Fatigue Behavior of Prestressed Concrete Bridge Girders Strengthened with Various CFRP Systems

by O.A. Rosenboom and S.H. Rizkalla

Synopsis: Carbon Fiber Reinforced Polymer (CFRP) materials provide a solution for the classical challenge facing bridge maintenance engineers: the upgrading of bridges whose service life has been exceeded but due to evolving demands must stay in service. Of particular concern are rural short span prestressed concrete bridges that may be required to carry loads above the initial design value. The CFRP strengthening systems presented in this paper have the potential to increase the ultimate capacity of such bridge girders. This research project is aimed at investigating the fatigue performance of CFRP strengthening systems for prestressed concrete bridge girders. Five 9.14 meter prestressed concrete bridge girders were tested under fatigue loading conditions: one as a control specimen and four strengthened with various CFRP systems including near surface mounted (NSM) bars and strips, and externally bonded sheets and strips. Results show that CFRP strengthened prestressed concrete bridge girders can withstand over two million cycles of fatigue loading equivalent to a 20 percent increase in live load with little degradation.

Keywords: bridge girder; externally bonded; fatigue behavior; fiber reinforced polymers; flexural behavior; near surface mounted; prestressed; strengthening

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INTRODUCTION

Research objectives

Through examination of various parameters, this research aims to provide the prestressed/precast concrete industry as well as transportation agencies with economical and effective strengthening systems for prestressed concrete using CFRP materials. The feasibility of using carbon fiber reinforced polymers (CFRP) strengthening systems to upgrade the load carrying capacity of 40 year old prestressed concrete bridges is investigated. Although there is an ever-expanding research database of reinforced concrete structures strengthened with different CFRP systems, information on various strengthening techniques for prestressed concrete structures is very limited. The first phase of the research began was an investigation of the static behavior of prestressed concrete bridge girders strengthened with CFRP which can be found elsewhere[1].

Background and literature review

Many bridges in the United States do not conform to current design standards. If they are not strengthened they may need to be replaced to accommodate the increase in load. One of the major concerns to departments of transportation is short-span prestressed concrete bridges in rural areas which have exceeded their design life but due to evolving industry demands may be required to carry loads above the initial design value. CFRP systems have the potential for cost-effective retrofitting of prestressed concrete bridges by increasing the load-carrying capacity thus extending their service life.

The use of externally bonded and NSM CFRP systems to repair or strengthen reinforced concrete beams in flexure has been well researched[2-4]. Takacs and Kanstad[5] showed that prestressed concrete girders could be strengthened with externally bonded CFRP plates to increase their ultimate flexural capacity. Reed and Peterman[6] showed that both flexural and shear capacities of 30 year-old damaged prestressed concrete girders could be substantially increased with externally bonded CFRP sheets. Reed and Peterman also encouraged the use of CFRP U-wraps as shear reinforcing along the length of the girder in externally bonded systems to delay debonding failure. The use of NSM CFRP in prestressed concrete bridge decks was explored by Hassan and Rizkalla[3] and found to be a viable alternative to externally bonded systems.

Debonding of externally bonded FRP systems has been noted by many researchers often at the termination point of the FRP plate/sheet for members with a short span, and at the midspan section for long span members. Many models have been proposed to predict the failure loads of FRP strengthened reinforced concrete members due to plate-end debonding[7,8], yet the midspan debonding mechanism has not been as extensively researched[9,10]. U-wrap CFRP reinforcement has been recommended for use at the termination point of the main CFRP strengthening system, but the benefits of providing this extra reinforcement throughout the length of the girder is not known[9]. One of the benefits of NSM FRP strengthening is to reduce the propensity for debonding failure. Models to predict this debonding load have been characterized from earlier plate-based work[2].

The fatigue behavior of reinforced concrete beams strengthened with externally bonded CFRP systems has been investigated[11, 12], yet no work has been done on prestressed concrete members strengthened with CFRP and tested in fatigue. The fatigue behavior of prestressed concrete was extensively examined in the 1960's and 1970's[13, 14] with results showing that very little fatigue degradation occurs if the girder remains uncracked. If the fatigue load is above the cracking load and the stress range in the prestressing strands is high, failure will be due to rupture of the prestressing strands.

