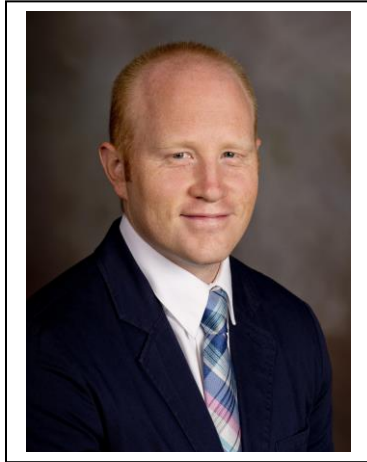


# **FRACTURE RESILIENCE AND REDUNDANCY OF BUILT-UP STEEL GIRDERS**



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## **BIOGRAPHY**

Dr. Matthew Hebdon is an Assistant Professor of civil engineering at Virginia Tech. He is a licensed professional engineer. Dr. Hebdon has research experience in fatigue and fracture of steel structures, bridge monitoring and testing, redundancy of structural systems, historic fabrication methods and materials, and large scale testing of structures.

Dr. Robert Connor is an Associate Professor of civil engineering at Purdue University. He is also the Director of the Steel Bridge Research, Inspection, Training, and Engineering Center (S-BRITE). Dr. Connor has research experience in fatigue and fracture of steel structures, field testing and remote monitoring of structures, and member and system-level redundancy of steel bridges.

## **SUMMARY**

Internal member redundancy provides built-up steel girders with the ability to resist total member failure in the event an individual component fails. Anecdotal evidence of in-service performance has historically shown this to be the case in many bridges. However, due to the lack of experimental data, these members are currently required to be inspected as fracture-critical when deemed non-redundant. The full-scale experimental and analytical research program described in this paper provides needed information on parameters that affect the ability of built-up members to arrest a fracture, as well as describing the length of the remaining

fatigue life. The results from this study have been used to develop recommended assessment procedures for built-up flexural members when a component has failed. Proposed evaluation guidelines will permit bridges with built-up steel girders where sufficient capacity exists, and the fracture critical designation can be removed, to be inspected using a rational in-service interval and level of detail.

Considering the large number of riveted fracture critical bridges in the inventory, both highway and railroad bridge owners will benefit from this research since it allows for implementation of a more rational inspection strategy without compromising safety and reliability. The strategy provides a more integrated approach to inspection that accounts for the probability of detection capabilities, fatigue life, and fracture resilience. Further, new members utilizing high-strength bolted built-up members have the potential to be used without the penalty of being classified as fracture critical in terms of inspection

# FRACTURE RESILIENCE AND REDUNDANCY OF BUILT-UP STEEL GIRDERS

## Introduction

Beginning in the 19<sup>th</sup> century the primary fabrication method for large steel cross-sections consisted of fastening shapes (typically plates and angles) together with hot-driven rivets. Members were typically fabricated using hydraulic-driven rivets in the shop, and pneumatically-driven rivets in the field. During this era of fabrication a large number of bridges were constructed across the country, many of which remain in the current inventory.

Bridge systems utilizing two main load carrying members (e.g. two-girder, truss, tied-arch, etc.) were commonly constructed during the same era that built-up members were used as the primary fabrication method. Today, most of these two-member systems are considered fracture-critical. Anecdotal historical evidence suggests that there are a number of bridges in which individual components of built-up members have failed, yet the member maintained significant load carrying capacity. In other words, failure of one component, either by fatigue or fracture, did not result in the failure of the entire member. As a result, built-up members have been recognized as having a level of redundancy due to the discontinuity separating adjacent components. However, until recently, little research has been focused on this aspect of built-up members. Due to this lack of experimental and analytical evidence, current regulation prohibits utilizing internal member redundancy when determining whether or not a member should be classified as fracture critical. This prohibition subsequently prohibits setting rational inspection levels and intervals [1].

