Fatigue Testing of Two Full-Size Pre-Cracked AASHTO Bridge Girders

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Synopsis:

This paper presents the results of an investigation on the fatigue performance of two full-size pre-cracked prestressed concrete bridge girders. One AASHTO Type III girder and one AASHTO Type V girder were tested under 1,000,000 cycles of repeated service load intermingled with 2,500 cycles of repeated overload. The behavior of the girders was monitored after each 200,000 cycles of service load as well as each 500 cycles of overload. At the end of the fatigue tests, the girders were tested to failure to determine their ultimate strengths.

The test results demonstrated that the fatigue loadings had virtually no effect on the girder behavior. The girders showed no degradation in stiffness or strength after 1,002,500 cycles of fatigue loading. Both girders showed considerable ductility, and their ultimate loads and maximum deflections exceeded the predicted values.

Keywords: bridge; cracking; cyclic load; fatigue; fatigue strength; fatigue testing; girder; prestressed concrete; repeated load

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INTRODUCTION

During the production of large long-span prestressed concrete bridge girders in the pre-tensioning plant, there is an inherent production problem that often causes two or more transverse cracks within the middle third of the girder. The cracks would start at the top flange of the girder and extend downward through the web, and often into the bottom flange of the girder.

In a previous study by Zia and Caner (5), (6), it was determined that just before de-tensioning the large tensile force in the prestressing strands restricts the contraction of the concrete girders in their initial cooling period. This restrained thermal contraction causes tensile stresses in the concrete girders, large enough to exceed the tensile strength of the concrete at its early age, thus causing the transverse cracks. Upon de-tensioning, the cracks would be closed and become virtually invisible. Zia and Caner also observed that given time and moisture, the cracks would heal and the concrete would regain nearly its full compressive strength and most of its tensile strength.

However, the degree of concrete healing is uncertain, and bridge engineers have been concerned about the long-term behavior and strength of the girders impaired by the transverse cracks. Accordingly, this investigation was undertaken to assess the fatigue performance of two full-size pre-cracked AASHTO prestressed concrete girders.

RESEARCH SIGNIFICANCE

This research provides much needed data on fatigue performance of full-size prestressed concrete bridge girders, especially with pre-existing cracks. Such information is valuable to bridge designers when they consider the fatigue resistance of such girders. The research also demonstrates the validity of the AASHTO design requirement for stress limit under fatigue loading.

EXPERIMENTAL PROGRAM

Test Specimens

Two AASHTO girders were tested under fatigue loadings. One was a Type III girder of 19.95 m (65 ft. 5 ½ in.) long and the other was a Type V girder of 19.8 m (65 ft.) long. The Type III girder (Fig. 1) was prestressed with thirty-four 12.7 mm (1/2 in.) low-relaxation strands, each tensioned initially to 138 kN (31 kips). Twenty-two of the strands were straight, and the remaining 12 strands were draped with hold-down points at 1.85 m (6 ft. 1 in.) on each side of the mid-span. In the top flange of the girder, there were two additional straight strands initially tensioned to 13.3 kN (3 kips) each. These two strands were used primarily to support the stirrups and help control cracks, and were not considered as active prestressing elements.

The Type V girder (Fig. 2) was prestressed with thirty-six 12.7 mm (1/2 in.) low-relaxation strands, each with initial tension of 138 kN (31 kips). All the strands were straight with 32 strands located in the bottom flange and 4 strands located in the top flange ("B" Strands). Also located in the top flange were two additional straight strands ("A" Strands) initially tensioned to 4.45 kN (1 kip) to support stirrups and help control cracks. Again, these two strands were not considered as active prestressing elements.

Both girders showed transverse cracks when they were delivered to the laboratory. Figs. 1 and 2 show the cross-sections of the two girders. Other details on the cross-sectional properties and material characteristics of the two test specimens are described elsewhere by Zia, et al. (7). The compressive strengths of the concrete for the two girders are given in Table 1.

Test Procedure

Both girders were simply supported by elastomeric bearing pads and a single point load was applied at mid-span by a MTS actuator with a 1,984 kN (446 kips) fatigue capacity and 1,016 mm (40 in.) stroke. The span was 19.6 m (64 ft. 4 in.) for the Type III girder and 19.4 m (63 ft. 8 in.) for the Type V girder. The girders were tested without a cast-in-place deck.

Initially, each girder was loaded statically until flexural cracks were observed in the bottom flange of the girder, and the load-deflection data were recorded. Then the fatigue service load was applied to the girder for a total of 1,000,000 cycles. After each 200,000 cycles of service loading, a fatigue overload of 500 cycles was applied to the girder, for a total of 2,500 cycles of overload. Following these fatigue loadings, the girder was tested to failure for its ultimate strength. At the end of each segment of these fatigue loadings (service load as well as overload), a static test was conducted to obtain the load-deflection data in order to assess the behavior of the girder. In addition, the crack development at the various loading stages was also monitored.