EXPERIMENTAL PROGRAM

Test girders

As part of a research program funded by the North Carolina Department of Transportation, ten prestressed concrete C-Channel type bridge girders were tested at the Constructed Facilities Laboratory at North Carolina State University. Five of the girders were tested under fatigue loading conditions: one as a control specimen (F0), two strengthened with near surface mounted CFRP bars (F1) and strips (F2) and two strengthened with externally bonded CFRP strips (F3) and wet lay-up sheets (F4). Five identical girders were tested under static loading conditions and details of the results of these experiments can be found elsewhere [1]. Table 1 shows the properties of the CFRP systems used determined from coupon tests as well as the concrete strength found from the testing of core samples. All girders were 9.14 m long C-Channel type prestressed concrete bridge girders (see Fig. 1). The girders were taken from a decommissioned bridge in Carteret County, NC, USA, which was erected in 1961. Each girder was prestressed with ten 1725 MPa seven-wire stress relieved, 11 mm prestressing strands (five in each web) and had a 125 mm deck with minimal reinforcing. The measured camber of the girders at midspan, due to prestressing, ranged from 32 to 38 mm. Details of the various strengthening systems are provided in Figure 1.

Design of strengthened girders

The design of the strengthened girders proceeded after testing the control girder under static loading conditions[1]. The objective of the strengthening was to achieve a 20 percent increase in the ultimate load carrying capacity with respect to the control girder, except for F4 which was designed for a 60 percent increase in the ultimate load carrying capacity to examine the behavior at a significantly higher range. Each strengthened girder was designed using a cracked section analysis program, Response 2000[15]. For the design, the manufacturer's properties were used to model the FRP materials. The prestressing steel and concrete material properties were taken from the provided specifications.

Flexural failure, defined as rupture of the FRP or crushing of the concrete in compression, was the desired mode of failure. The shear stresses in the concrete at the plate end were significantly lower than the shear strength of the concrete (according to equations proposed by Malek, et al[16]) so debonding at the plate end was not a concern. However, to delay FRP delamination-type failures along the length of the girder, 150 mm wide U-wraps were installed at 900 mm spacing for all externally bonded strengthened girders. This arrangement was selected to simulate typical anchorage details commonly used by the construction industry for reinforced concrete members strengthened with FRP.

Test setup, loading and instrumentation

All girders were tested using a 490 kN MTS hydraulic actuator. The actuator was mounted to a steel frame placed at the midspan of the girder. Typical prestressed concrete bridges constructed in the early 1960s usually have a substructure of wooden piles that is difficult to mimic in the laboratory. However, in order to simulate small displacements at the supports, the girder was supported at both ends on a 64 mm neoprene pad which in turn rested on a 25 mm steel plate. Hydrostone was used at the supports for leveling purposes. The width of the neoprene pad was 216 mm which provided a clear span of 8710 mm for the tested girder.

The behavior of the girders during testing was monitored using three sets of string potentiometers, placed at quarter spans, and two linear potentiometers to measure vertical displacement over the supports. The compressive strain in the concrete was measured using a combination of PI gauges (a strain gauge mounted to a spring plate) and electrical resistance strain gauges located beside and between the loading tires. PI gauges were also placed at the level of the lowest prestressing strand to measure the crack width and determine the strain profile through the section. The tensile strain in the CFRP reinforcement was measured using six strain gauges: two at midspan, two at 150 mm from midspan and two at 305 mm from midspan as shown in Figure 2. A picture of the test setup is provided in Figure 3.

The loading sequence for the tested girders started by increasing the applied load to a load level slightly higher than the cracking load, unloading, and then reloading again at a rate of 2.5 mm/min up to the load at which the crack at midspan reopens. This loading sequence was selected to determine the effective prestressing force in the girders by observing the re-opening of the flexural cracks[17]. The girders were then cycled between two load values at a frequency of 2 Hz. The dead load for all the girders was the same, 8.9 kN. The live load used for the control specimen was 40 kN. This was based on the service load the original girder was designed for (HS15 type loading) and includes the appropriate distribution and impact factors. For three of the strengthened girders tested in fatigue (F1, F2, F3), the live load was increased 20 percent to 49 kN and for F4 it was increased 20 percent for one million cycles and 60 percent for the next one million cycles.

