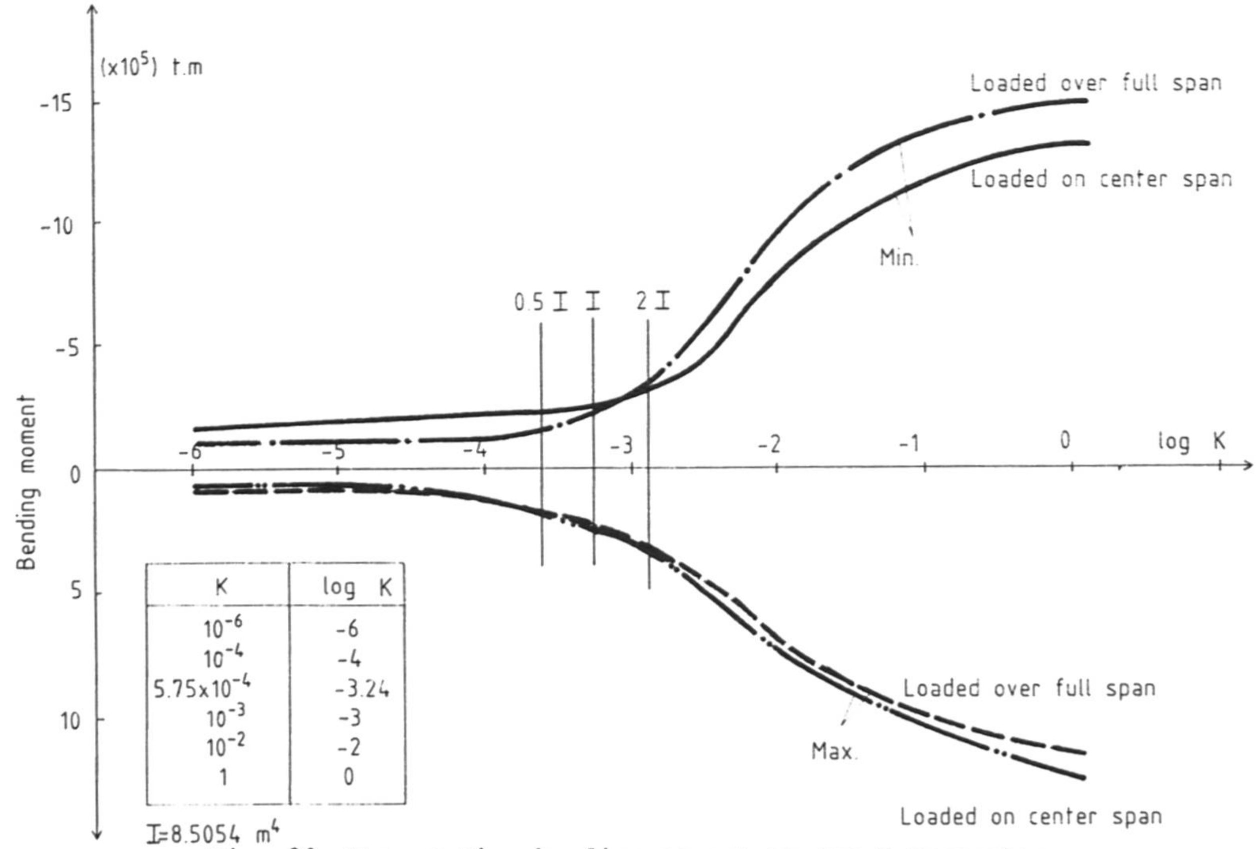


Fig. 20 Max. & Min. bending moment versus K parameter

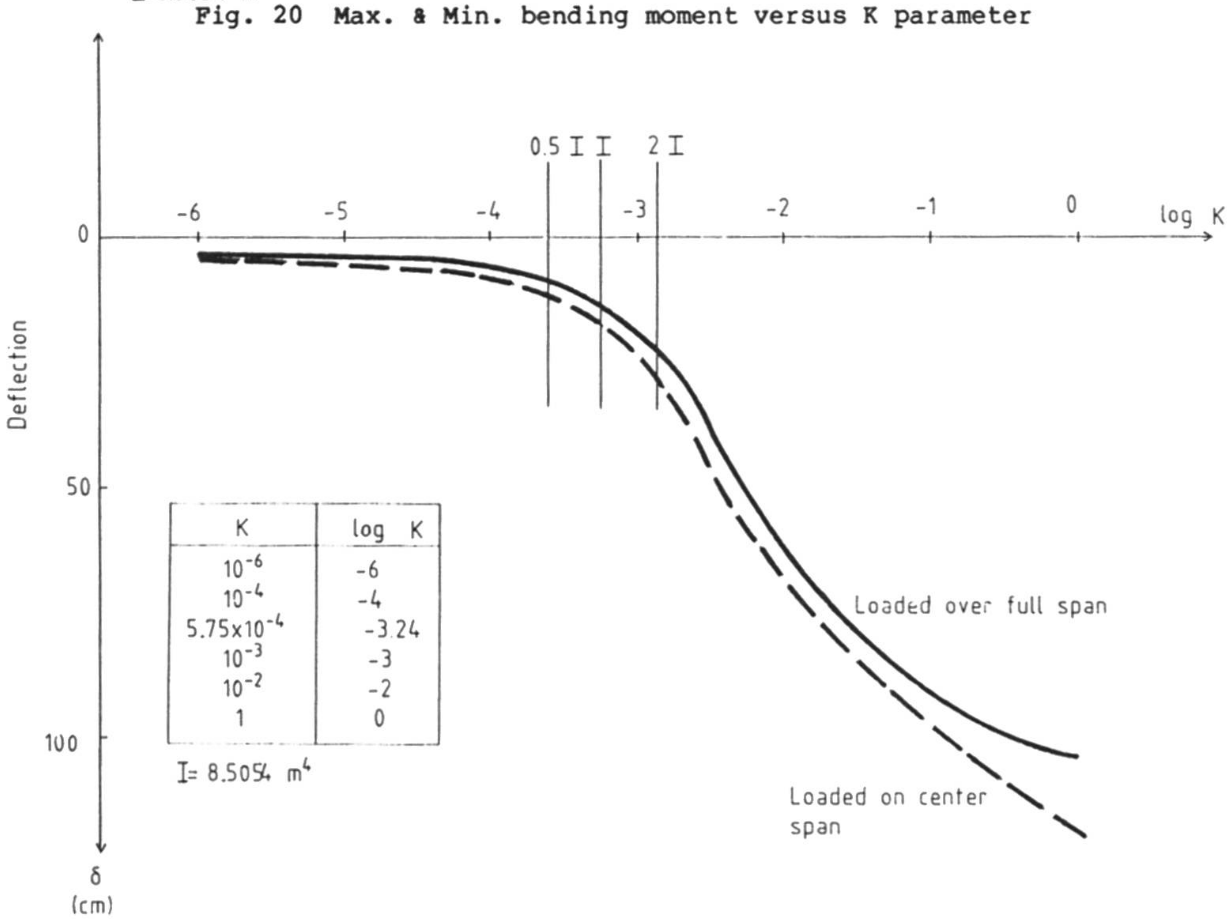


Fig. 21 Deflection versus K parameter curve in the middle of center span

In Figs. 20 to 22, K values corresponding to the values of I, 0.5I, 2I of a stiffening girder for the calculation model are plotted. Such diagrams are convenient, since changes in the bending moment and other factors corresponding to the changes of I can be grasped. Though this structural system is within the range which is little affected by I, it is definitely preferable to choose a smaller value of I.

5.5 Discussion

Since this chapter aims at studying the effects of flexural rigidity on the statical elastic behavior of a cable-stayed bridge, the flexural rigidity ratio parameter K, is changed to limit values, without consideration of the material strength and buckling strength of structural members.

It is a general characteristic of a cable-stayed bridge that the effects of flexural rigidity of a stiffening girder is conspicuous in each member. It is clearly seen in the above figures in which the results of calculation are shown.