The purpose of this research was to experimentally and analytically investigate the member-level redundancy of built-up steel members subjected to flexure. First, built-up steel members were evaluated to demonstrate their ability to resist total member failure during a component fracture, and second, the remaining fatigue life of partially failed built-up cross-section was determined through experimental testing. Criteria for design and

evaluation to ensure acceptable performance were also developed.

## Experimental Research Program

Fifteen large-scale built-up specimens were tested as part of this research. Specimen lengths ranged from 24'-0" to 40'-0", with web depths ranging from 23" to 46". Three historical specimens were obtained from a bridge which was being removed from service (see Figure 1). The remaining twelve specimens were fabricated with new material. Thirteen of the specimens used component dimensions and proportions typical of built-up construction. Two of the specimens, intending to determine an upper bound to fracture resilience, used proportions which were extraordinary and unrealistic. Both riveted and high-strength bolted specimens were evaluated. Other parameters which were investigated included the number of bottom flange cover plates per specimen, the contribution of friction between built-up member components, and the method of hole fabrication (punched vs drilled).



Figure 1: Typical historical built-up specimens

Specimens were loaded in four-point bending using two 220 kip hydraulic actuators as shown in Figure 2. This created a constant moment region which facilitated the measurement of stresses in bottom flange components both before and after a component failure. Foil strain gages were used to measure the stresses at key locations in each of the bottom (tension) flange components both

longitudinally and transversely. This resulted in the ability to evaluate the localized stress distribution near a failed component both in and out-of-plane.

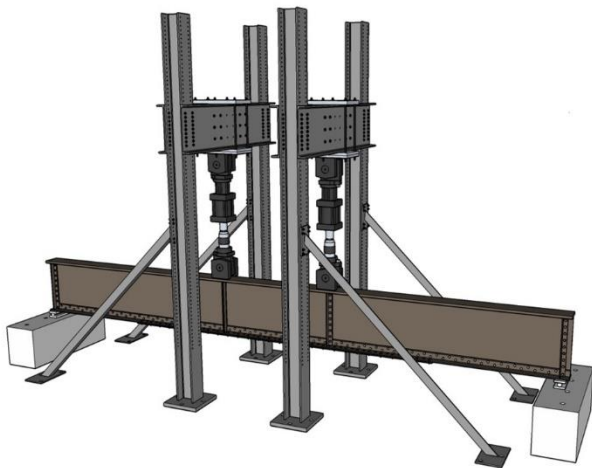


Figure 2: Test setup

A single component of each specimen was notched with a small cut near mid-span (within the constant moment region) to control the location of failure. This allowed the instrumentation to be strategically placed to capture any stress redistribution. A fatigue crack was then grown from the notch tip to create a high stress concentration and increase the likelihood of fracture at that location. The final crack length was determined based on linear elastic fracture mechanics, plate geometry, and material properties.

## Fracture Resilience

In order to evaluate the ability of built-up members to resist total member failure, each specimen was subjected to a cooling and loading protocol under extreme conditions which was expected to result in a component fracture. An initial load was applied to each actuator which reflected a simulated dead load. Each cracked specimen was then cooled to a temperature in which lower shelf brittle behavior of all bottom flange components could be obtained. This temperature ranged from  $-60^{\circ}\text{F}$  (the AASHTO zone III lowest anticipated service temperature) to  $-120^{\circ}\text{F}$  resulting in CVN impact energies that were less than 10 ft-lbs and in most cases, 3-5 ft-lbs. The load was then rapidly increased to a simulated live load, corresponding to  $0.55F_y$  on the original net-section. Based on LEFM and the length to which each crack

had been grown, this load was expected to produce a fracture of the cracked component.

No specimens fractured at the initial critical crack length. A few different approaches were taken to create a fracture in the cracked component. The first method used was to grow the crack to a more critical length and repeat the cooling and loading protocol. In six of the seven specimens tested in this manner the cover plate never fractured, even after repeated attempts at varying crack lengths. The specimen which did fracture had cracks which were beyond their calculated critical length, however the fracture did not propagate into any other components of the cross section.