FATIGUE TEST RESULTS AND DISCUSSION

Control girder

Cracking of the control specimen occurred at a load of 55.6 kN. After unloading and reloading, the flexural crack at midspan reopened at a load of 45.4 kN, which shows an approximate loss of prestress of 6.7 percent. After the initial loading cycles, the girder was cycled between 8.9 kN and 49 kN as described above. The load deflection behavior of the control girder is shown in Fig. 4. The stress range in the lower prestressing strand, which can be defined as

$$SR = \frac{f_{ps2} - f_{ps1}}{f_{pu}}$$

where f_{ps2} and f_{ps1} are the upper and lower stress in prestressing subjected to cyclic loading conditions and f_{pu} is the ultimate strength of the prestressing, is shown in Fig. 5 versus the number of cycles. The control girder survived 2 million cycles with very little degradation. The girder was then loaded to failure, which occurred at a load of 142 kN, a 3.64 percent decrease from the ultimate strength achieved in the static test of the control girder.

Near surface mounted CFRP strengthened girders

One girder strengthened with NSM bars and another strengthened with NSM strips (identical to two girders tested under static loading conditions) were tested in fatigue, F1 and F2 respectively. After the initial loading the strengthened girders were tested at a frequency of 2 Hz between 9 kN and 57.6 kN. The cracking load of the NSM bars and strips strengthened girders occurred at loads of 54 kN and 51 kN respectively. For both girders, the largest degradation in stiffness occurred between the secondary loading sequence (used to determine the prestress losses) and 10,000 cycles. Even with a substantial increase in stress range (see Figure 5) compared to the control girder, after 2 million cycles there was very little degradation in the load versus deflection plot for either girder. The load versus deflection plots for both girders are very similar – the one for the girder strengthened with NSM bars is shown in Figure 6.

After 2 million cycles the girders were tested up to failure and showed little difference between the girders tested under static loading conditions. For the NSM bars strengthened girder tested in fatigue, failure was due to crushing of the concrete at a load of 178 kN, compared to the statically tested girder which failed at a load of 181 kN (see Figures 7 and 8). The NSM strip strengthened girder tested in fatigue also failed due to crushing of concrete at a load of 163 kN, compared to the statically tested girder which failed at a load of 180 kN (40.6 k).

Externally bonded CFRP strengthened girders

Two girders strengthened with externally bonded CFRP systems were tested in fatigue: one girder strengthened with externally bonded strips (F3) and another strengthened with externally bonded sheets designed for a 60% increase in capacity compared to the control (F4). These were identical to two girders tested under static loading conditions[1].

The girder strengthened with externally bonded strips had a cracking load of 54 kN. After 10,000 cycles, the behavior of the strengthened girder changed markedly from that of the NSM strengthened girders. Whereas the stress range and the maximum deflections of the other girders stabilized, F3 showed a steady increase in both these quantities. At 625,000 cycles a large crack was noticed near midspan which led to localized delamination of the CFRP sheets from the concrete substrate. This crack was due to rupture of a prestressing strand. The test was continued and catastrophic failure occurred at 908,000 cycles due to progressive rupture of the prestressing strands followed by debonding of the CFRP strips. The rupture of the first prestressing strand constitutes failure of the girder, since the deflection under service loading exceeded the limits specified for this type of girder. Gradual degradation of the bond between the concrete and FRP due to the fatigue loading could have resulted in a loss of girder stiffness therefore increasing the stress range in the lower prestressing strands precluding failure. Although very little corrosion was detected in the prestressing strands in an examination after failure, this could also have been a cause of the early fatigue rupture. Load versus deflection behavior of this girder is shown in Figure 9.

A girder strengthened with externally bonded CFRP sheets designed to achieve a 60% increase in the ultimate load carrying capacity of the control girder was tested in fatigue (F4). The cracking load of this girder was measured to be 70 kN, much greater than any of the previously tested girders due to a higher effective prestressing force. Due to the uncertainty in calculating the stress range in the lower prestressing strand for the girder, it was decided to test this girder identically to the other strengthened girders. Very little degradation was noticed between the initial loading sequences up until 1 million cycles. At this stage it was decided to cycle between the load range of 8.9 kN to 72.7 kN, representing a 60 percent increase in live load. A static test performed at one million cycles showed very little change in stiffness up to 72.7 kN. Another 250,000 cycles degraded the girder so that a secondary stiffness can be seen after reopening of the crack as can be seen in the load versus deflection plot in Fig. 10.

