Transformative Approaches for Evaluating the Criticality of Fracture in Steel Members

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ABSTRACT

There has been considerable research and interest in the topic of fracture-critical members (FCM) during the past decade. As a result, the entire concept of what constitutes an FCM is being revisited, and many long-standing ideas and opinions related to this classification of members are being shown to be overly conservative. Significant advances in the understanding of fracture mechanics, material and structural behavior, fatigue crack initiation, fatigue crack growth, fabrication technology, and inspection technology have allowed other industries to address fracture—or, more importantly, control of fracture—in a more integrated manner. After years of research, new stand-alone AASHTO guide specifications that give codified direction on how to perform 3D system analysis to verify system redundancy, as well as guide specifications to evaluate internal member-level redundancy of mechanically fastened built-up members for both new and old bridges, have been proposed. Additional research demonstrating the benefits of exploiting the improved toughness of modern high-performance steel (HPS) grades has been completed. Through these advances, it is now possible to create an integrated fracture control plan (FCP) combining the original intent of the 1978 FCP with modern materials, design, fabrication, and inspection methodologies. Further, an integrated FCP provides economic benefits and improved safety to owners by allowing for a better allocation of resources by setting inspection intervals and scope based on sound engineering rather than based simply on the calendar. In summary, an integrated FCP encompassing material, design, fabrication, and inspection can ensure fracture is no more likely than any other limit state, ultimately allowing for a better allocation of owner resources and increased steel bridge safety. This paper summarizes some of the recent advancements related to the topic of the FCM and provides a suggested approach to providing more rational treatment of such members without compromising reliability.

Keywords: fracture critical members, steel bridges.

INTRODUCTION

Despite the perception surrounding bridges classified as having fracture critical members (FCM), there is actually very little evidence that would suggest such bridges have been more unreliable than other types of steel bridges. In fact, the term *fracture critical* conjures up images of certain failure to many. In contrast, truss bridges, which are classified as having FCM, also have what the author refers to as BCM, or "buckling critical members." However, nonredundant compression members in trusses exist without any additional concern regarding their criticality, nor do they require any special fabrication or in-service inspection. What is interesting is that it is often much more difficult to redistribute compression forces into other members. For example, a compression member, when subject to tension, will generally be able to carry such forces without becoming unstable, presuming there is enough steel in the cross section. However, a slender tension member will likely buckle should compression forces be applied following failure of a nearby compression member.

Nevertheless, as an industry, there is little concern that nonredundant compression members will suddenly fail in an unstable buckling mode. The reason is simple: The bridge engineering community as a whole believes the results of many years of experimental research and analysis. Thus, such members are not treated differently in design or during in-service inspection whether they are redundant or not. For some reason, the same is not true for tension members. While isolated failures of tension members have occurred, the bridge engineering community is very reluctant to accept the decades of research and advancements that have been made in the understanding of fracture mechanics, the availability of steels with superior toughness, advances in fabrication methods, and nondestructive testing. This paper attempts to demonstrate that it is time to move forward and accept integrated fracture control plan concepts that have been widely accepted and proven to be reliable in the aerospace and pipeline industries.

CURRENT VIEWS ON FCM

It is the observation of the author that the majority of the papers published since 2007 that are related to fatigue and fractures issues or structural health monitoring (SHM) in highway bridges begin by citing the I-35W collapse, which

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occurred in August 2007. The papers typically go on to recite how the I-35W bridge was a "fracture critical bridge." (*It is noted that there is no such thing as a fracture critical bridge per the AASHTO Specifications, but only fracture-critical members.*) The obvious problem with such papers is that the failure of the I-35W bridge had nothing to do with the fact that the bridge had members classified as FCM (NTSB, 2008). Rather, the failure was due to a serious error that occurred during design that resulted in undersized gusset plates at several locations on the trusses. It is also worth noting that the bridge was built circa 1967, long before (1) the implementation of the modern fatigue design provisions for highway bridges and (2) the implementation of the AASHTO/AWS D1.5 fracture control plan (FCP).

Another common failure cited in papers is of course the Silver Bridge (aka the Point Pleasant Bridge). This bridge opened in 1928 and collapsed in 1967 (NTSB, 1968). Despite the fact the bridge carried traffic for nearly 40 years, the sudden failure of one of the eyebars led to a sudden catastrophic failure of the bridge, resulting in the loss of 46 lives. While it is true that the failure of the Silver Bridge was due to brittle fracture of a nonredundant eyebar, it cannot be overlooked that the circa 1928 high-strength steel used in the eyebar was extremely brittle and would never be permitted for use in highway bridges built since at least the mid-1980s.

