FULL SCALE DESTRUCTIVE TESTING OF AN ADJACENT PRESTRESSED CONCRETE BOX BEAM BRIDGE

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ABSTRACT

The paper discusses the full scale destructive testing of an adjacent prestressed concrete box beam bridge. The bridge was located in Fayette County, Ohio and had been in service for 43 years. The non-composite bridge consisted of three spans each 47’10” with an asphalt overlay. Each span consisted of 9 box beams that were 36” x 21” and contained 27 – 3/8” diameter strands. The facia beams on all spans showed significant levels of deterioration. Only one interior beam (in the east span) showed deterioration with a longitudinal crack and minor spalling of concrete. The center span was tested in an undamaged state as a baseline, then the east and west spans were manually damaged to different levels of severity and tested. The full scale testing involved loading each span with hydraulic cylinders supported by a frame anchored through the abutments and/or piers. Instrumentation was used to measure deflections and strains across the width of each span and to monitor the load during the testing procedure. Various loading patterns were used to understand behavior. The results showed significant load transfer between the beams until the shear keys and lateral ties failed. Failure of the spans typically occurred with crushing of the top flange on some of the beams within the span. Preliminary calculations showed that the total capacity could be predicted by adding together the calculated damaged capacity of each beam.

Keywords: Bridge Testing, Adjacent Prestressed Concrete Box Beams, Damage, Deflections, Failure.

INTRODUCTION

This paper describes the results from the full scale testing of the three spans of a 43 year old adjacent prestressed concrete box beam bridge. This research was the second phase of an overall project to evaluate box beams with deterioration. The first phase involved the forensic study and destructive testing of damaged individual beams removed from a similar type of bridge[1]. The results of phase I showed the differences in behavior of the damaged beams on an individual basis. However, these beams do not behave as single members in adjacent member bridges. For adjacent box beam bridges, the beams are placed adjacent to each other and tied together through shear keys as well as transverse tie rods. This adjacent placement causes a significant difference in the behavior of the bridge system compared to an individual beam.

Though some full scale destructive bridge testing has been performed[2-4], research does not currently provide experimental results for adjacent prestressed concrete box beam bridge system behavior. This lack of analytical and experimental verification on the behavior of damaged prestressed box beam bridge systems affects economics. An overly conservative approach to evaluating a bridge based on individual member capacity and undamaged load distribution behavior may lead to premature load restriction of the bridge, closing of the bridge, or replacement of the bridge. In addition, replacement of a bridge may be considered necessary when in fact a more economical solution may be repair of the bridge to extend its service life. Finally, not completely understanding the behavior of the bridge system compared to individual member behavior could lead to unexpected premature failure leading to damage of public property, personal injury, or worse. This occurred in December of 2005 when an outside girder from the Lake View Drive Bridge over I-70 failed[5-6]. It has also been noted that during removal of some damaged beams, collapse of the beams under their own weight have occurred once shear keys and transverse ties have been cut.

BACKGROUND

The bridge was identified with the assistance of the Ohio Department of Transportation technical liason, Mike Loeffler and Fayette County Engineer, Steve Lubbe. The bridge was slated for replacement in 2010 and located on Fayette County Road 35 (FAY 35- 17-6.80) northeast of Washington Court House. The bridge was put into service in 1967 and crossed the North fork of Paint Creek (see Figure 1). According to the plans each span was 47’ 10” in length with a left forward 15^{o} skew (see Figure 2). Transverse ties, 1” in diameter, were located approximately at the third points of the spans. Each span consisted of nine prestressed concrete box beams. The spans were referred to as the East, Center and West spans and the beams in each span were number 1-9 from south to north.

The beams were 21” deep and 36” wide with 5” thick flanges and webs. The voids were created with cardboard forms and drawings for the bridge showed a total of 27 - 3/8” diameter 250 ksi prestressing strands. A total of 3 rows of strands existed, with 14 strands in the bottom row, 9 strands in the middle row and 4 strands in the top row. Mild reinforcement also existed at the top of the beam in the form of 4 #5 bars (see Figure 3). Shear reinforcement on the bottom of the beam consisted of 6 - #4 bars spaced at 7½" at each end of the beam and then 15" throughout the remaining length of the beam. The shear reinforcement on the top of the beam consisted of #4 bars at 7 ½" spacing. The bottom row of strands existed below the bottom shear reinforcement. The connection of the beams over the pier involved the lap splice of a #4 bar with a length of 2 feet.

