REPAIR OF PRESTRESSED CONCRETE BRIDGE GIRDERS WITH FRP

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1 INTRODUCTION

Impact damage by over-height vehicles to reinforced concrete (RC) and prestressed (PS) concrete bridge girders is an ever recurring problem facing bridge maintenance departments throughout the United States, and worldwide. According to one report, 81 percent of damaged prestressed girders were caused by lateral overheight vehicular loads [1]. Aside from the obvious structural damage to the bridge resulting from the impact, the damage also results in long traffic delays during maintenance, potential safety hazards, large financial burdens on maintenance budgets, and negative psychological effects on highway users.

As much of a problem as impact damage may be, roughly 86 percent of the occurrences only cause minor to moderate damage to the girder. Minor to moderate damage can be defined as anything ranging from isolated cracks and shallow spalls, up to large spalls that expose undamaged prestressing stands. Only approximately 14 percent of impact damage occurrences are severe enough that they can cause rupture of prestressing strands and remove large portions of the concrete section [2]. The current research is focused on these severe cases.

For most transportation departments, the common practices for dealing with severely impact-damaged bridge girders is to saw-cut the deck, remove the girder, and replace with a new girder. Several options have been examined in the past, as well as new solutions proposed and researched by transportation departments, to eliminate the need for complete removal of severely impacted PS girders. This research project was proposed to evaluate the effectiveness of externally bonded Carbon Fiber Reinforced Polymer (CFRP) sheets as a repair system for impact-damaged PS bridge girders.

The use of FRP materials began after WWII primarily in military applications. Significant use then expanded within the aerospace and automotive industries. The use of FRP materials within the transportation infrastructure, either as a retrofit material or in construction of new structures, gained popularity since the 1980's. Today, CFRP bars, strips, tendons, and wet lay-up sheets are commonly used in the repair and retrofit of concrete structures. Some of the major benefits of CFRP include its high strength to weight ratio, high fatigue endurance and the ease of fabrication, manufacturing, handling and installation.

One of the most common uses of CFRP materials is to externally bond the material directly to the concrete surface. The sheets are typically bonded to the tension side of the members to increase the flexural strength and/or on the sides of the member to improve the shear capacity. Installation of this technique is relatively simple and can be achieved in a very short time. The system must be designed to avoid premature failure due to possible delamination or debonding of the CFRP material from the concrete surface, a failure mode that many researchers are currently investigating [3]. Several research groups [4,5] have explored the usage of CFRP systems to restore damaged prestressed concrete members, both in the laboratory and field tests. Most groups were able to successfully restore most of the flexural capacity of damaged members, but rarely was the full undamaged capacity successfully restored. Lastly, very little research has been performed on large-scale PS girders repaired and tested under shear-critical loading conditions. The current research project will demonstrate that the repair of severely damaged PS girders, with up to 18 percent of strands ruptured, can restore the ultimate flexural and shear strength of the undamaged PS girders.

2 EXPERIMENTAL STUDY

2.1 Test girders

Three Type II AASHTO girders (AASHTO1, AASHTO2, and AASHTO3) were tested to examine the flexural behavior of CFRP repaired girders damaged by vehicular impact. The girders were damaged, either accidentally or artificially, near midspan location to examine different damage scenarios, including varying the amount of prestressing strands ruptured.

Along with the three flexural tests, two other short span girders were tested under shear-critical loading conditions. Following the flexural testing of AASHTO2, the girder was saw-cut into thirds, with the two identical end pieces used for the shear study. One section of the girder was used as a control specimen (AASHTO2C), while the other specimen was damaged near the support and repaired with CFRP sheets (AASHTO2R).

AASHTO1 was the first girder tested in this research project. The test girder was an AASHTO Type II girder prestressed with sixteen 12.7 mm (0.5 in) – 7 wire 1860 MPa (270 ksi) straight prestressing strands. The total length of the tested girder was 16.71 m (54.84 ft) with a center-to-center support span of 16.26 m (53.34 ft). The girder, after impact damage from an overheight vehicle, is shown in Figure 1. The approximate volume of damaged concrete was equal to 0.1 m^3 (4.8 ft^3). In addition to the loss of concrete, one prestressing strand on the bottom layer was ruptured on the front side, resulting in a 6.3 percent loss of prestressing force. The full 2.13 m (7.0 ft) composite bridge deck was not part of the test specimen since it was cut through in order to remove the damaged girder from the bridge superstructure. The average width of the composite deck of the test girder was 388 mm (14.9 in). Cross section and elevation drawings of AASHTO1 are shown in Figure 2.