The limits of each load cycle of the fatigue loadings are given in Table 2. For the Type III girder, the lower load limit would produce the same moment at the mid-span by a composite deck of 2.44 m (8 ft.) wide and 203 mm (8 in.) thick. Similarly, the lower load limit for the Type V girder would produce the same moment at the mid-span by a composite deck of 2.7 m (9 ft.) wide and 213 mm (8 1/2 in.) thick.

For both girders, the upper load limit of the fatigue service load would produce a nominal bottom fiber stress of $0.25\sqrt{f_c'}$ MPa ($3\sqrt{f_c'}$ psi) at the mid-span of the composite girder if a concrete deck had been cast on the test girder. The stress is the design value under service load used by NCDOT. Since the girders were tested without a cast-in-place deck, the actual nominal bottom fiber stress was $0.27\sqrt{f_c'}$ MPa ($3.24\sqrt{f_c'}$ psi) for the Type III test girder and $0.26\sqrt{f_c'}$ MPa ($3.15\sqrt{f_c'}$ psi) for the Type V test girder. So the actual nominal stress for the two test girders was 8% and 5%, respectively, higher than the design value.

Also for both girders, the upper load limit of the fatigue overload would represent the effect of over-weight vehicles allowed to use a bridge with special permit issued by NCDOT, which is based on 75% of the ultimate strength of the girder with a composite deck. Under this fatigue overload, the actual nominal bottom fiber stress was $1.6\sqrt{f_c'}$ MPa ($19.23\sqrt{f_c'}$ psi) for the Type III test girder and $0.94\sqrt{f_c'}$ MPa ($11.34\sqrt{f_c'}$ psi) for the Type V test girder. As expected, these stresses were significantly higher than the stresses under the fatigue service load.

ANALYTICAL STUDIES

Analytical studies were also conducted to model the behavior of the test girders by using two separate computer programs, one called *Cracked Beam* and the other *Response 2000*. The former was developed by Ellen (3) and Longo (4) using Microsoft Excel and the latter was developed by Bentz (2). The analytical results are presented below for comparisons with the test results. Other details on the analytical studies and the computer programs can be found in reference (7).

RESULTS

Tests for Initial Cracking

Figs. 3 and 4 show the selected load-deflection curves of the two girders after different loading cycles. It is noted in both figures that the ultimate load test included loading and unloading several times. From both figures, it can be seen that for the initial static test (zero cycle) that was conducted to determine the flexural cracking load, both girders behaved elastically before cracking. After the flexural cracking load was determined, the girders were unloaded and reloaded in order to determine the load at which the flexural crack reopens. Based on the slope of the load-deflection curve, the initial cracking load, and the load at crack reopening, the girder stiffness E_cI, the elastic modulus E_c and the flexural modulus f_r of the concrete, as well as the prestress loss for each girder were calculated. These results are summarized in Tables 3 and 4.

It can be seen from Table 4, the elastic modulus E_c determined from the slope of the load-deflection curve is quite close to the value predicted by the ACI Code using the 28-day concrete compressive strength. On the other hand, the flexural modulus f_r obtained from the test differs much more from the predicted value by the ACI Code.

Regarding the loss of pretress, the magnitude is quite small. Since the girders were tested not too long after they were cast, so the losses due to creep and shrinkage would be negligible. The values given in Table 4 would represent mostly the loss due to elastic shortening.

Fatigue Test

During the fatigue test under service loading, crack development was very limited since the upper limit of the service load cycle was lower than the maximum load applied to the girder during the initial static test. However, the first 500 cycles of overloading caused much more significant cracking because the upper limit of the overload cycle was substantially higher than the maximum load applied to the girder during the initial static test. For all subsequent service load and overload cycles, the effect on crack development was gradual and minimal.

At the end of the fatigue test of the Type III girder, cracks were observed in the lower part of the girder within the central region of 5.8 m (19 ft.). The maximum crack length was 546 mm (21.5 in.), extending well into the web of the girder. The maximum crack width under a load of 667 kN (150 kips) was 0.33 mm (0.013 in.) and the average crack spacing was 140 mm (5.5 in.) on the side of the girder.

For the Type V girder, cracks were observed within the central part of 4.12 m (13 ft.) of the girder at the end of the fatigue test. The maximum crack length was 1194 mm (47 in.) near the midspan. The maximum crack width was 0.64 mm (0.025 in.) under a load of 1,130 kN (254 kips) and the crack spacing ranged from 203 to 254 mm (8 to 10 in.)

As seen from Figs. 3 and 4, all the load-deflection curves for each test girder are virtually parallel in the elastic range, which indicates that there was no stiffness degradation after the girder was subjected to 1,000,000 cycles of service load and 2,500 cycles of overload. In addition, in each figure, the continuous shift of the load-deflection curves from the origin suggests that there was a gradual, but permanent reduction in the camber of the girder.

