Evaluation and Repair of Damaged Prestressed Concrete Girder Bridges

FRANCESCO M. RUSSO, TERRY J. WIPF, F. WAYNE KLAIBER, AND RYAN PARADIS

Every year, numerous prestressed concrete (P/C) girder bridges are damaged by overheight vehicles. When this happens, bridge engineers are faced with numerous questions relative to the behavior and strength of the bridge. These questions must be answered so decisions can be made concerning traffic restrictions and future maintenance actions. Results of an investigation of damaged P/C bridge behavior and damaged P/C beam strength are briefly presented in this paper. In this project, two P/C bridges carrying I-680 near Beebeetown, Iowa were tested. The westbound (WB) bridge was accidentally damaged and tested in the damaged state and following replacement of the damaged beams. The eastbound (EB) bridge was not damaged and was used as a reference. Following testing of the bridges and removal of the damaged beams, one of the beams was tested in an "as-removed" condition while the other was strengthened with carbon fiber reinforced polymer (CFRP) longitudinal plates and external CFRP stirrups. Both beams were tested to failure. Additionally, a three beam laboratory P/C beam bridge model was tested. The model had a total length of 40 ft-4 in. (12.29 m) and a width of 18 ft (5.49 m). The bridge model was tested 180 times to study the effects of incremental damage and load placement on the behavior of a controlled specimen. Following testing of the model, two of the beams were removed, the first was tested in an undamaged condition, the other following intentional damage and CFRP repair. Both beams were tested to failure. An additional component of the work was the development of several analytical models of the damaged and repaired bridges. Both three-dimensional grillage (downstand grillage) and stiffened plate models were created. The models were calibrated using the experimental deflections recorded during both bridge tests. The analytical models were used to describe the live load distribution patterns in both the damaged and undamaged bridges. Significant redistribution of moment away from the damaged beams to the adjacent undamaged beams and curb/rail section was observed. Key words: prestressed concrete, FRP, repair, finite element, load testing.

INTRODUCTION

In 1996, an unknown overheight vehicle struck the center span of a 3-span prestressed concrete (P/C) bridge carrying I-680 over County Road L34 near Beebeetown, Iowa. Due to concerns about the remaining strength of the two most severely damaged beams, unknown effect of the damage on the load distribution patterns in the remaining structure, and concerns regarding the durability and effectiveness of any proposed repair, it was decided that the beams would be replaced. Frequently the decision to replace a damaged prestressed beam is made because of a lack of knowledge about the reserve strength of the bridge rather than from calculations that definitively indicate that the bridge has been compromised. The damaged bridge provided an opportunity to perform an in-place assessment of load distribution in damaged and undamaged bridges. The damaged beams were eventually tested following their removal from the bridge to determine the effect of damage on their remaining strength and to determine the effectiveness of carbon fiber reinforced polymer (CFRP) strengthening techniques. An additional aspect of this research was analytical modeling of the I-680 bridges in the damaged and repaired conditions so that the effect of damage on load distribution could be quantified. The models were calibrated using the experimental results.

In addition to the tests conducted on the I-680 bridges, a 40 ft-4 in. (12.3 m) long and 18 ft (5.5 m) wide P/C bridge model was also tested. The model was damaged in small increments to record the relative changes in bridge behavior due to incrementally applied damage. A total of 180 tests were conducted with various load placements and levels of damage. Two beams from the bridge model were then tested as isolated specimens, one undamaged and the other following intentional damage and CFRP repair. Due to space limitations, results of the bridge model testing and tests conducted on the isolated beams removed from the model are not presented in this paper. For additional details concerning all aspects of this research, refer to Klaiber, Wipf, Russo, Paradis and Mateega (*1*).

The objectives for this project were as follows:

· Determine the load distribution patterns in undamaged and damaged bridges.
· Ascertain whether the live load distribution in damaged bridges can be predicted with reasonable accuracy.
· Establish the effect of damage on the remaining strength of P/C beams.
· Determine whether damaged beams be economically and effectively repaired with strength as the controlling factor.