Figure 1: FAY 35-17-6.82

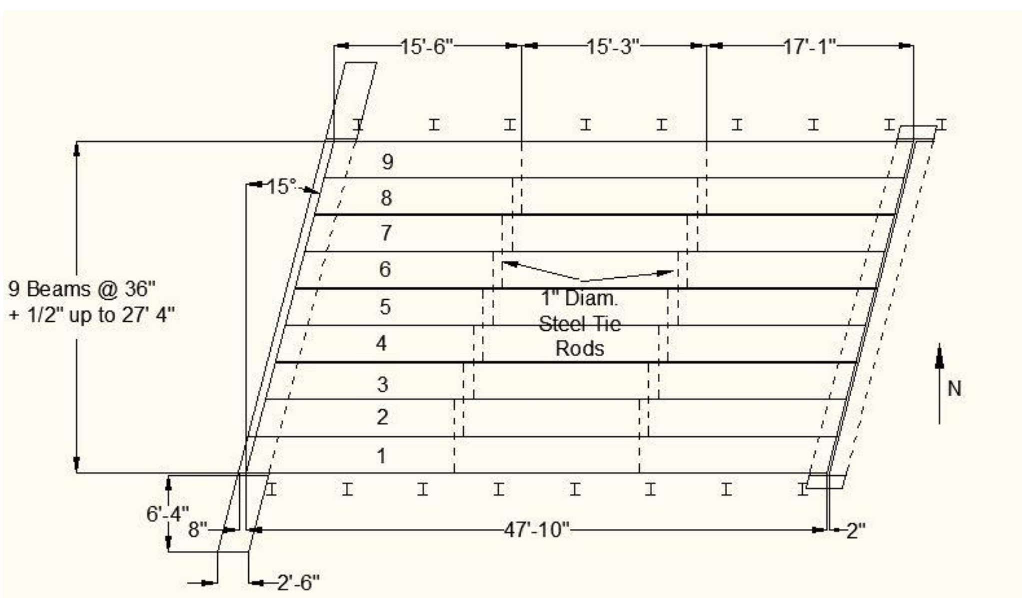

Figure 2: FAY 35-17-6.82 Bridge Span

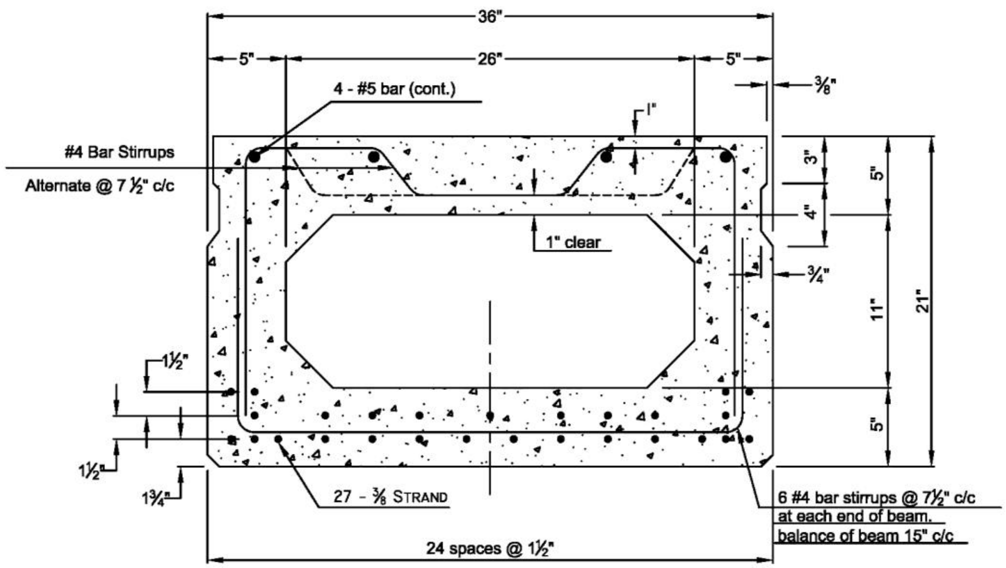

Figure 3: FAY 35-17-6.82 Beam Section

TEST SETUP

In order to perform the testing, three steel test frames were designed and fabricated. Each steel test frame consisted of two W36 x 260 longitudinal beams approximately 50 feet in length and spaced 18" apart. Each end of each frame was supported on two W33 x 118 approximately 4'6" in length and spaced approximately 4" apart. Figure 4 shows the frames in place on the first span. The test frames were anchored through the abutment or pier with a 1 ¾" Dywidag post tensioning Threadbar with an ultimate capacity of 400 kips at each end of the frame. The Threadbars were anchored to the top of the W33 x 118 and the bottom of the abutment/pier with plates and nuts (see Figure 5). A hydraulic cylinder with a capacity of 350 kips was mounted between each test frame (see Figure 6). The load from the cylinders was transferred into two small W6x25 spreader beams. The length of the spreader beams was approximately 6 feet for the east span. The lengths were reduced for the center and west span testing to 4 feet to concentrate the load more.

Figure 4: Test Frames on East Span

Figure 5: Abutment Test Frame Anchorage

Figure 6: Hydraulic Cylinder

TESTING PROCEDURE

EAST SPAN

The East span was damaged by removal of concrete with jack hammers to expose strands. The strands were then flame cut with a torch (see Figure 7). Damaging occurred near mispan for a length of approximately 50 to 60 inches and a width of approximately 9". Figure 8 shows the damage pattern for the East span. It should also be noted that Beam 3 had longitudinal cracking near the pier and Beam 9 had the exterior strand exposed and broken.

One strain gage was installed on the top and one on the bottom of each beam at instrument lines G and H (see Figure 8). A small frame was erected on each side of the bridge and supported on the pier and abutment. Steel tubes attached to the frames ran across the bridge along the instrument lines. String potentiometers were mounted to the steel tubes and the strings were attached to the bridge with anchors, one per beam per instrument line (see Figure 9). The instrumentation frame allowed deflections to be measured independent of bridge deformations. All instrumentation from the East span was connected to a high speed data acquisition system to monitor and record data during the testing. Loading the east span consisted of applying 50 kips of load to a cylinder, removing the load, applying 50 kips of load to another cylinder, and so on until each cylinder was loaded and unloaded. The process was repeated to a load of 100 kips per cylinder. A load of 50 kips was then applied to each cylinder consecutively without unloading, followed by an increment to 100 kips and so on until the bridge was no longer able to maintain additional load. All loading was performed in a displacement control mode to assure safety.

