Performance of channel-shaped precast PC slabs under moving truck load

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Summary

Channel-shaped precast PC slabs (CPC slabs) prestressed in two directions were developed for decks on steel highway bridges. It has been shown in previous studies that the CPC slabs have large load carrying capacity and high durability against the traffic load. The CPC slabs were applied to the Kuwazai Bridge, a three span continuous box girder bridge constructed five years ago. Moving truck load tests were carried out on this bridge for evaluation of performance and behaviour of the CPC slabs after five years of service. The experimental data obtained by the tests were compared with FEM analytical values to ensure soundness of the slabs. Flexibility of composite action in the bridge was also analyzed. The results showed that the CPC slabs were still sound, with their concrete cross sections remaining fully effective.

Keywords: precast PC slab; performance evaluation; moving truck load test; durability; composite action

1. Introduction

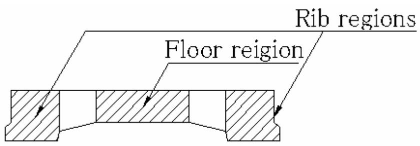

Fig. 1 Channel-shaped precast PC slab, cross section

Precast PC slabs are widely used as a way to save field work labour. The CPC slab, shown in Fig. 1, is a new design of that technique which has been already applied to some newly built steel girders of major highways. Although no problems have been reported since they were put into service, it would be useful to test an existing bridge under actual wheel load and evaluate performance of the CPC slabs in service under the heavy traffic.

Moving truck load tests were carried out on the Kuwazai Bridge of the Prefectural Chuo Loop in Osaka in order to examine behaviour or the CPC slabs in that bridge and to evaluate the current performance of them which have been under wheel loads of actual traffic. Elastic analysis was also performed using the FEM with a model of the whole bridge in order to reproduce the behaviour observed during the experiments and investigate flexibility of the composite action between the slabs and steel girders.

2. Outline of the experiment

2.1. Kuwazai Bridge

The Kuwazai Bridge is a three span continuous non-composite steel box girder bridge with a bridge length of 114.000 m (34.575 m + 44.000 m + 34.575 m spans) and an effective width of 10.750 m. The bridge has two main box girders with varying cross sections and a stringer at the centre of the bridge cross section, with three one-way lanes for southbound traffic on it. Figures 2 and 3 show the general views of the superstructure.

The CPC slabs were used in this bridge for a shorter work period and smaller superstructure weight. Each CPC slab panel was a pretensioning member with a width of 1.500 m in the direction of the bridge axis, a length of 11.850 m in the direction perpendicular to the bridge axis and thicknesses of 27 cm at the rib regions and 17 cm at the floor region. A continuous slab structure was completed by placing 73 panels on the erected steel girders using crawler cranes, filling shrinkage-compensating mortar in the joints between the panels, then introducing longitudinal prestress by a postensioning system.

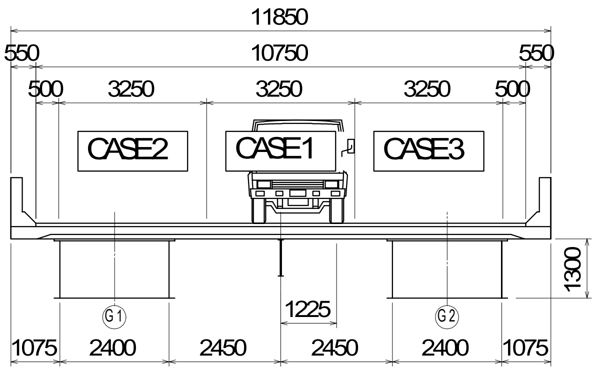

Fig. 2 Kuwazai Bridge, standard cross sectional view

The CPC slabs were combined with the steel girders after the completion of longitudinal prestressing, by filling shrinkage-compensating mortar in the notches in the slabs which were provided to receive the studs projecting from the girder flanges. Design of the CPC slab was decided in accordance with the load bending moment equations specified in the Japanese Specifications for Highway Bridges. The slab was designed to be fully prestressed in the direction of the slab span and to allow concrete tensile stress within a limit in the direction of the bridge axis. Design strength of the concrete was $\sigma_{ck} = 50\ N/mm^2$. Prestressing steel was SWPR7A 1S15.2 mm for the direction of the slab span and SWPR19 1S21.8 mm for the direction of the bridge axis.