The stress range in the prestressing strands, as shown in Fig. 5, was lower than that of the control girder up to one million cycles (due to the higher prestressing force), but increased dramatically after the 60 percent increase in live load was applied. Between 1.25 million cycles and 2 million cycles very little change was observed in the cracking pattern or the load-deflection behavior. After 2 million cycles the girder was tested to failure. Like the girder tested under static loading conditions, failure was due to rupture of the CFRP sheets followed by crushing of concrete. Due to the higher prestressing force observed in this girder, the girder tested in fatigue failed at a load of 245 kN, greater than the ultimate load observed for girder S6.

CONCLUSIONS

Five 40-year-old 9.14m long prestressed concrete girders have been tested under fatigue loading conditions. One was tested as a control specimen and four were strengthened with various CFRP systems. The cyclic loading was designed to simulate loads on an actual bridge girder, from the dead load to the dead load plus live load. Based on the results, the following conclusions can be drawn:

1. Prestressed concrete girders strengthened with NSM CFRP systems to achieve a 20 percent increase in ultimate load carrying capacity can withstand over two million cycles of a loading equivalent to a 20 percent increase in live load.
2. Girders strengthened with externally bonded CFRP sheets to achieve a 60 percent increase in ultimate load carrying capacity can withstand over one million cycles of loading equivalent to a 60 percent increase in live load.
3. The girder strengthened with externally bonded CFRP strips performed worse under fatigue loading conditions than either the NSM systems or the externally bonded sheet strengthened systems, although this performance could be due to other circumstances such as corrosion of the prestressing strands. More testing needs to be done to corroborate these findings.
4. The influence of the CFRP U-wraps placed along the length of the girder for preventing fatigue initiated debonding is not known and requires further research.

Future fatigue testing of similar prestressed concrete girders will involve the testing of an externally bonded CFRP sheets girder strengthened to achieve a 40 percent increase in strength, as well as a girder strengthened to achieve a 20 percent increase in strength using externally bonded high modulus sheets.

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REFERENCES

1. Rosenboom, O.A., and Rizkalla, S.H., "Static behavior of 40 year-old prestressed concrete bridge girders strengthened with various FRP systems. *Proceedings of the Second International Conference on FRP Composites in Civil Engineering*, Adelaide, Australia, Dec. 2004.

2. Hassan T., and Rizkalla, S., "Investigation of Bond in Concrete Structures Strengthened by Near Surface Mounted CFRP Strips," *ASCE, Journal of Composites for Construction*, V. 7, No. 3, 2003, pp. 248-257.

3. Hassan T., and Rizkalla, S., "Flexural Strengthening of Post-Tensioned Bridge Slabs with FRP Systems," *PCI Journal*, V. 47, No. 1, 2002, pp. 76-93.

4. De Lorenzis, L., and Nanni, A., “Characterization of FRP Rods as Near-Surface Mounted Reinforcement,” *Journal of Composites for Construction* V. 5, No. 2, 2001, pp. 114-121.

5. Takács, P. F., and Kanstad T., “Strengthening Prestressed Concrete Beams with Carbon Fiber Reinforced Polymer Plates,” *NTNU Report R-9-00*, Trondheim, Norway, 2000.

6. Reed, C. E., and Peterman, R. J., “Evaluation of Prestressed Concrete Girders Strengthened with Carbon Fiber Reinforced Polymer Sheets,” *ASCE, Journal of Bridge Engineering*, V. 9, No. 2, 2004, pp. 185-192.

7. Smith, S. T., and Teng, J. G., “FRP Strengthened RC Beam I – Review of Debonding Strength Models,” *Engineering Structures* V. 24, No. 4, 2002, pp. 385-395.

