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Experiment on Non-prestressed Continuous Composite Beams

Expérience sur poutres composites continues et non-précontraintes

Versuche an schlaff bewehrten durchlaufenden Verbundbalken

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1. Introduction

In deck girder bridges with concrete slab, continuous girder bridges have many advantages such as economical about the steel weight, comfortable driving, high ultimate strength and seismic proof. In designing continuous composite girder bridges, prestressing in negative moment regions has usually been required in Japan. But prestressed continuous composite girder bridges have disadvantages of involving complex procedure in design and taking a long time in construction.

In order to get rid of such disadvantages, non-prestressed continuous composite girder bridge is proposed. The design concept is as follows. The composite section is effective to positive moment, while the slab concrete can not resist tensile stress for negative moment, therefore, only steel girder with bar reinforcement (Fig. 1) proves effective, in case an adequate number of shear connectors are provided.

This is similar in design to continuous composite girder in U.S.A. However, it differs from that of U.S.A. in that no consideration is taken of negative and positive moment regions.

Statical experiment has been carried out in order to ensure this design, and items investigated are as follows.

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*) The author died on February 10, 1969, at the age of 60 years. This paper, written in June 1968, represents Professor Tachibana's last contribution to his field of activity. Possible enquiries regarding the paper will be answered by T. Mukaiyama, Akashi Technical College, Akashi, Japan.

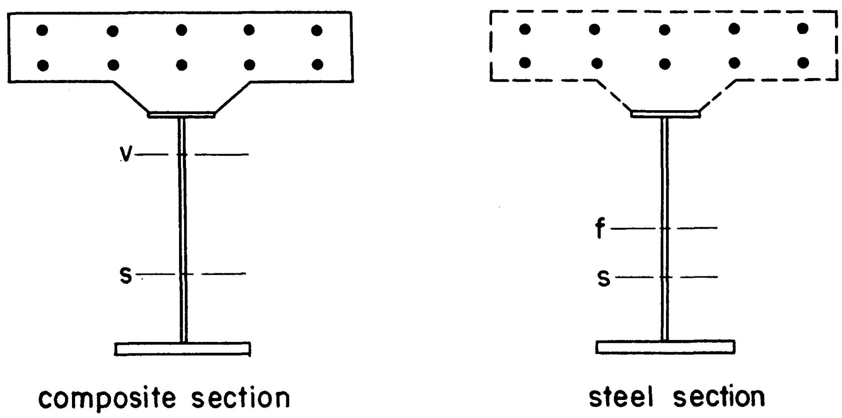

Fig. 1.

1. Relation between the width of slab cracks caused in negative moment regions and the amount of bar reinforcement, diameter of bar, and the ratio of perimeter of bars to gross slab concrete area.
2. Co-operation between longitudinal reinforcement and steel beam (including the study of effective width of reinforced concrete slab).
3. Composite action of composite beam with cracked slab for positive moment (including the study of value $n = E_s/E_c$ and effective width of composite beam with cracked slab).
4. Amount of shear connectors suitable for negative moment.
5. Confirmation of ultimate strength of non-prestressed continuous composite beams.

Concerning dynamical experiment, a series of similar model tests are now being made by our co-operators.

2. Test Specimens and Testing Procedure

Four sets of composite beam specimens are shown in Table 1, Fig. 2a and Fig. 2b, loading conditions beeing also indicated there.

2.1. Beam A

The amount of bar reinforcements and shear connectors differ from each other in beams A 1, A 2 and A 3. Beam A 2, A 4 and A 5 are not the same in the number of reinforcement, but nearly equal in the amount of reinforcement and shear connectors, of which design is made by the criteria in section 3.4.

2.2. Beam B

Beam B is designed in the same way as beam A 2, except the pitch of shear connectors, which are provided for positive moment. Beam B 1 is tested under positive moment during any test, while beam B 2 is tested under negative

Table 1. Test specimens

Beam		Number	Purpose of experiment
A	A 1 A 2 A 3 A 4 A 5	2 2 2 2 2	1. Relation between stress of reinforcement and crack width of concrete slab 2. Adequate amount of shear connectors for negative moment 3. Co-operation of longitudinal reinforcement with steel beam
B	B 1 B 2	2 2	Composite effect of composite beam with cracked slab for positive moment (including the study of n value)
C	C 1 C 2	2 2	Effective width
D		2	Ultimate strength of beam