Fig. 19 shows the bending moment of a stiffening girder, over the range of K = 10^{-6} to K = 1, when the uniform load, w 1.0 t/m, is applied over the full span. K value of one (K = 1) is a limit value reached when the moment of inertia of the stiffening girder becomes relatively large, in which case the bending moment close to that of a three-span continuous girder not stiffened with cables, occurs in the stiffening girder. K value of 10^{-6} (K = 10^{-6}) is a limit value when the flexural rigidity becomes relatively small, in which case the bending moment close to that of a multiple-span continuous girder fully supported at the cable anchor points occurs. K values of ordinary cable-stayed bridges are within a range of 10^{-3} to 10^{-4}, and a smaller bending moment occurs with a smaller value of K, that is, with a smaller value of I. This means that an optimum K value in terms of both the cable's extensional rigidity and the stiffening girder's flexural rigidity is within 10^{-3} to 10^{-4}. It is thus understood that the selection of an optimum stiffening-girder flexural rigidity is considerably advantageous to an economical design.

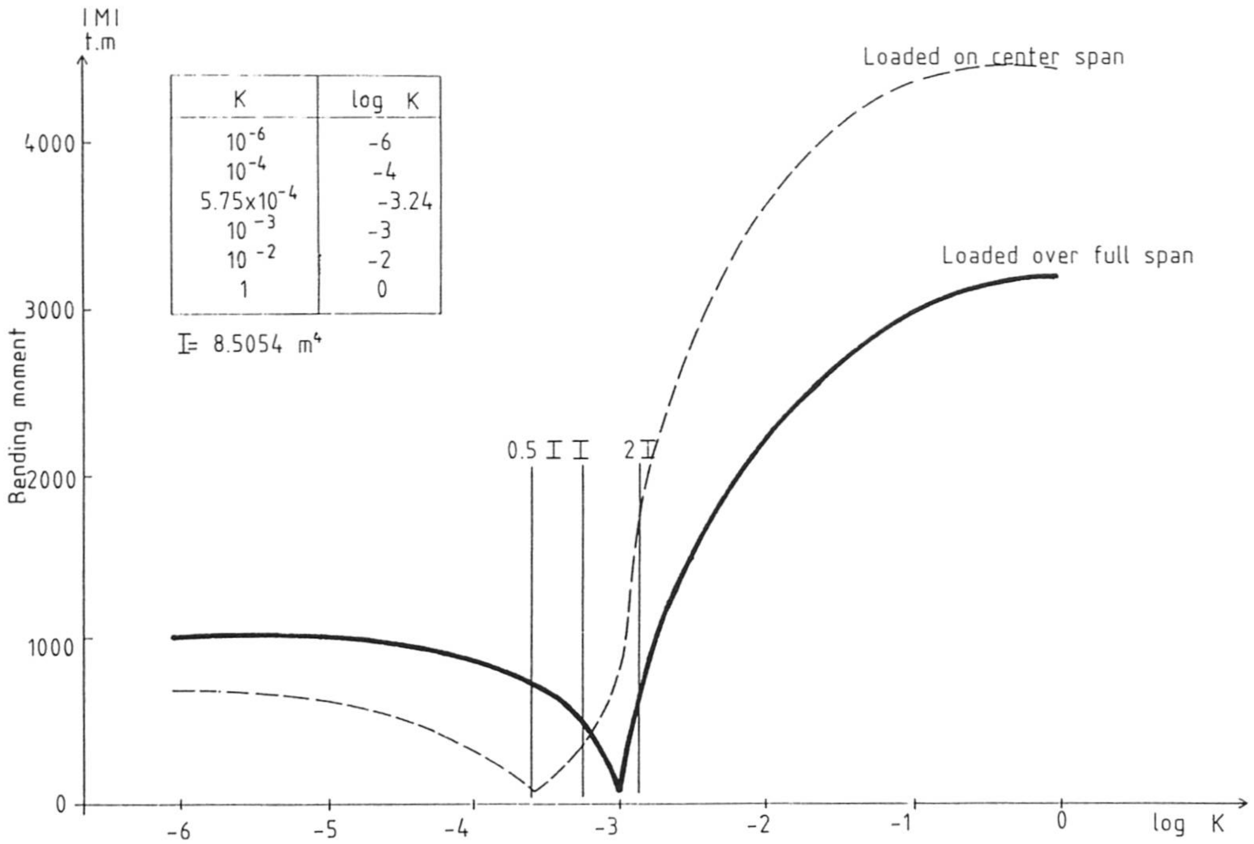
No bending moment diagram is given here for the case in which the load of w = 1.0 t/m is applied to the center span only, because it is similar to the one mentioned above.

