

## **Towards a Better Estimation of Inspection Intervals for Cracking Steel Railroad Bridges**

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### **ABSTRACT**

In order to best prioritize limited funds for railroad bridge capital and maintenance, it is important for the bridge engineer to have an accurate estimate of service life for existing steel bridges. In the context of the paper, “service life” is affected by several damage modes such as corrosion, fatigue and fracture as well as other factures such as redundancy. Use of conservative bridge design loads and conservative computation methods can result in an estimated steel bridge life that is inappropriately short, resulting in less-than- optimal allocation of limited funds. Methods recommended to achieve more appropriate life estimates are based on recent research on riveted steel bridge members, as well as inspection and life estimation procedures used in the aircraft and pipeline industries. Collectively these practices are sometimes referred to as Fitness-for-service (FFS). Implementation of FFS allows consideration of inspection policies and trending of inspection, redundancy, material toughness, and statistical variability of loads and member performance. Considerable efforts underway for highway bridges are providing valuable input to the process. Testing of steel bridges at FAST and in revenue service, as well as laboratory testing at Purdue University, is providing additional data for development of new recommendations for better life assessment of steel railway bridges. The recommended procedures utilize an implementable FFS approach that links inspection, fatigue, and material fracture toughness to perform a risk-based assessment of an individual bridge or an entire inventory.

### **INTRODUCTION**

As the population of aging steel bridges increases, there is a need to utilize methods to more accurately and safely determine the strength capacity and fatigue life of in-service bridges. While over half of the bridges used by railroads in North America are composed of steel construction, a large number of these are older than their original 80 year design life (Unsworth, 2010). Most of the older rail bridges have seen a dramatic increase in the frequency of trains and the magnitude of loading since their original construction. It is critical that bridges perform as intended to ensure smooth movement of commerce. Further, budgets for replacement and heavy repair can drive the need for extended life of the bridges. Therefore, reliable methods of determining remaining capacity and functional life are necessary to accurately predict the remaining service life of a bridge and ensure public safety.

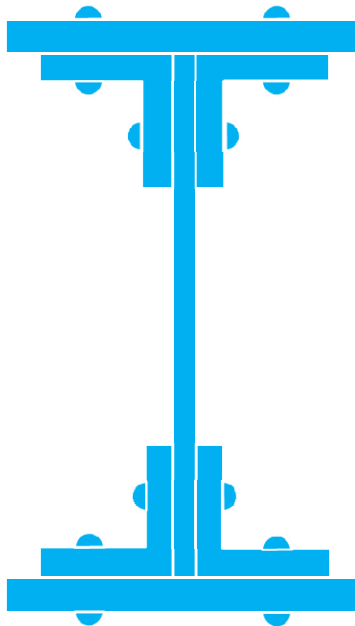
One common method in many industries to examine the capacity of steel structures is to utilize what is referred to as Fitness-for-service (FFS) evaluation. The British Standards Institute (BSI) has developed guidelines which are often used in the evaluation of steel structural members abilities to prevent a fracture: BS 7910:2013, ‘Guide to methods for assessing the acceptability of flaws in metallic structures’ (BSI, 2013).

To perform an FFS assessment of any kind, specific properties of the materials such as the fracture toughness and yield strength are required. For older railroad bridges, such information, in particular the toughness, is generally unknown. Obtaining material coupons is one way to collect such data. However, this requires destructively removing samples from the bridge and engaging a material testing facility.

Further, the ability to locate and size existing cracks or flaws is also essential. Inspections of bridges are critical in this step in finding cracks and measuring their progress over time. However the likelihood of finding cracks through visual and even non-destructive testing on in-service bridges, which can be very large and often difficult to access, has long been a concern among the engineering community. Recent research on the probability-of-detection of cracks suggests that the crack detection is highly variable between different inspectors and is independent of crack length, experience, atmospheric conditions, and equipment used (Snyder, Whitehead, Connor, & Lloyd, 2015).

As stated, reliable data regarding in-situ material properties and defect size and location are required to perform and have confidence in the results of FFS. Unfortunately, obtaining such data on a large scale (i.e., possibly hundreds of bridges) is very difficult. Further, it may not be practical or reasonable to think an inspector could find a 1-inch crack emanating from a single rivet hole in the presence of severe corrosion among tens of thousands of rivets.

Rather than utilize a traditional FFS approach which relies on detecting small cracks and knowledge of material toughness, a different strategy is presented in this paper. It is well known that a large number of steel bridges (virtually all steel bridges constructed prior to the early 1960s) were fabricated with mechanically fastened built-up steel members. Built-up members are composed of multiple components, typically plates and angles fastened with hot-driven rivets (see Figure 1). More recently high-strength bolts have been used in built-up member fabrication. The number of components in a built-up member and the faying surfaces between each component create multiple load paths and therefore an inherent redundancy in the member. It is proposed in this paper that rather than attempt to detect small cracks before they become critical, it may be possible to exploit this internal redundancy within the member and evaluate the strength and fatigue performance of the member assuming failure of any one component has occurred. This redundant behavior has been recognized by the engineering community. However it has not been utilized to its full potential because, until recently, no known research has investigated the ability of built-up members to redistribute load between components.



**Figure 1 Typical built-up girder cross-section**

The authors believe that the approach described herein offers the following advantages over traditional FFS approaches.