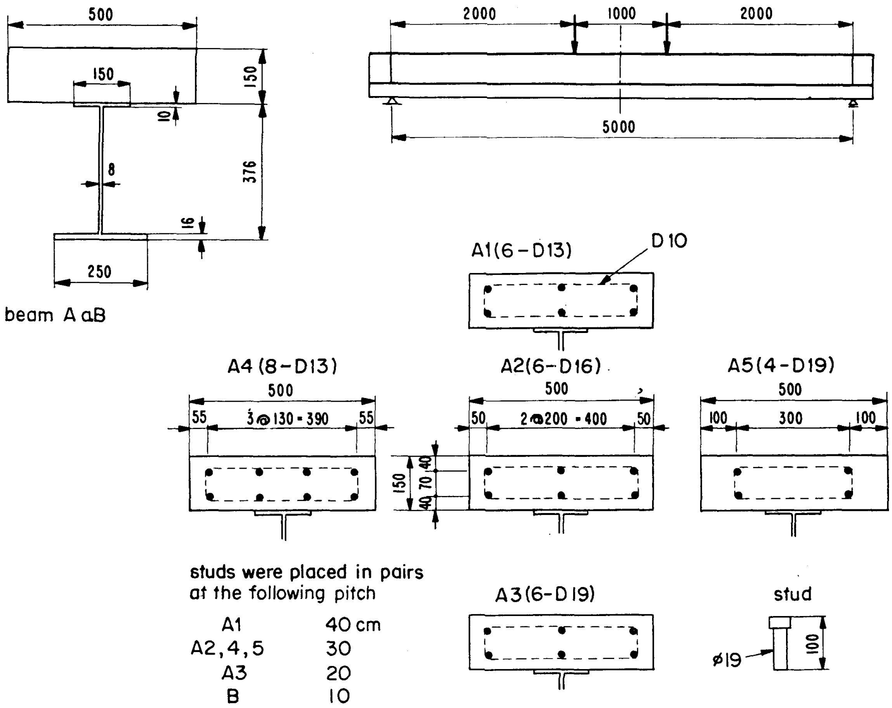

Fig. 2a. Beam specimens.

moment which causes the stress about 1,200 kg/cm^2 in reinforcement and causes slab crack before testing under positive moment. These tests aim at the comparison between the composite behaviour of composite beam with cracked slab and that with uncracked one.

2.3. Beam C

Four beams with wide slab have been designed in order to investigate the effective width of slab for negative moment as well as the effective width of cracked slab for positive moment.

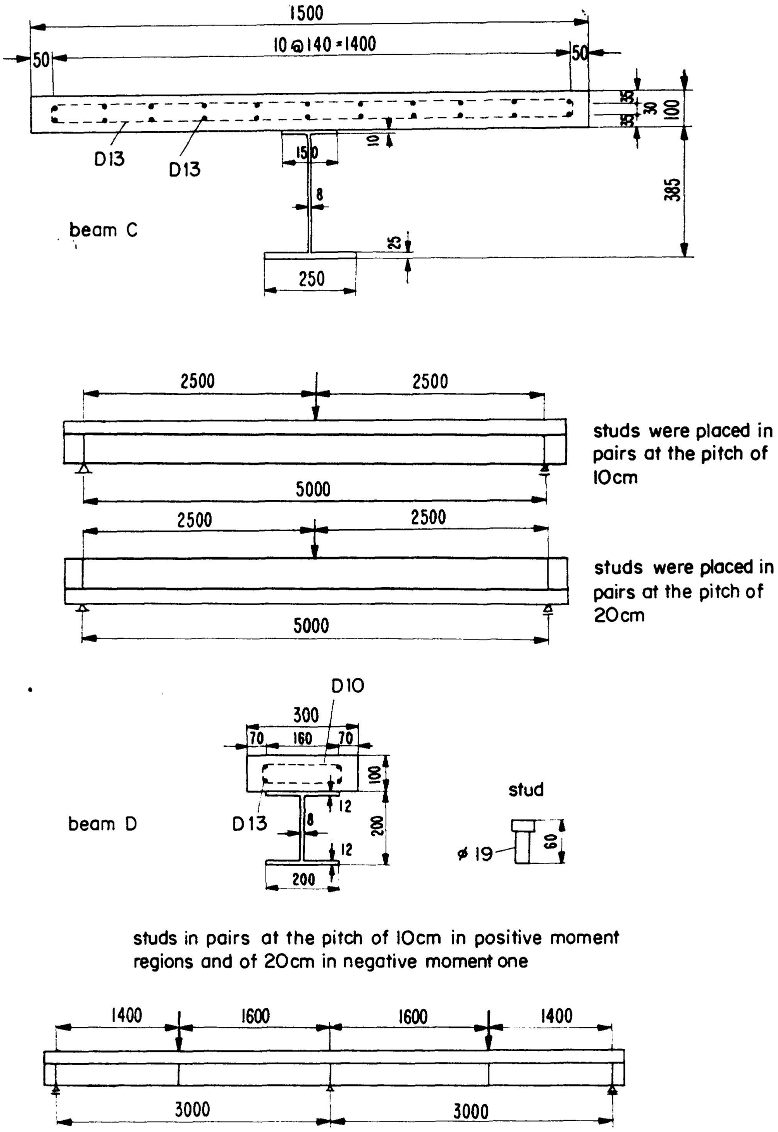

Fig. 2b. Beam specimens.

