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**IX****Behavior of First-Built Composite Girder Bridge in Japan**

Le comportement du premier pont-mixte au Japon

Das Verhalten der ersten Verbund-Balkenbrücke in Japan

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SUMMARY

Kanzaki Bridge, Osaka, Japan, was removed after 25 years' service as the first-built steel-concrete composite girder bridge in Japan. Lessons from the load history investigation of the bridge and from the strength tests of the girders and materials, are discussed in relation with the soundness or deterioration of the bridge after its 25 years' use.

RESUME

Le pont Kanzaki à Osaka, premier pont-mixte acier-béton au Japon, a été enlevé après 25 ans de service. La contribution présente le résultat de recherches sur la sollicitation effective du pont et son état de détérioration après 25 ans de service.

ZUSAMMENFASSUNG

Die Kanzaki Brücke in Osaka, Japans erste Verbund-Balkenbrücke, wird nach 25 Jahren Benützung wieder abgebrochen. Die Belastungs- und Beanspruchungsgeschichte der Brücke wird beschrieben und anhand der tatsächlich eingetretenen Abnützung besprochen.

1. INTRODUCTION

Kanzaki Bridge, Osaka, Japan, was the first steel-concrete composite bridge^{1),2)} built in 1953 to carry the Osaka Prefectural Road over Kanzaki River. Photo 1 shows the general view of the original bridge. The bridge was designed in accordance with the German Code of Practice for Composite Girder Bridges in 1950, because at that time Japan didn't have her own design specification for composite girder bridges. The bridge was removed in 1978 after 25 years' service to meet a further increase in traffic and the control of flood tide in the river. Then, it was replaced by a new non-prestressed composite box girder bridge consisting of 3-spans of 76.5 + 88.0 + 76.5 m with the effective width of 22.5 m. Photo 2 shows the new bridge by the side of the old bridge which was being demolished. Since full-size tests of composite girders which were subjected to a long history of loadings and of weather conditions, have hardly been reported, various tests of the girders of Kanzaki Bridge were carried out or are in progress, in the same way as the tests previously done at the time of the original construction, as well as an investigation on the load history of the bridge. The paper presents lessons from the behaviors of the composite girders after 25 years' use.

2. OUTLINE OF KANZAKI BRIDGE

Kanzaki Bridge consisted of 18 spans of 12 m length of composite I-beams, 7 spans of 12 m length of composite welded girders and 2 spans of 10 m length of non-composite I-beams, making the total bridge length of 320 m as seen in Fig. 1. It was provided with 15 cm thick reinforced-concrete slab with 5 cm thick asphalt-concrete pavement. Inverted T-shaped reinforced-concrete abutments and pile-bent type reinforced-concrete piers were set on pile foundations. The effective width was originally 6.0 m and later in 1962-63 it was enlarged to a roadway of 8.0 m plus a footway of 2.0 m to meet an increase in traffic. The design live load was 13 ton trucks as the 1st-class bridge following the Japanese Bridge Design Specification in 1939.

Photo. 2 Picture of New KANZAKI Bridge by the side of Old Bridge which was being removed.