The second and third AASHTO Type II test girders, AASHTO2 and AASHTO3, were not directly damaged by the vehicular impact. AASHTO2 was older than the others; based on the presence of a large endblock and 28 - 11.1 mm (7/16 in) diameter 1724 MPa (250 ksi) prestressing strands it was determined the girder was cast in the 1960's. The girder was prestressed with twenty four straight and four harped prestressing strands using a hold down system at midspan. The total length of the girder was 16.61 m (54.5 ft), with a span length of 16.17 m (53.06 ft) from center-to-center of the supports. Based on measurements taken at two foot intervals, the average width of the composite deck was determined to be 419 mm (16.5 in). AASHTO3 was comprised of 16 - 12.7 mm (0.5 in) diameter 1862 MPa (270 ksi) straight prestressing strands. The total length of the girder was 16.61 m (54.5 ft). The testing span length of this girder was shortened to avoid the damage at one end of the girder, caused during transportation. The center-to-center length between the testing supports was 14.94 m (49.0 ft) with an average composite deck width of 343 mm (13.5 in).

Fig. 1 AASHTO1 at NCDOT yard after impact damage.

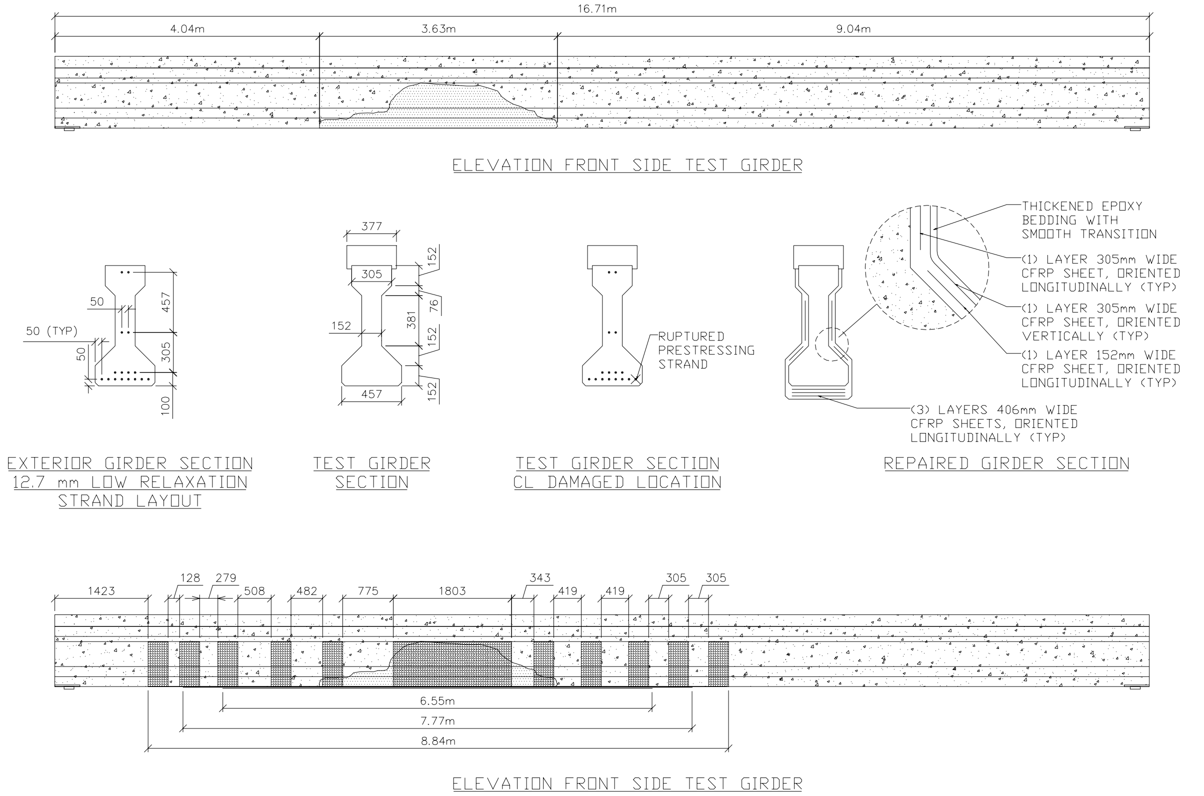

Fig. 2 AASHTO1 girder cross section, elevation, and CFRP design details.