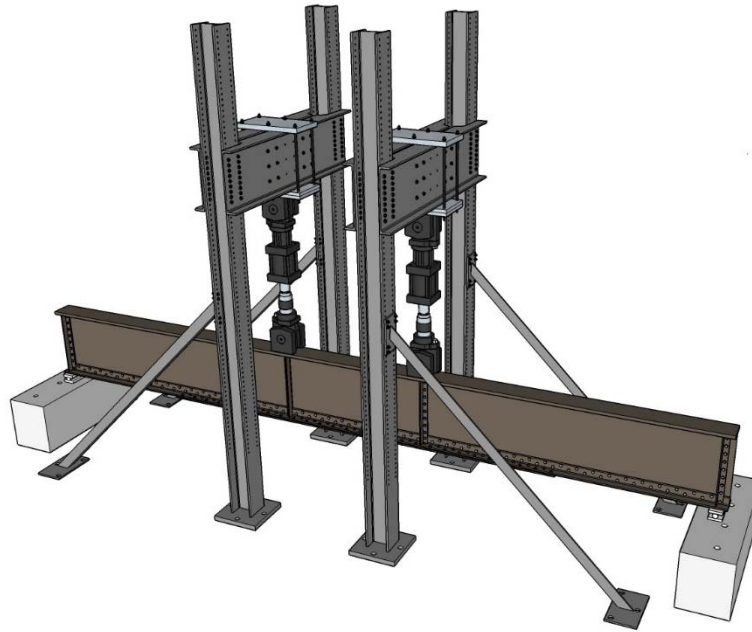
- No specific information is required regarding material toughness, as it is explicitly assumed that any single component has already fractured. As will be described, all testing from large-scale members was on the lower shelf, with CVN impact energies less than eight to ten ft-lbs.
- Since failure of an individual component is assumed to have occurred (due to fatigue or fracture), it is not critical to locate small cracks prior to them becoming critical. Rather, it is only necessary for an inspector to detect a completely failed component (i.e., an entire angle or cover plate), which is not believed to be an unreasonable expectation, even on severely corroded members.
- The approach can be used to establish a rational interval for detailed inspection where fatigue and fracture are the focus. Assuming a single component has failed, based on the fatigue resistance in this faulted state the expected life until cracking in the *second* component can be estimated. Thus, once the remaining life after failure of the first component is established, the inspection interval for fatigue and fracture would be set at some conservative period less than this calculated life. This and subsequent inspections would simply need to detect a failed component. While general safety inspection would still occur at routine intervals, there would be no need for in-depth inspection to find small cracks during those inspections.

## EXPERIMENTAL PROGRAM

Recent research at Purdue University has evaluated the member-level redundancy of built-up steel girders (Hebdon, 2015). The research was intended to provide information to answer two specific questions: (1) Does a member which experiences a brittle fracture of a component have the ability to redistribute the stresses and prevent a fracture from propagating into adjacent components, thereby preventing a catastrophic failure of an entire member, and (2) if a built-up girder experiences a complete failure of a single component, what is the remaining fatigue life of the damaged cross-section?

In order to investigate this behavior, large-scale built-up bridge girders have been tested at the Robert L. and Terry L. Bowen Laboratory for Large-Scale Civil Engineering Research (see Figure 2 for a schematic of the test setup). Experimental specimens consisted of 14 girders, three of which were from a recently retired bridge (originally built in 1925), and the remaining specimens were fabricated for the research test. The specimens from the retired bridge consisted of 23"x3/8" web plate, with two 5x3 1/2"x7/16" flange angles (long legs vertical) on the top and bottom flange. A 12"x1/2" cover plate was added to the bottom flange of two specimens to increase the redundancy. A 12"x5/8" cover plate was added to the bottom flange of the remaining specimen. Three different geometries were used for the fabricated specimens. Because the bottom (tension) flange was the area of interest, each of the fabricated specimens had a 14"x2" top flange which was welded to the web plate. A single specimen was fabricated with a 30"x1/2" web plate, 6x6x3/4" flange angles and a 14"x1" cover plate. Five specimens were fabricated with a 36"x1/2" web plate, 6x6x3/4" flange angles and two 14"x3/4" cover plate. Five specimens were fabricated with a 46"x1/2" web plate, 6x6x3/4" flange angles and a 14"x3/4" cover plate.

Each of the test specimens were tested in four-point bending to create a constant moment region (see Figure 2). This facilitated the measurement of the redistribution of stress at each stage of the test through strategic strain gage placement in the constant moment region. The goal of the project was to first fracture a single component of each specimen to observe its fracture resilience – its ability to arrest a crack without other components fracturing. Next, each specimen was cyclically loaded to determine the fatigue life after failure of one of the components occurred.