Figure 7: Exposed and Cut Strands – East Span

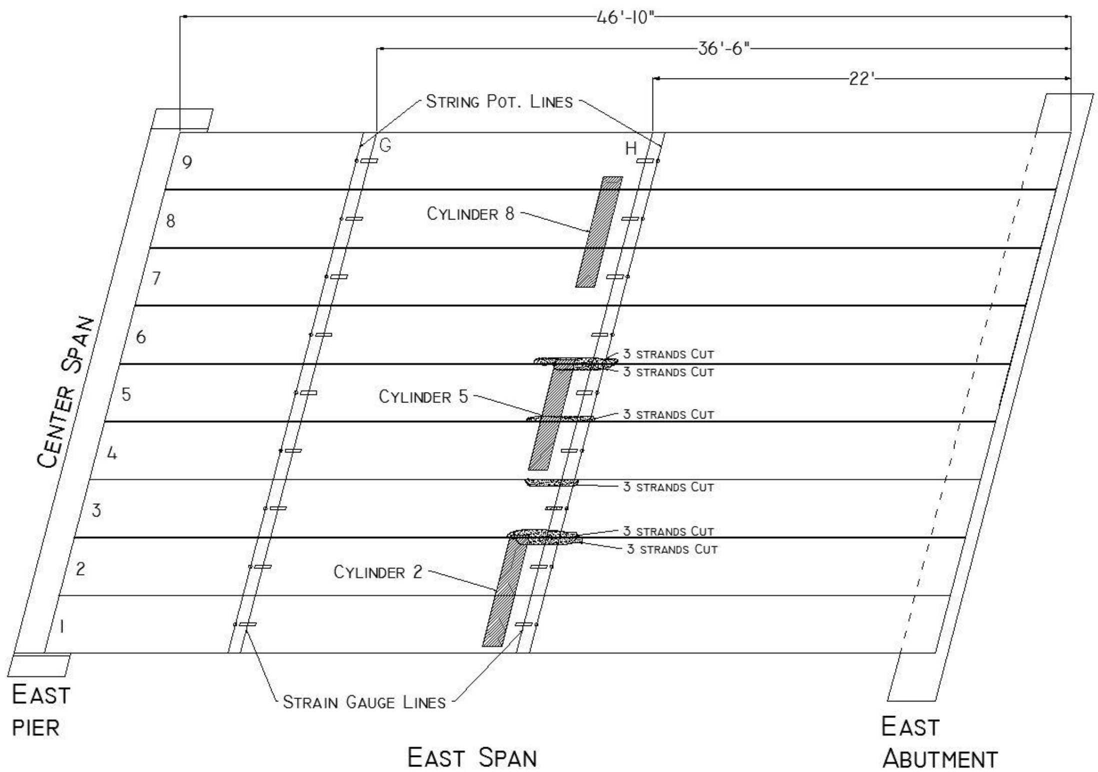

Figure 8: Damage of East Span

Figure 9: Steel Tubes with String Potentiometers

CENTER SPAN

No damage of the Center span was induced in order to obtain data for the existing condition of the span and to serve as a control. Only strain gages along instrument line E (see Figure 10) were mounted to the bottom of the bridge since this span was over water and had limited access. On the top surface, part of the asphalt was not removed and hence did not allow the installation of strain gages to Beams 1-3. The asphalt was not removed at the request of the County Engineer to see the effect of the asphalt overlay. Loading was similar to that performed on the East span.

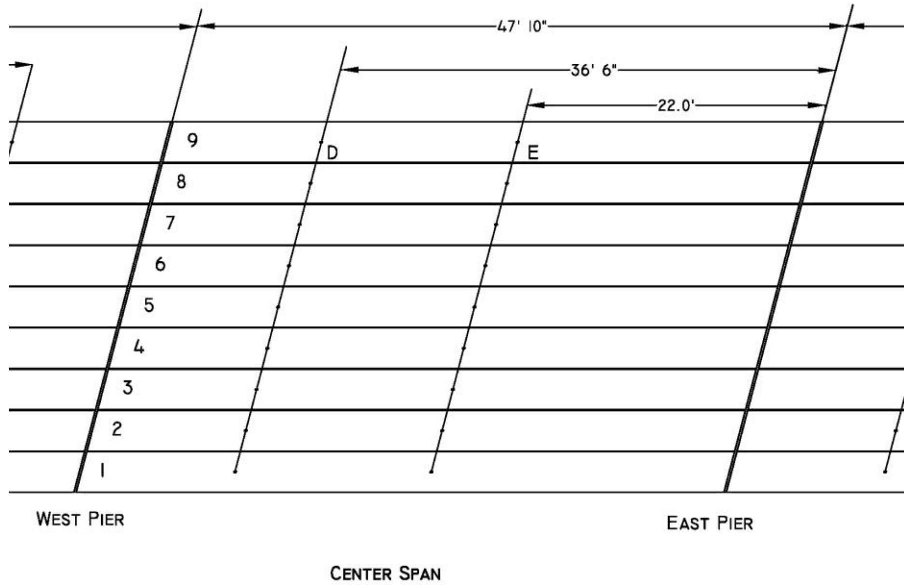

Figure 10: Instrumentation Locations for Center Span

WEST SPAN

The damage imposed on the West span consisted of cutting through the bottom of the concrete and through all of the bottom row strands in the three center interior beams (Beams 4-6). Two cuts were made approximately 3 feet apart about the mid-span of the beams to assure no redevelopment of the strands (see Figure 11). Instrumentation (see Figure 12) and loading on the West span was similar to the East span. Since the West span was the last span to test, loading continued until the span collapsed.