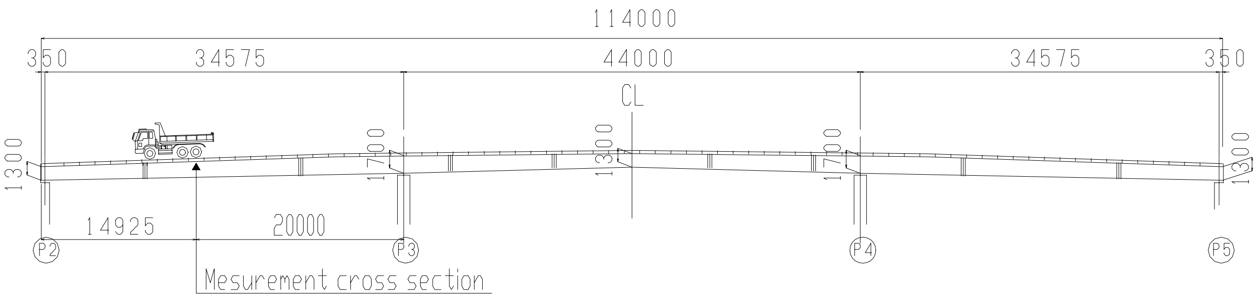

Fig. 3 Kuwazai Bridge, side view

2.2. Moving truck load tests Truck loads

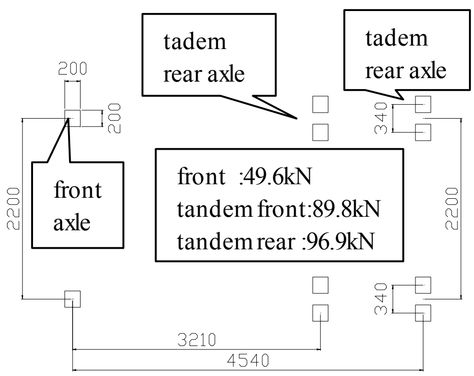

Fig. 4 Wheel positions and axle loads

Since the bridge was actually in service and difficult to close for performing static loading tests, the authors adopted dynamic loading tests using a moving three-axle truck. Truck scale measurements were 236 kN for gross weight, 49.6 kN for front axle load, 89.8 kN for tandem front axle load, and 96.9 kN for tandem rear axle load. Figure 4 shows the wheel positions and individual axle loads. Three cases were tested, using each of the three traffic lanes. A line was assumed in the direction of the bridge axis at the centre of an inner slab span supported by the stringer and one of the box girders, and the truck was driven with its left wheels travelling on that line in Case 1. The truck travelled at a constant speed of 30 km/h, four times per case.

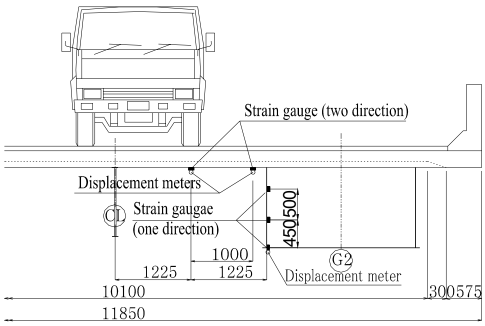

Fig. 5 Measurement locations, cross sectional view

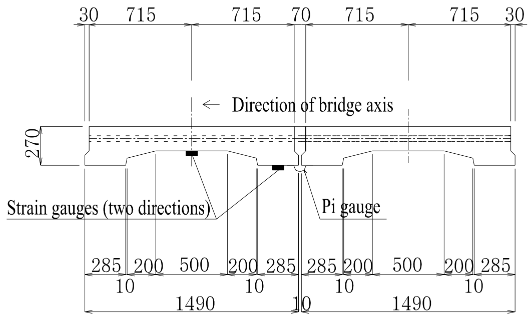

Fig. 6 Measurement locations, slab side view

Measurement items

Focus of measurement was placed on two CPC slab panels located around the centre of a side span of the bridge (indicated by a black triangle in Fig. 3).