Ultimate Load Test

Under the ultimate load test, the load-deflection curves for both girders as shown in Figs. 3 and 4 indicate that initially each girder maintained its original stiffness. However, as more extensive cracking developed in the girder under increasing load, there was a gradual reduction in the girder stiffness.

For the Type III girder, as the ultimate load of 907 kN (204 kips) was approached, the girder deflection was 132 mm (5.19 in.). Since the girder was tested without a composite deck slab, its failure mode was sudden and explosive. Failure occurred primarily from crushing of concrete in the compression zone on one side of the steel loading plate. As soon as the compression zone was lost, the large force in the prestressing strands put the remaining concrete area in the web and the bottom flange under a very high compressive stress which, in turn, caused concrete crushing in those areas as shown in Fig. 5.

A similar behavior was observed for the Type V girder. As the ultimate load was reached at 1,680 kN (377 kips), the top flange failed in compression at about 0.91 m (3 ft.) from the midspan. Immediately following the crushing of the top flange, the web also crushed explosively. A view of the girder after failure is shown in Fig. 6.

The behavior and the strength of the two girders were analyzed by the two computer programs, *Cracked Beam and Response 2000.* The comparisons between the experimental and the analytical results are shown in Tables 5 to 7. Similarly, the experimental and predicted load-deflection curves are compared as shown in Figs. 7 and 8.

It is worth noting that, as shown in Table 6, *Cracked Beam* predicted slightly higher stress in the bottom layer of strands than *Response 2000.* This is because the tensile stress in concrete was neglected in *Cracked Beam* analysis, whereas it was considered effective in the analysis by *Response 2000.* Therefore using *Cracked Beam* would be more conservative.

Also from Table 6, it can be seen that based on *Cracked Beam* the stress range in the bottom layer of strands for the Type III girder was 134 MPa (208.2 ksi – 188.7 ksi = 19.5 ksi) under the fatigue service load, and 202 MPa (218 ksi – 188.7 ksi = 29.3 ksi) under the fatigue overload. Similarly, the stress range in the bottom layer of strands for the Type V girder was 97 MPa (197 ksi – 183 ksi = 14 ksi) under the fatigue service load, and 269 MPa (222 ksi – 183 ksi = 39 ksi) under the fatigue overload.

According to the provision (Section 5.5.3.3) of the AASHTO LRFD Bridge Design Specifications (1), the limit on the stress range in straight strands subjected to fatigue loading is 124 MPa (18 ksi). Therefore, the two test girders were subjected to similar stress ranges under the fatigue service load, but with substantially higher stress ranges under the fatigue overload.

CONCLUSIONS

1. After 1,000,000 cycles of fatigue service load and 2,500 cycles of fatigue overload, both girders showed no degradation of stiffness and strength. No prestressing strand in either girder showed any signs of fatigue or failure. The measured and calculated deflections demonstrated that the ductility of the girders was not affected by the fatigue loadings.

2. For both girders, cracking and permanent deflection progressively increased with each segment of 500 cycles of fatigue overloading.

3. Based on the initial cracking load from test, the calculated values of prestress loss, the modulus of elasticity of concrete, and the flexural modulus of concrete were reasonably accurate.

4. The analytical results from both computer programs were sufficiently accurate in predicting the structural performance of the girders. In general, the predictions made by *Cracked Beam* were more conservative than the predictions made by *Response 2000*.

5. The research demonstrated that the limit of 124 MPa (18 ksi) on the stress range in strands subjected to fatigue loading as specified by AASHTO is a suitable design criterion.

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Table 1 Compressive Strength of Concrete

Test Specimen	Compressive Strength (psi)		
	1 day	7 day	28 day
Type III Girder	5,476	6,962	7,698
Type V Girder	4,572	7,293	9,439

Note: 1 MPa = 145 psi or 1 ksi = 6.895 MPa

Table 2 Load Limits of Fatigue Loadings

Test Specimen	Fatigue Service Load (kips)	Fatigue Overload (kips)
Type III Girder	26 to 107	26 to 152
Type V Girder	30 to 186	30 to 254

Note: 1 kip = 4.448 kN

Table 3 Girder Stiffness, CrackingLoad, and Load at Crack Reopening

Test Specimen	Girder Stiffness E_cI (in 10^6 kips-in^2)	Cracking Load (kips)	Load at Crack Reopening (kips)
Type III	731	134	114
Type V	2,695	208	140

Note: 1 kips-in^2 = 2,870 kN-mm^2 1 kip = 4.448 kN

Table 4 E_c, f_r, and Prestress Loss

Test Specimen	E_c (in 10^3 ksi)		f_r (in psi)		Prestress Loss
	Test	ACI	Test	ACI	
Type III	5.83	5.39	842	658	9%
Type V	5.18	5.93	526	729	10.4%

Note: 1,000 ksi = 6.895 GPa or 1 MPa = 145 ksi