Because a key component of the test was to determine the resilience of built-up steel sections when a single component fractured, additional methods were employed in the remaining specimens to ensure that a fracture would occur. A combination of brittle welds and driven wedges were used to increase the stress concentration at the crack tip. Additionally, it was found that fasteners near the crack were required to be removed to prevent constraint at the crack location, thereby allowing a significant fracture to occur. The use of these additional methods to create a fracture in a material already at lower-shelf behavior illustrated the fracture resilience of built-up members.

Of the eight specimens in which a component was fractured, six specimens did not experience fracture in adjacent components. The remaining two specimens were extreme cases which were designed such that failure was expected. These specimens were used to confirm the limitations of the design and evaluation recommendations that were developed. The first of these specimens had extensive cracking (after several fracture attempts had already taken place) in all of the bottom flange components. The remaining net-section was beyond yield stress capacity. The second specimen had unrealistic component proportions as well as fatigue cracks in all of the bottom flange components. This specimen experienced more than three times the calculated mean fatigue life. As stated, these two specimens were tested to understand at what point built-up steel members are no longer capable of resisting member failure.

The remaining six specimens were all fractured using the methods described. In each of

the cases, when a fracture occurred, the fracture did not propagate into any adjacent components. The applied load resulted in a stress approximately 80%-100% of the nominal yield stress of the remaining components. Of the twenty-eight fracture attempts, only eight resulted in fractures of a member component. The testing clearly demonstrated that 1) fracture of cracked components in built-up members is highly unlikely and 2) in the extremely unlikely event that a cracked component suddenly fractures, the fracture will not instantly propagate into the remaining components, assuming specific evaluation criteria are met.

### Fatigue Life of Partially Failed Cross-Sections

Twelve of the specimens were subjected to a constant amplitude cyclic loading program to determine their remaining fatigue life in the partially failed state (i.e., when a single flange component was completely failed). The remaining fatigue life was defined as the number of cycles after the first component failure (whether by fatigue or fracture) until a second component failed by fatigue. It is noted that a second component failure did not immediately result in total member failure. However, due to the built-up specimen geometry, two failed components resulted in a significant loss in the load capacity of the tension flange, and the capacity of the resulting net-section was drastically reduced.

The specimens were tested at calculated net-section stress ranges (using traditional mechanics of materials calculations on the partially failed cross-sectional geometry) which varied from 6.6 ksi to 25.6 ksi. The range of stress ranges was used in an effort to capture fatigue behavior over a wide spectrum. After experimental and analytical evaluation, it was found that localized stresses were amplified near the failure region. Therefore, the resulting amplified stress ranges (which reflected true conditions in the components) were between 7.8 ksi and 30.5 ksi. The resulting fatigue data was plotted against the AASHTO fatigue curves and can be seen in Figure 3.

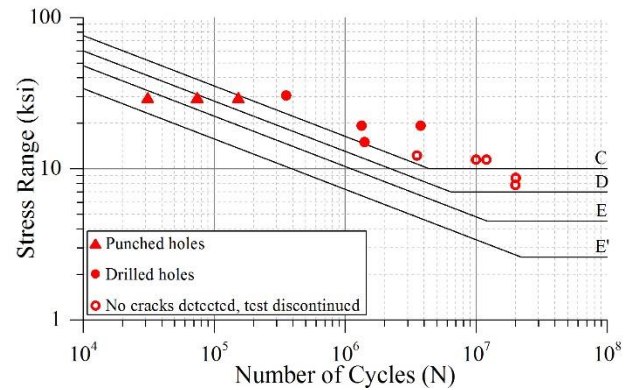


Figure 3: After-failure fatigue life

The fatigue data from the different specimens was evaluated to determine which parameters impacted the remaining fatigue life of built-up steel specimens. The parameter that had the largest impact on the remaining fatigue life was the method of hole preparation in the built-up specimen components. Other parameters investigated that did not appear to have an influence on the remaining fatigue life were: the contribution of friction, non-symmetric cross-section, the number of tension flange components, and the fastener type (rivets vs. high-strength bolts).