8. Smith, S. T., and Teng, J. G., “FRP Strengthened RC Beam II – Assessment of Debonding Strength Models,” *Engineering Structures* V. 24, No, 4, 2002, pp. 397-417.

9. Teng, J.G., Chen, J.F., Smith, S.T., Lam, L., “FRP Strengthened RC Structures”, John Wiley and Sons, New York, NY, 2002.

10. Oehlers, D.J., Seracino, R., “Design of FRP and Steel Plate Structures” Elsevier, New York, NY, 2004.

11. Heffernan, P.J., Erki, M.A., “Fatigue Behavior of Reinforced Concrete Beams Strengthened with Carbon Fiber Reinforced Plastic Laminates”, *Journal of Composites for Construction,* V. 8, No. 2, 2004, pp. 132-140.

12. El-Hacha, R., Filho, J., Rizkalla, S., and Melo, G., "Static and Fatigue behavior of Reinforced Concrete beams strengthened with different FRP Strengthening techniques", *Proceedings of the 2nd International Conference on FRP Composites in Civil Engineering* (CICE 2004), Adelaide, Australia, December 8-10, 2004.

13. Overman, T.R., Breen, J.E., and Frank, K.H., “Fatigue of pretensioned concrete girders’, *Report 300-2F*, Center for Transportation Research at the University of Texas at Austin, 1984.

14. Rao, R. and Frantz, G.C., “Fatigue tests of 27-year-old prestressed concrete bridge box beams”. *PCI Journal*, 1996, V. 41, No. 5, pp. 74-83.

15. Bentz, E. C., “Section Analysis of Reinforced Concrete Members,” Ph.D. Thesis, University of Toronto, 2000.

16. Malek, A. M., Saadatmanesh, H., and Ehsani, M. R., "Prediction of Failure Load of RC Beams Strengthened with FRP Plate Due to Stress Concentration a the Plate End," *ACI Structural Journal*, V. 95, No. 1, 1998, pp. 142-152.

17. Zia, P., and Kowalsky, M. J., "Fatigue Performance of Large-Sized Long-Span Prestressed Concrete Girders Impaired by Transverse Cracks," *Federal Highway Administration Report, FHWA/NC/2002-024,* 2002.

Table 1. Material Properties of Tested Girders

Girder Designation	F0	F1	F2	F3	F4
Strengthening Technique	none	NSM Bars	NSM Strips	EB Strips	EB Sheets
FRP Details	--	1 #3 per web	2 strips per web	1 strip per web	4 plies per web
A_{FRP} (mm^2)	--	130	129	119	1960
Ultimate Tensile Strength of FRP (MPa)	--	2070	2070	2800	340
E_{FRP} (GPa)	--	124	124	165	46.5
FRP rupture strain	--	0.0167	0.0158	0.0170	0.00734
f'_c (MPa)	Not available	66.7	48.0	61.4	67.8

