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Precast Pretensioned Girder Bridges with Continuous Situ-cast Decks and Diaphragms

Ponts à poutres précontraintes d'éléments préfabriqués avec tablier continu et diaphragmes coulés sur place

Vorgespannte, vorfabrizierte Balkenbrücken mit an Ort hergestellten durchlaufenden Fahrbahnplatten und Querträgern

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1. Introduction

Utilization of simply supported precast pretensioned girder bridges in Japan started about in 1955. Since then a large number of this kind of the bridges with about 20m-spans or less have been constructed.

In highways the precast girder bridges were used relatively large in number in the early days, but it was found that many expansion joints were not desirable for riding quality and smooth passage of vehicle, in addition to difficulties of assuring good maintenance of highways. Therefore instead of precast girders the situ-cast continuous bridges have been often employed. But recently precast girders have been again approved more excellent with respect to productivity and quality control. And the method has been developed which enables to construct continuous girder bridges with precast members by casting in situ the decks and diaphragms over intermediate piers.

Thus, in planning multi-span viaducts of about 20m-spans in the urban areas, priority is now usually given to this type of bridges.

This paper reports how to connect precast girders over intermediate piers, the design concept and the flexural test results.

2. Construction method of girder connection for continuity

In Japan there are the standardized precast pretensioned beams available for simply supported beam bridges. The cross section of beam with, say, about 20m-spans is shown in Fig.1. And the bridge is composed of the beams placed in position at spacing of 1.05m and prestressed transversely through the diaphragms and slabs.

In connecting the precast beams the similar method is used.

Details of the method are as fol-

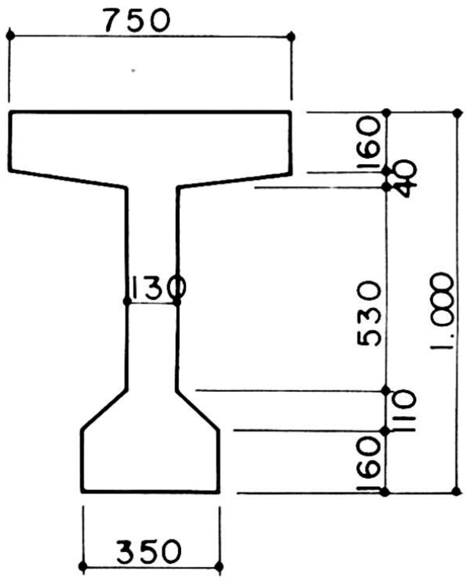

Fig.1 Typical cross section

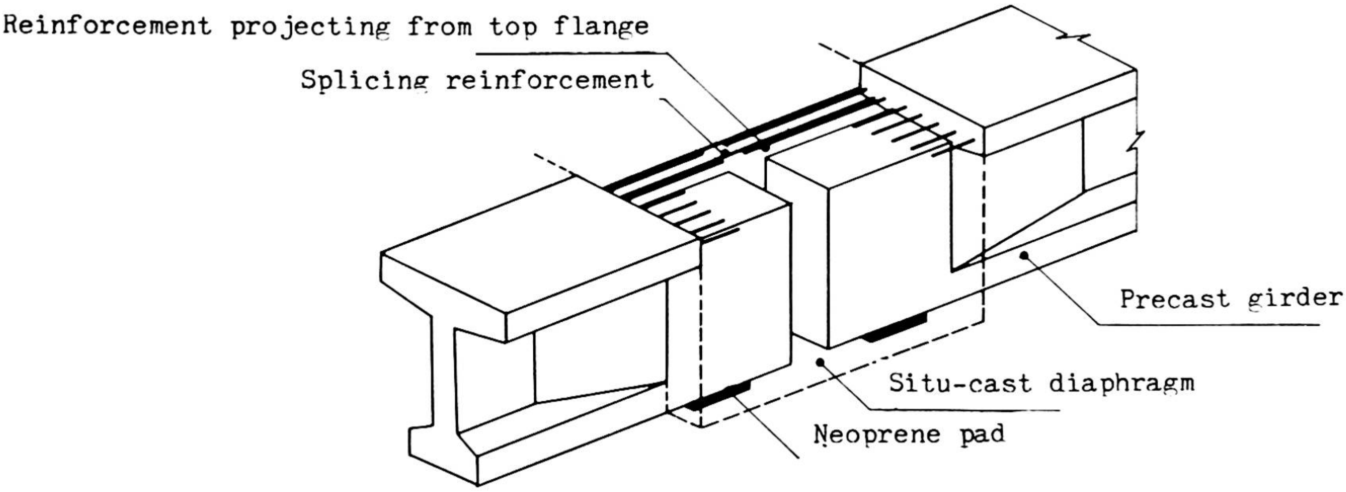

Fig.2 Schematic view of continuity zone

lows; The end of precast girders is designed as schematically shown in Fig.2. Continuity reinforcements are arranged longitudinally in the upper flanges of precast girders and along the projecting length the additional splicing reinforcements are lapped. Then concrete is placed in situ into fairly massive diaphragms including both ends of the precast girders. Then by using lateral prestressing steels provided in the diaphragms the adequate prestress is transferred to the connection zone.