It was found that specimens with drilled holes had a substantially higher fatigue life than specimens with punched holes. This corroborates previous research in which punched holes were found to have a lower fatigue life than drilled holes or holes that were subpunched and reamed [2]. The AASHTO category E' fatigue curve was found to be a reasonable lower bound for specimens having punched holes, and the AASHTO category C fatigue curve was found to be a reasonable lower bound for specimens having drilled holes. In all cases, the remaining fatigue life tests illustrated that built-up members with partially failed cross-sections are able to sustain a substantial amount of fatigue cycles prior to any other component damage.

### Analytical Program

An analytical program was undertaken to more fully understand the localized stress distribution of a built-up steel member having a single failed component. Three dimensional finite element models were created in Abaqus CAE version 6.13. The dynamic explicit models incorporated contact modeling between the member

components, and fastener pretension to account for all of the parameters which were expected to contribute to the stress transfer in the member. The initial models were first benchmarked using data from the experimental test program. Subsequently, more simplified models were developed for use with a parametric study, which were able to accurately describe the member behavior, while significantly reducing the total computation time.

The longitudinal stress distribution in the components adjacent to a failed component was found to vary significantly throughout the transverse direction as well as the thickness of individual components (see Figure 4). In order to account for this variation throughout each component cross-section, the net-section stress of each component was found by summing the stress over the entire cross-section and then dividing by the net cross-sectional area. The resulting net-section stresses of each component were compared at discrete locations along the length of the member, with relation to the component failure location. It was found that the largest localized net-section stress was found in adjacent components in the same plane as the failure, and in the plane of the nearest fastener on either side of the failure plane.

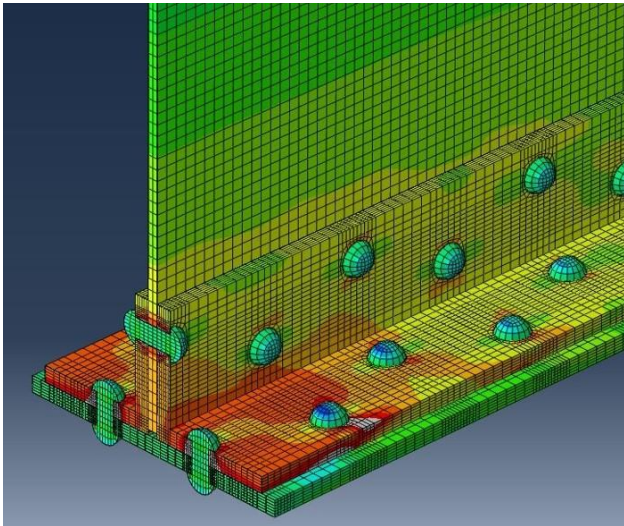


Figure 4: Stress distribution at failed cover plate

### Parametric Study

Throughout the parametric study the resulting net-section stress was determined for each model and then compared. Four different parameters were investigated as part of the study: Non-symmetric cross-sections, tension flange lateral

bracing spacing, height of web plate, and quantity of tension flange components.

### Non-Symmetric Cross-Sections

The effect of non-symmetric cross-sections on the localized stress distribution near a failed component was investigated to determine whether the resultant stress increase could affect the remaining fatigue life. Two scenarios were considered in which a non-symmetric cross section could result: the failure of a tension flange angle, and the partial failure of a tension flange cover plate. A series of ten different failure situations were evaluated for three different beam sizes; a fully failed flange angle, and nine different stages of cover plate failure. Figure 5 shows a portion of a typical model set.