**Figure 2 Test setup**

### **Fracture Resilience Test**

Thirteen of the specimens were tested for fracture resilience. Notches were cut into one of the components of each specimen to control the location of subsequent cracking. Seven specimens had components with cracks placed at the edge of fastener holes (to simulate a typical fatigue crack). Six specimens were fabricated with components which had cracks at the edge of the plate. Each notched specimen was prepared by creating a sharp crack tip (with different methods) to increase the stress concentration at the tip of the notch. The hydraulic actuators applied a force corresponding to a simulated dead load moment prior to the fracture test. Specimens were then cooled with liquid nitrogen to a temperature of  $-60^{\circ}\text{F}$  (or below) while dead load was maintained. In other words, the warmest test temperature was  $-60^{\circ}\text{F}$ . At this temperature all material was on the lower shelf and brittle behavior was expected if fracture occurred. In fact, Charpy impact energies were less than 8-10 ft-lbs for all specimens at  $-60^{\circ}\text{F}$ . Each specimen was then loaded further such that the bottom flange stress was  $0.55 F_y$  of the original net-section. Based on linear elastic fracture mechanics (LEFM) and the specific crack size in each specimen, this loading created a condition in the cracked component that should have resulted in brittle fracture. Of the thirteen specimens tested, only one experienced a brittle fracture of a component from pure bending stresses on the first attempt. Interestingly, much of the cover plate was already significantly cracked in fatigue and the crack was well beyond the calculated critical crack size using LEFM. In fact, in seven of the specimens, complete failure of the first component occurred through fatigue. For these seven specimens, after the first attempt at inducing a fracture, the crack was grown further in fatigue, the specimen cooled and the fracture test repeated. After several attempts, the crack component failed in fatigue. Based on the instrumentation and finite element studies, it was found that as the crack grew in the component, it shed load to the stiffer, uncracked components. Thus, as the crack grew, the load in the plate decreased thereby lowering the likelihood of fracture.

In the remaining five specimens, fractures were introduced through the help of an external device which used driven wedges to magnify the stress concentration at the crack tip. Additionally, it was found that the specimens consistently fractured only when fasteners adjacent to a crack were removed, decreasing the constraint provided by the adjacent plates in the cracked region.

In short, it proved very difficult to produce a fracture in a cracked component at extremely cold test temperatures. In fact, without the use of the external device used to amplify the stress at the crack tip, the authors do not believe brittle fractures were likely to have occurred in any of the specimens when the

fatigue crack had grown to the calculated “critical” size using LEFM. This is actually very desirable behavior as sudden brittle fractures in individual components are unlikely to occur and hence eliminates the potential for brittle fractures “jumping” from one component to another. Further, all specimens were capable of attaining the yield moment ( $M_y$ ) on the net section which included the failed component.

### After-Failure Fatigue Test

After the failure of the first component, whether due to fatigue or fracture, each specimen was cyclically loaded to determine the remaining fatigue life. The results of the fatigue tests were plotted with the AASHTO fatigue design curves and can be seen in Figure 3. The stress range is calculated based on the after-fracture remaining net section (i.e., including holes and not including the failed component).

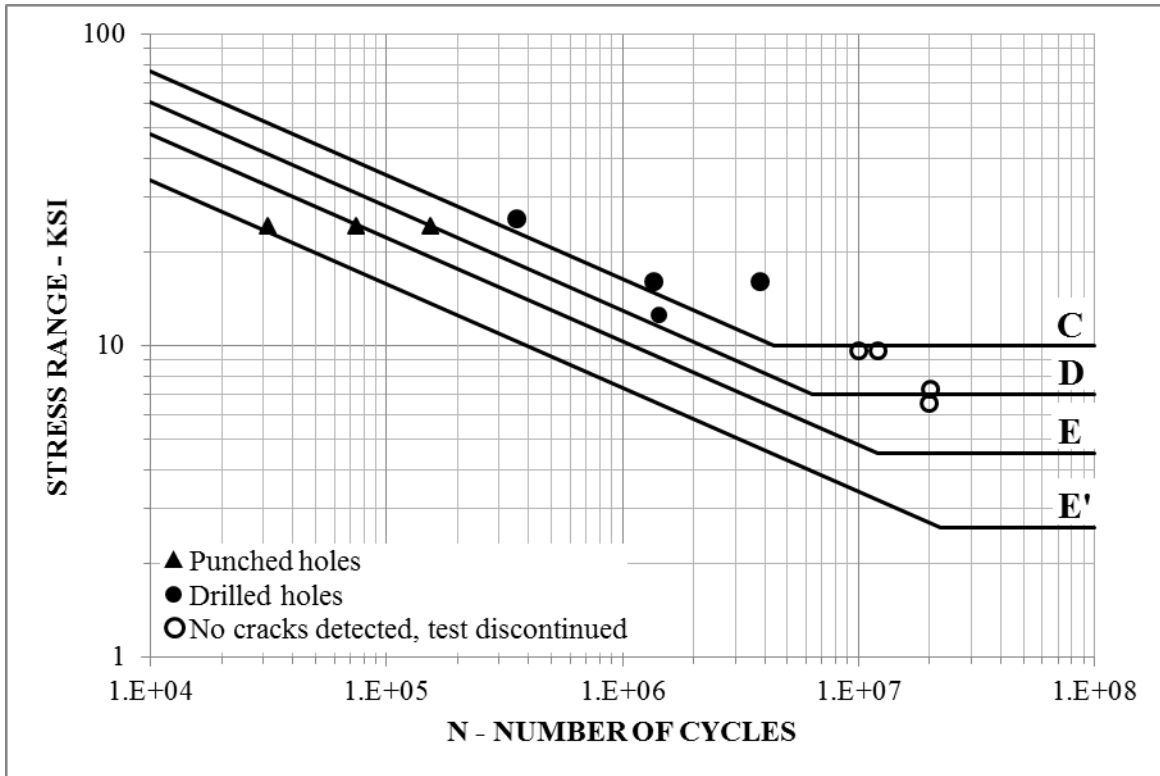


Figure 3 After-failure fatigue life

Specimens included both drilled and punched holes. As shown in Figure 3, the AASHTO Category E' fatigue curve represents a conservative lower bound for specimens containing punched holes, and the Category D fatigue curve is a conservative lower bound for specimens containing drilled holes. The stress range is calculated based on the remaining cross section using basic mechanics of materials.