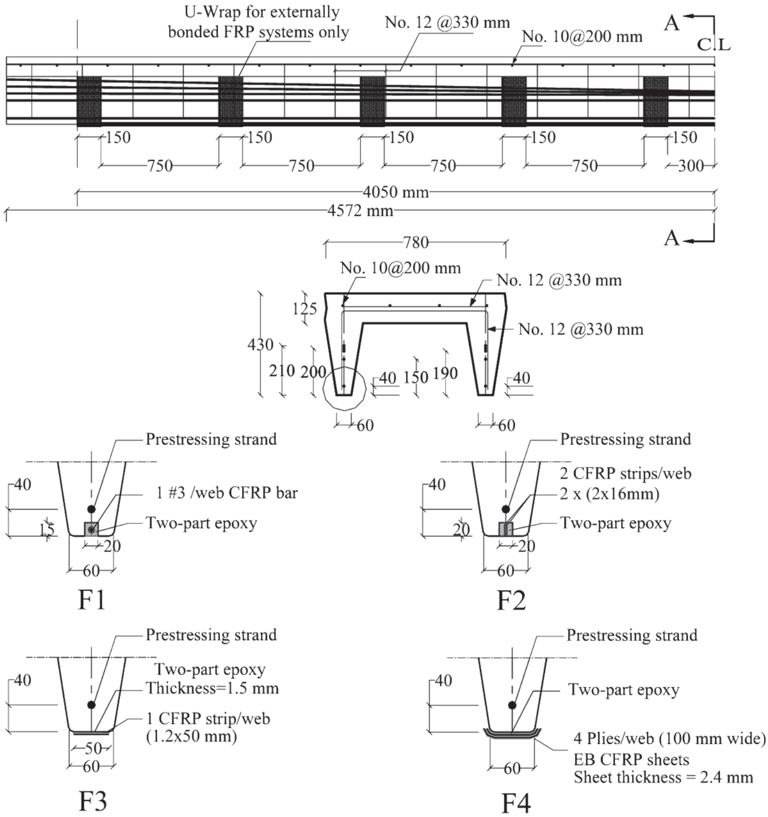

Figure 1 – Reinforcement and strengthening details of the C-Channel girders

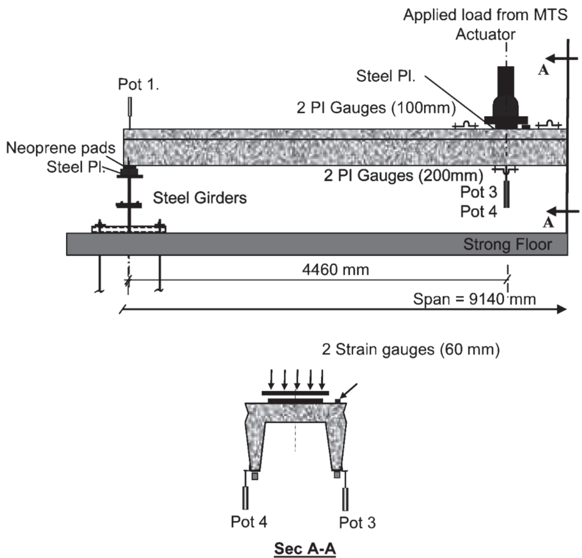

Figure 2 – Test setup for girders tested under fatigue loading conditions

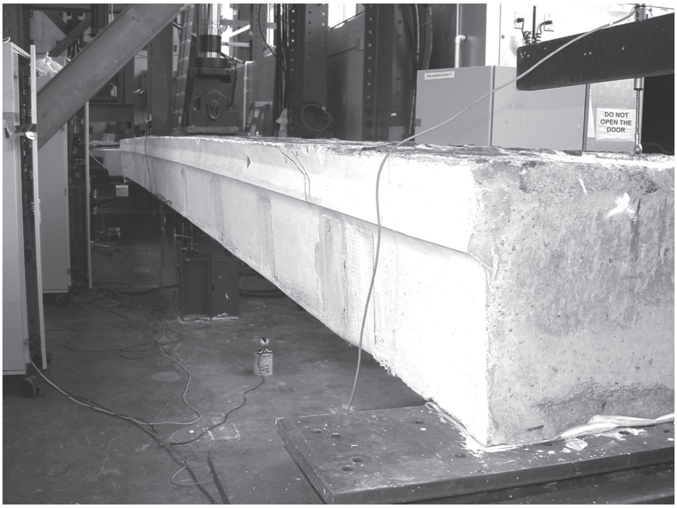

Figure 3 – Typical test setup for girders tested in fatigue

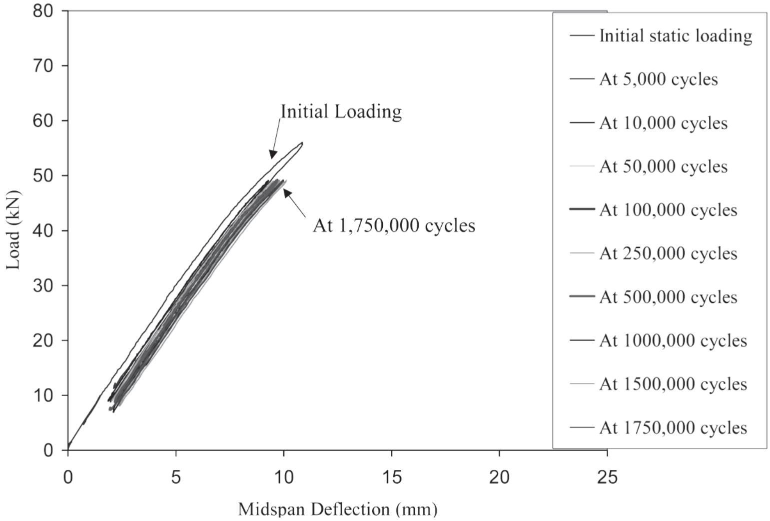

Figure 4 – Load versus deflection for control girder (Fo)

