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Prestressed Bridge Girders after 20 Years of Service

Poutres en béton précontraint après 20 ans de service

Vorgespannte Brückenträger nach 20 Jahren Beanspruchung

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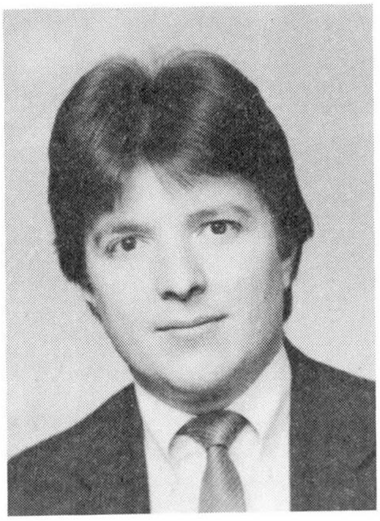

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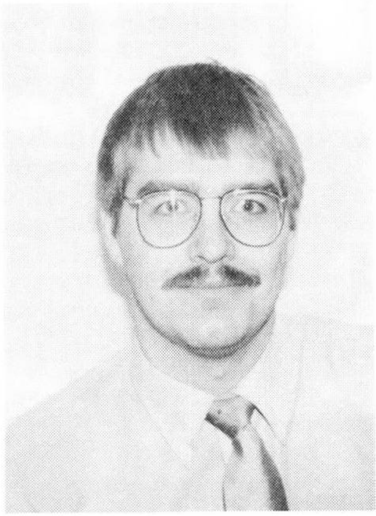

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SUMMARY

The prestress losses, material properties, fatigue, and structural performance characteristics of four prestressed bridge girders are described. The girders had been in service under actual traffic conditions for twenty years, and showed little corrosion or durability problems. Prestress losses with time were approximately equal to those predicted by current American codes.

RÉSUMÉ

Les pertes de précontrainte, les propriétés des matériaux, la fatigue, et les caractéristiques structurales de quatre poutres de pont en béton précontraint sont décrites. Les poutres ont été en service sous des charges de trafic durant vingt ans, et n'ont pas eu de problèmes de corrosion ou de durabilité. Les pertes de précontrainte avec le temps ont été pratiquement égales à celles prescrites par les codes américains actuels.

ZUSAMMENFASSUNG

Die Vorspannungsverluste, Materialeigenschaften, Ermüdung und das konstruktive Verhalten vier vorgespannter Brückenträger sind beschrieben. Die Träger waren für ca. 20 Jahre unter Verkehrslasten in Gebrauch und zeigen nur geringe Korrosions- oder Gebrauchstauglichkeitsprobleme. Die Vorspannungsverluste im Verlauf der Zeit entsprechen etwa den Werten der zur Zeit gültigen amerikanischen Normen.

1. INTRODUCTION

The determination of the actual material and structural properties is a key step in the rating of structures. In the case of most steel bridges the assessment of the material properties is quite straight forward. This is not the case, however, for concrete and prestressed concrete bridges where significant time effects are involved. In prestressed concrete bridges, in particular, loss of prestress due to relaxation, creep, shrinkage, and cyclic loads is very difficult to predict. In this paper the results of a study on four prestressed girders subjected to real traffic for twenty years is reported. The main objectives of the study were:

(1) To determine the actual prestress losses of the girders and to compare them to those given by current code-prescribed equations.
(2) To assess the remaining fatigue life of such girders.
(3) To determine the material properties of the girders and components and to compare them with non-destructive test results.
(4) To determine the amount of impact damage that such a girder can sustain before replacement or repair is required.
(5) To assess the performance of two types of strand repair techniques subjected to fatigue loading.

Due to space limitations only the results of the first two girder tests will be addressed with respect to items (1)-(4).

2. DESCRIPTION OF THE SPECIMENS

In 1986 a four-span county road bridge over an interstate highway in Minneapolis, Minnesota, was removed due to road realignment work. Four of the girders removed from the center spans of this bridge were brought to the University of Minnesota Civil and Mineral Engineering Structures Laboratory for testing. The girders were originally fabricated in July of 1967. At the time of removal they had been in service for approximately twenty years.