### FINITE ELEMENT PARAMETRIC STUDY

A finite element (FE) parametric study was performed to:

- Explore the behavior of built-up members when a cover plate was only partially failed or if only one angle was failed to explore if local out of plane (weak axis) amplifications in stress were observed.
- Explore local effects in girders with non-traditional cross sections (e.g., large cover plates such as 26 inches x 1 inch thick) resulted in unexpected amplifications that

could lower the fatigue performance of the girder by affecting stress distribution in the girder.

The FE models were first benchmarked with stress data obtained from the experimental tests. Figure 4 shows a contour plot of longitudinal stresses in a half symmetry FE model with a failed cover plate. Note the increase in the localized longitudinal stress in the flange angles when the cover plate has completely fractured. The longitudinal stresses in the flange angles begins to dissipate as the distance from the fracture increases. This is as expected since load is transferred back into the cover plate through friction and the fasteners.

Based on the parametric study, it was found that the primary factors influencing the stress distribution in partially failed built-up members were the number of components (flange angles and cover plates) and out-of-plane effects resulting from failures that resulted in a non-symmetric cross section (i.e., a broken angle or cracking across only a portion of a cover plate).

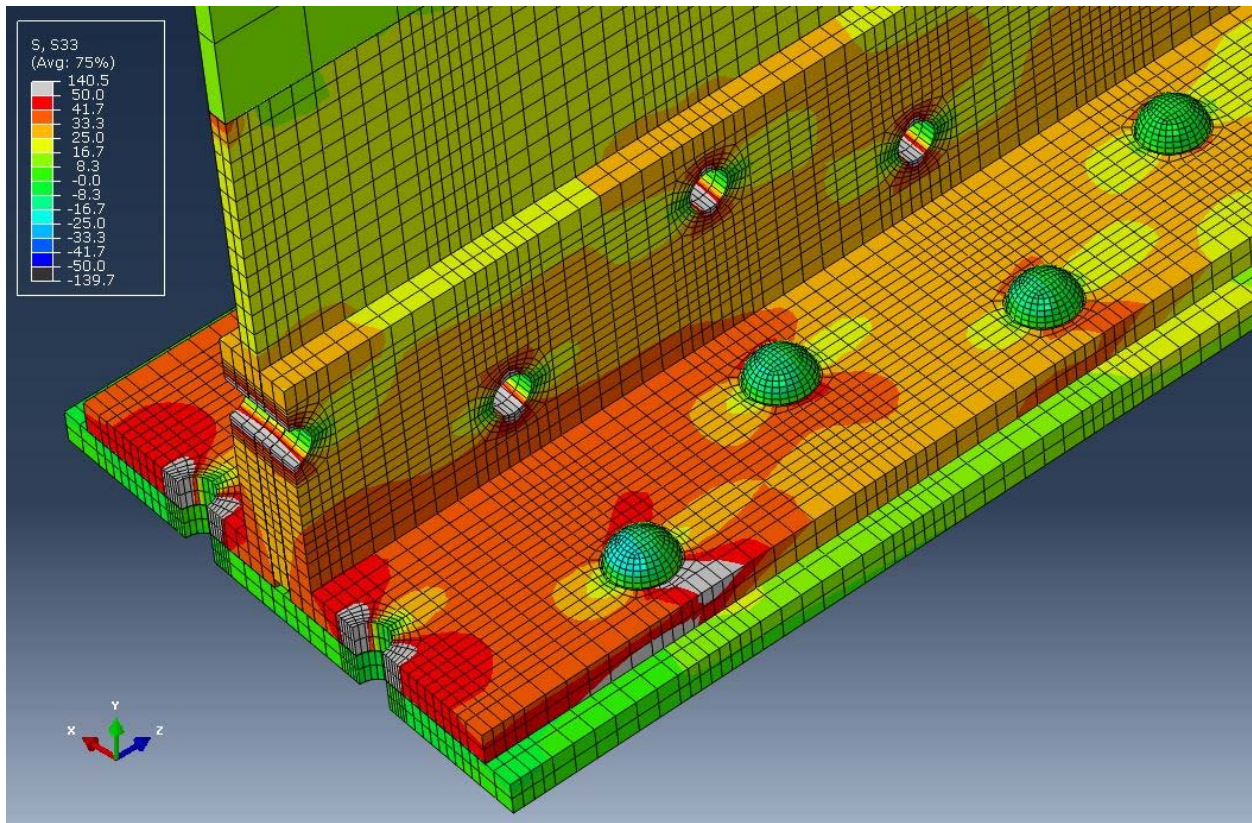


Figure 4 Typical FE model with failed cover plate



A non-symmetric cross section could result from a failed flange angle or a partially failed cover plate. The stress increase resulting from a non-symmetric cross section was not significant for a failed flange angle. Interestingly, it was found that a partially cracked cover plate with a crack extending more than 60% of the width had an influence greater than the fully failed cover plate. Note though, the amount of time it takes the crack to grow from 60% of the width to full width is very short due to the fact that such large cracks grow very quickly. Based on a study of crack growth rates in plates of various widths, it was determined that the rate at which the stresses were increased were an insignificant portion of the fatigue life and hence, the fatigue cycles which would accumulate at these higher stress ranges due to the non-symmetric cross-section could be ignored.