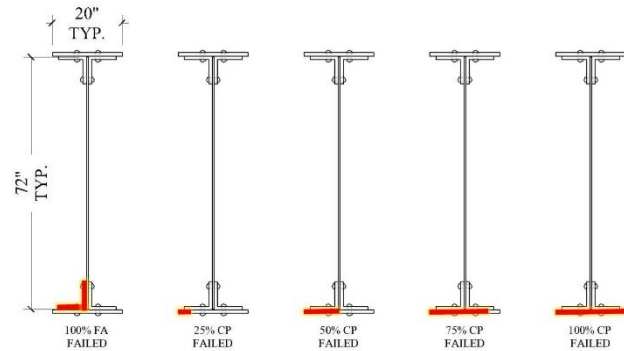


Figure 5: Non-symmetric cross-section model set

The net-section stress in a single flange angle was used to compare the resulting stress increase from each of the failure scenarios. When the various stages of failure were analyzed, it was found that only scenarios where the cover plate was more than 60% failed was the stress increase in the angle greater than what would result from a fully failed cover plate (considered a symmetric cross-section). Since this scenario would represent a case where a fatigue crack was of substantial size, the crack growth rate would be expected to be rapid at this stage. The total crack length was plotted based on number of cycles for various experimentally tested specimens. The resulting duration in which the crack length would be expected to be greater than 60% of the cover plate width was found to be approximately 5%. Because of the minimal duration at which the stress increase was found to apply, the effect of non-symmetric cross-sections was

considered insignificant in the evaluation of remaining fatigue life of built-up steel members.

### Tension Flange Unbraced Length

The effect of the unbraced length of the tension flange on the stress increase was evaluated in conjunction with the non-symmetric cross-section. Three different unbraced lengths were used (10'-0", 20'-0", and 30'-0"). The models evaluated did not indicate that the unbraced length was a factor in the stress increase of partially failed built-up steel members.

### Web Height

Due to the results of previous parameters, only symmetric cross-sections were considered in the web height parameter study. Four different web heights were evaluated, (24", 48", 72", and 96") as shown in Figure 6. Each of the models had a single cover plate on both the tension and compression flanges. A completely failed cover plate was simulated for each of the models. The remaining net-section stress was measured in the tension flange angles adjacent to the failed cover plate. The resulting net-section stress in each of the models was found to be less than 3% between each of the models. Therefore the height of the web was not considered to be a significant parameter in the stress distribution of partially failed built-up members.

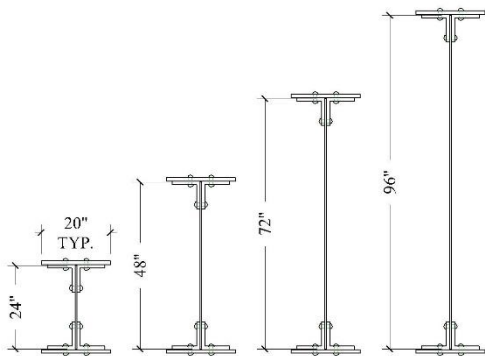


Figure 6: Web height model set

### Quantity of Tension Flange Components

The effect of the quantity of tension flange components (cover plates and angles) was investigated to determine their effect on the redistribution of stresses adjacent to a failed component. Models were evaluated with 1, 2, 3, and

4 cover plates for three different model sizes (see Figure 7 for typical configuration).

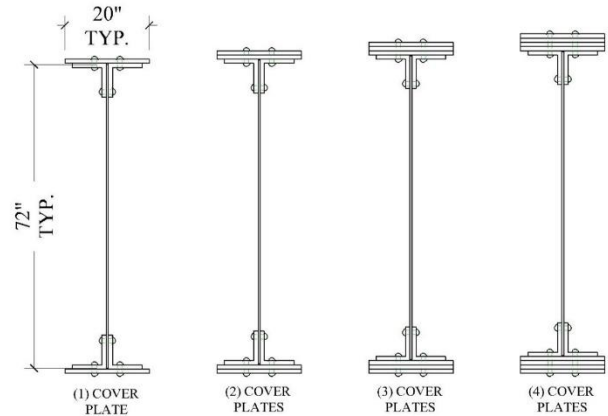


Figure 7: Quantity of flange components model set

It was found that the number of cover plates of a steel built-up member has a significant effect on the stress distribution in the localized region near a failed component. Figure 8 shows the stress distribution at the failure plane for a model with three cover plates in which the lower cover plate is failed. When a component of a built-up member is failed, the load is transferred through the fasteners, into adjacent components. Based on the results of this model set, the majority of the load was found to be distributed to the component most adjacent to the failed component (e.g. for a member with two cover plates which experiences a failure of the lower cover plate, the majority of the load is distributed into the upper cover plate, while the flange angles see a minimal increase in longitudinal stresses).