Beam C 1 is tested as follows: First the beam is loaded to study the effective width for positive moment. Secondly, it is overturned and tested under negative moment causing stress about 1,200 kg/cm^2 in the reinforcement. Thirdly, it is overturned again and tested under positive moment. After comparing the effective width of cracked slab with that of uncracked one, it is loaded to failure.

Beam C 2 is tested under positive moment to examine elastic behaviour and then it is overturned and tested to failure under negative moment.

2.4. Beam D

Beam D is a non-prestressed continuous composite beam having two equal spans. Elastic test and failure test are carried out to obtain cracking behaviour of a slab in the negative moment region and ultimate load.

2.5. Materials of test beams

According to material test results, average yielding stress of structural steel is 2,800 kg/cm^2, that of deformed bar reinforcement 3,800 kg/cm^2, average compressive strength of concrete 370 kg/cm^2, and value n 7.5.

3. Test Results and Discussion

3.1. Relation between crack width and reinforcement

Crack width of each load was measured by contact type strain gauge at both sides of slab. As load increased, cracks gradually diffused, till the cracks became approximately from 10 cm to 20 cm apart when the test came to an end. Relation between calculated stress of reinforcement and crack width is shown in Fig. 3. In Table 2, the maximum crack width and corresponding stress of each beam are summarized.

The table shows the largest crack width that occurred in beam A 1 and the smallest one in beam A 3: Thus the more reinforcement are, the smaller the crack width becomes.

The best result of crack width showed itself in beam A 4. Among the beams with the same amount of reinforcements, when reinforcement of smaller diameter made better results of crack width were obtained.

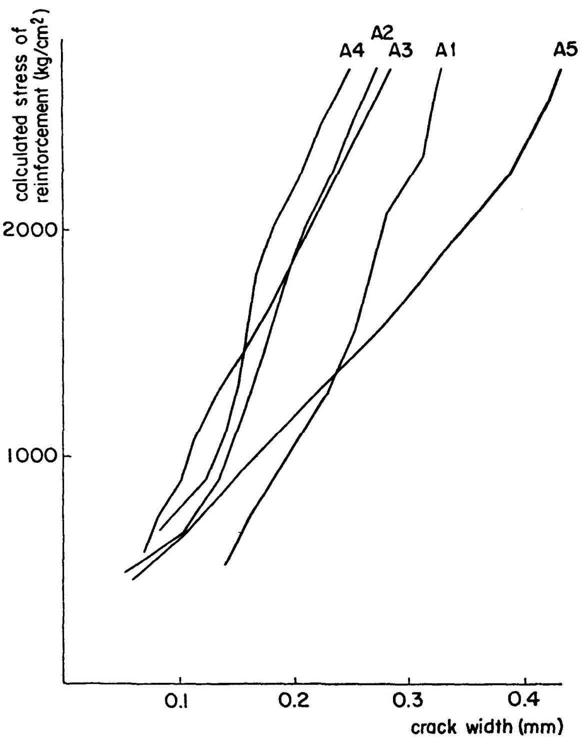

Fig. 3. Relation between calculated stress of reinforcement and crack width.

Table 2. Maximum crack width

Beam	Rein-forcement	Area ratio of reinforcement to slab (%)	Perimeter ratio cm/cm^2	Maximum crack width (mm) at	
				$\sigma_s = 1{,}000\,\text{kg/cm}^2$	$\sigma_s = 2{,}000\,\text{kg/cm}^2$
A 1	6 - D 13	1.02	0.032	0.194	0.276
A 2	6 - D 16	1.58	0.040	0.141	0.208
A 3	6 - D 19	2.28	0.048	0.110	0.212
A 4	8 - D 13	1.35	0.043	0.132	0.180
A 5	4 - D 19	1.52	0.032	0.168	0.346
B 2	6 - D 16	1.58	0.040	0.133	—

Mean value of 2 beams

D: Deformed bar reinforcement

Perimeter ratio is generally related with diameter of reinforcement. Beams A 2, A 3 and A 4 with larger perimeter ratio showed better results than beams A 1 and A 5.

Pitch of shear connectors is 30 cm in beam A 2 and 10 cm in beam B. Comparing their crack width at the stress of 1,000 kg/cm^2, we found that the difference of pitch of shear connectors did not affect the property of cracking.

Now, according to the proposal of the European Concrete Committee, admisible maximum crack width should be 0.2 mm for non protected members of usual structure.