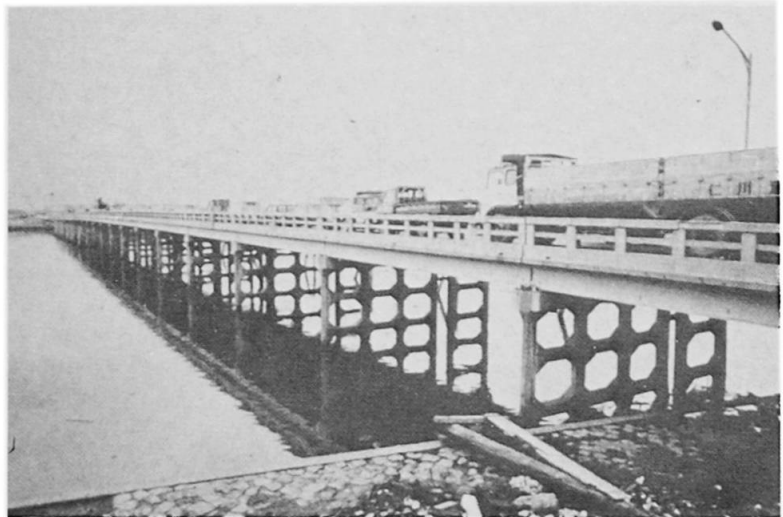


Photo. 1 Picture of Old KANZAKI Bridge



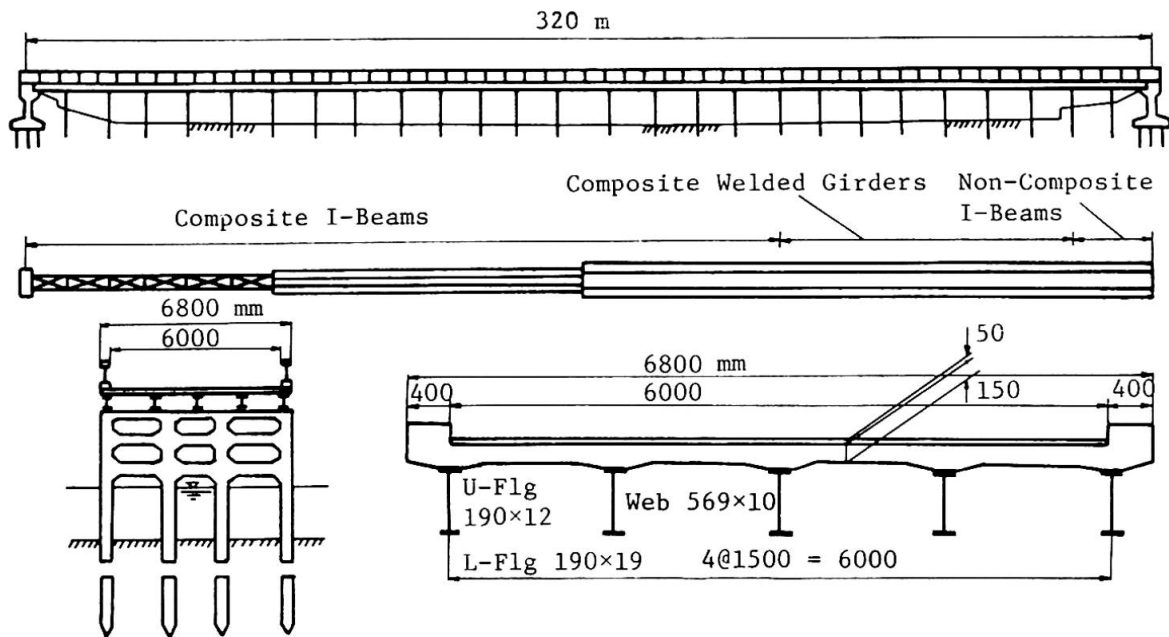


Fig. 1 General View of Old KANZAKI Bridge

3. LOAD HISTORY SURVEY

Kanzaki Bridge was designed against 13 t trucks, but since 1956, 20 t trucks have been specified for a live load of a 1st-class bridge in Japan. Table 1 shows working stresses due to the present 20 t trucks, to be compared to the allowable stresses specified for the original design against 13 t trucks. The working stresses are a little larger than the original allowable stresses, but this has to be discussed from the point of the actual working stresses. Traffic surveys from 1965 to 1977 showed the average number of passing vehicles on up and down lanes were approximately 30,000 per day regardless of the year. The reason for such uniform numbers is the existence of traffic crossings at the both bridge ends which could control the running of heavy vehicles, resulting in the saturation of traffic in the daytime. Fig. 2 gives the result of traffic frequency surveys carried out in 1970. It shows that the maximum actual working stress which could have occurred once per day reaches about 650 kg/cm², but less than 846 kg/cm² which is a design stress due to 20 t live load. It is also known that the occurrence frequency of a stress equivalent to 20 t truck can be estimated about 0.1 number of vehicle per day. Therefore, if it were assumed that the traffic volume and the loading pattern didn't change, the number of occurrence of 20 t live load stress would have probably been less than 1000 within 25 years.