The bearing shoes used temporarily for precast girders remain left behind in order to serve for the continuous girders when the connection has been completed. Therefore there exists a pair of shoes located slightly apart on each pier top.

3. Design concept on continuity

The amount of continuity reinforcement over piers may be not always enough compared to that of conventional continuous girders, because , as described before, the reinforcement for continuity are placed only in top flanges.

The zone of continuity tends to crack at lower load level than the other zones do and so it can be said that the former has the lower flexural rigidity. It is considered better to analyse this type of bridge as a group of simply supported beams, even though it behaves almost as continuous girder. In this respect it is different from conventional continuous girder.

A pair of bearing shoes are used by taking into consideration the fact that the connection zone can be constructed with more ease and that the safety of bridge could be assured even if the continuity of girder is completely lost. And a pair of bearing shoes, which is usually specified to use neoprene, is expected to work together alike one shoe because of their elastic deformation in vertical direction. It is found desirable by the analysis that the intensity of elastic constant of neoprene pads is less than 350 t/cm. This condition may be always satisfied when the pads are designed conventionally to correspond to the reactions and deformations of simply supported beams. The amount of continuity reinforcement that carries the loads (dead loads such as pavement and handrails, and live loads) imposed on the bridges after the completion of continuity, is calculated by the conventional reinforced concrete theory on the assumption that there exists only one bearing shoe on each pier top.

Generally, in case of spans about 20m long, deformed reinforcing bars of 22 mm dia. are placed at spacing of 10cm, so that the calculated stress of bars is less than 1600 kg/cm^2.

It is expected that the secondary bending moment will take place due to creep of precast girders, resulting tensile stresses in the lower parts of girder sections in case of standardized precast girders. That is, it works to relieve the negative moment acting on the zone of continuity, and furthermore the secondary deformation due to creep is estimated to be less than the deformation of simply supported girders. Because of this reason the moment due to creep is not fully taken into consideration. However, some amount of reinforcement is provided in the lower part of the diaphragms in the direction of bridge axis in order to resist this creep moment.

Thus the precast pretensioned girder bridges with situ-cast continuity zone over piers constructed in this fashion is substantially of simply supported beam system, although progressively taking into consideration the incompleteness of continuity.

This system has both the advantage of simplisity in construction like simply supported beam system and the same good riding quality as assured in continuous beam system.

4. Test results

As described above, this system of bridge can be considered as continuous beam system from riding quality, even though it has incomplete continuity zone from view of design concept.

Consequently, the zone of continuity should have sufficient flexural rigidity so that pavement on both sides of it can be kept in good condition under service load.

A series of tests were carried out to make sure the point as indicated above. The main purpose of these tests is to observe how the test beams crack under service load or so, and to pursuit their behaviors until failure.

Three test beams prepared were 9m long with the same cross section, reinforcement and continuity zone at center of spans as those of actual beams. The method of bending test of beams is shown in Fig.3.

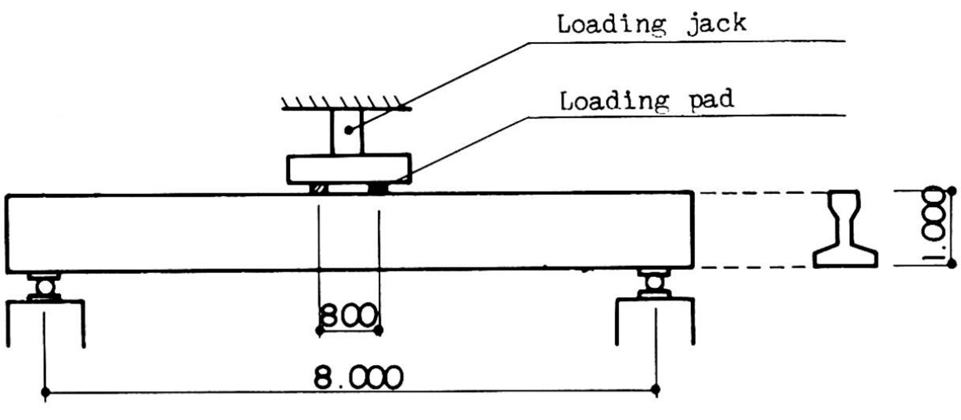

Fig.3 Loading method

The test results are as follows;

(1) Cracks were observed, firstly at both construction joints in decks, then in decks between both joints and finally in decks outside them. The load level at which cracks were observed at each stage were clearly different.