Figure 11: Damage Created in West Span Beams 4-6

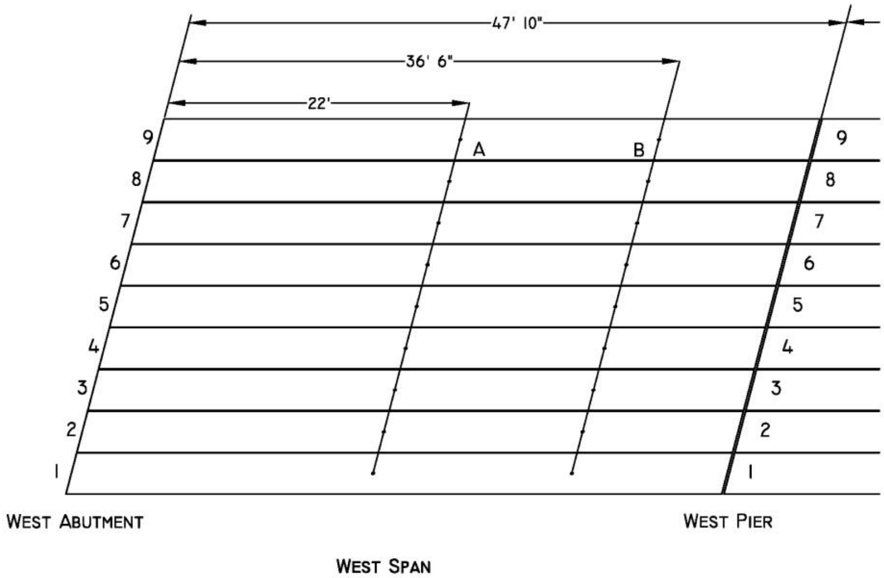

Figure 12: Instrumentation Locations for West Span

RESULTS

For brevity, results are provided only when all cylinders are loaded. In addition, results are provided at instrument lines near midspan. The other instrument lines show similar results with lower magnitudes in deflections and strains.

EAST SPAN

The deflection data recorded from the East span was very erratic due to what was believed to be excessive movement of the support frame for the string potentiometers. However, quality load and relative strain data still provided significant information on the behavior of the span.

The top strains along instrument line H across the width of the bridge while loading was applied to all cylinders is shown in Figure 13. Strain was not monitored in Beam 9 since the poor condition of the concrete did not allow the mounting of a top gage to the beam. In general, the strains increase with increasing load. However, Beam 4 has significantly higher strain as the total loading increased above 249 kips. Table 1 provides the cylinder loads for each strain plot in Figure 13. The cylinders are numbered based on the Beam they were primarily above. Though the load is highest on cylinder 5 for some of the total load plots, it is not significantly higher than the load in cylinder 2 or 8. At the 364 kip total load, the load on cylinder 5 is even lower than the load on cylinder 8. Therefore, it is concluded that a larger portion of the load is being transferred into Beam 4. This makes sense since Beam 4 was not damaged.