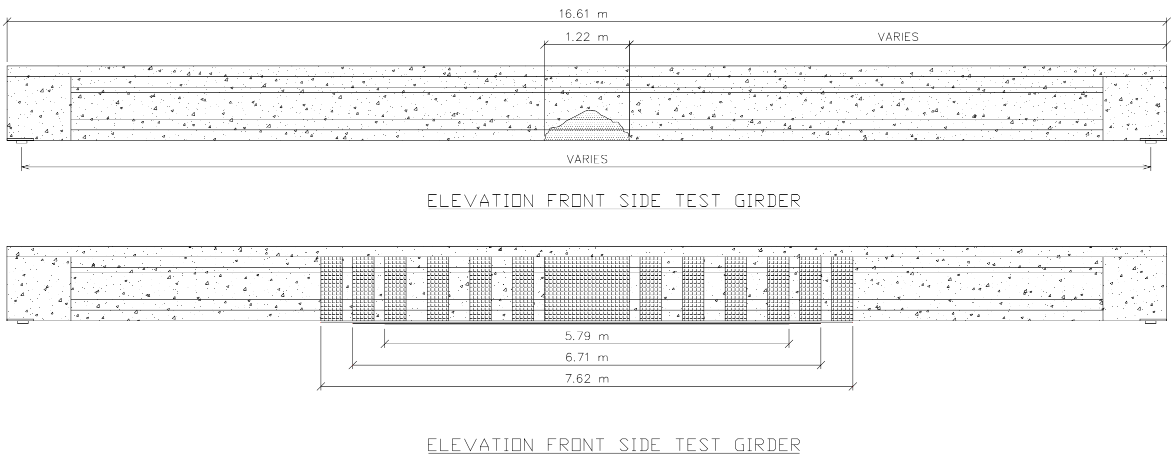

Fig. 3 AASHTO2 and AASHTO3 girders' damaged and repaired elevations.

Both AASHTO2 and AASHTO3 were received by the researchers undamaged. In order to simulate impact damage exhibited by AASHTO1, a large portion of the tension flange of the girders was removed, with four and three prestressing strands being removed from AASHTO2 and AASHTO3, respectively. The damage caused a 14.3 and 18.8 percent reduction in prestressing force, respectively. Approximately 0.06 m^3 (2.2 ft^3) of concrete was removed from both girders. The damaged and repaired elevations of AASHTO2 and AASHTO3 are shown in Figure 3.

The first girder tested during the shear study, AASHTO2C, was tested monotonically to failure as a control specimen. The girder was prestressed with 28 - 11.1 mm (7/16 in) diameter 1724 MPa (250 ksi) strands. The total length of the girder was 6.30 m (20.67 ft), with a span of 5.43 m (17.83 ft) from center-to-center of the supports. The average width of the composite deck was determined to be 419 mm (16.5 in). AASHTO2R had the same member characteristics, but the concrete was damaged and prestressing strands ruptured. The girder was tested under the same loading conditions as AASHTO2C for comparative purposes.

In order to simulate impact damage in AASHTO2R, a large portion of the tension flange of the girder, including a small section of the web, was removed, and four prestressing strands were ruptured corresponding to a reduction in prestressing force of 14.3 percent. A total of approximately 0.04 m^3 (1.5 ft^3) of concrete was removed. Figure 4 shows the elevation of both the AASHTO2C and AASHTO2R girders.