BEEBEETOWN BRIDGE TESTS

The I-680 bridges cross county road L34 near Beebeetown, IA (see Figure 1). The bridges are asymmetric three-span bridges designed by the Iowa DOT in August 1965. The spans are 43 ft-1 ½ in., 56 ft-3 in., and 47 ft-3 ½ in. (13.14 m, 17.14 m, and 14.41 m) long from east to west between the substructure centerlines. There are eleven beam lines in each structure. The first seven beam lines adjacent to the median are on 5 ft (1.52 m) centers, typical of Iowa DOT practice at the time these bridges were designed and constructed. To account for a ramp taper on both the eastbound (EB) and westbound (WB) bridges, there are also flared beam lines. These beams are spaced at 3 ft-6 in. (1.07 m) centers as a minimum and flare out to 5 ft (1.52 m)

F. Russo, (formerly of the Iowa Department of Transportation), URS Corporation, 671 Moore Road, King of Prussia, Pennsylvania 19406-1318. T. Wipf, Department of Civil and Construction Engineering, Center for Transportation Research and Education, Iowa State University, 420 Town Engineering Building, Ames, Iowa 50011. F. Klaiber, Department of Civil and Construction Engineering, Iowa State University, 422 Town Engineering Building, Ames, Iowa 50011. R. Paradis, HDR Engineering, Inc., 8404 Indian Hills Drive, Omaha, Nebraska 68114-4049.

on center at their widest point. The measured thickness of the slab-in-place including overlay exceeded 9 in. (230 mm) when the damaged beams were removed from the WB bridge; the overlay was approximately 3 in. (75 mm) thick.

FIGURE 1 Elevation view of WB bridge, looking West (damaged Beam 1W in center span)

The impetus for this research project was the collision of an unknown vehicle with the north three beams of the overhead WB structure in July 1996 (see Figure 2). Damage was centered ±5 ft (1.5 m) west of the midspan diaphragm of the center span. The damage to the WB bridge was such that approximately 6 ft (1.8 m) of the bottom flange was spalled or fractured from the north fascia beam, Beam 1W, exposing numerous prestressing strands. Several of the strands in Beam 1W seemed to be lax; however, no strands were severed during this collision. There was a preexisting severed strand from a 1993 collision. There was less damage on the first interior beam, Beam 2W. Significant cracking of the bottom flange as well as fracturing of the core concrete was present in both beams, but to a lesser extent on Beam 2W. Web cracking spread over the west half of Beams 1W and 2W. Cracking seems to have been arrested by the midspan cast-in-place concrete diaphragm. The second interior beam, Beam 3W, was also damaged, but not as severely, with the damage consisting of the spalling of a patch installed following prior collisions with the bridge.

FIGURE 2 Underside of damaged WB bridge, Beam 1W on left

Experimental Results

A large number of static load tests were conducted on the dual bridges in question. As previously noted, the WB bridge was damaged by overheight vehicle impact while the EB bridge was undamaged. An initial series of 43 static tests were conducted on each bridge to characterize the response under load and to determine the effect of various load placements on the relative distribution of load in the bridge. An additional 35 tests were conducted on the WB bridge following replacement of the damaged beams. Four test lanes were established. Lane 1W/1E was located so that the truck was directly over the flared beams (damaged beams of the WB bridge) and as close to the rail as possible (see Figure 3). The second lane, Lane 2W/2E is parallel to Lane 1W/1E but offset laterally by 12 ft (3.6 m). The third lane, Lane 3W/3E is located adjacent to the centerline of the through traffic lanes while the fourth lane, Lane 4W/4E, is adjacent to the median railing. In addition to the various load placements, a variety of data was collected from the bridges including quarter point and midspan deflections, as well as strains on the exposed strands of the damaged beams, on the diaphragms, and at the ends of the beams.

FIGURE 3 Lane 1W loaded with a single test truck

For the tests conducted in Lane 1W/1E, the position where the test truck is closest to the edge beams on the flared side of the bridge, the center span data indicate a different deflected shape in the WB and EB bridges with the damaged WB bridge deflecting more over a number of beam lines including those known to be undamaged. The deflections indicate that load is "shed" from the damaged beam lines in the WB bridge; this was later confirmed analytically. The maximum center span midspan deflection in the WB and EB bridges was measured to be 0.064 in. (1.6 mm) and 0.053 in. (1.3 mm), respectively. Diaphragm strains in both bridges were small during the Lane 1W/E loading, the maximum being approximately +15$\mu\varepsilon$ in both bridges, and agree with the findings of others that the diaphragm plays an insignificant role in live load distribution. Replacement of the damaged beams results in the WB repaired bridge behaving essentially the same as the undamaged EB bridge, thus, the change in behavior of the original WB and EB bridge data can be directly attributed to the presence of isolated main member damage.