Now, the service load of the test beams is designated as P-1800, which means the load at which the stress of continuity reinforcement has reached 1800 kg/cm^2 as calculated by the conventional theory, while the allowable stress of reinforcement in actual beams is specified to 1600 kg/cm^2.

The initial crack at construction joints in top flanges occurred before the stress of reinforcement reached 1800 kg/cm^2. But the crack width remained almost unchanged thereafter, and at P-1800 the width was less than 0.1mm. At

P-3000 (the load at which the stress of reinforcement has reached the nominal yield stress 3000 kg/cm^2) the width was less than 0.2mm. In the zone between the two construction joints, no crack was found even at P-1800, and first crack occurred at the calculated stress level of 2000 to 2400 kg/cm^2.

Then the load was removed. When the beams were reloaded, the crack width in the zone between both construction joints was 0.1mm at P-1800, and became rapidly as large as 0.4mm at P-3000. This crack developed and caused the beams to fail finally as shown in Fig.4. Outside the both joints, there existed no crack before P-3000.

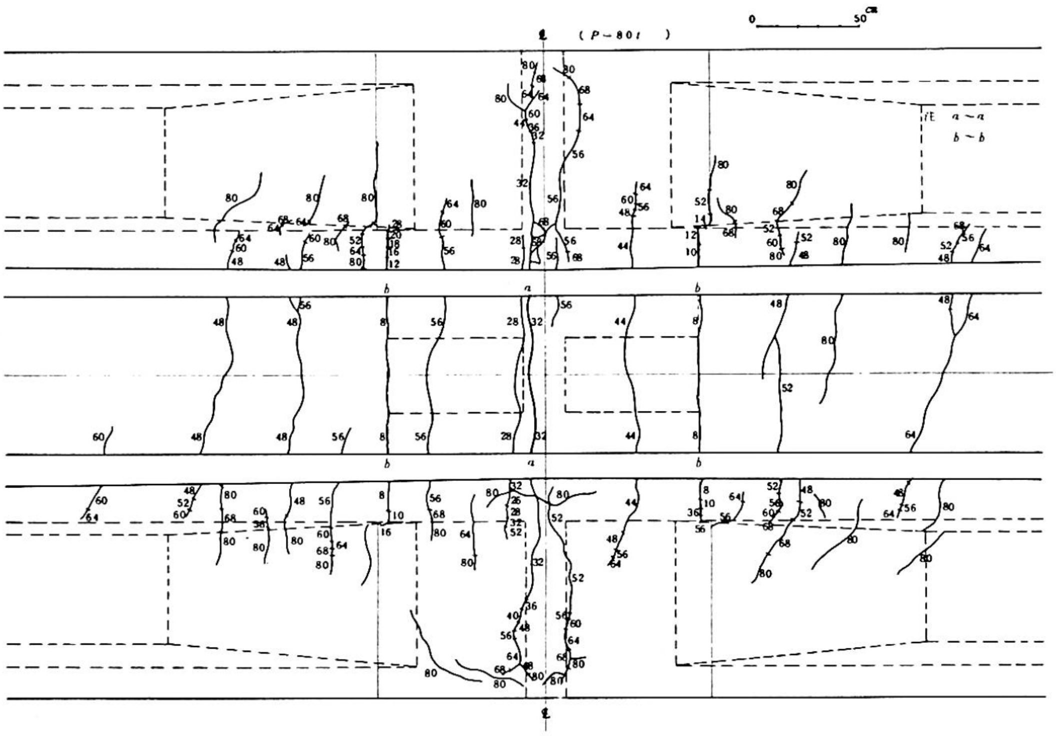

Fig.4 Typical final crack pattern

(2) The reduction in flexural rigidity at the zone of continuity as load increased was estimated by measuring deflections of test beams. The crack at construction joint in top flanges gives little influence on the flexural rigidity of beams. The flexural rigidity of test beams began to decrease after the cracks occurred in the middle part between both joints at the load higher than P-1800 as indicated in Fig.5. Until the test beams failed any slippage was not observed in the splicing reinforcement having lap lengths of over 30 times dia. each along the reinforcement projecting from top flange of girders.

The test beams failed due to excessive compression of top flange after the tip of crack between both joints had reached almost the top fiber. No abnormal behavior to be attributed to the performed continuity was observed throughout these tests.

The description above is the excerpts from the test results, which show that the zone of continuity does not give any worse influence on pavements at service load level and at the same time that not any special behavior were observed which might be resulted from the performed continuity. Consequently it is concluded that the precast girder bridge with this kind of the continuity works completely well.