## **INSPECTION STRATEGY**

In light of the above, it is proposed that inspections intended to detect small cracks in built-up members may not be necessary. As discussed, built-up members have been shown to have adequate resistance to brittle fracture propagation when an entire component fractures. Brittle fracture in one component does not result in failure of the entire member if the remaining net cross-section has adequate strength. Thus, if an inspector “misses” a small crack that results in subsequent fracture of a component immediately after the inspection, failure of the entire girder is not likely. A rational inspection interval can be set by calculating the fatigue life assuming a single component has failed and setting the inspection frequency to something less than this interval, such as 75% of the estimated life. Since the inspector must only identify completely cracked components, detection is much more likely.

If the member possesses infinite life or significant remaining fatigue life (say over 100 years) prior to the assumed fracture occurring, the authors believe the approach can be applied directly. The member must also possess adequate strength to carry the applied dead load and live load in this condition. In cases where insufficient fatigue life remains, the approach must be applied cautiously using sound engineering. For members where insufficient fatigue life exists (i.e., negative fatigue life) prior to the assumed fracture occurring, it is recommended to obtain a more accurate estimate of in-service stress ranges to better estimate the remaining life. It is possible that the actual in-service stress ranges are less than calculated and as a result the actual remaining fatigue life is greater.

Further, an owner must be confident that no cracks exist, especially in such cases where calculated fatigue life has been exhausted. Hence, an inspection should take place prior to implementing the strategy described herein in order to provide confidence that no existing cracks are present. It is emphasized that regardless of the inspection technique (e.g., visual vs. NDE), the engineer must be confident in the findings and have some knowledge regarding probability of detection for the given situation. . If fatigue cracks are present in multiple components at a given cross section (i.e., wide spread fatigue damage) or if there is severe section loss due to corrosion, the remaining components may not have adequate capacity to carry the applied loading should a given component fail.

## CONCLUSION

In summary, the after-fracture performance of built-up member subject to flexure was studied. The research revealed the following:

1. Brittle fracture of a component of a built-up girder is unlikely without the aid of external influences to reduce localized constraint and increase the stress concentration at a crack.
2. Brittle fracture of a single component in the built-up girders tested did not propagate into other components and did not result in a catastrophic failure of the entire member.
3. In the damaged state, all girders tested were capable of attaining yield moment ( $M_y$ ) on the remaining (i.e., after fracture) net cross section.
4. After the failure of a component of a built-up member, significant fatigue life may remain. The remaining fatigue life can be characterized by the AASHTO Category D fatigue curve for drilled holes, and the Category E' fatigue curve for punched holes.
5. A general approach to establish rational inspection strategy focused on the fatigue and fracture limit states for built-up members was presented.

## ACKNOWLEDGEMENTS

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## REFERENCES

- BSI. (2013). *BS 7910 - Guide to methods for assessing the acceptability of flaws in metallic structures*.
- Hebdon, M. H. (2015). *Member-Level Redundancy of Built-up Steel Girders*. Purdue University.
- Snyder, L. R., Whitehead, J. M., Connor, R. J., & Lloyd, J. B. (2015). *SPR 3820: Probability of Detection Study for Visual Inspection of Steel Bridges*. West Lafayette, Indiana, USA.
- Unsworth, J. F. (2010). *Design of modern steel railway bridges*. Boca Raton, FL: CRC Press.



# TOWARDS BETTER ESTIMATION OF STEEL RAILROAD BRIDGE LIFE

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AREMA 2015 Annual Conference



Minneapolis, MN | October 4-7, 2015



## Steel Bridge Inventory

- More than ½ of rail bridges are steel
- Significant number are beyond design life



[www.historicbridges.org](http://www.historicbridges.org)

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## Determine Remaining Capacity

- Fitness For Service
  - Material properties
    - Fracture toughness
    - Yield Strength
  - Loading history
    - Historical data
    - Fatigue cycles to-date
  - Locate and size flaws and cracks
    - Questionable reliability and accuracy



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## Construction Practices

- Primarily used from 1870's to 1960's
- Fastened with hot-driven rivets



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## Motivation

- Internal redundancy widely recognized in built-up members
  - AASHTO Manual for Bridge Evaluation (2010)
  - Based on observed performance
- Research needs
  - Experimental and analytical evidence
  - Demonstrate benefits of multiple components
    - Increased fracture resistance
    - Fatigue life after component failure



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## Research Objectives

- Evaluate redundancy of built-up steel girders in resisting failure
  - Brittle fracture of a single component will not result in catastrophic failure of the entire member
  - Significant fatigue life after a component failure (fatigue or fracture)