The correlation between the effects of two trucks placed in adjacent lanes or in the same lane (i.e., multiple trucks on the bridge) is excellent compared to the effects of linear superposition of individual truck test results. The most significant deflection for Beams 1 and 2 in both bridges occurred when the two trucks were placed end-to-end in Lane 1, L1W/E-P4&P6 (see Figure 4). The deflection of the center span of the WB bridge was 0.085 in. (2.16 mm), approximately L/7,900 and was 0.083 in. (2.11 mm), L/8,100 in the EB bridge. Other beams in the WB bridge deflected more than their counterparts in the EB bridge by a small amount, i.e., 0.01 in. (0.25 mm) for most of the beams. The beams in both bridges typically deflect as if they are simply supported and have uniform stiffness or only slightly non-uniform as for the WB bridge. The maximum exposed strand strains were also recorded during the test with two trucks in Lane 1. The maximum strain recorded in Beam 1W was +186$\mu\varepsilon$ and in Beam 2W, +169$\mu\varepsilon$. These strains correspond to a stress range of approximately 5,300 psi (36.5 MPa) and 4,800 psi (33.1 MPa) in the two strands, respectively. These stress ranges are small and represent a stress range of less than 2% of the ultimate strength of the strand.

FIGURE 4 Truck location L1W-P4and P6

Analytical Results

In order to further understand the experimental results and quantify the effect of damaged beams on the load distribution pattern of the WB bridge, analytical models were created and calibrated using the experimental data. First, a series of undamaged models were created to predict the response of the repaired WB bridge and the undamaged EB bridge, which as previously mentioned, behaved essentially the same. Once comfortable with the correlation, damage was introduced into the model so that the analytical and experimental behaviors were in agreement. The results of these analyses indicate the difference in response of the damaged and repaired bridges.

Figure 5 depicts the analytical model and an example load placement for the repaired WB bridge. The figure depicts a stiffened plate model created using the STAAD-III software program. The deck was modeled using a shell element while the beams were modeled using eccentrically linked beam elements having the properties of the P/C beams. Loads were applied as point loads on the surface of the deck. In addition to the stiffened plate model, three-dimensional grillage models were also created in order to test the ability of different modeling techniques to capture the experimental response. Both types of models were able to reasonably simulate the experimental behavior. For the load case of two test trucks placed side-by-side in adjacent lanes as close to the damaged beams as possible, the analytical model predicts a live load moment in the most heavily loaded beam, Beam 2W, of 191 ft-kips (259 kN-m) as opposed to 252 ft-kips (342 kN-m) using the AASHTO distribution factor of S/5.5 for multi-lane loading. This demonstrates the significant conservatism of the AASHTO formulas for this bridge. A significant amount of moment is carried in the curb and rail section adjacent the loaded edge of the bridge.

Following the creation of the undamaged models, a damaged model was created using the “best” undamaged model as the starting point. Simulation of the amount and extent of damage is crude at best in that it is difficult to truly describe the extent and effect of the impact of the properties of the remaining beam. A simple procedure of ignoring a portion of the bottom flange based on visual inspection of the extent and severity damage was able to reasonably simulate the effects of the damage on the bridge performance. A comparison of the transverse deflected shape of the bridge as recorded in the field and predicted analytically is presented in Figure 6. The figure presents the measured deflection at midspan of Beams 11, 9, 7, 5, 3, 2, and 1W of the damaged WB bridge with two test trucks side-by-side with their rear tandems centered at midspan. The excellent agreement of the experimental and analytical results is apparent. Further examination of the