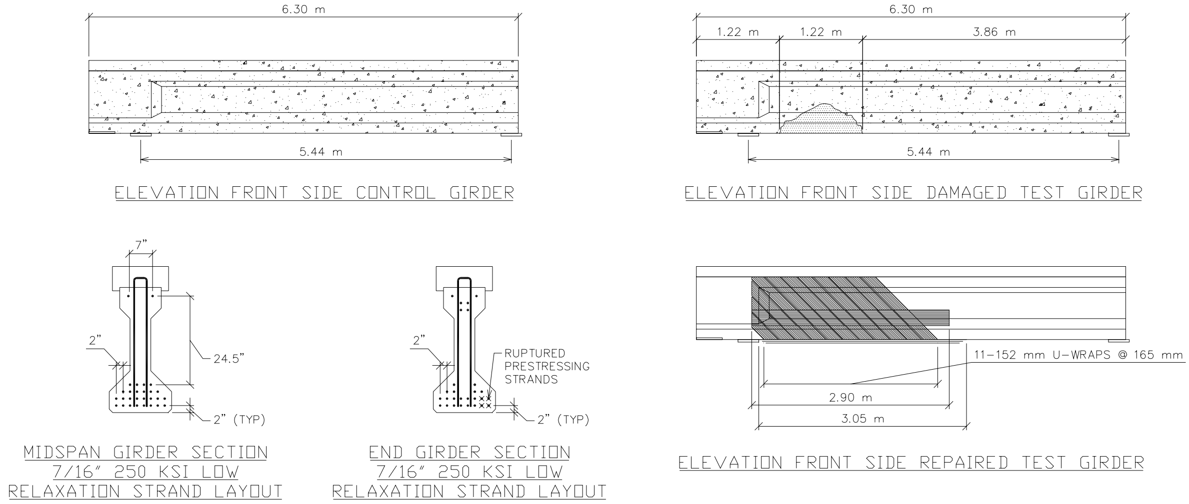

Fig. 4 AASHTO2C and AASTHO2R elevations.

2.2 Material properties

Material tests were performed on prestressing strands, concrete core samples, and CFRP sheets for each girder and repair system. The prestressing strands were tested in tension according to ASTM A416 specifications using a 979 kN (220 k) MTS closed-loop universal testing machine used to apply a constant rate of displacement. Several core samples were taken from both the girders and deck slab by qualified NCDOT personnel, then tested in accordance with ASTM C42 in an MTS closed-loop universal testing machine used to apply a constant rate of displacement. All CFRP coupons were tested in accordance with ASTM D3039 with a width of 25 mm (1.0 in) and gage length of 152 mm (6.0 in). All material test results can be found in previous research [6]

2.3 Design of CFRP repair systems

The purpose of applying CFRP to the various damaged AASHTO girders was to restore their original undamaged ultimate flexural and shear strengths. Based on previous research [7], it was decided to use externally bonded CFRP wet lay-up sheets to restore the original flexural and shear capacity of all the girders. The wet lay-up sheets were chosen for two reasons: 1) transverse U-wraps would be needed to encapsulate the restored concrete section to stabilize crack growth, thus meaning that two CFRP systems would be needed if wet lay-up sheets were not chosen as the main longitudinal reinforcement and 2) based on previous research [7], and in consultation with NCDOT officials, CFRP wet lay-up sheets were found to be the most cost-effective repair system, and thus most likely to be used by the NCDOT in future field applications.

The first task in the design of the CFRP flexural repair systems for AASHTO1, AASHTO2, and AASHTO3 was to determine the original flexural capacity of the girder, which was predicted using RESPONSE 2000©, a cracked section analysis program [8]. For AASHTO2 and AASHTO3, initial static tests were performed prior to damaging. The RESPONSE predictions were then calibrated to match the elastic and inelastic behaviour of these two girders. To restore the original ultimate flexural capacity of each girder, a cracked section analysis was performed in iterations until the repaired flexural capacity equalled the predicted undamaged capacity. The analysis concluded for each girder that three layers of 406 mm (16 in) wide longitudinal CFRP layers were required, with the design of AASHTO1 being conservative. In order to minimize the initiation of new cracks in the repaired concrete, additional longitudinal sheets were installed; a 152 mm (6 in) wide sheet was placed on the top of the bottom flange and a 305 mm (12 in) wide sheet was placed in the middle of the web. These additional longitudinal sheets were placed on both sides of the girder to ensure symmetry of the section. To ensure proper utilization of the CFRP's strength and strain capacity, the damaged concrete section was restored using a two part repair mortar as well as adequate development length of the longitudinal sheets was provided. The final design of the CFRP repair system for AASHTO1 is shown in Figure 2, while girders AASHTO2 and AASHTO3 are shown in Figure 3.