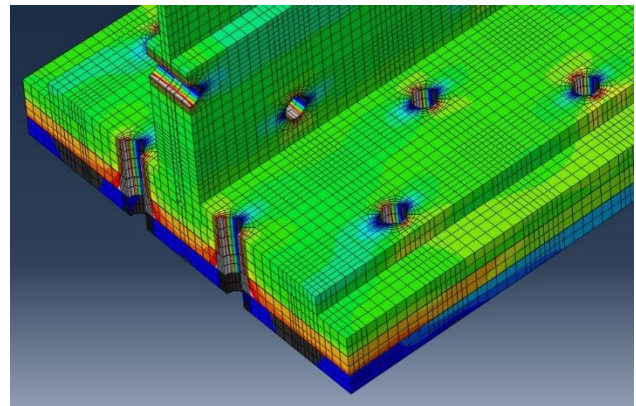


Figure 8: Three cover plate model with one cover plate failure

In order to describe this behavior in a way that could be used during evaluation, a relationship

was found which relates the actual net-section stress of an adjacent component to its calculated net-section stress. The calculated net-section stress of a partially failed built-up member in bending can be found using simple mechanics of materials calculations ( $\sigma = My/I$ ) in addition to the resulting cross-sectional geometry. The intent was to determine an amplification factor which could accurately describe the large stress distribution which is transmitted into the adjacent component. In order to determine the relationship, the data for twelve different models with 1, 2, 3, or 4 cover plates was collected (see Figure 9). For each model, the resulting net-section stress was normalized by the calculated net-section stress which is calculated using the resulting partially failed geometry of the member. The linear relationship  $\beta_{AF}$  (Equation 1) should be used to amplify the calculated net-section stress to conservatively account for the resulting net-section stress in a partially failed built-up member in flexure as shown in Equation 2.

Equation 1: 
$$\beta_{AF} = 1 + 0.2 \left( 1 + \frac{N}{4} \right)$$

Where:

N = Number of cover plates

Equation 2: 
$$\sigma_{net-AF} = \beta_{AF} \frac{M}{S_{x-net-AF}}$$

Where:

$\sigma_{net-AF}$  = After-failure net-section stress

M = Applied moment

$S_{x-net-AF}$  = After-failure section modulus

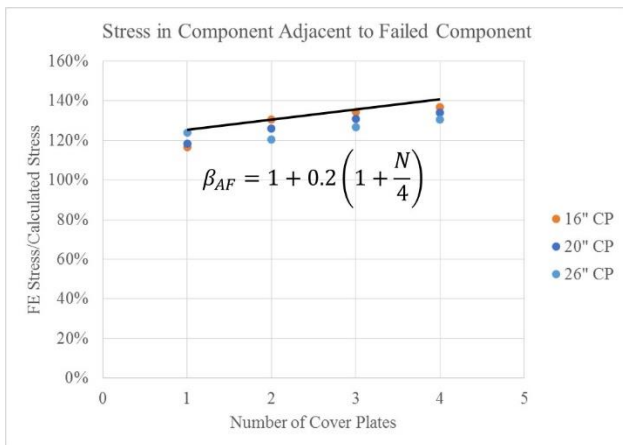


Figure 9: After-failure stress distribution factor

## Evaluation Methodology

In order to facilitate the application of the given research, an evaluation methodology was developed which was intended to be used by bridge owners and practicing engineers. The methodology was developed to calculate the redundancy of built-up steel girders in design as well as existing members which may have been previously subjected to some amount of fatigue cycles. For either case, the methodology can determine the remaining fatigue life of members in the partially-failed state. From the remaining fatigue life, a rational inspection period can be determined and implemented to more accurately meet the needs of the member.