By taking this suggestion into consideration, it can be confirmed that the maximum crack width due to the stress 2,000 kg/cm^2 of reinforcement will not exceed 0.2 mm, if the area of reinforcement is about 1.5% of that of a concrete slab.

3.2. Co-operation of Reinforcement with Steel Beam

The strain of reinforcement in beam A 2 is shown in Fig. 4, from which it is clear that the strain of reinforcement, when the load is small, is lower than the calculated value, enabling us to see that concrete resists tensile stress to some extent. After cracks appeared, strains of reinforcement in the place where cracking occurred rapidly increased. As cracks increased, the average value of strain became nearly equal to the calculated value by using the steel section. The measured strain in steel beam in beam A 2 is plotted in Fig. 5. Above load $P = 15$ ton, which makes upper flange of beam yield, measured strain coincided fairly well with calculated one. It follows then that the longitudinal reinforcement is effective to the negative moment.

Next, Fig. 6 shows the deflection and the variation of the neutral axis of beam A 2 as a function of load. Composite section turns into steel section by applying rather small load $P = 5 \sim 7$ ton to test beam, i. e. the stress of reinforcement is from 1,000 to 1,200 kg/cm^2.

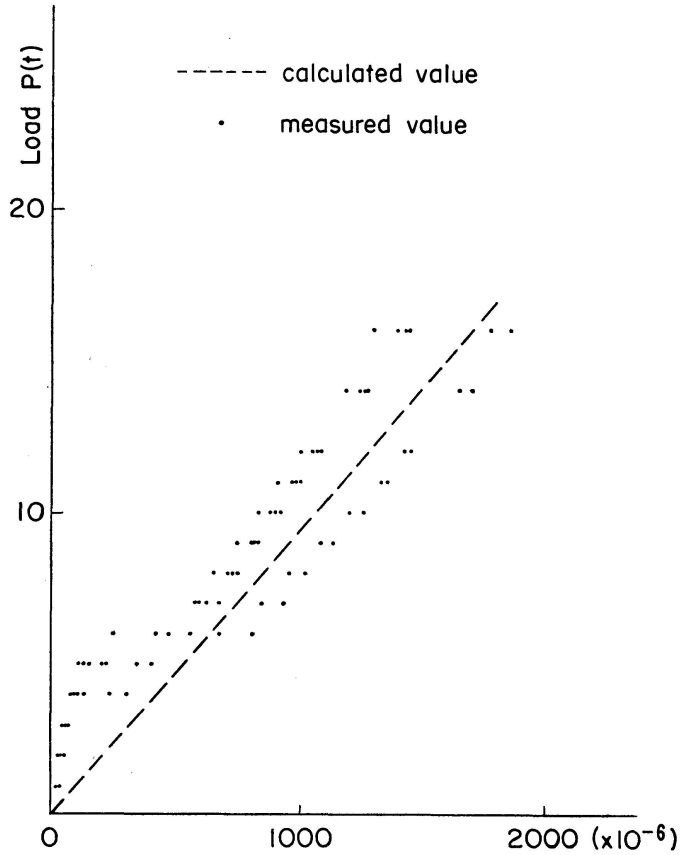

Fig. 4. Strain of reinforcement in beam A 2.