The design of the CFRP repair system for AASHTO2R consisted of two phases; the flexural repair and the shear repair. The flexural analysis and repair was performed in the same manner described above. The shear repair was performed using the current Precast/Prestressed Concrete Institute (PCI) Handbook [9] to determine the shear capacity along the entire length of the member for the undamaged and then damaged section. After the damaged concrete section was restored with repair mortar, the required width and spacing of transverse diagonal CFRP sheets was determined using ACI Committee 440 [10] guidelines.

2.4 Test setup

The three flexural Type II AASHTO specimens were loaded to failure using a 980 kN (220 k) MTS hydraulic actuator. The actuator was mounted to a steel frame which was located at midspan of the girder. The load was applied to the girder through a recommended [11] 254 mm by 508 mm (10 in by 20 in) steel loading plate placed in contact between the actuator and girder.

Both shear critical Type II AASHTO specimens were loaded at a span to depth ration, a/d, of 1.57. The same load plate previously described was used for both specimens. The control specimen was tested using a 1960 kN (440 k) MTS hydraulic actuator mounted to a steel frame. The actuator was replaced with a 2670 kN (600 k) hydraulic jack for AASHTO2R in anticipation of the ultimate load capacity exceeding that of the actuator.

2.5 Instrumentation

The displacement profile along the length of each of the three flexure specimens was measured using string potentiometers. Strain gauges measured the tensile strain in the longitudinal CFRP sheets and the compressive strain in the concrete. PI gauges (a strain gauge mounted to a spring plate) were also used to measure compressive strains along the height of the section. The location of the instrumentation was selected to determine: 1) the strain profile at the damaged region, 2) the behavioral differences between the damaged and undamaged section, if applicable, and 3) the tensile strain in the CFRP to determine the bond characteristics between the CFRP and concrete.

The displacement beneath the load point of each shear-critical specimen was measured using two string potentiometers. Compressive strains were measured using both PI gauges and strain gauges. The shear crack widths were measured using three sets of two linear potentiometers oriented perpendicular to each other. The location of the instrumentation for both girders was selected to determine: 1) the strain profile below the load point, and 2) to compare the crack widths of the control specimen with those of the repaired specimen. AAHTO2R was instrumented with strain gauges along the CFRP diagonal struts to determine the transverse strain distribution along the length of the girder.

2.6 Test procedure

The loading of AASHTO1 consisted of three stages that all occurred after the installation of the CFRP repair system: 1) initial loading, 2) fatigue loading, and 3) final static loading to failure. During the initial loading phase, the girder was loaded up to a level of 227 kN (51 k), applied using displacement control loading. This load was selected to evaluate the observed stress ratio in the lower prestressing strands and load-deflection behavior of the girder at this load level. The fatigue loading regime was performed by oscillating between 83.2 kN and 201 kN (18.7 k and 45.2 k) to simulate the dead load and the dead load plus live load, as determined in previous research [6]. The final static test of the girder consisted of several intermediate cycles of progressively increasing load up to a failure.

The loading of AASHTO2 and AASHTO3 consisted of four stages: 1) initial loading of undamaged specimen, 2) loading after removal of concrete section, 3) loading after cutting of prestressing strands, 4) final static loading to failure after CFRP repair. During the initial loading of AASHTO2 and AASHTO3, each girder was loaded beyond cracking, unloaded, and then reloaded again beyond cracking. The purpose of the loading was to examine the cracking load, crack reopening load, and load-deflection behaviour of each undamaged specimen. After the desired amount of concrete was removed from each girder, and again following the cutting of the prestressing strands, both girders were loaded to examine any changes in the load-deflection behaviour. The final static test of each girder consisted of two cycles up to failure.

The loading of AASHTO2C and AASHTO2R was performed in one cycle up to failure. No initial tests were performed after the removal of the concrete or cutting of prestressing strands due to the brittle nature of the shear failure.

3 TEST RESULTS AND DISCUSSION

3.1 Flexural study

Three AASHTO Type II girders (AASHTO1, AASHTO2, and AASHTO3) were damaged (either by impact or simulated impact) at or near midspan, and repaired using CFRP sheets. The girders were damaged and repaired as previously described. A Summary of the test results for the flexural tests are provided in Table 1.