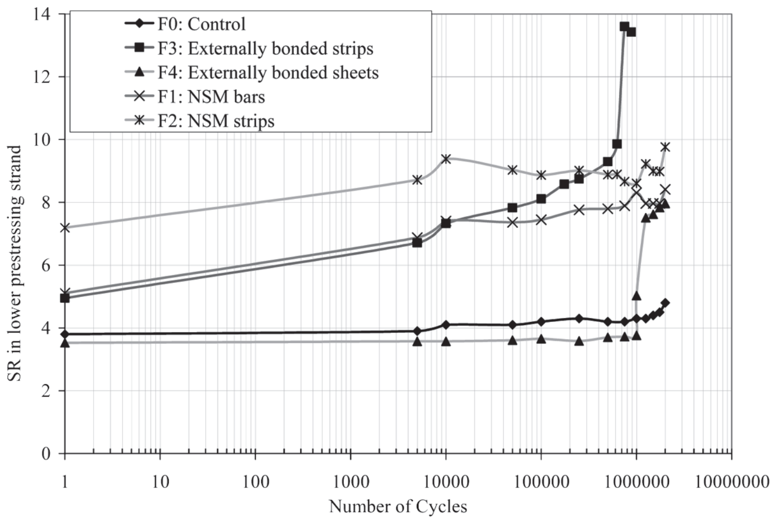

Figure 5 – Stress range versus the log of the number of cycles for girders tested in fatigue

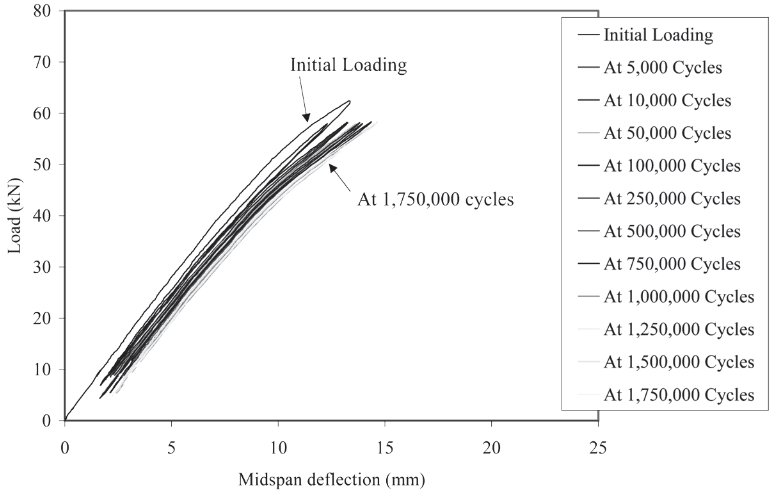

Figure 6 – Load versus deflection for girder strengthened with NSM bars (F1)