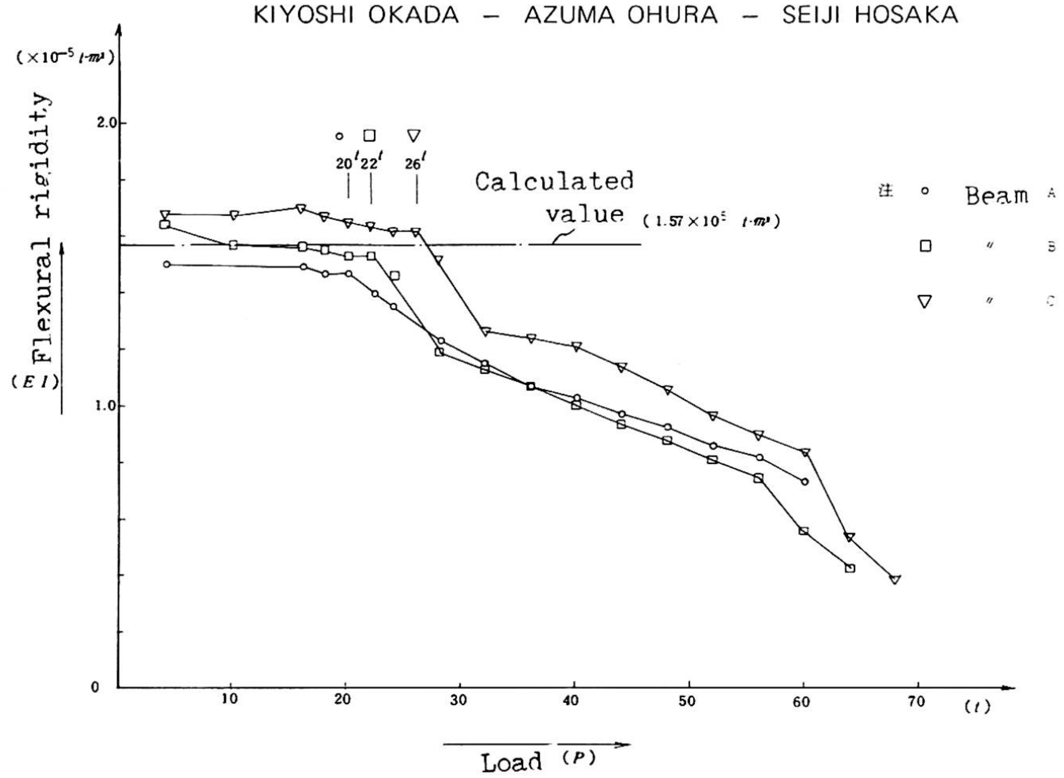

Fig.5 Reduction of flexural rigidity estimated by deflections measured

5. Remarks

A large number of viaducts have been constructed in urban areas in Japan. The engineers concerned must pay attention for the people living along the job sites to the shortening of construction periods and various pollution problems during construction.

From this point of view, the utilization of factory-made precast beams as reported here is to be approved to a large extent. The authors are happy if this report may give any help to the engineers concerned.

SUMMARY

The precast pretensioned girder bridges with continuous situ-cast decks and diaphragms over intermediate supports are the structures which behave actually as indeterminate continuous beams, but which are assumed to consist of determinate simply-supported beams with respect to structural analysis. Consequently they are completely different on this point from conventional continuous beam bridges. The method of girder connection for continuity, the design concept and the flexural test results are presented.

RESUME

Les ponts à poutres précontraintes d'éléments préfabriqués, avec tablier continu et diaphragmes coulés sur place sont des structures qui se comportent en fait comme des poutres continues indéterminées, mais qui sont considérées dans le calcul comme des poutres simples statiquement déterminées. En ce qui concerne l'analyse, ces ponts sont donc complètement différents des ponts conventionnels à poutres continues. La méthode d'assemblage des poutres pour assurer la continuité, la conception du projet et les résultats des essais de flexion sont présentés.

ZUSAMMENFASSUNG

Vorgespannte, vorgefertigte Balkenbrücken mit an Ort hergestellten durchlaufenden Fahrbahnplatten und Querträgern verhalten sich praktisch wie durchlaufende statisch unbestimmte Träger. In der Berechnung wird allerdings statisch bestimmte Lagerung als einfacher Balken vorausgesetzt. In dieser Beziehung unterscheiden sich solche Brücken demnach von normalen konventionell hergestellten Tragwerken. Die Verbindung der Balken über den Zwischenstützen, die grundlegenden Entwurfs-Gedanken sowie die Ergebnisse eines Biegeversuchs werden dargestellt.

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