Effects of Damage on the Behavior of Pretensioned/Prestressed Concrete Beam (PPCB) Bridges

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In June 1996, a series of field-tests were conducted on twin bridges carrying I-680 over county road L34 in Beebeetown, Iowa. The westbound bridge was damaged by an over-height load. The load fractured a portion of the bottom flange and web of the first two beam lines of the 11-beam structure. A majority of the strands were exposed in the process but no strands were severed. Several strands appear lax. The eastbound bridge is undamaged. The Iowa DOT decided to replace the two damaged beams due to uncertainties regarding their capacity and long term serviceability. Prior to removal of the beams, the westbound and eastbound bridges were load tested. Results from the tests are still being interpreted but the bridges do appear to behave differently. Results indicate that the differences may be only partially due to damage; the other causes being differences in end restraint of the center span and participation of the barrier rail in edge stiffening. Grillage models of an undamaged bridge have been developed and are shown to reasonably agree with the experimental results of both the damaged westbound and undamaged eastbound bridges. The analytical models are more flexible than the experimental results. This is likely due to neglect of the barrier rail stiffness and the treatment of the beams as simply supported. The influence of intermediate diaphragms has been analytically shown to have little effect on the distribution of loads in exterior lanes. Key words: prestressed, bridge, damage, load testing, analysis.

INTRODUCTION

This paper describes the results of a field test conducted in June 1997, on two similar pretensioned/prestressed concrete beam (PPCB) bridges located in Beebeetown, Iowa. The bridges carry I-680 eastbound and westbound over county road L34 in Pottawattamie County. Figures 1 and 2 present the geometry of the two bridges, instrumentation and test vehicle locations, as well as test vehicle geometry and load.

The impetus for this research project was the collision of an unknown vehicle with the north three beams, beams 1W - 3W, of the westbound structure in July 1996. Damage was centered ± 5' (1525 mm) west of the mid-span diaphragm of the center span.

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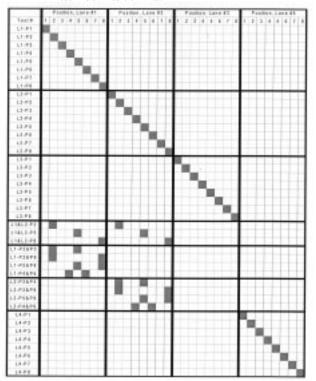
Approximately 6' (1830 mm) of the bottom flange spalled from the north fascia stringer, beam 1W, exposing the bottom two layers of 0.5" (12.7 mm) diameter strands. Several of the strands on beam 1W seem to be lax but no strands were severed during this collision. There was a preexisting severed strand from a 1993 collision. Cracking of the bottom flange and web as well as fracturing of the core concrete is present on both beams, but to a lesser extent on beam 2W. Cracking appears to have been arrested by the presence of the 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.

Prior to the initiation of this research project, the IADOT decided that beams 1W and 2W would be replaced while beam 3W would be patched. The decision to replace beams 1W and 2W was made due to uncertainties concerning the remaining strength, effect of damage on load distribution, and long-term serviceability of the damaged beams. IADOT's decision to support a study on load distribution and remaining strength in damaged PPCB bridges is an attempt to develop a more refined criteria through which the effects of damage on the behavior and strength of such bridges can be more accurately assessed. The project has two objectives: determine the effects of damage on load distribution and assess the remaining strength of a damaged member.

FIELD TESTING OF THE I-680 BEEBEETOWN BRIDGES

The focus of the load testing portion of this project was to track the flow of forces in a damaged and complementary undamaged PPCB bridge. The project endeavored to answer the following questions:
- Are redundant load paths available to assist in load sharing?
- What load is on the damaged beam(s)?
- Has the redistribution of load following damage overloaded any elements that were not originally damaged?

The answer to the first question is evident. It is well known that typical multiple stringer bridges are highly redundant in that "failure" of an individual superstructure element does not constitute collapse of the structure. This is primarily due to the interconnection of adjacent stringers through a common slab and secondarily due to the presence of intermediate diaphragms. However, the answer to the load distribution questions could only be determined through testing and subsequent data analysis of the two bridges.

The static tests conducted on the Beebeetown bridges used loads of known magnitude and configuration. The tests employed two IADOT supplied maintenance vehicles (dump trucks) described in

TABLE 1 Test Truck Position Matrix

Figure 2. Most of the tests were conducted with a single truck, IA-4280, at various longitudinal and transverse positions. The tests were designed to place the truck at a specified transverse location on the bridge, i.e. a specified "lane" location, and incrementally move the truck while taking readings at predetermined positions. The predetermined truck locations are illustrated in Figure 2 and described by Table 1.