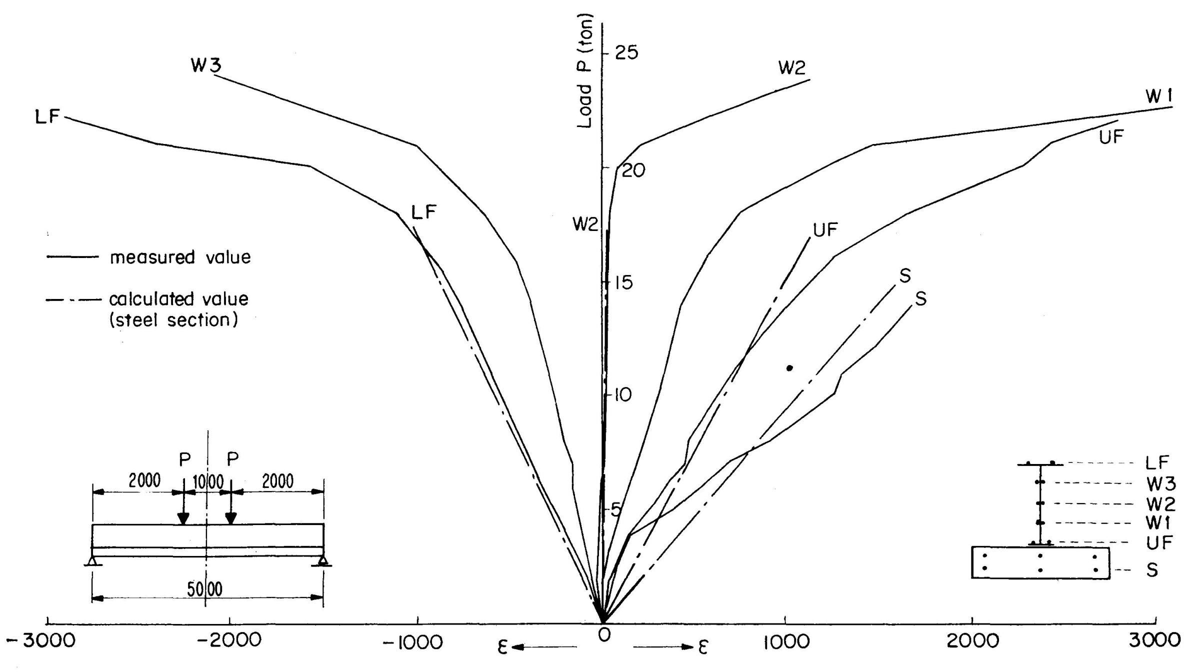

Fig. 5. Measured strain in steel beam in beam A 2.

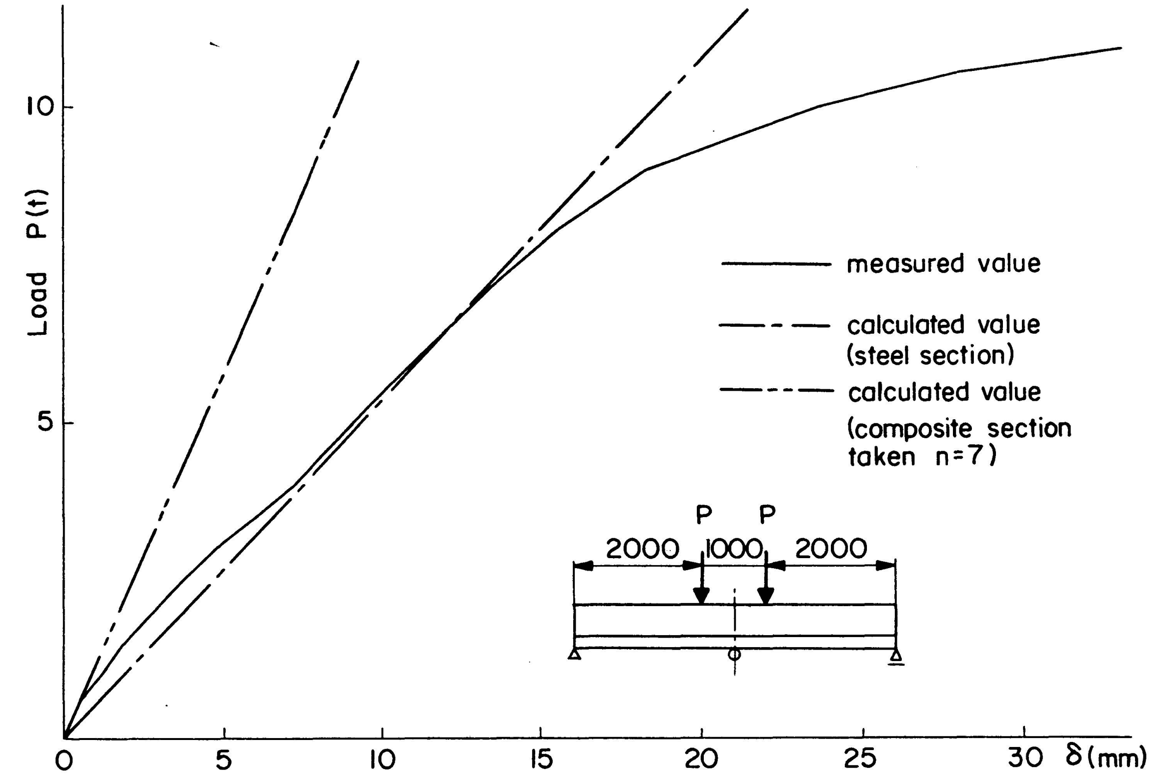

Fig. 6a. Deflection of beam A 2.

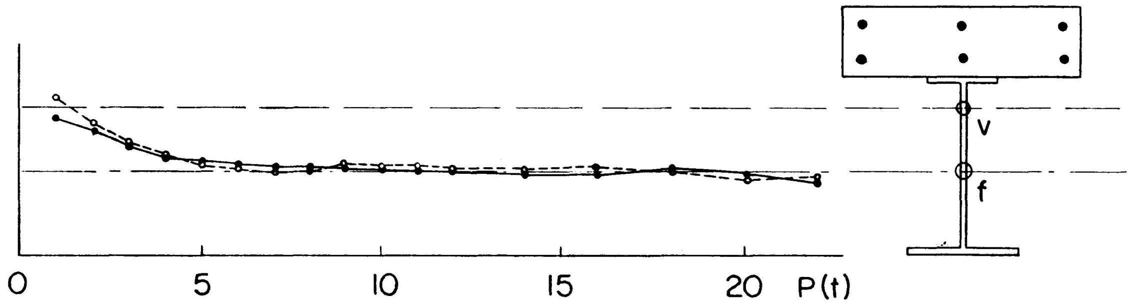

Fig. 6b. Variation of the neutral axis of beam A 2.