Table 1 Stresses due to 20 t Live Load (kg/cm²)

| Member | Stress | Original Allowable Stress | Working Stress | | Total Stress |
|--------------|---------------|---------------------------|------------------|------------------|--------------|
| | | | Dead Load Stress | Live Load Stress | |
| R.C. Slab | Concrete | 50 | 4 | 52 | 56 |
| | Reinforcement | 1200 | 92 | 1225 | 1317 |
| Steel Girder | | 1300 | 554 | 846 | 1400 |

4. STATIC STRENGTH AND FATIGUE TESTS

Various tests were done or are in progress for full-size specimens cut out of the original composite welded girders.

4.1 Material properties tests

No clear sign to show deterioration or drop of strength was observed at the compressive strength and carbonation of concrete, the tensile strength of steels and reinforcements, and at the push-out strength of shear connectors, compared with the original tests carried out in 1953. A shear connector was composed of a steel block of $40 \times 40 \times 150$ mm and an inclined ring-shaped steel bar of 12 mm diameter (Photo. 3).

4.2 Flexural strength tests of single-web Composite girders spanning 11.50 m (Photo 4)

Load versus strains, deflections, and slips were measured for two test girders (T-1 and T-2) up to their failure. Fig. 3 shows the relations of loads with deflections at the span center. Generally, the calculated strength of a girder is located a little below its measured strength, but the calculated strengths of T-1 and T-2 girders are a little above the measured values. Such reduction of the measured values shows the lowering of the flexural rigidities of the girders, probably due to existing cracks in their slabs.

4.3 Flexural strength tests of two-web (π -Type) composite girders spanning 11.5 m (Fig. 4)

Since a width-center loaded girder (π -1) collapsed due to punching shear, its failure strength was almost equal to that of the single-web compos-

Photo. 4 Test of Single-Web Composite Girder in Bending³⁾

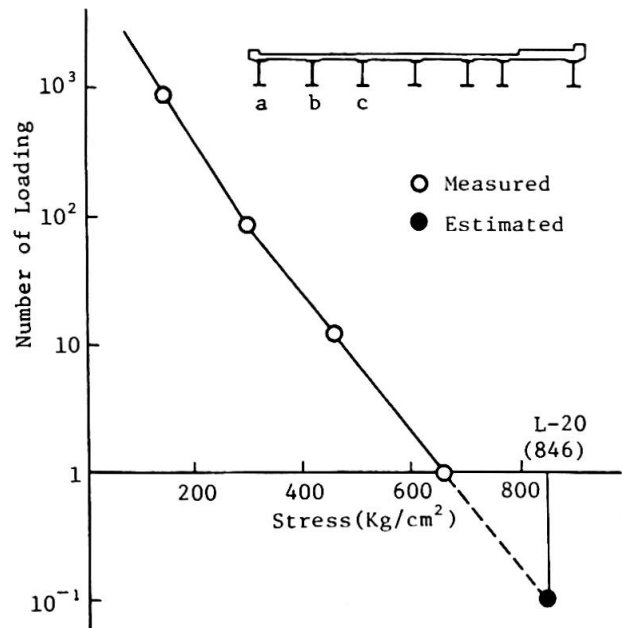


Fig. 2 Load Frequency versus Stress (Girder "C")