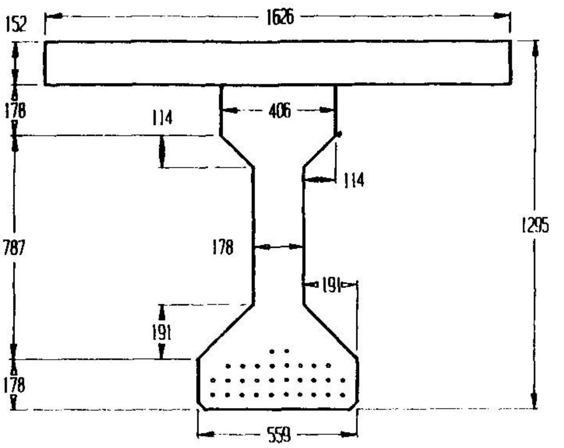

Figure 1 - Girder dimensions (in mm)

The girders are standard AASHTO-PCI Type III girders, and details of the girders are shown in Figure 1. The girders were 1143 mm. deep, 19.71 m. long, prestressed with thirty 13 mm. diameter 1724 MPa stress-relieved strands. Twenty-two of the strands had a straight profile in the bottom flange of the girders. The remaining eight strands were draped; the two hold-down points were located 1525 mm. either side of centerline. The strands were initially stressed to a design prestressing level of 1206 MPa. The construction records indicate that the girders were pulled to approximately 2 percent over this design prestress.

Each girder was tested individually, with a new slab cast to simulate the actual bridge. The new slab was 152 mm. thick, 1626 mm. wide, and reinforced in the same manner as the original bridge slab. The loads were applied by two actuators located at the hold-down points, resulting in a constant moment region over the middle 3025 mm of the girder.

3. PRESTRESS LOSSES

The girders arrived at the laboratory in an apparently uncracked condition. The only visible damage was located at the ends of the girders: a small amount of epoxy paint covering the ends of the strands had spalled and rust was evident on the ends of the strands. The first loading applied to the girders was a cracking cycle. This test was repeated several times to accurately determine the crack opening load. The measured load at first cracking was 378 kN, and the loading was continued to 623 kN, about 45% of its calculated ultimate capacity. Small cracks propagating up to 620 mm from the bottom fiber in the constant moment region were the only visible damage due to these loadings. Four techniques were used to estimate the prestress losses:

(a) Formation of the first crack: From the load at first cracking and tensile capacity of the concrete from cores, the prestressing can be estimated.
(b) Reopening of cracks: By carefully monitoring the opening of cracks with high-resolution LVDT's during reloading, the decompression load can be calculated.
(c) Discontinuities of the load-deflection curve: The change of stiffness when the cracks reopen can be obtained directly from load-deflection curves.
(d) Exposing, instrumenting, and cutting strands: By using strain gages, a direct measurement of the prestress level can be obtained.

While it is difficult to state the initial prestress because the construction documents are the only source available, the in-situ prestress levels as given by methods (a) to (c) were very close to one another. They indicated a remaining prestress of 895 MPa in the strands, or about 74% of the original prestressing. Method (d) gave somewhat lower values (64%), but this can be explained by slight misalignment of the gages, transfer length and differences in prestress from strand to strand. The total prestress losses calculated were about 310 MPa, very close to the lump sum prestress losses predicted by the current AASHTO specification [1].

4. FATIGUE LOADING

The complete load histories imposed on the two girders are shown in Table 1. The load history was selected to model past and current American bridge specifications [1] for prestressed girders. They are based on the bottom fiber stress. The oldest specifications allowed no tension there, while more recent editions allow between 0.8 f'_c and 1.6 f'_c, where f'_c is the compressive strength in kgf/cm^2.

Girder 1 - Following the static tests (G1PL) to investigate prestress losses, the girder was subjected to almost three million cycles of load at increasing levels of nominal tensile stress in the bottom fiber. The corresponding stress ranges in the strands for fatigue tests G1F1 through G1F4 were 55, 69, 90, and 207 Mpa, respectively. Only a small crack growth was noticed during the initial cycling for G1F1 and G1F2; in general cracks stabilized with the first 10 x 10^4 cycles at each level. Most new cracks developed during the static tests conducted intermittently to monitor the amount of damage accrued during from fatigue. Figure 2 shows the load-deflection curves after each of the loading runs. Very little, if any, fatigue damage was evident after this very severe load history. There was noticeable permanent set in the specimen only after loading G1F4.

Girder 2 - After an initial series of static tests (G2PL), the beam was cycled for two million cycles (G2F1 and G2F2) in its undamaged state. The concrete in the bottom flange at the centerline was then chipped away to expose four strands to simulate damage due to impact from a truck travelling underneath the bridge (G2D1). The beam was then subjected to fatigue loading (G2F3). Two bottom strands were then cut to simulate additional damage (G2D2), and the girder was fatigued again (G2F4). After this two more strands were cut (G2D3) and the fatigue load repeated (G2F5 and G2F6). One strand broke during G2F4, and at least another during G2F5. Figure 3 shows the load-deflection curves for static tests after each of these runs.