From deflection data of beam A and D in the negative moment region, only steel section contributed to the flexural behaviour of the beam.

In beam D, as the load increased, a great many cracks appeared in the regions near middle support, and measured values of deflection and strain were nearly equal to theoretical value of steel section (Fig. 7 and 8).

3.3. Composite Effect of Composite Beam with Cracked Slab

The flexibility of a beam with cracked slab is not different at all from that of a virgin concrete slab not yet cracked. From the measured deflection at the mid-point of span of beam B 2, it was apparent that the composite action had completely recovered after cracks were closed (Fig. 9), and thus we can assume $n = 7$.

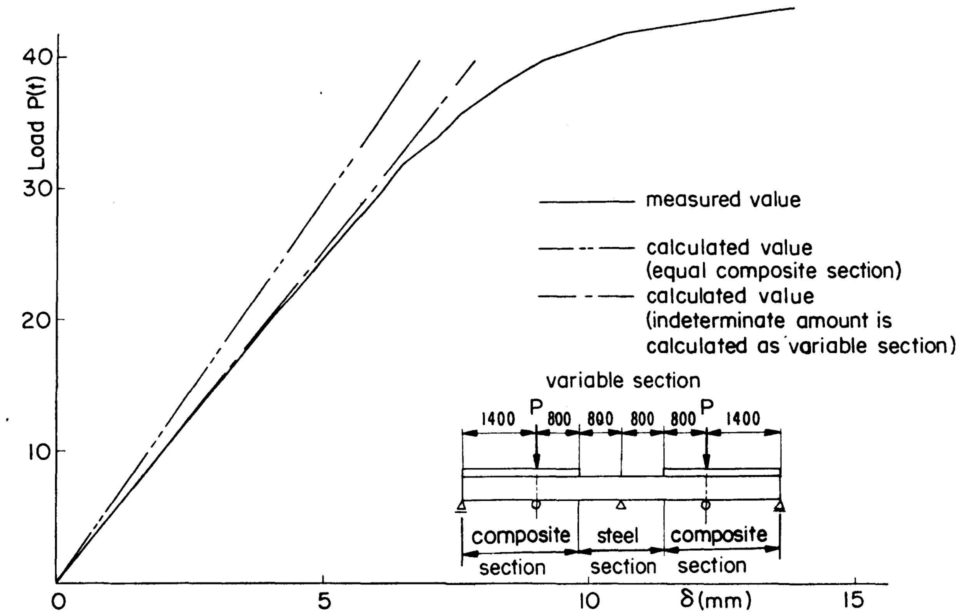

Fig. 7. Deflection of beam D.

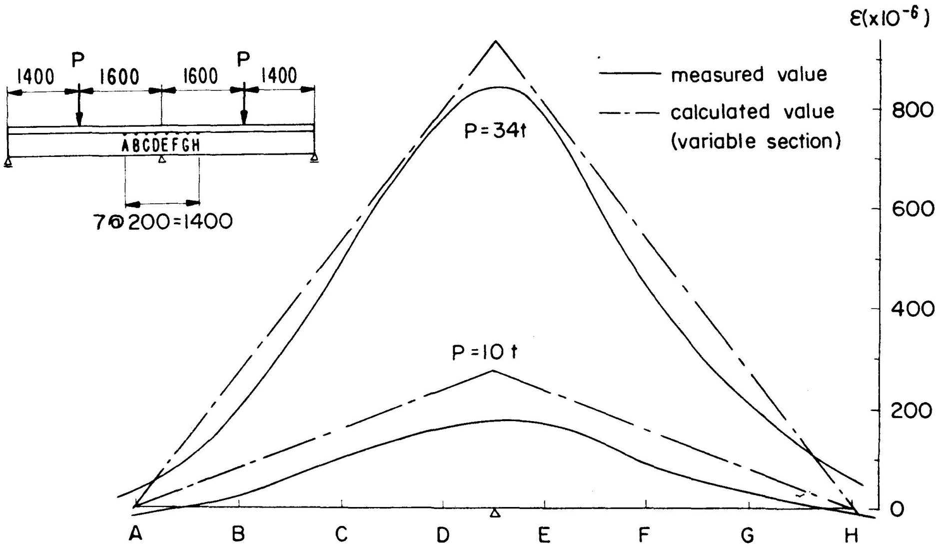

Fig. 8. Strain in upper flange of steel beam near the middle support of beam D.