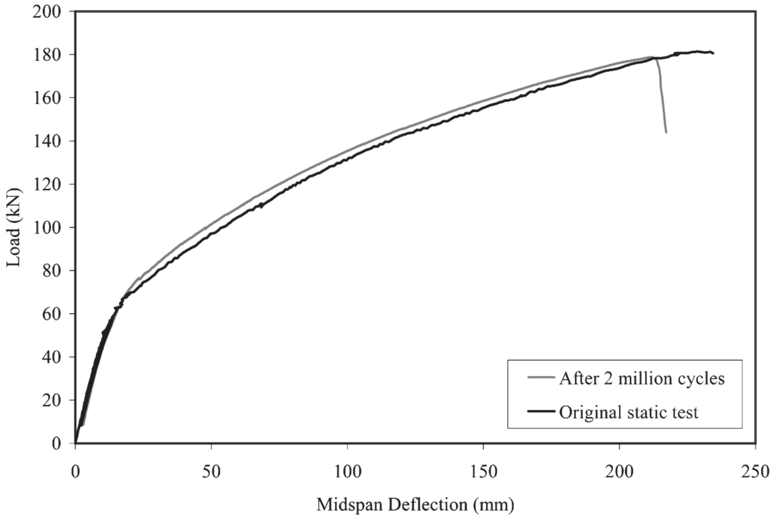

Figure 7 – Static test after two million cycles of girder F1

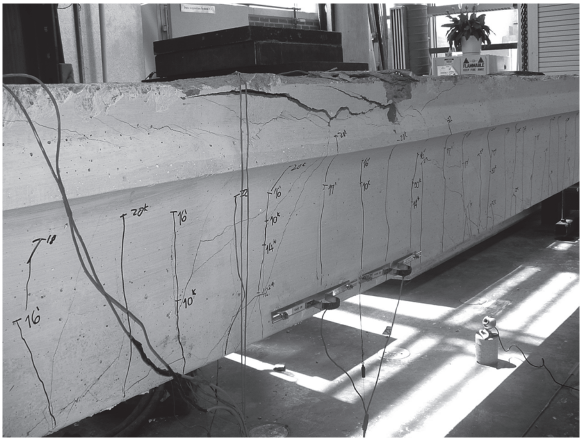

Figure 8 – Failure of girder F1 during final static test after two million cycles of fatigue loading

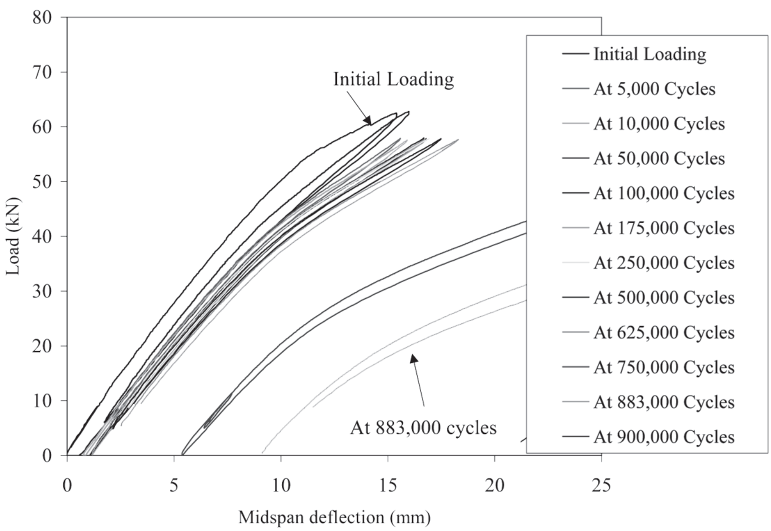

Figure 9 – Load versus deflection for girder strengthened with externally bonded CFRP strips (F3)

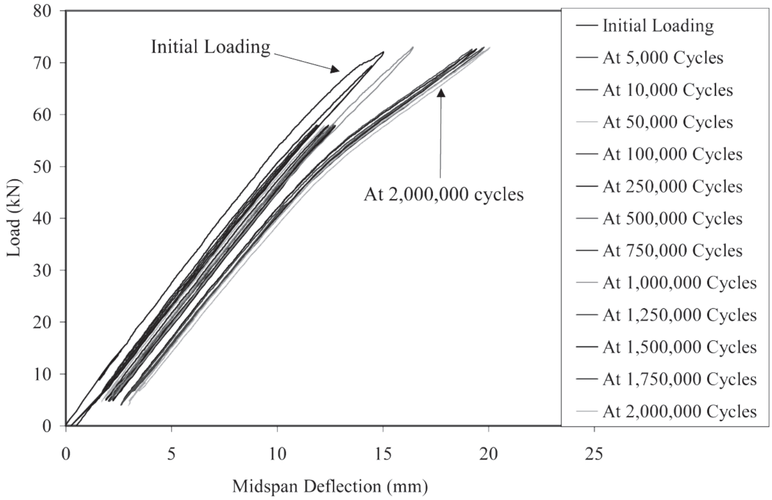

Figure 10 – Load versus deflection for girder strengthened with externally bonded sheets (F4)