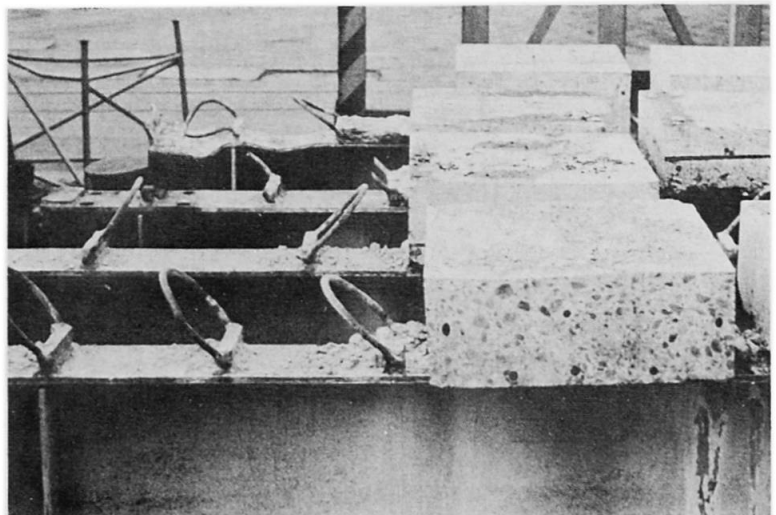
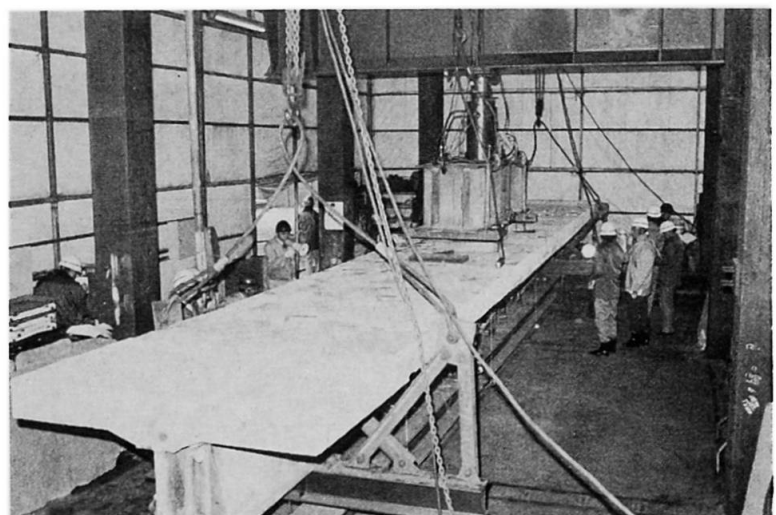


Photo. 3 Shear Connectors



ite girder. In fact, the effect of bending action of the slab on the load-carrying capacity of the π -1 girder was hardly observed. At a width-off-center loaded girder (π -2) (Photo. 6), cracking started on the upper surface of the slab at the load of 45 t. Then, the cracks were propagated in a fan-shape at the load of 80 t and finally, the failure occurred at the load of 125 t due to the development of large cracks around the loading plate. Fig. 5 shows the relations of load with vertical deflection, in which an increase in the deflection of K point is remarkable at about 80 tons. It is noticed that the width-off-center loading governs the ultimate strength of the composite girder more severely than the center loading. Photo. 7 shows the failure of the girder.

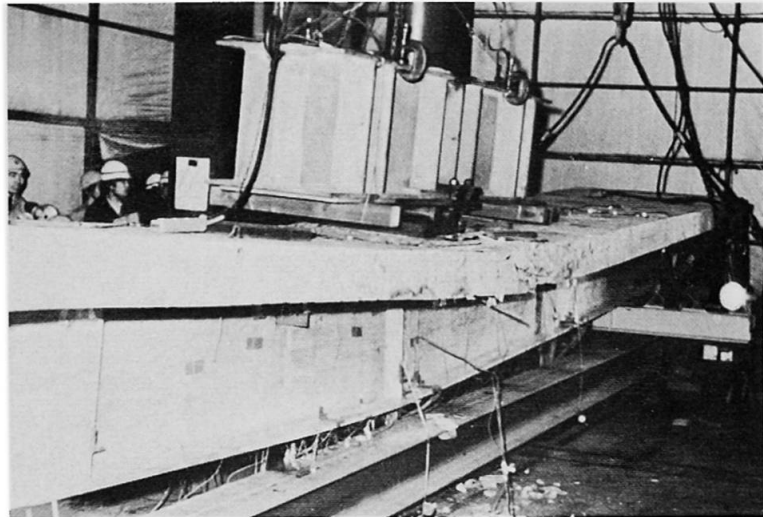


Photo. 5 Failure of Single-Web Composite Girder

4.4 Fatigue tests

The fatigue tests of steels, slabs and a single-web composite girder were carried out. They didn't show any decrease in their fatigue strength in terms of S-N curves even after 25 years' use, compared with their standard fatigue strengths, except the lowering of the S-N curve of a butt-welded joint of a tensile flange. Other fatigue tests of shear connectors and two-web composite girders are still going on.