## Anecdotal Evidence

- North Fork Mollala River Bridge, OR
  - Fracture in bottom flange of riveted built-up two-girder bridge



LeBejeux, 2001

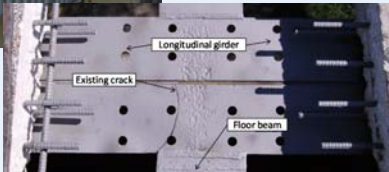


## Anecdotal Evidence

- UT Austin Research Project
  - Fatigue through cracks in flange angle of built-up member

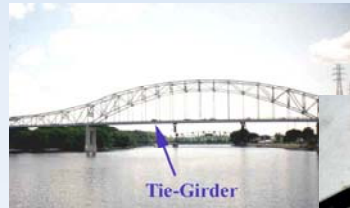


Fasl, 2013



## Anecdotal Evidence

- Hastings Bridge, MN
  - Fracture in web plate of riveted built-up tie-girder



Neimann, 1999



## Experimental Research Program

- Test process
  - Notch a component
    - Controlled location (angle/cover plate)
      - Not looking at initial fatigue life – already documented
    - Crack growth through fatigue to critical length (LEFM)
  - Cool beam → lower shelf behavior
  - Load to induce a fracture
    - 0.55 F<sub>y</sub> (Minimum)
    - Increase stress concentration when required
  - Examine stress redistribution
  - Determine fatigue life



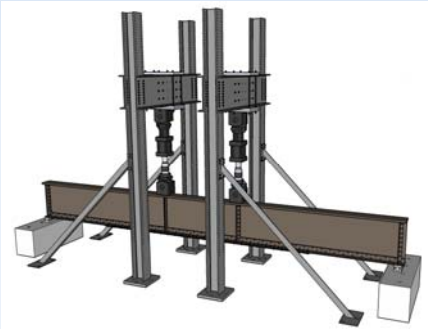
## Experimental Research Program (15 specimens)

Specimen Height	Failed 1 <sup>st</sup> Component	Bolted		Riveted	
		Friction	Low-Friction	Friction	Low-Friction
23 in.	Cover Plate				23-1D
	Flange Angle				23-2D 23-3D
30 in.	Cover Plate	30-1D			
	Middle Cover Plate	36-1P			
36 in.	Bottom Cover Plate	36-2D			
		36-5D 36-6D		36-3D 36-4D	
	Flange Angle				
46 in.	Cover Plate	46-2P 46-4D	46-1P	46-3P	
	Flange Angle			46-5D	



## Experimental Research Program

- Test setup



## Keep in Mind Test Conditions

- All material on lower shelf
  - Single digit ft-lbs
  - Test temperature -60F (warmest temp)
    - As low as -120F
- Applied stress 0.55 F<sub>y</sub> (MINIMUM)
- VERY challenging to obtain brittle fracture from cracked component
  - As fatigue crack grows in one component, load is shed to uncracked components
  - Resulted in development of “Dr. Fracture” to drive wedges into crack





### Keep in Mind Test Conditions

- Had to remove fasteners either side of the notch to increase strain energy stored in section of plate near crack (rubber band analogy)
- When fracture occurred, remaining cross section at  $0.8x F_y$  to  $>1.0x F_y$
- Summary:
  - With high applied load & material on lower shelf & in presence of fatigue crack, & fasteners removed...
  - Driven wedges **STILL** required to induce a fracture



### Experimental Research Program

- Driven wedge to increase stress concentration



### Experimental Research Program

- Fracture resilience



### Experimental Research Program

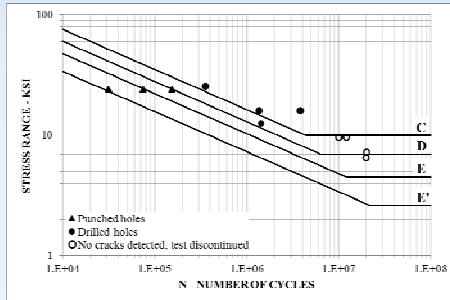
- Summary of fracture attempts

Specimen	# of Fracture Attempts	# of Successful Fracture Attempts	Method	Failure Mode of 1st Component
23-1	4	0	Fatigue Crack Growth	Fatigue
23-2	3	0	Fatigue Crack Growth	Fatigue
23-3	1	0	Fatigue Crack Growth	Fatigue
30-1	2	1	Driven Wedges	Fracture
36-1	4	0	Fatigue Crack Growth	Fatigue
36-2	1	0	Hardfacing Weld Fuse	Fatigue
36-2b	3	1	Driven Wedges	Fracture
36-3	1	1	Driven Wedges	Fracture
36-4	1	1	Driven Wedges	Fracture
46-1	1	0	Fatigue Crack Growth	Fatigue
46-2	1	0	Fatigue Crack Growth	Fatigue
46-3	1	1	Fatigue Crack Growth	Fracture
46-4	3	1	Driven Wedges	Fracture
Total	26	6		