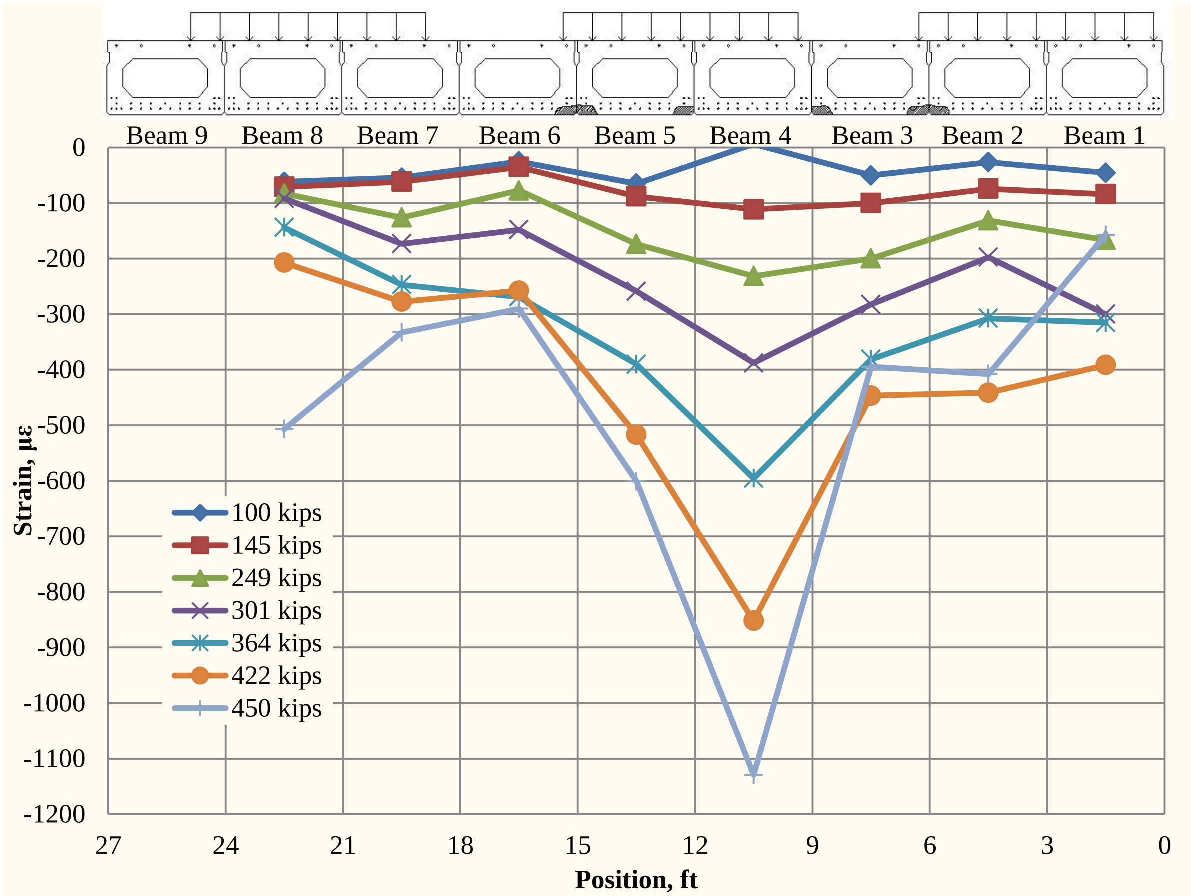

Figure 13: Top Strain Profile Instrument Line H

Table 1: Cylinder Load Distribution (East Span)

Cylinder Load (kips)	Cyl. 2	0	59	62	76	100	132	138
	Cyl. 5	52	45	97	118	125	156	156
	Cyl. 8	48	42	90	107	138	135	156
Total Load (kips)		100	145	249	301	364	422	450

During ultimate loading of the East span, flexural cracking was initiated in the beams and then resulted in water leaking from the cracks indicating the voids of the boxes contained water. This water was significant judging from the duration of leakage (see Figure 15). The bridge failed by crushing of the top flange concrete in Beams 1 and 9. Additional load was no longer able to be supported by the bridge and testing was terminated in order to assure damage to the loading system did not occur. Figure 16 shows the flexural cracking and concrete crushing failure of Beam 1.

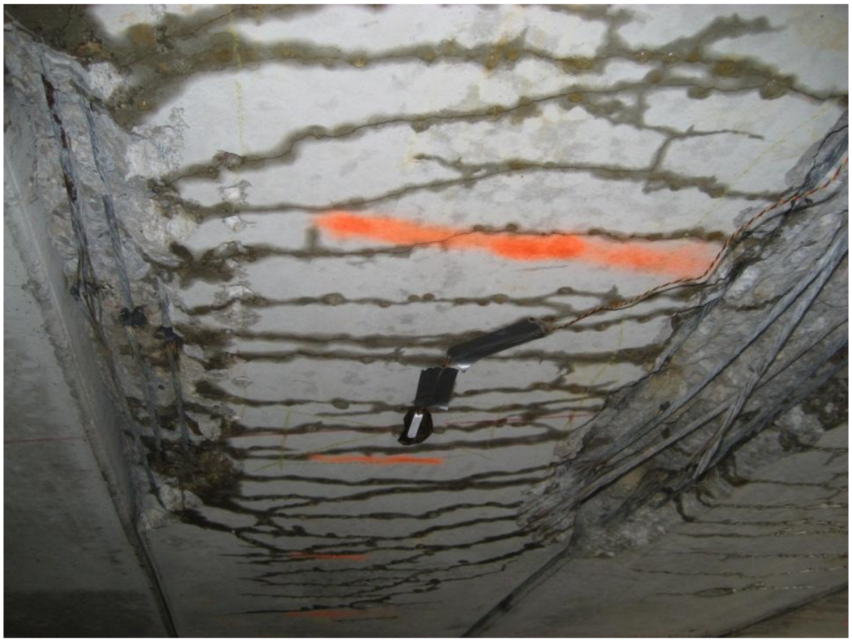

Figure 15: Flexural Cracking and Void Water Leakage

Figure 16: Flexural Cracking and Concrete Crushing in Beam 1

CENTER SPAN

Figure 17 shows the deflection across the bridge at instrument line E as the total load from all cylinders increased. Table 2 provides how the total load was distributed from the three cylinders. When the maximum load of approximately 450 kips was obtained, the bridge was no longer able to support higher total load and deflections increased. Initially, the deflections are fairly consistent across the width of the bridge. However, after the maximum load was reached Beams 1, 2 and 8 showed slightly higher deflections even though more load is applied from cylinder 5 to the middle bridge beams compared to the outside beams. The deflection profile remained the same even after the load was decreased. The deflection across the width of the bridge is much higher at lower loads after the peak loading was reached, resulting in permanent deformation. It should be noted that the data for the string potentiometer attached to Beam 5 was not usable from the test.