The proposed methodology applies only to built-up steel members subjected to flexure. In addition, members must meet specific criteria which are based upon experimental and analytical results in order to apply the calculations presented herein. Members must have a minimum of one cover plate on the tension flange. The member must also satisfy all other limit states in the partially failed condition (e.g. strength). In order to ensure that the remaining cross-section would have adequate capacity in the partially failed state, the net-area of any potentially fractured component must be less than or equal to 60% of the total net-area of the tension flange (the vertical leg of the flange angle shall not be included in the tension flange areas calculated). In addition, for evaluation of existing members with prior fatigue accumulation based on the AASHTO Manual for Bridge Evaluation (MBE), the initial fatigue life must be determined based on the AASHTO Category D fatigue curve (instead of the Category C fatigue curve as currently prescribed [3]).

Three potential scenarios describe the conditions that built-up flexural members may fall under. In each of the scenarios described below,  $\Delta f_0$  is the initial fatigue life, and  $\Delta f_{AF}$  is the fatigue life after a component failure. The scenarios as well as their steps are described as follows:

1. Case I: if  $\Delta f_0$  was below the Category D CAFL (7 ksi) AND  $\Delta f_{AF}$  is below the Category C CAFL (Category E' for punched holes).
  - a. This condition indicates that the partially failed member was, and

- still is in the ‘infinite life’ region of the S-N curve.
- b. Since an infinite fatigue life was found, this indicates that fracture critical inspections are not necessary. The member may be inspected with traditional inspection protocols used on non-fracture critical members. Routine inspections should focus on locating any broken components along the length of the member.
2. Case II: if  $\Delta f_o$  was below the Category D CAFL (7 ksi) but due to the reduced net-section,  $\Delta f_{AF}$  is above the Category C CAFL for drilled holes or Category E’ CAFL for punched holes.
    - a. This condition indicates that the partially failed member had ‘infinite life’ prior to the component failure, but after the failure the member is in the ‘finite life’ region of the S-N curve.
    - b. A finite fatigue life should be calculated using the AASHTO MBE and  $\Delta f_{AF}$ . No prior fatigue damage should have occurred since the member was previously in the ‘infinite life’ region. Therefore the fatigue life would be assumed to start at the time of the first component failure.
    - c. A hand-on fracture critical inspection interval should be based upon the calculated remaining finite fatigue life in the partially failed state.
  3. Case III: if  $\Delta f_o$  was above the Category D CAFL (7 ksi) and  $\Delta f_{AF}$  is above the Category C CAFL for drilled holes or Category E’ CAFL for punched holes.
    - a. This condition indicates that the partially failed member was, and still is in the ‘finite life’ region of the S-N curve.
    - b. A finite fatigue life should be calculated using the AASHTO MBE, the initial fatigue accumulation (based on  $\Delta f_o$ ) and  $\Delta f_{AF}$  using Miner’s Rule.

- c. A hands-on fracture critical inspection interval should be based upon the calculated remaining finite fatigue life in the partially failed state.

The scenarios above are intended to guide an owner or practicing engineer in setting the frequency of fracture-critical inspections. Less rigorous routine inspections should still occur to evaluate the condition of the member (as well as other details of the bridge).

## Conclusion

Built-up steel girders were found to have a high level of resistance to fracture based on experimental testing and performance under extreme loading cases. In addition, members in a partially-failed state were found to have a significant remaining fatigue life prior to any other component failure. Through a finite element model parametric study, it was found that the localized stress distribution that occurs in a partially-failed member has the largest impact on the component adjacent to the failed component. This relationship was evaluated and an amplification factor was found that can be used to modify the calculated net-section stress to reflect the stress increase in the adjacent component. An evaluation method was presented which discusses different scenarios that a built-up member may experience and the proper manner to evaluate remaining fatigue life for each of the three scenarios. Built-up steel members meeting the criteria developed herein, should not be deemed fracture critical members that must be inspected at arm’s length at the present arbitrary interval of 24 months. Rather, using these criteria, a rational interval for such inspections should be calculated. Additionally, because of member-level redundancy, inspections should focus on locating a failed component, rather than small cracks.

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