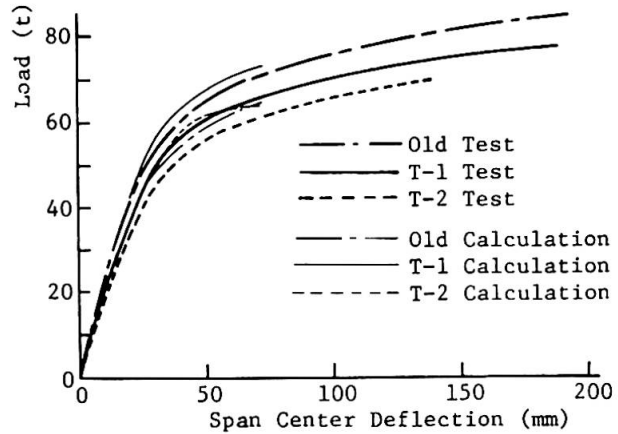


Fig. 3 Load versus Span Center Deflection

5. CONCLUSION

Cracks of 0.1 to 0.3 mm width were observed on the surface of concrete slab of the actual bridge at intervals of about 1.0 m and they went through the slab to its back side. So, a considerable decrease in the static and fatigue strengths of the composite girders was expected before the tests. The various tests, however, didn't show any par-

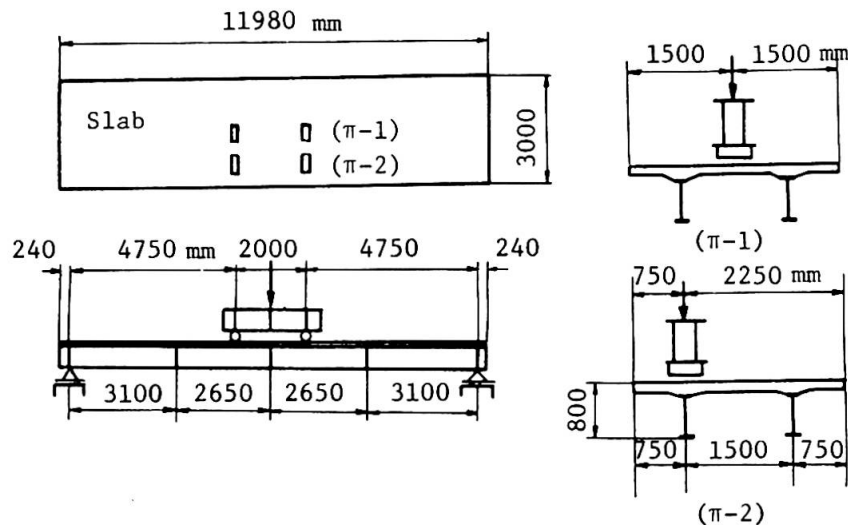


Fig. 4 Test Set-Up of π -Type Girders

particular sign of the reduction of strengths, although the ultimate strengths of the girders seem to be generally somewhat lower due to the existing slab crackings. The soundness of the girders even after 25 years' service is a great surprise, which could be explained as follows:

- The spacing 1.5 m of main girders is quite small compared with the present conventional design in Japan.
- At the original construction the quality of concrete was excellent and the works was done carefully.
- Through 25 years, maintenance works were made fairly well.
- Traffic crossings at the both ends of the bridge didn't allow the fast running of heavy trucks, resulting in keeping reserved strength against the overload 1.5 times as large as the original design live load. Fortunately, the authors could investigate what happened with steel-concrete composite girder bridges which experienced 25 years' service. In Japan, reinforced concrete slabs of highway bridges have been suffering from damages due to the propagation of cracks, but more careful design and construction would guarantee more than 25 years' life of bridges under controlled traffic.