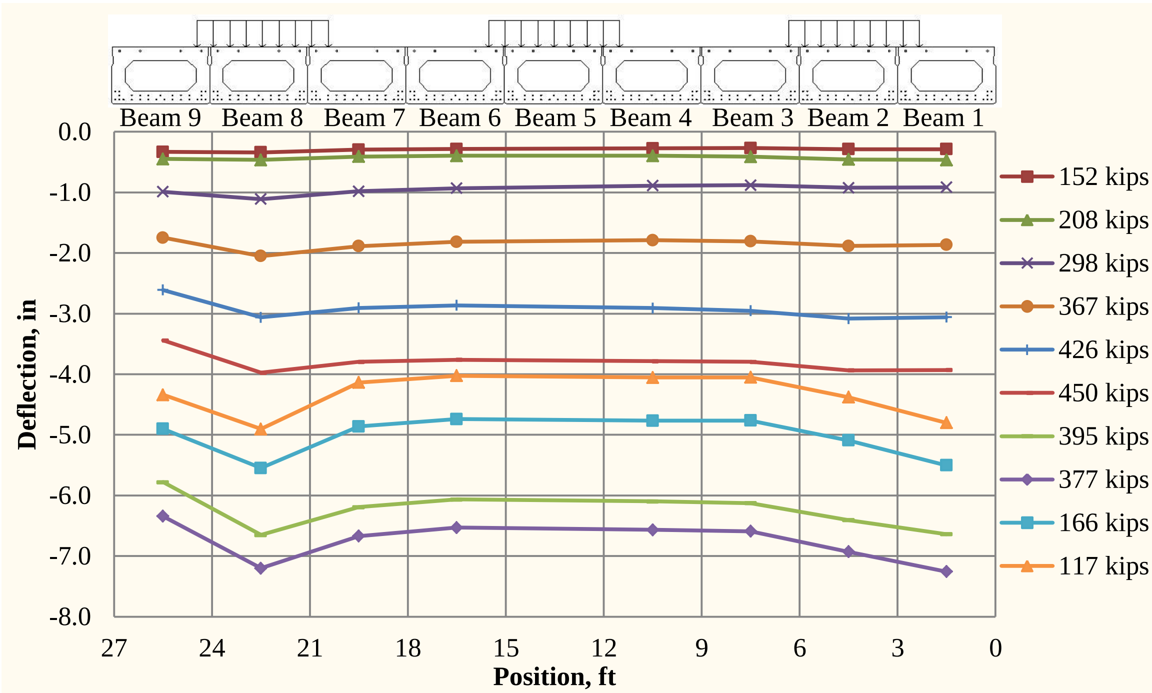

Figure 17: Total Load and Instrument Line E Deflection (Center Span)

Table 9: Cylinder Load Distribution (Center Span)

Cylinder Load (kips)	Cyl. 2	52	83	100	132	163	149	100	97	35	21
	Cyl. 5	45	59	87	104	135	159	180	183	93	73
	Cyl. 8	55	66	111	132	128	142	114	97	38	24
Total Load (kips)		152	208	298	367	426	450	395	377	166	118

The Center span failed to carry more load after the girders failed by crushing of the top flange and flexural cracking of the bottom flange. Figure 18 shows large relative deflections between Beams 2 and 3 caused by likely failure of the shear keys between the beams. Figure 19 shows the underside of Beams 8 and 9 along with the cracking and spalling of concrete.

Figure 18: Relative Deflection of Beams 2 and 3

Figure 19: Spalling and Cracking of Beams 8 and 9

WEST SPAN

Figure 20 shows the deflection across the bridge at instrument lines A as the total load from all cylinders increased. Table 3 provides how the total load was distributed from the three cylinders. When the maximum load of 363 kips was obtained the bridge was no longer able to support higher total load and deflections increased. Initially, the deflections are fairly symmetric about Beam 5. The deflection of Beam 9 begins to show less of an increase in deflection compared to all the other beams after a total load of 251 kips. At the maximum load and loads afterward, the deflection of Beam 9 relative to the remaining beams is very noticeable. The behavior is likely due to loss of the shear key for Beam 9. Beams 4-6 show the largest deflection as the load is highest in cylinder 2. With the exception of Beam 9, the deflection is relatively uniform across the bridge even after the maximum load has been reached.

Figures 21-22 show the west span prior to collapse. The large relative deflections shown between Beams 8 and 9 and also Beams 6 and 7 in Figure 21 imply the loss of the shear key between those beams. The shear key between Beams 3 and 4 has also failed (Figure 22). However, the bridge was still able to maintain and transfer load throughout the system. This was likely due to the transverse tie rods existing between the beams as well as some shear friction between the shear keys and beams.

Loading was stopped and the string potentiometers were removed to prevent damage. Loading was again applied to the span without being able to exceed the previous maximum total loading. Eventually the span completely collapsed in a ductile manner (see Figure 23 - 24). Beams 1-8 failed near midspan close to the loading locations and Beam 9 failed near the abutment. Beams 7 and 8 pulled sufficiently away from the pier and fell to the ground near the pier. The transverse tie between Beams 8 and 9 (see Figure 24) and Beams 6 and 7 fractured allowing Beams 7 and 8 to be separated from the remaining beams. Beam 9 had lost its top flange which was a patched section at this location. A combination of separation of the top flange, poor concrete, and shear resulted in the failure of Beam 9.