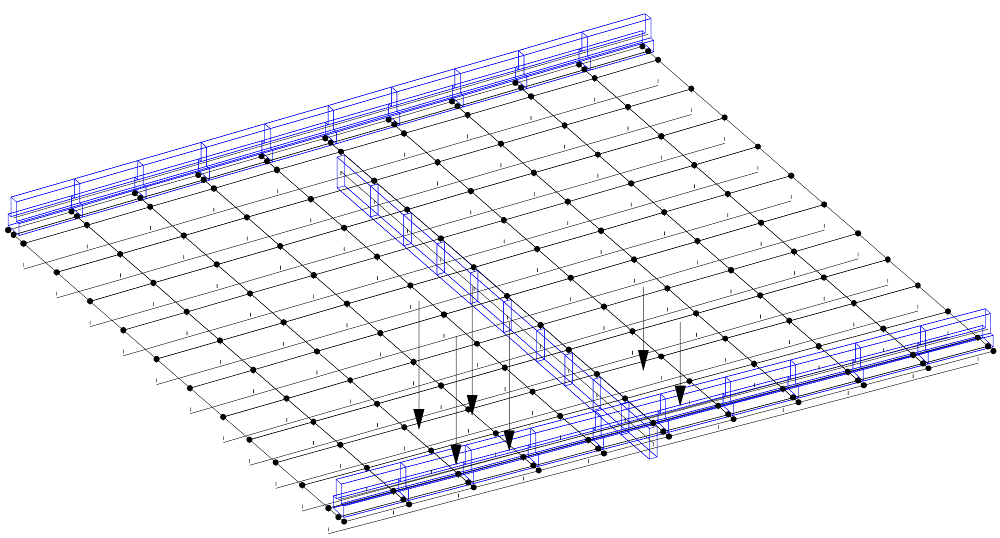

FIGURE 5 Analytical model of the repaired WB bridge center span; load at L1W-P4

analytical results indicates that for the isolated main members in this bridge, that is to Beams 1W and 2W, for loads placed over the damaged beam lines, a significant redistribution of moment occurs, much of it being taken up by the adjacent curb and rail as well as the nearest undamaged beam lines. The amount of load carried by beams remote from the damage is insignificant. The primary means of load redistribution is via transverse flexure of the slab.

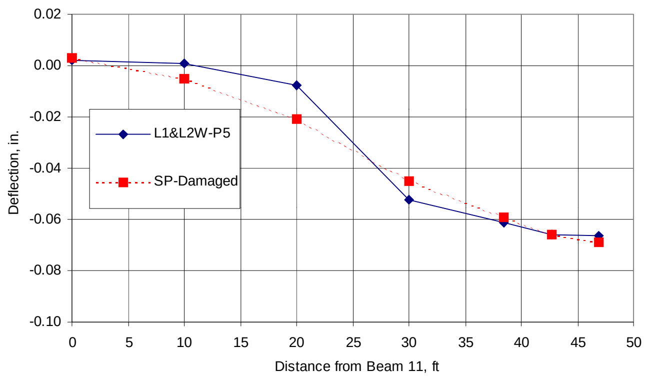

FIGURE 6 Damaged WB bridge experimental vs. analytical deflections; L1 and L2-P5

ISOLATED BEAM TESTING

The second important aspect of this project was to determine the feasibility of using CFRP laminates to restore the strength and to a lesser extent stiffness properties to damaged P/C beams. Although the experimental and analytical work conducted on this bridge indicated that in all likelihood the beams did not need to be removed for strength considerations, there may be instances in which structural strengthening is needed. The use of CFRP materials was seen as an attractive solution to the repair problem due to their high strength/weight ratio and their ease of installation. Long-term environmental performance of the materials was not studied.

Beam 1W and 2W were removed from the I-680 bridge in Beebeetown in October 1997. Following their removal, they were transported to the structural testing facilities of the University of Nebraska at Omaha located at Wilson Concrete in LaPlatte, Nebraska. Beam 1W was tested "as-is" as a baseline specimen while Beam 2W was to be intentionally damaged further and then repaired using CFRP materials. A photograph of Beam 1W in the test frame is presented in Figure 7.