Figs. 20 and 21 show the maximum and minimum bending moments in the stiffening girder and deflection in the middle of the center span which are caused by the change in K. It is clear from the figures that whether or not the choice of K or I value is suitable is considerably influential, causing changes in the values to be obtained. This is also true with Fig. 22.

In the above discussion no consideration was given to the stay-cable's pre-stress.

6. COMPARATIVE STUDY ON STIFFNESS CHANGE OF VERTICAL MEMBERS

Here is discussed effects of the stiffness of vertical members of a Vierendeel-type girder in the flexural rigidity of the stiffening girder. It was discussed in detail in Chapter 5, how important a wide range of selection in the stiffening girder's flexural rigidity of a cable-stayed bridge is for an economical design.



K	log K
10^{-6}	-6
10^{-4}	-4
5.75×10^{-4}	-3.24
10^{-3}	-3
10^{-2}	-2
1	0

Fig. 22 K parameter versus bending moment at tower base

Also, as stated in Chapter 2, the Vierendeel-type is very convenient for a free selection of the flexural rigidity of a stiffening girder in a cable-stayed bridge of the double deck type. The following discussions will center on this point.

It can be assumed that dimensions of the box girders to serve as the upper and lower chord members in a Vierendeel-type girder are determined by various structural conditions, economy, and other factors. Then, the flexural rigidity of a Vierendeel-type girder can be controlled only by the size of the vertical members.

Therefore, a comparative study on changes of the flexural rigidity of a Vierendeel girder in terms of the moment of inertia I_v in the girder plane of the vertical members, will be useful.

6.1 Bending stiffness of Vierendeel-type girder

In general, the moment of inertia of a beam taken so that the same load as that applied to a structural system may have a beam deflection equal to that of the structural system, is called the "equivalent moment of inertia (i_e) for the beam of the said structural system."

Now the moment of inertia and the sectional area of upper and lower chord members of a Vierendeel-type girder are denoted by I_o and A_o, respectively. The equivalent moment of inertia I_e of the Vierendeel-type girder is defined as follows:

$$I_e = 2\ I_o + 2(\alpha A_o) \cdot (\frac{H}{2})^2$$

$$= 2\ I_o + \frac{1}{2}\alpha A_o H^2,$$

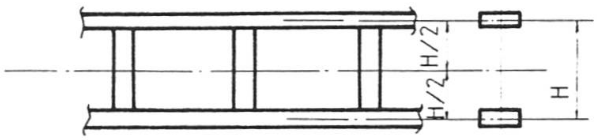


Fig. 23 Vierendeel-girder

where, α is called "the equivalent section coefficient." This formula enables us to study effects of the vertical member chord member stiffness ratio I_v/I_o, the differences between types of structural systems and the load conditions, on the equivalent moment of inertia I_e, namely the equivalent section coefficient α, as shown below.

Two types of structural systems were used in the study: one with both ends supported simply and the other with one end fixed and the other end free. Two loaded conditions were considered: consisting of a concentrated load and full uniform loading in the middle of the span. The combination of these types gave four cases, and differences among them were studied. Since the differences were not so large, the results for only a typical case are mentioned here, with all other omitted.

6.2 Correlation between I_e and I_v

- I_e values were obtained when the sectional area A_v of a vertical member was changed only by 0.19 to 0.030% with changes in A_v.

- The change in I_e, namely in α, was studied when I_v was changed with A_v kept constant. The change of α corresponding to that of I_v is shown in Fig. 26, where non-dimensionalization is effected by introducing I_v/I_o. Fig. 26 shows the value of β against I_v/I_o with the definition of $\beta = \frac{2I_o}{I_e} \times 100$ (%).

6.3 Discussion

- When I_v is constant, I_e changes only with a maximum of 0.3% if A_v is changed from $0.1A_o$ to A_o. This means that A_v has only a very slight effect on I_e.

- When I_v is changed from 1/8 I_o to 1.0 I_o with the constant value of A_v, I_e becomes larger with an increase in I_v. In detail, I_e can be obtained from α in Fig. 24, but α decreases quickly if I_v becomes smaller against I_o. However, if I_v is decreased, the value of I_v as small as the limit of formation of a laminated beam ($I_e \rightarrow 2I_o$) can be obtained.

- Fig. 24 shows the ratio β of the moment of inertial I_o of the upper and lower chord members themselves to I_e. It is seen that I_e does not increase above a certain level with an increase in I_v (β = 5.7%).