Table 1 presents the test name and indicates by shading the position of the center of the rear tandem of the test vehicle(s). Tests in which only one block is shaded are tests in which only one test vehicle, IA-4280, was employed. In the tests in which multiple trucks were used in the same lane, i.e. L1-P2&P5, truck IA-4280 was the lead vehicle. In tests where trucks were side-by-side in adjacent lanes, i.e., L1&L2-P5, IA-4280 was closest to the flared edge of the structure.

A number of deflection transducers and electrical resistance strain gages were used to instrument the bridges. Celesco PT101 and Unimeasure HX-PA displacement transducers were used at various locations as shown in Figures 1 and 2. Micro Measurements 032UW 'C Feature' gages were used to acquire strains in four of the exposed strands of the westbound bridge. Precision Measurements F-2400-06 foil gages were installed on the center span intermediate diaphragms connecting beams 1 through 4. F-2400-6 gages were also installed at the end regions of beams 1 and 2 near the pier diaphragm. These gages were intended to detect any strains at the ends of the beams which would be indicative of a degree of continuity. Refer to Figure 1 for partial details of the instrumentation layout.

TEST RESULTS

Selected results from the tests on both the eastbound and westbound bridges will be discussed in this section. The results are still being interpreted and compared to analytical predictions at the time of this writing. The intent of the experimental/analytical comparison is to develop computer modeling guidelines for damaged structures that will allow practicing engineers to more accurately assess the effects of localized damage on the overall behavior of damaged PPCB bridges. The following text will selectively comment on the observed behavior of the two bridges tested. Results from only the center span will be presented.

Figure 3 is a plot of several transverse deflected shapes taken at midspan of the damaged center span of the westbound bridge and the complementary location in the eastbound bridge. L1W-P5(A) and (B) are tests performed at the same location of the westbound bridge and illustrate repeatability of the experiment. L1E-P5 is a similar test of the eastbound bridge. The accompanying series entitled STAAD-1 (w/o) and (w) are computer models where the effects of the midspan diaphragm were included (w) or excluded (w/o) from the model. The model presumes that the members are undamaged, there is no edge-stiffening participation from the barrier rail, and that all beam ends are simply supported (i.e. continuity with the tail spans is ignored). It is reasonable to initially assume that there is no continuity because of the lack of pintles in the curved sole plate pier bearings along beam lines 1, 2, 10, and 11, along with the fact that the pier diaphragm does not encase the beam ends.

Figure 3 illustrates several key points. First, there is close agreement between the undamaged analytical models and the behavior of the damaged westbound bridge. This may either indicate that the damage is such that it has no measurable effect on the response of the bridge or that a number of other factors such as barrier rail stiffening or interior beam line continuity result in a deflected shape that appears undamaged. If in fact the analytical model is too "soft," the behavior of a more representative model may tend to more closely match that of the eastbound bridge and one could conclude that the damage was sufficient to alter the behavior of the westbound bridge. Figure 1 also illustrates the negligible change in deflected shape due to the presence of a midspan intermediate diaphragm. This would tend to indicate that the exterior beam line intermediate diaphragms have little effect on overall deflection patterns.

Figure 4 depicts the experimental and analytical transverse response when loads are moved to the second longitudinal lane line. In this figure it is again apparent that the analytical models are more flexible than either of the bridges tested. The westbound bridge appears to be resisting the majority of load in a localized area surrounding the point of application more so than the other scenarios. The presence of intermediate diaphragms has a more pronounced effect on the response than when loads are placed in lane 1.