FIGURE 7 Beam 1W in test frame; damage under left (west) actuator

Beam 1W (Baseline) Testing

At the time of the test, Beams 1W and 2W were inspected for the first time since the original damage inspection approximately two years prior. Following the initial damage and eventual removal of the beams from service, a protective tarp was installed to prevent loose debris from falling onto the county road under the bridge. It is known that the beams were hit several more times prior to removal due to the presence of several tears in the tarp. The amount of additional damage could not be documented. The notes from one of the researchers written the day of the test indicated that it was possible to see completely through the web. "With a minor amount of effort it would have been fairly easy to create a large void in the web simply by removing the fractured concrete." The researcher goes on to communicate that "the tension region damage is extreme…one load point is right over the damage. The damage extends over three stirrups and most of the three stirrups are exposed…I think this will be the source of failure in this test." These comments indicate damage much more severe than apparent from the initial inspection photos and description.

A baseline service load test and an ultimate load test were conducted on this beam. Prior to testing, the beam had one initially severed strand and an additional strand was intentionally severed in an attempt to measure the effective prestressing force in the strands. The service load test applied a maximum constant live load moment between the actuators of approximately 718 ft-kips (973 kN-m), approximately twice the design live load and impact moment for this beam during service. The load-deflection response of the beam was linear during the service test. Upward movement of the neutral axis was noted at midspan during the test, likely due to the influence of cracks. The neutral axis location at the undamaged quarter points was relatively constant throughout the service test. It is notable that when Beam 1W was still part of the I-680 WB bridge, the recorded exposed strand strain was +150 $\mu\varepsilon$ under the action of a single truck producing a moment of 655 ft-kips (888 kN-m) distributed to several beams. The corresponding deflection was 0.064 in. (1.63 mm). The strain and deflection in the isolated beam under a similar *directly applied* moment are +1,920 $\mu\varepsilon$ and 0.92 in. (23.37 mm), an increase of 12.8 and 14.4 times, respectively. The in-situ strains and deflections are only a small fraction of those measured in the isolated beam under similar applied moments. This dramatic increase in recorded strains in the isolated beam demonstrates the inherent load distribution and system redundancy in the damaged WB bridge.

Figure 8 depicts Beam 1W at the ultimate applied load. At ultimate load, the live load moment was 2,067 ft-kips (2,802 kN-m) and the midspan deflection was 8.62 in. (219 mm); the response of the beam was ductile. Failure of the beam was through the development of a large shear crack under the actuator placed over the damaged region of the beam. At ultimate load, the overlay partially debonded from the original slab. The flexural strength of this beam was 14% greater than that predicted by standard AASHTO code equations for a beam with three severed strands.

Beam 2W (CFRP Strengthened) Testing

The purpose of testing Beam 2W was to determine the effectiveness of CFRP retrofit techniques on the strength of a damaged and repaired P/C beam. Beam 2W was first further damaged by severing several strands then patched with a cementitious patching material,

FIGURE 8 South face of Beam 1W in the damaged region following the ultimate load test

CFRP longitudinal plates to replace the tensile capacity of the severed strands, and CFRP stirrups to help maintain bond between the longitudinal plates and the concrete (see Figure 9).

FIGURE 9 Beam 2W after repair with CFRP longitudinal plates (not shown) and CFRP stirrups

Following a series of service load tests in which it was determined that the repair had stiffened Beam 2W considerably as compared to its pre-retrofit response, an ultimate load test was conducted. It should be noted that in addition to the difference between Beam 1W and 2W in terms of one beam being damaged and the other repaired, Beam 2W had a considerably narrower composite slab, and the slab was more heavily damaged than in Beam 1W. For these reasons an exact A vs. B comparison is not possible for the two beams.