Measurement items for behaviour of the CPC slabs were strain in the underside of the slabs in the directions of the slab span and bridge axis (at both the rib and floor regions), gap between the adjacent panels (by a pi gauge) and deflection in the slabs (by displacement meters).

Deflection in the steel girders and strain in them in the direction of the bridge axis were also measured in order to examine the composite action between the slabs and steel girders. Figures 5 and 6 show individual measurement locations.

Dynamic strain gauges were used at a recording frequency of 100 Hz. Each measurement begun immediately before the truck entered the bridge and continued for about 30 seconds until the truck completely passed through the bridge.

3. Elastic analysis using the FEM

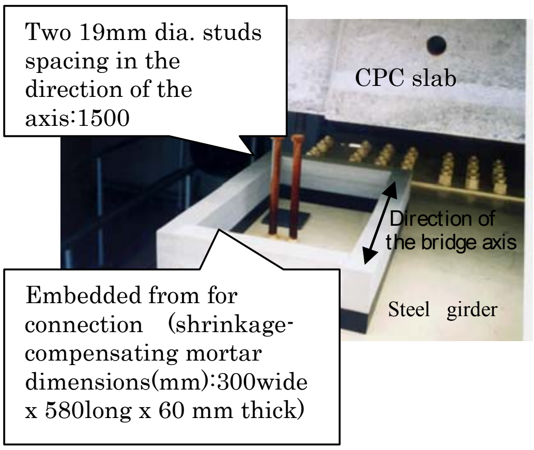

Fig. 7 Stud layout in the bond region

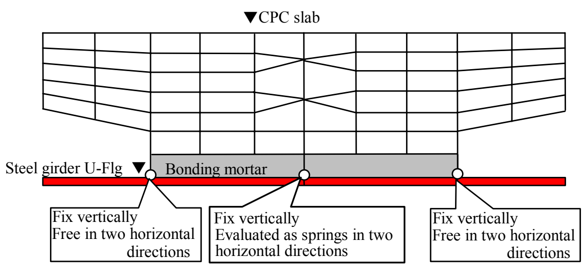

Fig. 8 Details of the bond region

Elastic analysis was performed using the FEM with a model of the superstructure of the measurement area shown in Fig. 9. Young's modulus was $E_c = 3.3 \times 10^4$ N/mm^2 for the concrete and shrinkage-compensating mortar and $E_s = 2.0 \times 10^5$ N/mm^2 for the steel girders. In the P2-P3 range 3-D 8-node and 6-node solid elements were used for the slab concrete, and 4-node and 3-node plate elements were used for the steel girders. All of the solid and plate elements were isoparametric. Beam elements were used in the other ranges to make the girders continuous. The slabs in this bridge were bonded to the steel girders using studs and shrinkage-compensating mortar as shown in Fig. 7. It is known that a bridge behaves as an elastic composite girder when this technique is used [2]. In this analysis the shrinkage-compensating mortar in the bond region (Fig. 7) was evaluated as an element shown in Fig. 8. Double nodes were used at the interface between the steel girder element and the bond region element, assuming rigid bond vertically and horizontal springs in the two horizontal directions. The degree of composite action was determined qualitatively and quantitatively, with a spring constant of the studs varied as shown in Table 1.

Table 1 Shear spring constant for the 19 mm dia. studs

Code	Analysis cases by composite actions Composite	Spring constant (kN/cm/spring)
FA-1	Full composite action (full bonded)	∞
FA-2	Full composite action (bonded at the ribs)	∞
FA-3	Elastic composite action (1 spring)	2695
FA-4	Elastic composite action (1/2 spring)	1348
FA-5	Elastic composite action (1/3 spring)	898
FA-6	Elastic composite action (1/10 spring)	270
FA-7	Non-composite action	0