3.4. Shear Connectors

Shear connectors for all beams in this experiment are designed according to the horizontal shear computed by the following formula.

$$\tau = \frac{QS}{I}, \tag{1}$$

where τ: Horizontal shear, per unit length at the junction of slab and steel beam.

Q: Vertical shear acting on cross section.

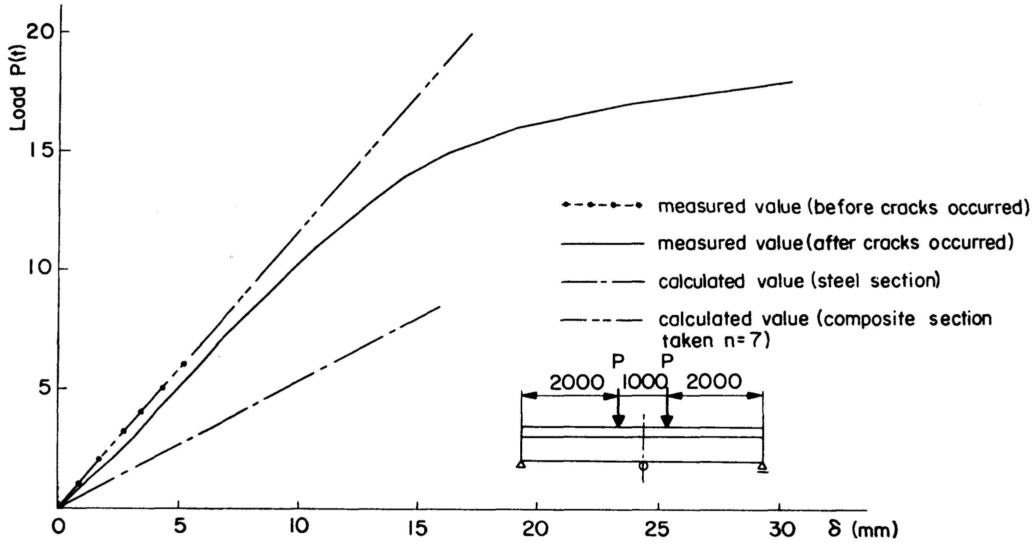

Fig. 9. Deflection of beam B 2.

S: Statical moment of the transformed section about the neutral axis of the composite beam for positive moment, or statical moment of reinforcement about the neutral axis of the steel section (Ref. Fig. 1).

I: Moment of inertia of the composite section for positive moment, or that of the steel section for negative moment.

When the working horizontal shear increased to allowable maximum load strength of an individual shear connector, the maximum residual slip between concrete slab and steel beam A was less than 0.02 mm.

Then, it is required that at failure of the beam the shear connectors should resist the horizontal force H, equal to the yield strength of the longitudinal reinforcement of slab.

$$H = A_s \sigma_y, \tag{2}$$

where H: Horizontal shear force.

A_s: Total area of longitudinal reinforcement in the slab.

σ_y: Yielding stress of reinforcement.

In each beam, working horizontal shear force per one shear connector computed by using the Eq. (2) is less than the value got from the Eq. (1).

3.5. Effective Width of the Slab

The comparison between the strain distribution in reinforcement of beam C 1 with uncracked slab and that in the same kind of beam with cracked one is made in Fig. 10a. All the cracks caused by negative moment are closed at a load stage of $P=12$ ton. White dots indicate the strains for load $P=16$ ton at virgin concrete slab. Black dots indicate the strains for load $P=16$ ton obtained by subtracting the strains at load $P=12$ ton from the strains at load $P=28$ ton. As these strain distributions are alike and moreover they are

