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## Evaluation of Member and Load-Path Redundancy on the US-421 Bridge Over the Ohio River

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This document is a technical summary of the unpublished Federal Highway Administration (FHWA) report *Evaluation of Member and Load-Path Redundancy on the US-421 Bridge over the Ohio River*, available through the National Technical Information Service at [www.ntis.gov](http://www.ntis.gov).

### Introduction

The sudden collapse of the Silver Bridge in 1967 demonstrated that failure of a single member could result in failure of the entire bridge. This failure and other fractures led to the development of fracture critical member (FCM) provisions for design, fabrication, and inspection. The design and fabrication aspects were originally published by the American Association of State Highway Transportation Officials (AASHTO) as a guide specification in 1978.<sup>(1)</sup> The guide specification was withdrawn in 1989 and largely adopted into the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications, and American Welding Society (AWS) D1.5 Bridge Welding Code.<sup>(2,3)</sup> Regulations regarding the field inspection requirements of fracture critical members are mandated by federal law.<sup>(4)</sup>

The AASHTO LRFD Bridge Design Specifications describe an FCM as “a component in tension whose failure is expected to result in the collapse of the bridge or the inability of the bridge to perform its function.”<sup>(2)</sup> Bridges containing FCMs require a “hands-on” inspection every 24 months.<sup>(4)</sup> These inspections are more rigorous than routine inspection and often require closing down portions of the bridge to traffic to gain access. As of December 2011, there were 18,770 bridges with FCMs, out of a total 91,850 steel bridges in the National Bridge Inventory (approximately 20 percent).<sup>(5)</sup>

Under the AASHTO specification, FCMs are non-redundant steel members either partially or wholly in tension. The specification defines redundancy as “the quality of a bridge that enables it to perform its design function in the damaged state,” and redundant members are those in which

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“failure does not cause failure of the bridge.”<sup>(2)</sup> However, determining redundancy is often difficult, as demonstrated by bridges in service where full-depth fracture of an FCM did not result in collapse of the structure as described in detail within NCHRP Synthesis 354.<sup>(6)</sup> In fact, many bridges have carried traffic for some time prior to a fracture being discovered. The performance of these structures after fracture implies that redundancy, though unaccounted for, does exist by backup mechanisms not considered in the engineering design.

Redundancy is often separated into three different types: internal redundancy, structural redundancy, and load-path redundancy, described as follows:

- Internal (member) redundancy describes multiple parallel elements within a member, such as a built-up member made from many different plates and other structural shapes that are bolted or riveted together. If one element were to fracture, the crack is expected to arrest and not propagate to the adjoining elements within the member.
- Structural redundancy is based on static indeterminacy of the structure as a whole; often, continuous-span structures would have structural redundancy. Further, the system performance of the bridge also provides structural redundancy, such as the participation of secondary members, the deck, parapets, etc. Though similar to load-path redundancy in some ways, structural redundancy is not always as obvious as load-path redundancy.
- Load-path redundancy is the simplest to identify because it is based on the number of primary load-carrying members in a span. Generally, to satisfy load-path redundancy constraints, there must be more than two primary load-carrying members. This form of redundancy is routinely recognized (e.g., multigirder bridges are not classified as fracture critical).

A technical memo published by FHWA with the subject “Clarification of Requirements for Fracture Critical Members,” dated June 20, 2012, provides guidance and recognition regard-

ing structural redundancy for members traditionally classified as FCMs.<sup>(7)</sup> This memo for the first time recognizes system performance (i.e., structural redundancy) as a method to assess redundancy and classify FCMs. The memo also encourages the use of internal redundancy as good detailing practice but does not allow it for classifying whether a member is fracture critical.

## Research Objective

The objective of the project was to assess the after-fracture performance of a two-line, simple-span truss bridge. The bridge was slated for explosive demolition and offered the ability to examine both internal and structural redundancy. Since this study focuses on a single bridge, the results and methodologies developed through this research are rather specific. Nevertheless, as will be discussed, they do show that the universal assumption of collapse following failure of an FCM is not always valid.

## Bridge description

The US-421 bridge spans over the Ohio River between Madison, IN, and Milton, KY, and is referred to as the Milton-Madison Bridge. It was constructed in 1921 and consisted of 19 spans of riveted, built-up members. An overall view of the bridge is shown in figure 1.

Figure 1 does not capture the approach-span trusses leading up to the river span. The first approach-span truss on the Indiana shore is the focus of this research effort. A picture of a test truss is shown in figure 2; the lower chord is close to the ground and spans over a city park property, making access for instrumentation easy. The test truss spans 149 ft and is in a Pratt configuration as shown in figure 3. The truss had some corrosion throughout, which was quite severe in some places and left holes and partially severed members, as seen in figure 4.

## Modeling and Instrumentation

A simple, three-dimensional analysis model of the bridge was created with beam elements, and shell elements for the deck.