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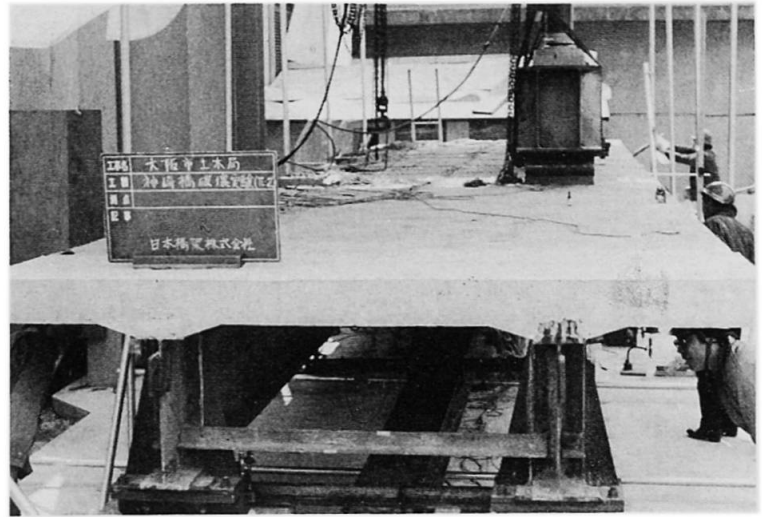


Photo. 6 Test of Two-Web Composite Girder

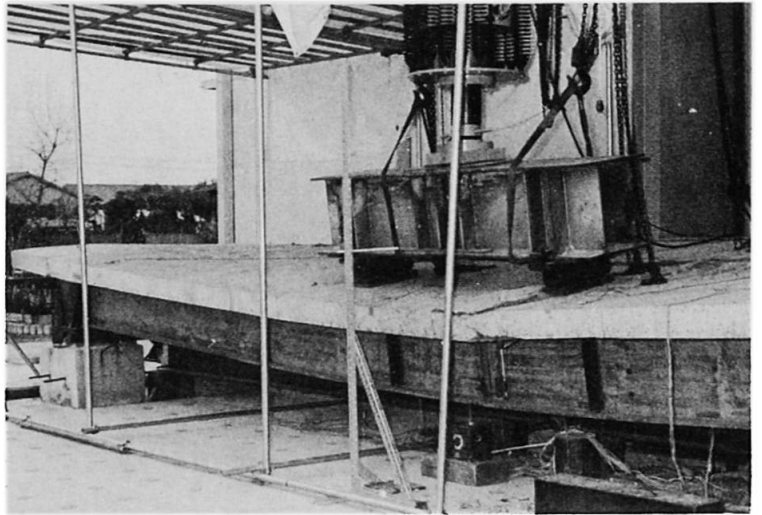


Photo. 7 Failure of Two-Web Composite Girder

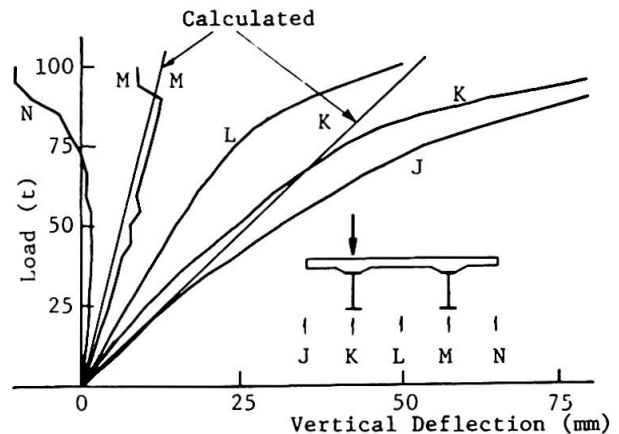


Fig. 5 Load versus Vertical Deflection