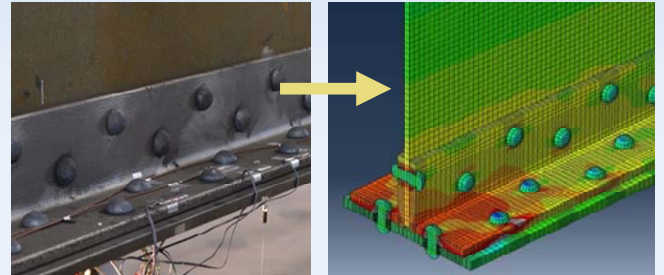


## Experimental Research Program

- CALCULATED Fatigue life AFTER 1<sup>st</sup> component failure



## Analytical Research Program



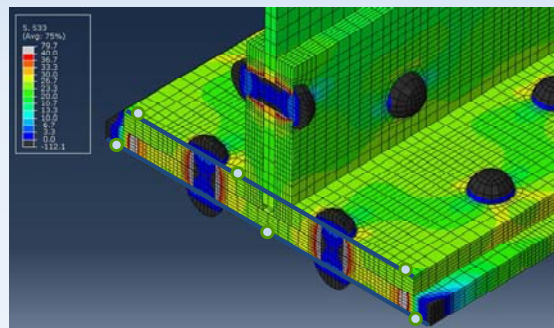
## Analytical Research Program

- Finite element model development
  - Full specimen models based off work performed by Martin (2014)
  - Abaqus
  - Nonlinear materials
  - Dynamic explicit
  - Contact modeling with friction between components
  - Pretensioned rivets



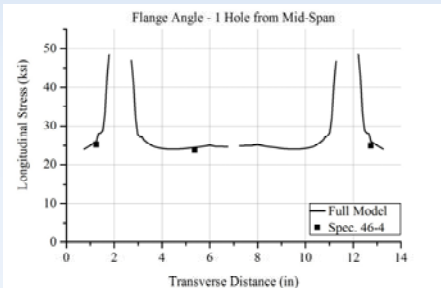
## Analytical Research Program

- Model validation



## Analytical Research Program

- Model validation
  - Average difference of 1.1 ksi (4.9%)

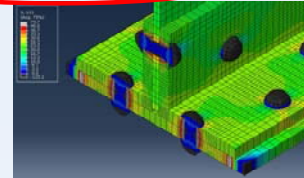


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## FE Parametric Study

- Investigate effects of:
  - ~~Web height~~
  - ~~Non symmetric cross section~~
  - ~~Tension flange unbraced length~~
  - Multiple flange components

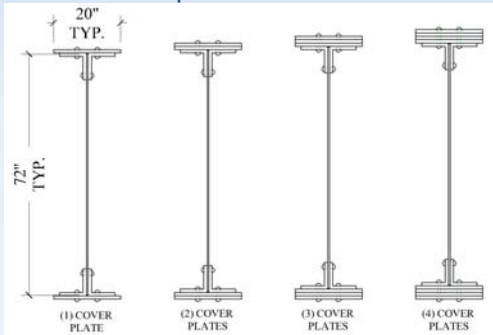


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## FE Parametric Study

- Number of cover plates

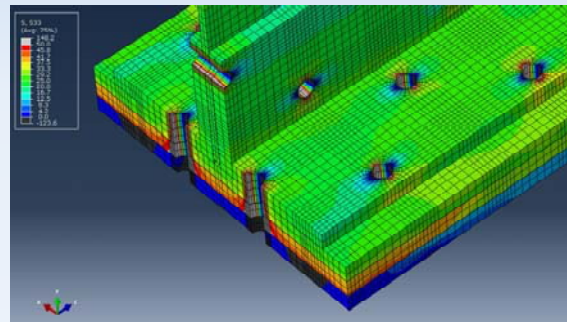


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## FE Parametric Study

- Number of cover plates



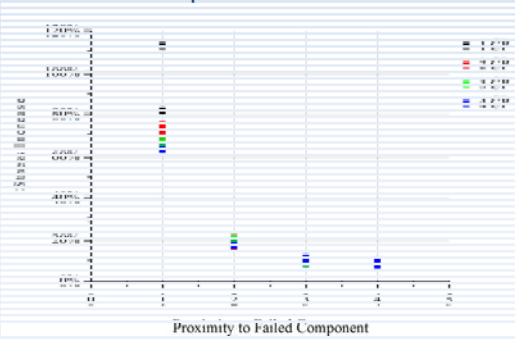
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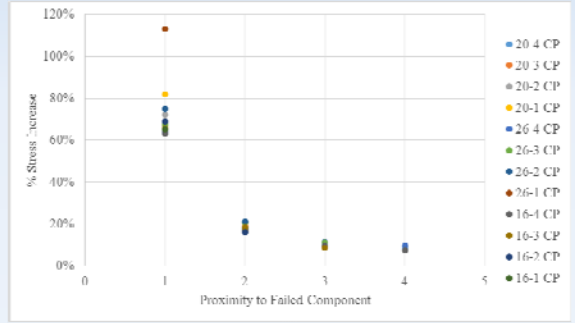
### FE Parametric Study