The bridge deck was replaced with a filled grid deck in 1996, and this was modeled with shell elements with different longitudinal and transverse properties. The model indicated which members should be

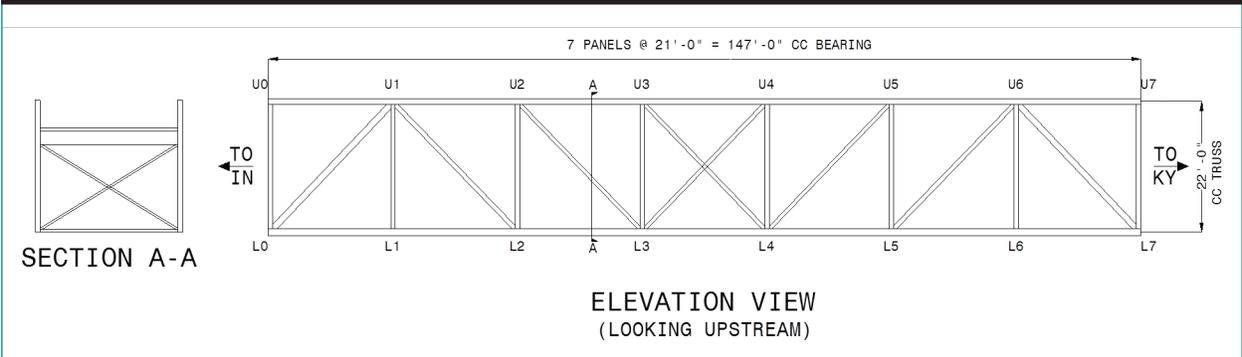
Figure 1. Overall view of bridge from Indiana shore.



Figure 2. View of the test span.



Figure 3. Elevation view of truss.



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removed in a simulated fracture event and guided placement of strain gauges.

Forty-eight weldable strain gauges were applied to select members on the truss. Remote monitoring was performed for five months under ambient traffic loading before the truss was removed from service. These data were used to conduct fatigue analysis from rain-flow stress histograms, which is covered in detail within the full report.

Additionally, static-load and slow-crawling tests were performed on the bridge with a sand truck of a known weight.

### Load Testing

Controlled load testing was performed on the bridge to understand its behavior, determine stresses in the bridge under a known load, and calibrate the finite element analysis. The two-part test included crawl tests and park tests, which were used to examine the response of the bridge to moving loads and to obtain static data (i.e., with the loading at defined locations)

that are helpful in calibrating the FE model. The gross vehicular weight of the test truck exceeded the posting on the bridge in order to ensure that sufficiently large stresses were produced. Analysis showed that the weight of the truck would not produce any overstressing in the bridge.

The results of the monitoring revealed that a substantial number of trucks that crossed the bridge produced stresses that were equal to or greater than those that the test truck produced. It was concluded that random vehicles crossing the bridge must have been at least as heavy as the test truck and that the posted weight limit for the bridge was exceeded on a daily basis.

The park tests were used to determine if the model could accurately capture the behavior of the bridge. The field measured stresses that were later compared to those predicted by the FE model for all nine park test locations. In general, the model yielded a conservative prediction of the stresses in the members.

Figure 4. Severe corrosion in lower chord near abutment.



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## Controlled Demolition

As far as this project was concerned, the controlled demolition plan was to completely sever the L3-L4 bottom chord on the upstream truss while monitoring the strain gauges. Sand was placed on the bridge to represent about  $\frac{2}{3}$  the original design live load. It was difficult to select this loading, as there is no guidance regarding what amount of live load a fractured bridge should be able to sustain. After much consideration, the selected load was deemed reasonable, as it is much higher than any loads seen during long-term monitoring and was thought to be a reasonable upperbound load. This resulted in 145 kips of sand distributed over the deck. However, this load was focused over the three interior panels of the truss (i.e., between nodes U2 and U5) and resulted in stresses in members that were approaching the original design loading. Figure 5 is a photo of the sand atop the bridge.

The chord member was fabricated from built-up channels that were laced together. Each built-up channel was comprised of two angles and a plate that served as the web. In some locations, two web plates were utilized. Each built-up channel was severed separately to evaluate the internal redun-

dancy of the member (i.e., each half of the member was cut separately). To facilitate this, the lacing was flame cut between the two sets of shape charges to ensure that no load was carried through the lacing. Shape charges, as shown in figure 6, were used to instantaneously sever the entire built-up channel, in order to represent a fracture event as much as possible. Figure 7 is a photograph taken after one half of the lower chord was severed.