Table 1 Summarized test results for AASHTO girders tested in flexure.

Specimen Designation	AASHTO1	AASHTO2	AASHTO3
Repair system	CFRP sheets	CFRP sheets	CFRP sheets
Initial cracking load*, kN	---	265	281
Cracking load**, kN	187	262	289
Prestress losses*, %	12.6	18.2	12.9
Effective prestress*, kN	123	68.9	122
Predicted ultimate load of undamaged girder, kN	514.7	529.8	562.7
Predicted ultimate load of repaired girder, kN	645.3	625.0	636.1
Measured ultimate load, kN	604.5	569.5	646.3
Maximum compressive strain in concrete, με	2600	2630	3090
Maximum tensile strain in CFRP, με	4680	4620	5760
Failure mode	Crushing of concrete	Crushing of concrete	Crushing of concrete

* For the girders which were tested prior to simulated impact damage
** The cracking load of the repaired material

AASHTO1

The applied load versus midspan displacement relationship for the fatigue loading between 83.2 kN to 201.1 kN (18.7 k to 45.2 k) for the repaired AASHTO1 girder is shown in Figure 5. The girder survived 2 million cycles with very little degradation or crack propagation in the repaired region after the initial cycles. Fatigue creep of concrete [12] caused small amounts of displacement degradation totaling 2.8 mm (0.11 in) after 2 million cycles of loading. No stiffness degradation in the girder was observed after completion of the fatigue loading regime.

Flexure-shear cracks, which initiated outside the termination point of the longitudinal CFRP in the undamaged region, propagated in an inclined fashion towards the compression zone and eventually caused failure of the girder near midspan. Test results indicated that the presence of the main longitudinal CFRP significantly increased the flexural capacity of the damaged zone of the girder causing the failure outside this region. The presence of the tension-strut CFRP reinforcement and transverse U-wraps controlled crack propagation within the damaged zone as well as enhancing the shear capacity of the strengthened section.

The overall behavior of the repaired girder far exceeded the predicted strength of the undamaged girder in both strength and ultimate displacement, as shown in Figure 5. The predicted response of the undamaged girder is also shown in Figure 5. Concrete crushing was the predicted failure of the damaged and repaired girder at an applied load of 645 kN (145.1 k), which corresponds well to the measured load at failure of 605 kN (136.1 k). The measured ultimate load was 17.6 percent higher than the ultimate load predicted for the undamaged girder, which could be caused by the slight over-designed CFRP repair system. The measured ultimate displacement of the repaired girder was 37 percent greater than the undamaged prediction and 5.2 percent greater than the repaired prediction.