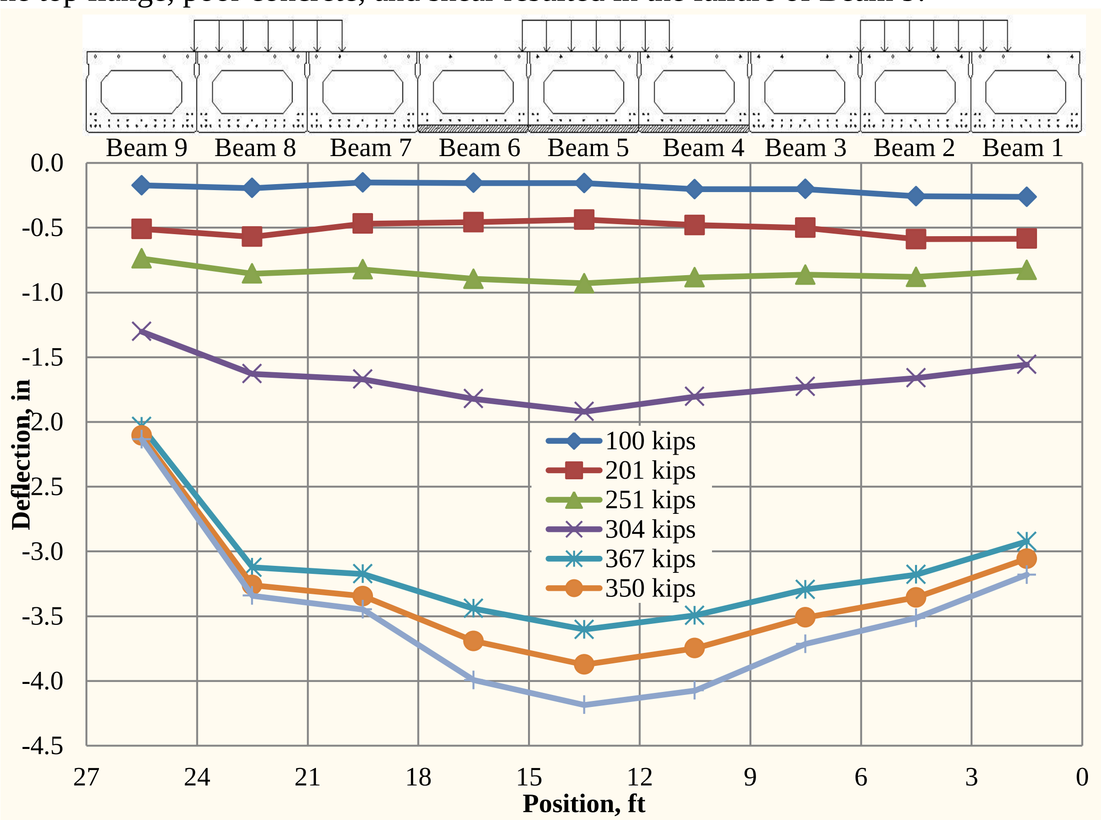

Figure 20: Total Load and Instrument Line A Deflection (West Span)

Table 3: Cylinder Load Distribution

Cylinder Load (kips)	Cyl. 2	58	101	53	36	83	52	31
	Cyl. 5	0	0	140	226	201	239	242
	Cyl. 8	42	100	58	42	83	59	49
Total Load (kips)		100	201	251	304	367	350	322

Figure 21: Shear Key Failures Between Beams 8-9 and 6-7 Prior to Collapse

Figure 22: Shear Key Failure between Beams 3 and 4 Prior to Collapse

Figure 23: Collapsed West Span

Figure 24: Fractured Transverse Bar (West Span)

SPAN COMPARISON

In order to compare the destructive test results for the different spans, the plots of the average midspan deflection for the Center and West spans versus the total applied load was created and is provided in Figure 25. As shown, both spans behave in a similar linear manner up to approximately 175 kips even though the West span had significantly more damage. After approximately175 kips, the West span begins to become nonlinear as the Center span continues to be linear to approximately 225 kips. At larger loading, the West span shows more average deflection. At the peak load for the West span, the average deflection is approximately twice that of the Center span. The Center span obtains the deflection of West span peak load at approximately 70 additional kips (430 compared to 360).

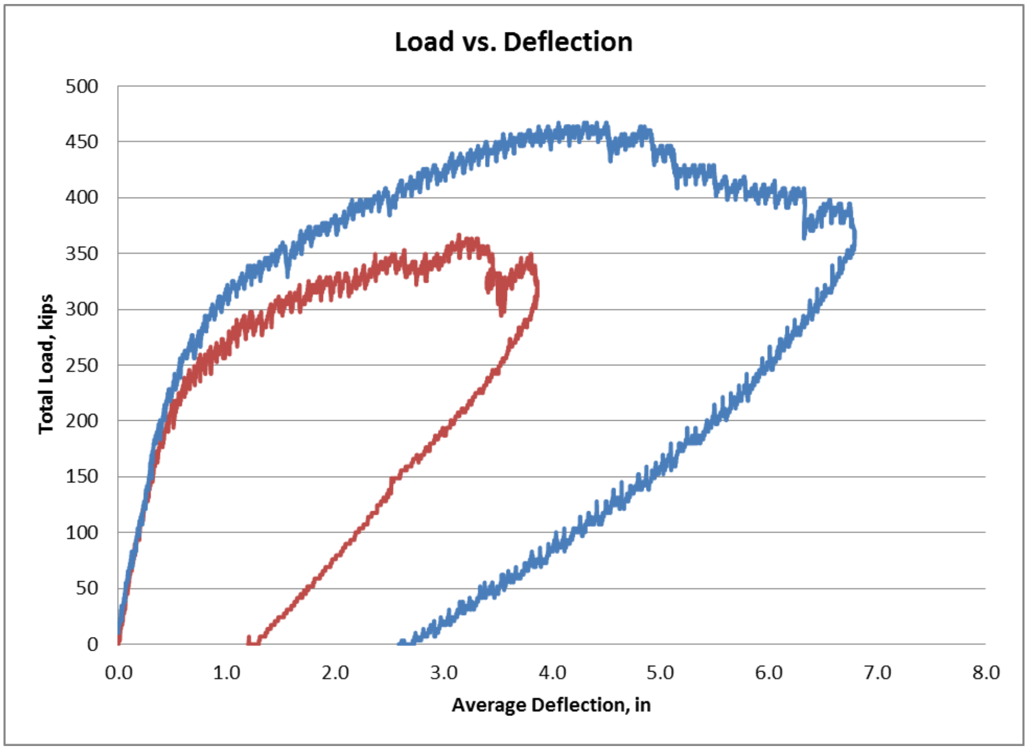

Figure 25: Average Mid-span Deflections vs. Load (Center &West Spans)

In order to compare the destructive test results for the East and West spans, the plots of the average midspan top strain versus the total applied load was created and is provided in Figure 26. As shown, both spans show initial linear behavior with the East span being slightly stiffer than the West span. The slope of both curves then change to another approximately linear portion with the East span again being stiffer than the West span. The change in stiffnesses occur at approximately 260 kips for the East span and approximately 225 kips for the West span. These differences can be attributed to the more severe damage in the West span compared to the East span.