Figure 8 shows a record from one of the strain gauges installed on the L3-L4 chord on the downstream truss during the two explosive events on the same member of the upstream truss. The plots include the effects of dead load and sand loads. Therefore, this plot shows the total stress in the member. The dead-load stresses were obtained from the computer model, while the stress from the sand load was measured. With the sand in place, the member was subjected to 9.4 ksi prior to the simulated fracture. After the first half of the upstream chord was severed, the stress increased to 9.9 ksi. After the member was completely severed, the stress further increased to 11.9 ksi. The dynamic amplification due to the instantaneous load release following the simulated fracture events was also observed.

Figure 5. Sand placed on deck before simulated fracture event.



Figure 6. Shape charges applied to one web plate of chord member.



For all members that were instrumented, the dynamic amplification from the first blast ranged from 1.08 to 1.30. For the second blast event, the amplification factors ranged from 1.17 to 1.41.

Vertical displacements were also monitored during the two blast events using high-resolution pictures and surveying equipment. Four truss nodes were monitored, and the results are shown in table 1. The measurements were taken at a distance of 164 ft and have a resolution of approximately 0.16 inches. The measured displacements were close to the resolution but are still thought to be representative of the real displacement. The agreement between the simple analysis model and the real bridge in terms of vertical displacement were reasonably close after the first blast event. However, after the second blast event, the model over-predicted vertical displacement by a factor of almost two. The breakdown in the correlation between the model and the test was likely due to the inability of the model to capture the complex alternate load paths used by the structure following removal of a member and the transfer of shear forces in the truss. This is believed to be a major result of the difficulty associated with modeling the filled grid deck and the rigidity of the connections to the primary members and floor system.

Figure 7. Web plate fracture after first explosive event.



## Conclusions

The following important conclusions were derived from this project:

- The internal redundancy of a truss chord indicated that it might be effective at preventing collapse. That is, the instantaneous removal of an element from a built-up FCM did not overload the remaining parallel element enough to cause total member fracture.
- Total removal of an FCM in this particular truss did not result in the collapse of the structure. In fact, the bridge likely could have remained functional under normal service loads. However, based on the small vertical displacement observed, the deficiency is not perceivable to vehicular traffic and not likely to be noticed.
- A simple analysis model was able to conservatively predict the behavior of the truss in the faulted state.
- There was not enough instrumentation applied to the bridge to completely assess the load redistributed within the truss after the imposed fracture. Nevertheless, the filled grid deck and floor system presumably played a major role in secondary support of load, which the simple finite element analysis model was not fully capable of capturing.

## Recommendations for Future Work

As a result of this research, recommendations for future research have been developed. These recommendations are as follows:

- A more detailed model of the truss bridge in this case study should be made to determine if better agreement with the test data could be achieved. This would likely focus on refined modeling on the floor and deck systems, partial composite action, and the truss connections.
- Similar case studies of in-service bridges should be performed using controlled demolition when opportunities present themselves. More case studies are needed to build a portfolio of data in order to determine if internal or structural redundancy can reliably prevent collapse after fracture. Although case studies alone cannot change codified practices, they will provide valuable data in support of any future changes.
- A larger factorial experiment should be performed on built-up members to assess the level of internal member redundancy. This project only provides one data point.
- A modeling standard should be developed so that bridge owners may study their own bridges in order to avoid unnecessary fracture-critical designation. A standard methodology is needed so that this task may be accomplished consistently and efficiently at a reasonable cost to bridge owners. Additionally, the use of a required standard will ensure safety and control for all bridges for which redundancy must be evaluated.

Figure 8. Data record of L3-L4 downstream chord during both explosion events.

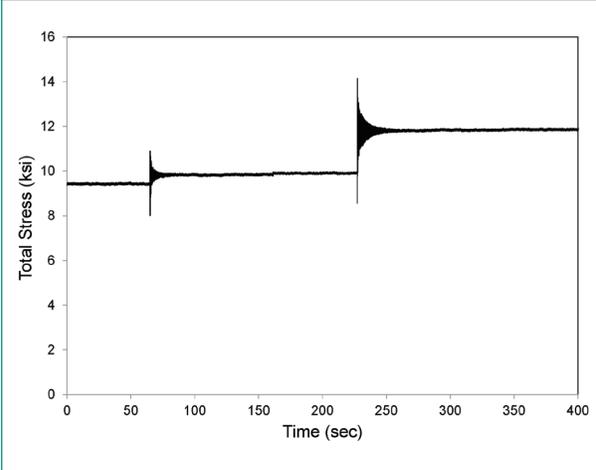


Table 1. Comparison of measured and predicted vertical deflections.

	Joint	Measured Vertical Displacement (inch)	Predicted Vertical Displacement (inch)
First Blast Event	U4	-0.08	-0.09
	U3	-0.08	-0.09
	L4	-0.16	-0.08
	L3	-0.12	-0.08
Second Blast Event	U4	-0.35	-0.71
	U3	-0.35	-0.71
	L4	-0.39	-0.65
	L3	-0.39	-0.65

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**Key Words**—Fracture critical, steel bridges, truss, internal redundancy, structural redundancy.

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