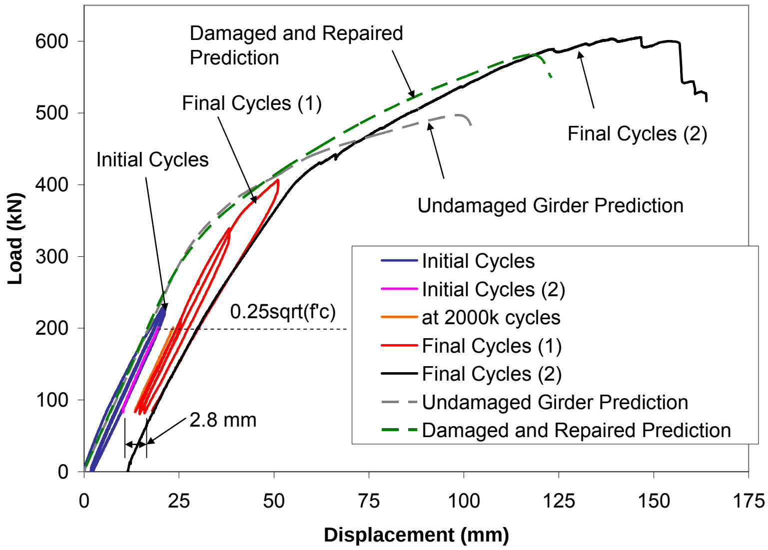

Fig. 5 Load vs displacement for AASHTO1

AASHTO2

Girder AASHTO2 was subjected to four cycles of initial loading; two of which were performed on the undamaged girder. One of these initial cycles loaded the girder up to 90 percent of the predicted ultimate load value. The third and fourth initial cycles of loading occurred after the simulated impact damage was applied to the girder. The third loading cycle followed the concrete removal around midspan, and the fourth cycle was after four prestressing strands were ruptured. A comparison of the stiffness of girder AASHTO2 in each of these cycles is provided in Table 2. The largest drop in stiffness was a result of the severe initial loading combined with the concrete damage, for a reduction of approximately 17 percent. A further reduction was observed after the cutting of the prestressing strands.

Table 2 Stiffness comparison of AASHTO girders tested in flexure (N/mm)

Specimen Designation		AASHTO1	AASHTO2	AASHTO3
Prior to CFRP repair	Initial cycle 1*	---	13.8	13.6
	Initial cycle 2*	---	14.2	13.6
	Post concrete damage*	---	11.9	11.5
	Post cutting of prestressing strands*	---	11.0	10.6
After CFRP repair	Final cycle 1	11.5	12.1	11.9
	Final cycle 2	10.5	11.8	12.3

Failure initiated in the compression region around the loading plate, where localized crushing led to out-of-plane behavior rapidly causing failure. The maximum measured value of concrete compressive strain was 2630 με. The relatively low value at crushing of concrete was likely a result of the poor condition of the concrete at midspan due to extant transverse cuts. The repaired girder far exceeded the strength and ultimate displacement of the original undamaged girder. The ultimate load of the repaired girder was 13.4 percent greater than the undamaged girder prediction, and was similar to the repaired girder prediction. The longitudinal CFRP system was able to adequately carry the tensile forces, and showed very little debonding or damage even after the failure event. The transverse CFRP sheets were also virtually unaffected during the loading, with only the U-wraps positioned directly below the loading area becoming damaged at failure.

AASHTO3

A comparison of the stiffness of girder AASHTO3 in each of the four previously mentioned loading cycles is provided in Table 2. Due to the simulated impact damage, the stiffness of the girder was reduced by 22.1 percent. Failure was due to concrete crushing in the compression region around the loading plate. The maximum measured value of concrete compressive strain at failure was 3090 με, significantly larger than the measured value for the previous AASHTO flexural tests. There was no evidence of debonding in the longitudinal CFRP at failure. The transverse U-wraps buckled at failure but contained most of the concrete damage. The predicted ultimate load of the repaired girder was 1.7 percent lower than the measured value.

3.2 Shear study

Two short span Type II AASHTO girders were tested under shear critical loading schemes. One girder was tested as a control specimen (AASHTO2C); the other girder was damaged and repaired using CFRP sheets and tested to failure (AASHTO2R). Summarized test results for the shear tests are provided in Table 3.

Table 3 Summarized results for AASHTO girders tested in shear

Specimen Designation	AASHTO2C	AASHTO2R
Repair system	None	CFRP sheets
Cracking load, kN	1007	1140
Average crack width at failure, mm	3.76	0.48
Failure load, kN	1840	1985
Predicted failure load, kN	1660	2160
Failure mode	Web shear failure	Shear in right transfer zone followed by flexure-shear
Max tensile strain in longitudinal CFRP, με	---	810
Max tensile strain in diagonal CFRP strut, με	---	1920
Max compressive strain, με	680	1400

AASHTO2C and AASHTO2R

Girder AASHTO2C and AASHTO2R were loaded in one cycle up to failure. During the loading process, several inclined cracks formed at 45 degrees from the support at various load levels. The number of cracks in the control specimen was visually observed, however the cracks were not visible in the repaired girder as a result of the diagonal CFRP struts. In order to compare the average crack widths of the two specimens, it was assumed that the same number of cracks formed in the repair specimen that was visible in the control specimen. The maximum crack width in AASHTO2C was 3.8 mm (0.15 in) and 0.5 mm (0.02 in) for AASHTO2R. It should be noted that the maximum crack widths formed closest to the loading point.