Figure 5 illustrates a large difference in the experimental and analytical responses of the bridges. In this series of tests, truck IA-4280 was positioned to be 2' (610 mm) from the median rail. The damaged region of the westbound bridge would have little to no effect on this load location and the eastbound bridge is not known to be damaged at all. As expected, the bridges behaved similarly with the westbound bridge being somewhat softer in the transverse direction. However, there is a great disparity in the predicted and measured response. The predicted response on the median side is
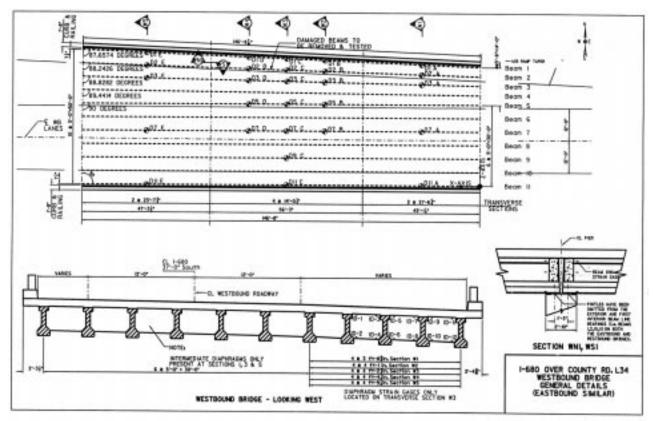
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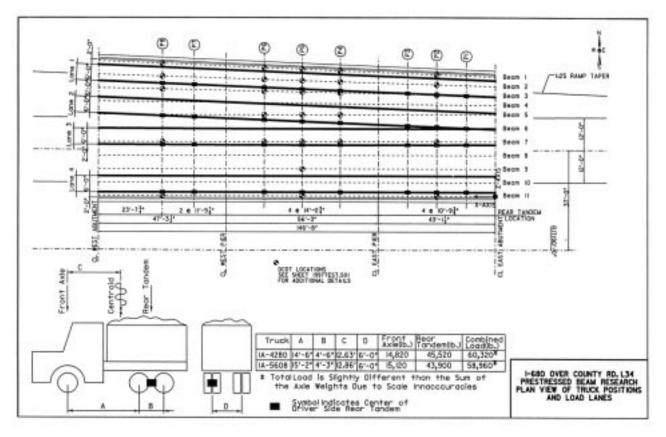**FIGURE 1 General plan and instrumentation layout.**

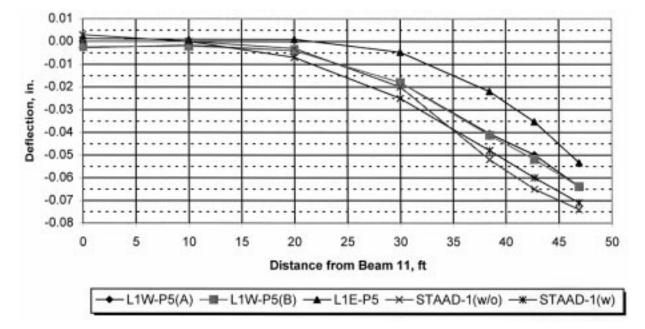**FIGURE 2 Test truck positions, geometry, and axle loads.**

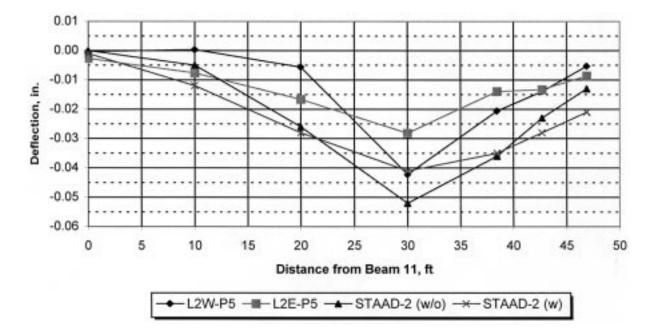**FIGURE 3 Transverse deflection, midspan, center span, lane 1 loaded.**

FIGURE 4 Transverse deflection, midspan, center span, lane 2 loaded.

expectedly larger than on the flared side due to the greater beam spacing. However, both the eastbound and westbound bridges deflected less on their median sides than the flared sides. The only explanation for this behavior is a much stiffer structure than analytically modeled. This stiffness is likely due to barrier rail participation and unaccounted for continuity effects.