It is also noted that a decrease in I_v causes a sudden drop of I_e. That is, the fact that I_e becomes smaller with a decrease in I_v, is considered to enlarge a possible range of selection of the flexural rigidity of a Vierendeel-type girder.

- With the Vierendeel-type stiffening girder, using flat, wide box girders as the upper and lower chord members as stated in Chapter 2, large chord members will have a large moment of inertia I_o, and relatively small I_v/I_o values. This means that the above-mentioned effect will greatly increase.

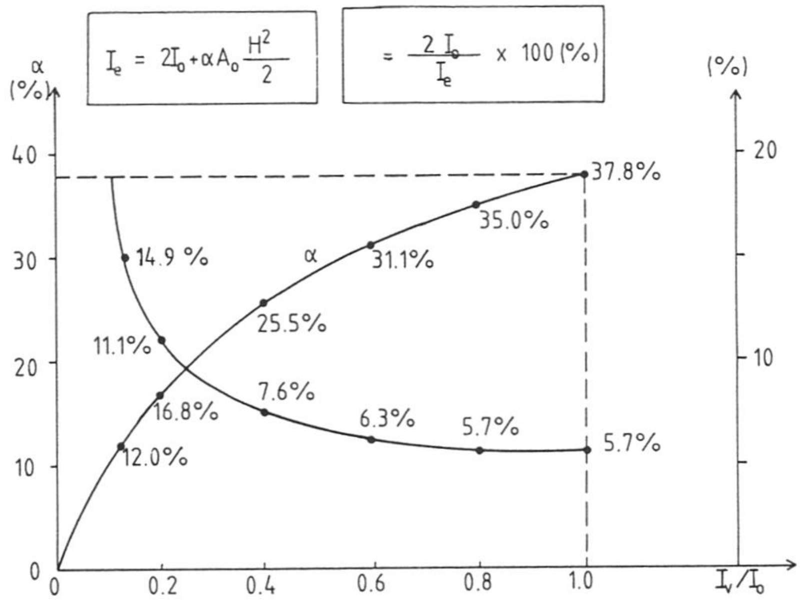


Fig. 24 Correlation between I_e and I_v

7. PROPOSED DESIGN

7.1 Design conditions

A trial design was carried out for a cable-stayed bridge having a center span of 468 m and a side span of 208 m. These values are close to the dimensions of a bridge currently planned in Japan. The required depth of the stiffening truss calculated from overhead clearance requirements for vehicles running on the lower deck is 9.5 m, which is the central distance between the upper and lower box girders.

The bridge has a road width as shown in Fig. 4; the total road width is 48 m and the live load is assumed to be T-20 or L-20 in Japan, which is respectively 20 tons truck load or 350 kg/m² uniform lane load plus a 5 t/m knife edge load, specified at the Standard Highway Bridge Design Specifications in Japan. The intensity of the total load is 37.1 t/m.

7.2 General view and cross section of the bridge at proposed design

A general view and cross section of the bridge at the proposed design, and a cross section of the stiffening girder are shown in figs. 2 to 4. Anchorage of the stay cable is shown in Fig. 5.

7.3 Member stresses of Vierendeel-type girder

A uniform load (10 t/m) is applied to structural systems which are divided into 2 systems; just before closing in the middle of the center span, and after completion.

Fig. 25 shows the case in which a load is applied over the full span, as an example to illustrate the member stresses of Vierendeel-type girder. Typical member stresses are shown in Fig. 25. Panel point 1 denotes the end support, and panel point 35 denotes the middle point of the center span. The symbol ↻ denotes the direction in which a bending moment acts.