During the ultimate load test, the load vs. deflection response for repaired Beam 2W was substantially stiffer than for the baseline specimen Beam 1W (see Figure 10). This is due to the repair. At a load per actuator slightly more than 105 kips (467 kN), a midspan/damaged region live load moment of 2,480 ft-kips (3,362 kN-m), the beam failed catastrophically. Beam 2W completely collapsed at ultimate load. Although the failure of the beam was catastrophic, significant inelastic behavior and large deflections preceded the failure. The mode of failure is somewhat analogous to that observed in Beam 1W, failure of the beam directly under the west load point in the damaged region. At the time of failure, the overlay in this beam had partially debonded and spalled. However, this was not until the ultimate load was approached. The failure was sudden so a primary cause of failure was not observed though it appeared from assessment of the beam following the test that a combined shear/compression failure occurred in the damaged region. Up to near collapse, the longitudinal plates appeared to be well-bonded to the beam. The numerous external confinement stirrups were effective at ensuring bond between the plates and beam though numerous sounds were heard throughout the load test as the epoxy bonding the stirrups and plates to the beam cracked. It appears that at maximum load, the plates began to debond. This is evident by the decreasing strain in the plates following maximum load. The strains in the three longitudinal 5 in. x 0.08 in. (127 mm x 2 mm) CFRP plates bonded to the underside of the bottom flange were an average of +7,200με, approximately 85% of the failure strain of the cured laminate. The composite stress in the laminate was computed to be 122,400 psi (844 MPa) and the total force in the three plates, close to 147 kips (654 kN), equivalent to the capacity of approximately 3.5, ½ in.ϕ (12.7 mm), 270 ksi (1860 MPa) prestressing strands. Having only removed 2 strands from the section, this retrofit attained its design goal of replacing the lost tensile capacity of the damaged strands. A lesser amount of CFRP would have also achieved this objective and ensured a greater amount of displacement ductility.

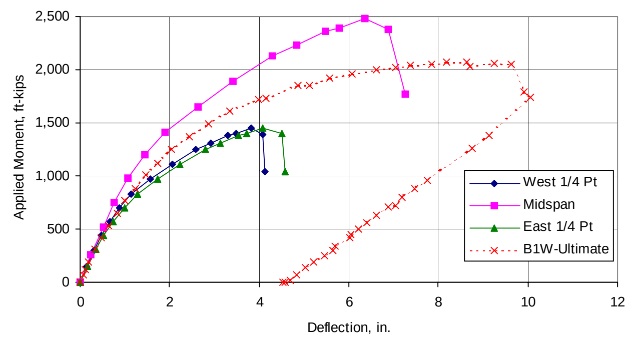

FIGURE 10 Load vs. deflection response; repaired Beam 2W vs. baseline specimen 1W

CONCLUSIONS

A sample of the results from a significant research program conducted at Iowa State University in the past several years has been presented concerning the field testing of damaged bridges and the performance of damaged beams. The results indicate the following:

· The damaged WB bridge behaved differently from the companion EB bridge and from the repaired WB bridge. The response of the repaired WB bridge and undamaged EB bridge is similar with the differences in response between these bridges and the damaged WB bridge attributable to the main member damage. Damage to the first two beam lines resulted in an observed experimental redistribution of load to beams otherwise undamaged.

· Analytical models of the repaired WB bridge were developed that correspond well to the measured response from the field tests. The analysis indicates that the moments in the most heavily loaded beam lines due to a variety of critical load placements are substantially less than those predicted by AASHTO equations. The models were then subjected to analytical representations of damage and the model calibrated to the field test results from the damaged bridge tests. It is observed that a significant amount of moment is redistributed away from the damaged beam lines, part being carried by the adjacent cub and rail and the rest by several other close beams.

- Following removal from service, two of the damaged beams were tested. Beam 1W, a control specimen, was not repaired but tested "as-is" to acquire the response of a damaged beam. The test indicated that the beam had sufficient strength to have remained in service. The failure was eventually in shear through the region of significant web damage, but only after the beam had deflected considerably. Beam 2W was repaired with CFRP longitudinal plates for flexural strengthening and external CFRP stirrups for bond and confinement. This beam was also tested to failure and attained a capacity, considering dead load effects as well, of approximately 12% greater than Beam 1W. The repair was successful in restoring strength and stiffness to the damaged beam. The fact that the failure was catastrophic is not to be misconstrued to imply there was no warning of impending failure. The significant deflections were clearly visible, and it must be remembered that there was no opportunity for redistribution of load as there would be had the beam been part of a complete bridge system.

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REFERENCES

1. Klaiber, F.W., T.J. Wipf, F.M. Russo, R.R. Paradis, and R.E. Mateega.. *Field/Laboratory Testing of Damaged Prestressed Concrete Girder Bridges – Final Report for Iowa DOT Project HR-397*. Department of Civil and Construction Engineering, Iowa State University. Ames, IA 1999.