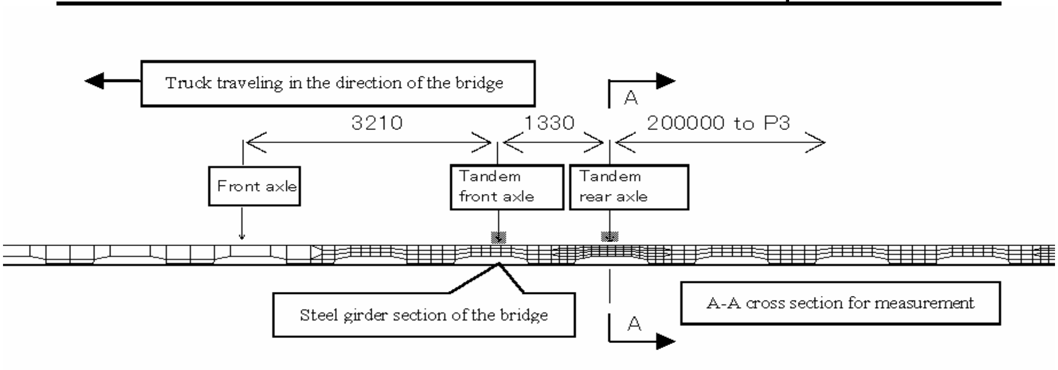

Fig. 9 FEM model of the superstructure of the measurement area

4. Experimental results and discussion

4.1. Examination on the composite action

The reference spring constant (1 spring) used in the analysis was an average value of previous punching test results with precast slabs, corresponding to a residual displacement of δ = 0.08 mm for 19 mm dia. studs[2]. The contact surface between the slabs and steel girders in this bridge was only 0.54 m per 1.5 m, or about 1/3 the upper flange length. The bond strength in that region might have been affected by dust or other contamination on the upper flanges at construction. Through the five years of service the bridge was continuously subjected to stress from the heavy traffic including an increasing number of overloaded trucks.

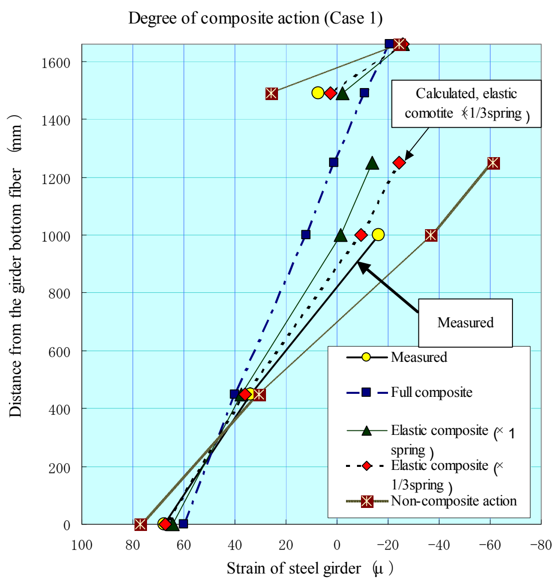

Fig. 10 Comparison of strain distribution in the main girder cross section (Case 1)

Based on these conditions the authors estimated that a spring constant of the studs in this bridge must be lower than a normal level for composite girders. In order to evaluate flexibility of the composite action in this bridge, horizontal spring constant for the studs was varied to four levels in the FEM analysis, and the obtained data was compared with the experimental results. Figure 10 shows strain distributions in the measurement cross section (A-A shown in Fig. 9) for each degree of composite action in Case 1. The values with the 1/3-spring constant (898 kN/cm/spring) best matched the measurement results also in other cases. Therefore, the authors adopted the 1/3-spring constant in the FEM calculation for the current slab analysis.

4.2. Soundness evaluation

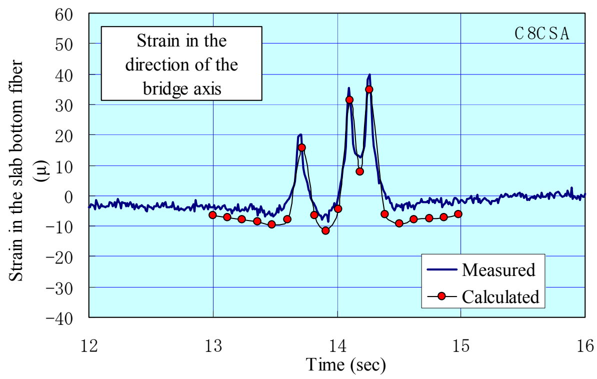

Fig. 11 Strain changes in the underside of the slabs in the direction of the bridge axis