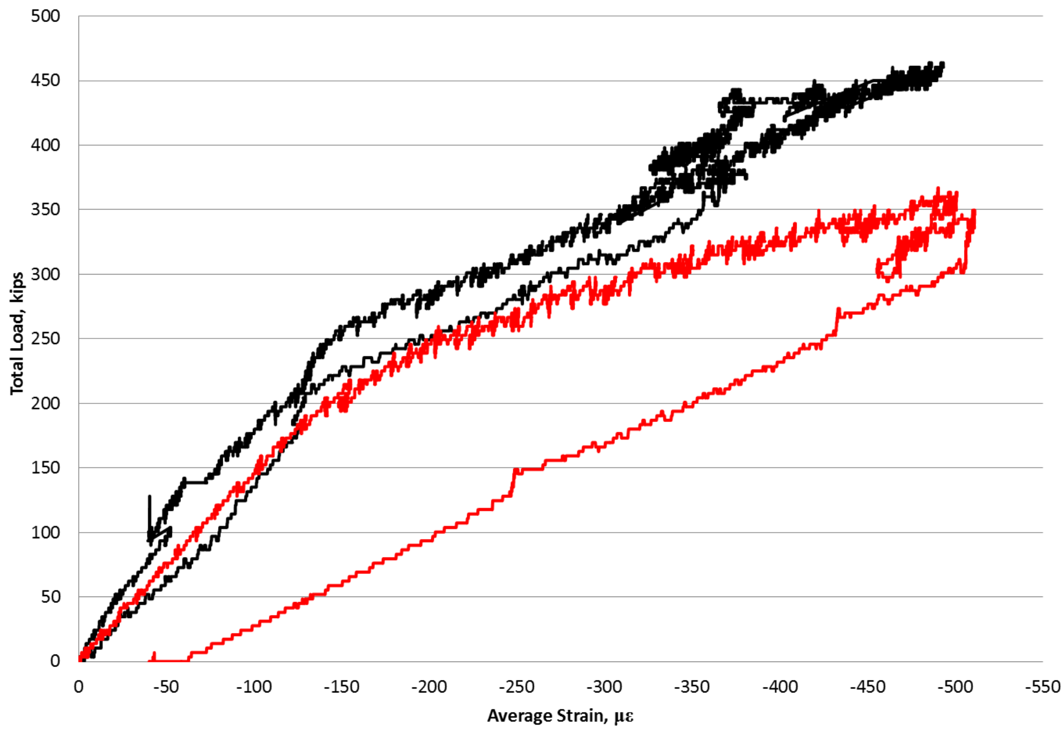

Figure 26: Average Mid-span Strain vs. Load (East &West Spans)

ANALYSIS

A procedure to estimate the total load capacity of the entire span was to sum the individual beam capacities determine by AASHTO LRFD. The individual beam capacities utilized in-situ concrete strengths determined from cores and dimensions per drawings. Each span had a various number of undamaged and damaged beams to different magnitudes. For the East span, Beams 3 and 5 had 6 strands cut and Beams 2 and 6 had 3 strands cut. Beam 9 on East Span also had 3 strands exposed and severely corroded prior to physically causing damage to the span. The remaining 4 beams had no strands cut. The Center Span had no strands cut on any of the beams. The West span had 14 strands cut on Beams 4-6 and the remaining beams did not have any strands cut. Based on these damage levels, the total capacity of the bridge for each span is shown in Table 4. In addition, the Table provides the total capacity of the bridge determined from the experimental results. The last column of the Table provides the percentage difference in the analytical and experimental results.

Table 15: AASHTO LRFD and Experimental Span Capacity

Bridge Span	AASHTO Capacity (kips)	Experimental Capacity (kips)	Percent Difference
East	344	464	+35
Center	382	467	+22
West	290	367	+27

CONCLUSIONS

The spans were able to distribute loads across the width of the bridge even at loads near capacity. The full scale testing showed that the East span and the Center span had nearly the same total experimental capacity even though significant damage was induced into the East span (18 out of 243 total strands cut = 7%). The West span had a lower capacity than the East or Center span, but the West span had more damage and the damage was concentrated to 3 beams (42 out of 243 total strands cut = 17%). As all spans reached their total capacity, the upper flange of several beams crushed and the bridge failed to carry any additional load. However, all spans were still able to support and distribute lower loads. In the extreme event where the load was continued to be applied well after the capacity of the bridge was reached, load was distributed across the bridge. The shear keys transferred the load among the beams and likely continued to do so even after failing due to the transverse tie bars producing friction between beams. Complete collapse did not occur until the transverse tie bars yielded and then fractured at several locations. All spans exceeded capacities determined by AASHTO LRFD Specifications (> 22%) when evaluating the span's total capacity as the sum of the individual capacity of the beams. However, the satisfactory performance of the shear keys and tie rods must be assured for system behavior and to meet the capacity.

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