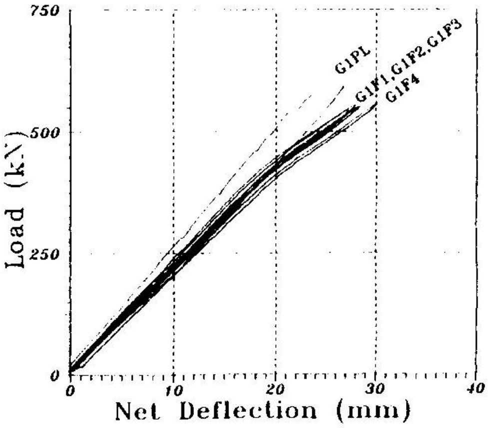

Figure 2 - Load-deflection for G1

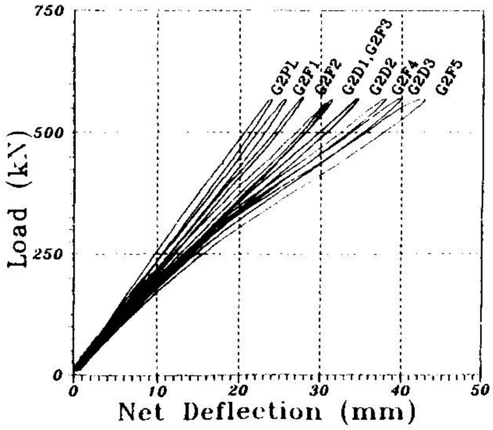

Figure 3 - Load-deflection for G2

The results of the tests on the first two specimens indicated that fatigue loading which produces strand stress ranges of less than 104 MPa has little or no effect on the ultimate strength and ductility of the member. The size of these girders and their close spacing in the field means that very little damage could have been done by fatigue since the sections were uncracked during their service life. The stress range in service was probably less than 20 Mpa.

TABLE 1 - Load Histories

Girder Number	Test Label	Load Type	Number of Cycles (N)	Bottom Fiber Stress	Purpose of Test
1	G1PL	Static	10	$3.2\sqrt{f'_c}$	Determine cracking load and losses
1	G1F1	Cyclic	5×10^5	0	Fatigue
1	G1F2	Cyclic	10×10^5	$0.8\sqrt{f'_c}$	Fatigue
1	G1F3	Cyclic	12×10^5	$1.6\sqrt{f'_c}$	Fatigue
1	G1F4	Cyclic	6×10^4	$3.2\sqrt{f'_c}$	Fatigue
1	G1U	Static	1	---	Test to ultimate
2	G2PL	Static	10	$3.2\sqrt{f'_c}$	Determine cracking load and losses
2	G2F1	Cyclic	5×10^5	$0.8\sqrt{f'_c}$	Fatigue
2	G2F2	Cyclic	15×10^5	$1.6\sqrt{f'_c}$	Fatigue
2	G2D1	Static	5	$1.6\sqrt{f'_c}$	Concrete around strands removed
2	G2F3	Cyclic	5×10^5	$1.6\sqrt{f'_c}$	Fatigue
2	G2D2	Static	5	$1.6\sqrt{f'_c}$	Cut two strands
2	G2F4	Cyclic	5×10^5	$1.6\sqrt{f'_c}$	Fatigue
2	G2D3	Static	5	$1.6\sqrt{f'_c}$	Cut two strands
2	G2F5	Cyclic	18×10^3	$1.6\sqrt{f'_c}$	Fatigue
2	G2U	Static	1	---	Test to ultimate

5. ULTIMATE STRENGTH TEST

After approximately three million cycles, both Girder 1 and 2 were tested monotonically to failure (Figure 4). In both cases the failure was initiated by the upward buckling of the longitudinal slab reinforcement at very large centerline deformations (530 mm or greater). The final failure occurred as crushing of the slab followed by an explosive outward failure of the poorly confined beam web. For Girder 1, the failure occurred at a load of 1303 kN and at a centerline deflection of 530 mm. The load at ultimate constituted approximately 95 percent of the ultimate capacity of the beam based on nominal material properties and the assumption that all steel yielded. By the time failure was reached the entire 10 ft at the center of the beam had formed a long plastic hinge, and inclined shear cracking had moved out from the constant moment region to the quarter points in the beam. The inclined shear cracks in this area had reached the top flange of the beam, while flexural cracking had progressed to the bottom of the slab. The fatigue loading did not appear to have affected the ultimate strength of the section.

The failure for Girder 2 was similar, except that it occurred at a much lower load (890 kN) and somewhat higher deflection (635 mm.). This was due to: (1) the cutting of the strands and loss of concrete section and (2) fracture of at least two additional strands during G2F4 and G2F5. The cracking in Girder 2 during the last three loading runs was very severe, with large portions of the bottom flange completely separated from the girder.