- Number of cover plates



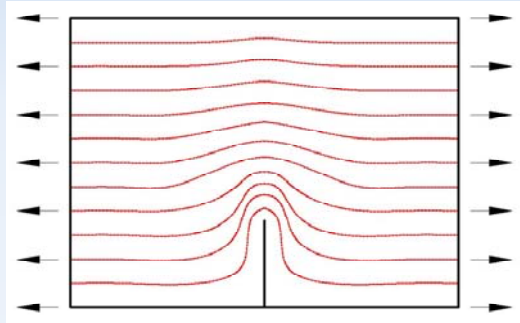
### FE Parametric Study

- Number of cover plates



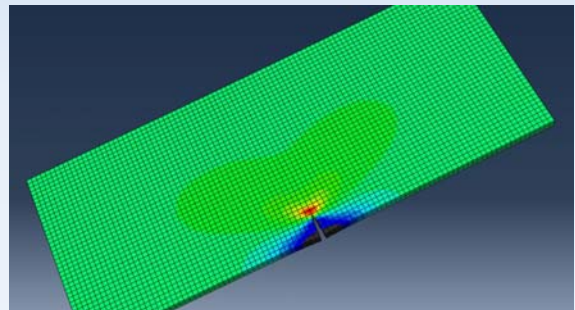
### FE Parametric Study

- Number of cover plates

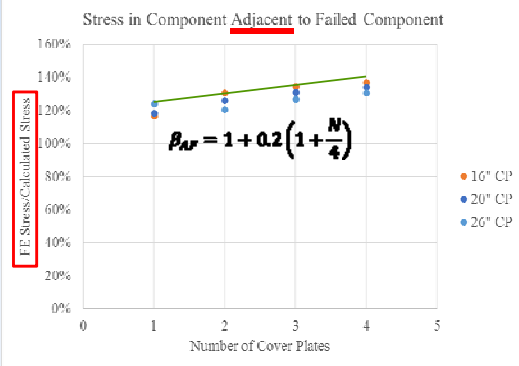


### FE Parametric Study

- Number of cover plates



### FE Parametric Study Effect of # of CPs on Stress Amplification



### Evaluation Methodology

$$\sigma_{Net-AF} = \beta_{AF} \frac{M}{S_{x-Net-AF}}$$

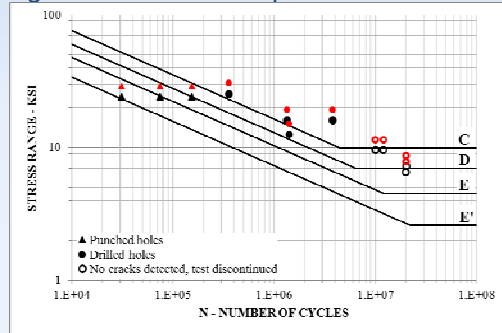
– Where:

- $\sigma_{Net-AF}$  = After failure net-section stress
- M = Applied moment
- $S_{x-Net-AF}$  = After failure section modulus
- $\beta_{AF} = 1 + 0.2 \left(1 + \frac{N}{4}\right)$
- N = Number of cover plates



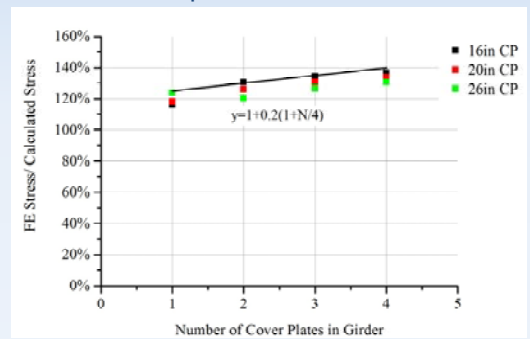
### Experimental Research Program

- Fatigue life after 1<sup>st</sup> component failure



### FE Parametric Study

- Number of cover plates



## FE Parametric Study Results

- Insignificant affect from:
  - Height of girder (web height)
  - Unbraced length of tension flange
- Non-symmetric failed cross-section
  - Due to crack growth rate, can be ignored
- Number of cover plates
  - Adjacent component has largest stress increase
  - Simple equation to account for this was developed



## Evaluation Methodology

- Objective
  - Provide methodology to evaluate existing built-up girders to determine remaining fatigue life in a partially-failed state
  - Evaluate level of inspection rigor necessary
  - Determine a rational inspection period



## Evaluation Methodology

- Applies to:
  - Members subjected to flexure
  - Minimum of 1 cover plate on tension flange
  - Partially failed member capacity must exceed all other failure modes
    - E.g., design load stress  $\leq$  net-section stress
  - Evaluation of initial life for Category D
    - AASHTO MBE's use of Category C implicitly recognizes member-level redundancy



## Evaluation Methodology

- $\Delta f_o$  = Original net-section stress range
- $\Delta f_{AF}$  = After failure net-section stress range
- Three potential scenarios
  - Case I:  $\Delta f_o \leq CAFL_D$  &  $\Delta f_{AF} \leq CAFL_{C,E}$
  - Case II:  $\Delta f_o \leq CAFL_D$  &  $\Delta f_{AF} \geq CAFL_{C,E}$
  - Case III:  $\Delta f_o \geq CAFL_D$  &  $\Delta f_{AF} \geq CAFL_{C,E}$



## Experimental Testing Application to In-Service Bridge



## Experimental Testing Application to In-Service Bridge



## Conclusions

- Fracture does not propagate into adjacent components
- Substantial remaining fatigue life with single component failed (Category C, E')
- Stress redistribution is localized
  - Adjacent component is most affected
- Methodology for evaluation can be implemented for determination of remaining fatigue life

## Questions