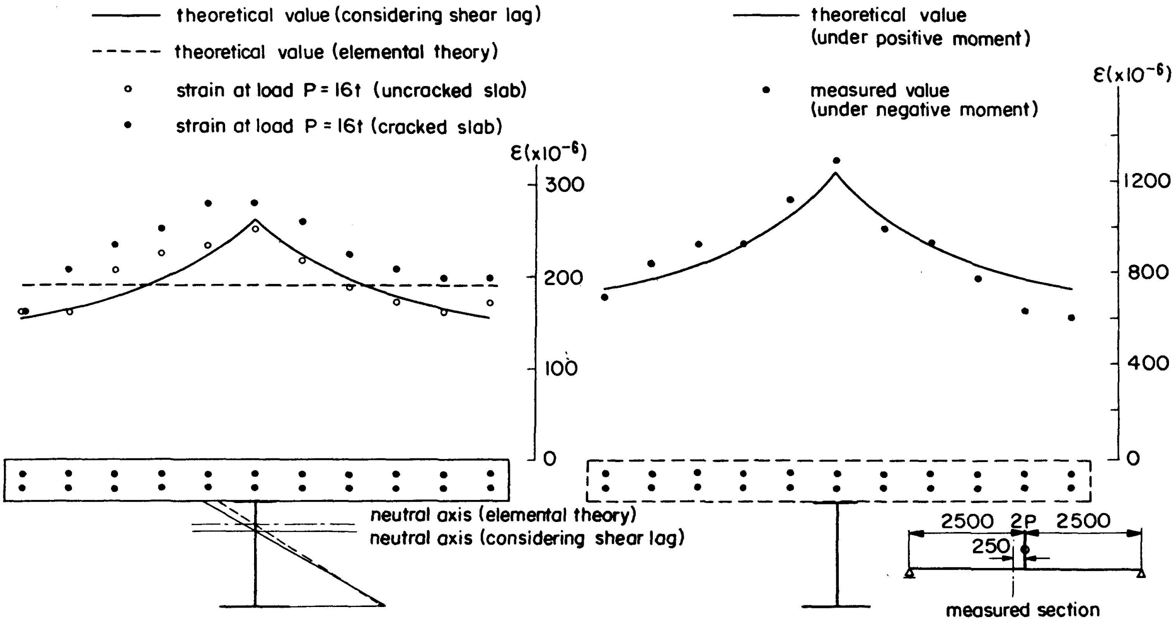

Fig. 10a. Distribution of strain in reinforcement under positive moment.

Fig. 10b. Distribution of strain in reinforcement under negative moment.

comparatively similar to the theoretical value (considering shear lag). It can be considered that the effective width of a cracked slab is the same as that of an uncracked one.

Fig. 10b illustrates the distribution of strain in reinforcement in the slab under negative moment. Experimental values are the average of the values measured at upper and lower reinforcement.

By comparing the measured strain distribution under negative moment with theoretical value under positive moment, we can see that the effective width under negative moment is nearly equal to that of positive moment.

3.6. Ultimate Strength of Composite Beam

Ultimate load of each beam is summarized in Table 3, where the theoretical ultimate load is computed by using the simple plastic theory. For continuous

Table 3. Ultimate load (ton)

Beam	A 1		A 2		A 3		A 4		A 5	
Experimental value	24.0	22.7	23.3	24.0	30.0	27.1	24.3	24.4	24.1	25.0
Calculated value	20.6	20.6	23.0	23.0	26.0	26.0	23.0	23.0	23.0	23.0
Ex./Cal.	1.17	1.10	1.01	1.04	1.15	1.04	1.06	1.06	1.05	1.09

Beam	B 1		B 2		C 1		C 2		D	
Experimental value	38.8	38.1	41.0	39.8	44.0	44.6	35.0	35.8	45.8	45.2
Calculated value	38.0	38.0	38.0	38.0	40.6	40.6	29.0	29.0	41.8	41.8
Ex./Cal.	1.02	1.00	1.08	1.05	1.07	1.10	1.21	1.23	1.10	1.08

beam D, it is assumed that plastic hinges are at middle support and at load points. Fully plastic moment at hinges in the negative moment region and positive moment region respectively correspond to steel section and to composite section.

No such reduction of ultimate load of a beam with cracked slab under positive moment can be found as in B 2 and C 1. So far as ultimate strength is concerned, longitudinal reinforcements also act effectively as steel beam.

Acknowledgement

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Summary

Prestressed continuous composite girder bridges have disadvantages of involving complex procedure in design and taking a long time in construction. In order to get rid of such disadvantages, non-prestressed continuous composite girder bridge is proposed.

Statical experiment has been carried out in order to ensure this design. According to the results of the test, we can find that this type makes possible a simplified and economical design in continuous composite girder.

Résumé

Les ponts faits de pièces composites continues et précontraintes ont les désavantages d'imposer des calculs complexes et des temps de construction prolongés. Afin d'éviter de tels inconvénients, on propose un pont qui serait fait de pièces composites continues mais non-précontraintes.

On a assuré cette conception y l'aide d'expériences statiques. En se reférant aux résultats des expériences, on peut se rendre compte que ce type rend possible une conception plus simple et plus économique des pièces composites continues.

Zusammenfassung

Durchlaufende, vorgespannte Verbundträgerbrücken haben den Nachteil, daß sie komplex zu berechnen sind und eine lange Ausführungszeit beanspruchen. Um diese Nachteile los zu werden, wird der schlaff bewehrte Verbundträger vorgeschlagen.

Statische Versuche wurden unternommen, um die Berechnung zu bestätigen. Gemäß der Ergebnisse können wir sagen, daß dieser Typ einen vereinfachten und wirtschaftlichen Entwurf für durchlaufende Verbundbalken ermöglicht.

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