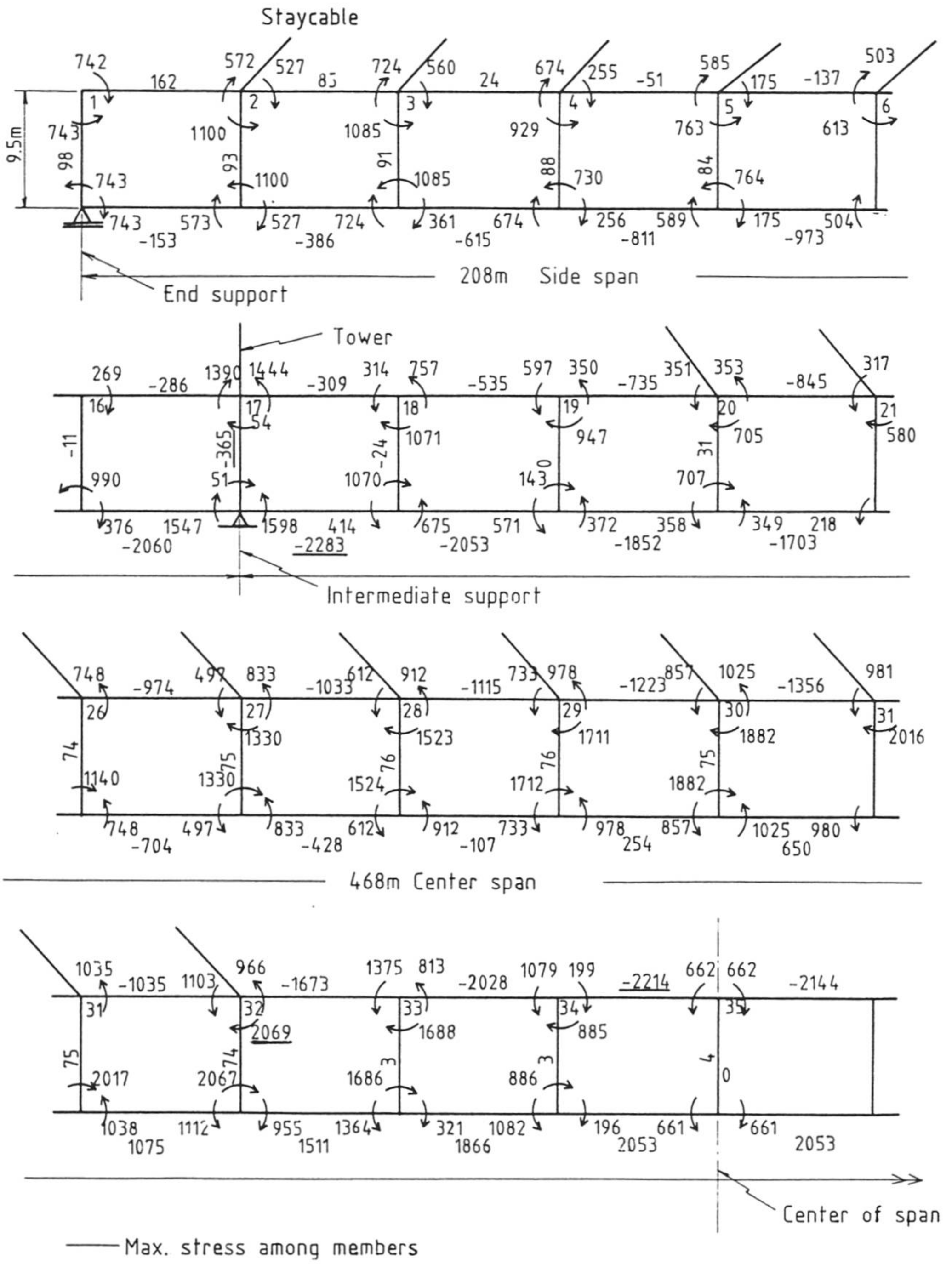


Fig. 25 Members stress of Vierendeel-girder member (Unit in t for axial stresses and t-m for bending moments)

The upper and lower box girders, serving as the upper and lower chord members of the Vierendeel-type girder, have the maximum axial stresses of -2,214 t and -2,283 t, respectively, and the maximum bending moments are 1,444 t-m and 1,598 t-m respectively. These stresses can be easily resisted by the upper and lower box girders with a sufficiently large width, while the maximum bending moment and axial stress in the vertical member are 2,069 t-m and -365 t, respectively. The value of the maximum bending moment is not so large as the other vertical members. As stated above, it is needless to say that these member stresses can be adjusted by giving prestresses to the stay-cable so that the peak stresses can be levelled.

7.4 Weight of steel used in stiffening girder

A preliminary calculation of the weight of steel to be used in a stiffening girder of the proposed design gives a value of 20,300 t, showing ten to several percent decrease compared with an ordinary type of truss designed based on the similar conditions.

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