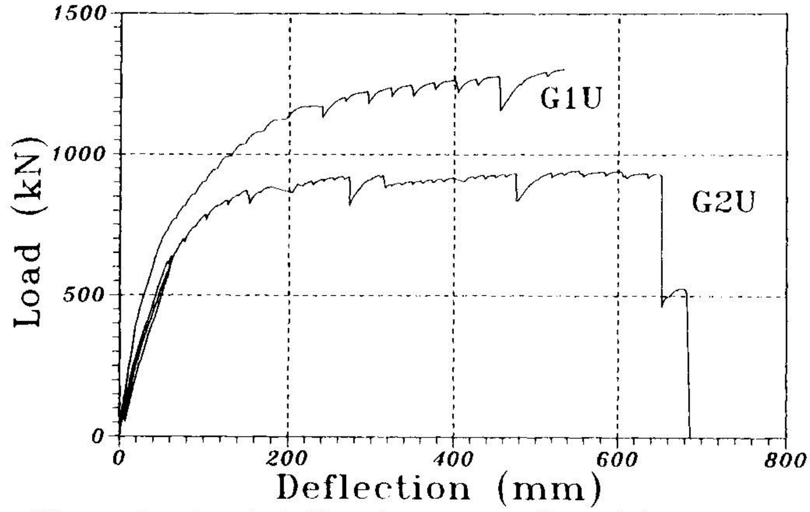

Figure 4 - Load-deflection curves for ultimate tests.

6. MATERIAL PROPERTIES

A series of nondestructive tests was conducted on Girder 1. The tests included a variety of uniformity/strength tests as well as chloride ion penetration investigations. Windsor probe (penetration test), Schmidt hammer (surface hardness), pulse velocity (compressive wave velocity) and breakoff tests (lateral pressure required to detach countersunk cylinder) were conducted and were correlated with 50 mm. and 100 mm. diameter cores drilled from the girder. In these tests, the concrete was found to be quite uniform throughout the girder with a compressive strength of about 58 Mpa. THe results showed a low coefficient of variation for all of the test methods with the exception of the break-off test and the 100 mm. cores. Problems with obtaining straight cores and good capping of the specimens help explain this variability.

Because the girders had been exposed to substantial amounts of deicing salts as spray from the interstate highway underneath, corrosion of the strands was considered a major potential hazard. The girders themselves received little or no deicing salts from above, as the original deck acted as a barrier. The top steel in the original deck was quite corroded, but the chlorides had not penetrated down to the girders. The strands which were exposed to check the effective prestress of Girder 1 were located in the flange near the end of the girder in an uncracked region. Evidence of some pitting corrosion appeared on one of the strand within 100 mm. of the end of the girder. Otherwise, the strands, which had a cover of 50 mm., appeared to be in excellent condition. Chloride ion penetration tests gave readings which were within the threshold limits of 250-350 ppm by weight of concrete usually assumed as the corrosion threshold. The highest reading obtained was 270 ppm, but the next highest readings were on the order of 120 ppm. It is interesting to note that the readings obtained from the bottom flange were consistently higher than those obtained from the web, and the readings obtained on one side of the girder were consistently higher than those obtained from the other side. It is expected that the girders which were exposed to the incoming traffic would have higher readings because the salt-concentrated mist tends to be carried under the bridge with the forward motion of the cars. Consequently, the side of the girder with the higher concentration of salts most likely faced the incoming traffic.

7. SUMMARY

The tests carried out so far indicate that the prestressed girders were in excellent condition after twenty years of service in an aggressive environment. There did not appear to be problems associated with corrosion of the girders, most likely because of the excellent concrete quality and the depth of cover. The prestress losses, estimated to be on the order of 310 Mpa, correlated well those predicted by the current AASHTO lump-sum method. The fatigue loading imposed indicated that for loadings up to HS20-44 or Type 3S2 vehicles the stress range was probably in the infinite life region of the fatigue curves. Thus the girders could be reused in other bridges if they are removed with care and NDT techniques indicate adequate material properties and no evidence of strand corrosion.

ACKNOWLEDGEMENTS

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REFERENCES

[1] American Association of State Highway and Transportation Officials (AASHTO), Standard Specification for Highway Bridges, Washington D.C., 1983.

[2] Leon, R.T., French, C.W., Olson, S.A., and Coggins, F.B., Reusability of 20-Year-Old Prestressed Bridge Girders, Proceedings of a Conference on Bridge Research in Progress, Iowa State University, Ames, Iowa, September 1988, pp. 103-106.