Figure 6 presents the analytical and experimental results for the case where both loaded test vehicles were placed side-by-side with their rear tandems at midspan. The westbound bridge appears somewhat softer than the eastbound, but the characteristic shape of the transverse response is the same. The analytical models are once again more flexible than the experimental response. It is of interest
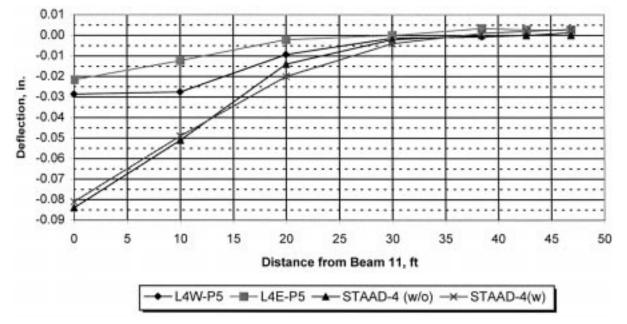
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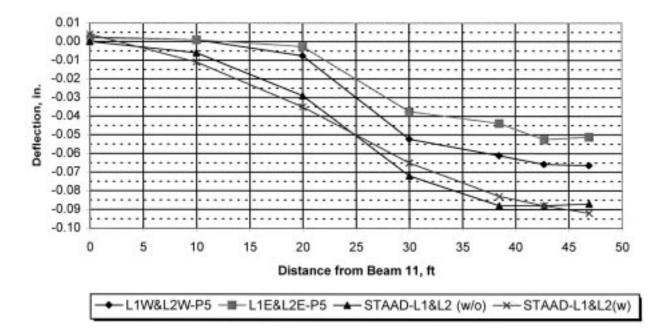**FIGURE 5 Transverse deflection, midspan, center span, lane 4 loaded.**

FIGURE 6 Transverse deflection, midspan, center span, lanes 1 and 2 loaded.

to note that the intermediate diaphragms appear to once again have little effect on the transverse deflected shape (load distribution) pattern of the bridge. The effect is only a slight flattening of the transverse response with a correspondingly small increase in deflection and moment to the exterior beam.

Examination of the diaphragm strain gages indicates that strain is consistent with deflection. Tests in which appreciable deflections were recorded in beams 1-5 tended to have higher strain readings in the diaphragms. Examples of these tests are the single and multiple truck tests in lanes 1 and 2. In these tests, diaphragm

strain readings as high as ±15 με were recorded in the westbound bridge and ±35 με in the eastbound bridge. When comparing similar load positions in the two bridges, the eastbound bridge diaphragm strain gages consistently read higher than the westbound bridge gages.

The beam end strain gages gave an intermittent indication of continuity though the indications were stronger and more consistent on the eastbound bridge. The most pronounced response in the westbound bridge was in beam 1 at the bottom flange strain gage on the center span beam. The highest reading under a single truck was with a truck positioned at midspan of the center span. The reading was -31 με. Side-by-side loads did not increase the strain response in beam 1. In a test specifically designed to check for continuity over the piers, loads were positioned at P5 and P8. In this scenario, the maximum strain recorded was -46 με. The highest strain readings were with trucks at P4 and P6, a reading of -48 με.

In contrast to the westbound bridge strain readings are those obtained on beam 2 of the eastbound bridge. In tests under a single truck in lane 1, the bottom flange strain gage indicated compressive strains of -165 με with a load at midspan of the center span. The corresponding bottom flange strain gage in the east tail span was not comparable. This is an indication of restraint due to factors other than full longitudinal continuity. In side-by-side tests, this strain increased to -200 με and when trucks were placed at P4 and P6 so as to maximize the amount of load in the center span, the bottom flange strain gage read -235 με. Although no conclusions can be drawn about the amount of restraint present in other uninstrumented beam lines or spans of the eastbound bridge, nor the source of such restraint, the existence of these high compressive strains lends credence to the argument that a certain degree of

negative moment capacity exists at the piers of this bridge. It is interesting to note that on both the eastbound and westbound bridges, only the bottom flange gages measured any appreciable strain. The web and top flange gages gave very little strain indication whatsoever.

SUMMARY

This paper has briefly described an ongoing investigation into the behavior of impact damaged ppcb bridges. As of this point, the bridges have been tested in the field and evaluation of that data continues. Tentative conclusions indicate that the eastbound and westbound bridges are behaving differently. This may either be de to damage or other differences in behavior. The deflections measured under known loads were small, the maximum being less than 0.1' (2.5 mm). Strains have been detected in both the intermediate diaphragms and at the beam ends. Strain values were higher in the eastbound bridge with the beam end strains being substantially so.

Further calibration of the analytical models is required to assess the sensitivity to factors such as loss of stiffness in a damaged member, effects of varying degrees of end restraint, and changes in structural behavior due to barrier rail participation. In the summer of 1998, the two damaged beams removed from the westbound bridge will be tested in order to determine the effect of the impact on their stiffness and strength. An additional three beams will be tested that have been damaged in the laboratory. Following these tests and the completion of a number of other analytical tasks, it is the intent of this project to provide a series of recommendations regarding when it is necessary to consider replacing a damaged ppc bridge beam.