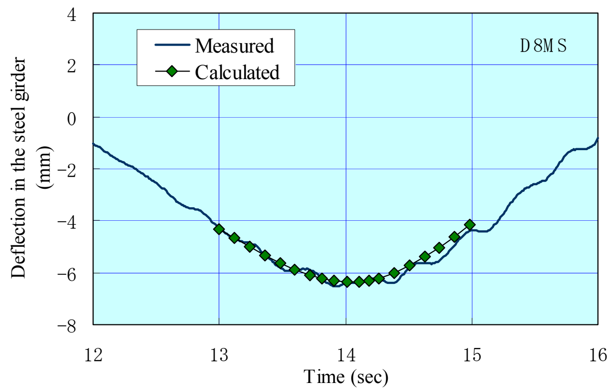

Fig. 12 Deflection changes in the main girder

Dynamic strain gauges were used to observe strain changes caused by the moving truck. Truck load in the FEM analysis was moved to reproduce the strain changes. Although measurements were influenced by the vibration of the superstructure, such effects were not included in the FEM analysis. Figure 11 shows strain changes in the underside of the slabs in the direction of the bridge axis, and Fig. 12 shows deflection changes in the main girder. The measured values used in these figures were taken from the first run of Case 1. The analytical results matched well with the measured values also in strain changes in the direction perpendicular to the bridge axis and those in the bottom fiber of the steel girders in the direction of the bridge axis, indicating that the FEM model used reproduced the response changes successfully. In the FEM analysis the CPC slab concrete was assumed to be a perfectly elastic body with a fully effective cross section. The good fit between the analytical results and measured values allowed to anticipate that the slabs in the existing bridge had behaved as a body with a fully effective cross section since its start of service. Gap between the CPC slab panels measured by the pi gauge was 0.025 mm (50 μm as strain) at the largest, which was almost equal to the strain in the ribs in the direction of the bridge axis.

The good fit with the response change curves obtained by the FEM analysis suggested no sign of cracking or other damage in the slab joints, supporting that longitudinal prestress functioned effectively and maintained the joints in sound condition. However, the test truck load was about 1/2 the design wheel load and must be different from a case where B live load is applied to the entire bridge. The fact that the bridge behaved as a perfectly elastic body under the current test load does not necessarily guarantee the same behavior of the bridge under larger loads. The findings from the current study can be understood as a proof that the slabs were not damaged or deteriorated under the load applied to present with their cross sections maintained fully effective and that the bridge remained in sound condition after being subjected to the load of this level. It is required to carry out ten- or twenty-year follow-up investigations after further exposure to loads and collect more data on damage or deterioration to the slabs and influences on the entire superstructure including decrease in the degree of composite action.

5. Conclusions

Moving truck load tests were carried out to determine behaviour characteristics of an existing bridge with CPC slabs and to evaluate soundness of the slabs after a five-year history of actual traffic loading. The authors also performed elastic analysis using the FEM with a model of the whole bridge for the purposes of reproducing the behaviours observed during the experiments, examining composite action between the slabs and steel girders in the bridge, and confirming the soundness of the slabs. The following findings were obtained from the experimental and analytical results:

The current bridge designed as a non-composite girder was found to behave as an elastic composite girder. Composite action was examined by elastic analysis using the FEM, with horizontal springs assumed for the slab-girder bond regions. The analytical results matched the measurement results when calculated with a spring constant of the 19 mm dia. studs being 1/3 the spring constant value which was known from previous punching tests to cause critical displacement. This indicated development of elastic composite action in the bridge.

Changes in response values were reproduced by the FEM analysis and compared with the measurement results to examine soundness of the slabs. The analytical results with the CPC slab concrete assumed to be a perfectly elastic body with a fully effective cross section matched the measured values of strain and deflection. This proved that the slabs with a five-year history of heavy traffic loading were still sound with the cross section maintained fully effective.

6. References

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[2] NAKAI H., *Design and Construction of Precast Slab Composite Girder Bridges* (in Japanese), Morikita Shuppan Co., Ltd., Japan, 1988.