The tensile strain at the center of each diagonal CFRP strut was also measured during the loading of AASHTO2R. The strain data indicated that the largest inclined crack openings were located approximately half way between the left support and load plate, as expected. Also, the strain induced in the diagonal CFRP struts was much lower than the ACI Committee 440 [10] rupture design value of 0.004 mm/mm, therefore not debonding or rupturing of the sheets did not occur in the repaired zone.

The ultimate failure of AASHTO2C initiated in the inclined cracks that formed on the shear-critical side of the test specimen from the support to the loading plate. The failure of AASHTO2R initiated beyond the longitudinal CFRP sheets after shear failure in the right transfer region had occurred. The diagonal CFRP struts located in the shear-critical region of the member remained intact after the failure of AASHTO2R. The ultimate load capacity of AASHTO2R was 7 percent higher than that of AASHTO2C, and the ultimate displacement was slightly enhanced. Figure 6 shows the failure of girders AASHTO2C and AASHTO2R.

Fig. 6 Ultimate failure of girders AASHTO2C (left) and AASHTO2R (right)

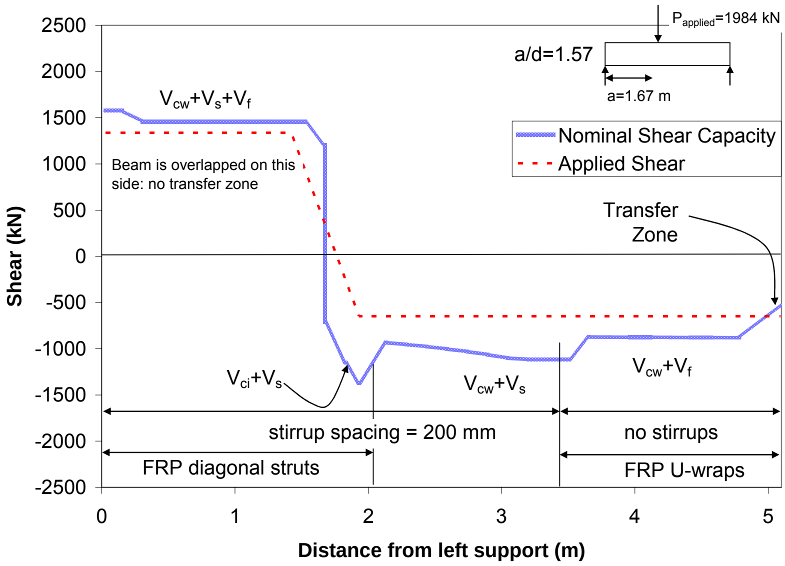

Fig. 7 Predicted shear capacity versus the applied shear load of girder AASHTO2R

The predicted shear capacity versus the applied shear demand for AASHTO2R is shown in Figure 7. The initial predicted shear failure load was 2309 kN (519 k), but the test specimen failed at 1985 kN (446 k). The initial prediction would have been accurate if shear failure to the left of the applied load, the shear-critical span, had occurred, however this was not the failure mode observed. From Figure 7, it is evident that the applied shear at the right end exceeded the allowable shear in the transfer region. Shear failure in the transfer region allowed the prestressing strands to debond, in turn lowering the moment capacity of the section on the right end. The moment capacity of the girder became much less after the contribution of the prestressing strands is progressively lowered due to debonding, leading to a flexure-shear failure in combination with plate end debonding at the termination point of the longitudinal CFRP sheets.

4 CONCLUSIONS

Following the experimental program, the findings from this research include:

- Based on the testing regime, it was possible to repair an impact-damaged girder with up to an 18 percent loss of prestressing force using CFRP sheets.
- Test results showed that the ultimate deflection of repaired girders can meet or exceed that of the same undamaged girder.
- Results from the fatigue test of AASHTO1 demonstrated that a CFRP repaired Type II AASHTO prestressed girder can withstand 2 million cycles of cyclic loading with very little loss of stiffness or residual deflection.
- Test results showed that a damaged Type II AASHTO girder with up to 14 percent loss of prestressing force and 25 mm (1.0 in) loss of web concrete can be repaired using CFRP sheets.
- The use of CFRP sheets reduced the shear crack widths of the repaired girder in comparison to the control specimen.
- Comparison of test results and predictions showed that PCI [9] and ACI Committee 440 [10] guidelines accurately predict the behavior of a prestressed girder repaired in shear.

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