In fact, a brief literature review will quickly reveal that the "classic" brittle fractures often cited have two things in common. First, they have only occurred in bridges designed and fabricated prior to the implementation of the AASHTO/AWS D1.5 FCP and modern AASHTO fatigue design provisions (AWS, 2015; AASHTO, 2017). Second, with the exception of the Silver Bridge (eyebar failure) and Mianus River Bridge (pin/hanger failure) (NTSB, 1968, 1984), which both utilized fundamental design approaches that have been completely abandoned by the U.S. bridge industry, all other cases where an FCM has fractured have not resulted in catastrophic collapse. Interestingly, the obvious implication of the first point is that bridges designed and fabricated *after* the implementation of these provisions are highly unlikely to experience a brittle fracture. Considering the FCP and modern fatigue provisions have been in place for more than 40 years with no noted failures suggests they are working quite well. The second point also illustrates that although failure of an FCM would be expected to possibly result in collapse of a portion of or the entire structure, history seems to prove otherwise in all but those systems that are truly nonredundant and, as stated, no longer utilized.

The preceding statement—that the lack of observed failures is due to the improvements in design, materials, and fabrication—is often questioned. Some believe that the lack of in-service fractures is primarily due to significant efforts spent on in-service inspections. While inspections are certainly important, the evidence does not suggest this is the primary reason for the lack of failures. In fact, with the exception of the Mianus River Bridge, fractures are almost always traced back to a flaw that could not be detected with the naked eye. For example, the fracture in the Silver Bridge was triggered by a small crack in the eyebar that could never be detected visually (NTSB, 1968). Other fractures, such as those observed in the Hoan Bridge, U.S. 422 Bridge, and others deemed to be due to so-called constraint induced fracture (CIF), were all triggered in the absence of any detectible fatigue crack (Fisher et al., 2001; Kaufman et al., 2004; Ellis and Connor, 2013). Hence, the evidence suggests that the improved performance is primarily due to the efforts of the FCP, better detailing, and better design rather than hands-on inspection. This is not to say inspection is not needed nor that it has not prevented failures in general. Rather, the role of inspection as the major preventer of sudden brittle fractures (in contrast with failures say due to corrosion) appears to be questionable.

Unfortunately, despite the excellent service record of FCM, even those designed and fabricated *prior* to the modern FCP and fatigue design provisions, many in the bridge industry seem to be of the opinion we are still building steel bridges no differently than we did in, say, the 1950s or even earlier. In effect, the perception is when the "FCM" term is evoked, it is thought that somehow, although the structure was designed and built in 2018, it is no less likely to experience a brittle fracture than a bridge built in, say, 1958, and it will, in fact, most likely experience a brittle fracture during its service life.

WHAT HAS CHANGED SINCE THE INTRODUCTION OF THE FCP?

Table 1 presents a short summary of a few high-profile examples where brittle fractures have been observed due to a variety of issues. These specific structures highlight some of the more common reasons fractures have been observed. (For example, poor weld quality, either during fabrication or during repair of a weld, is often a concern.) The use of plug welds or other fracture susceptible details (e.g., CIF details) has been identified to be the cause of several brittle fractures.

Table 1 clearly illustrates several important points. The most obvious is that all of the bridges were designed and fabricated prior to the introduction of the AASHTO/AWS D1.5 FCP introduced in the mid-1980s and the modern fatigue design specifications introduced about 1974. (It is noted that the FCP was first introduced in 1978 as an AASHTO Guide Specification, and many bridges were built to these provisions as early as this.) The details or materials that were utilized in these bridges are no longer permitted, as noted in Table 1. Another observation is related to all the bridges with FCMs that are *not* listed in the table—in other

Table 1. Summary of High-Profile Cases of Brittle Fractures							
Bridge	**Cause**	**Approximate Year of Destruction or Construction**	**Fabricated to FCP?**	**Designed for Fatigue?**	**Are these Details or Materials Permitted Today?**	**Would Field Inspection Have Prevented?**	**Did the Bridge Collapse?**
Silver Bridge (W. Va.)	Brittle high-strength steel	1928	No	No	No	No	Yes
Neville Island I-79 (Pa.)	Poor repair weld procedures	1970	No	No	No	No	No
Lafayette St. (Minn.)	Poor quality intersecting weld	1966	No	No	No	Maybe	No
Hoan Bridge (Wis.)	Constraint-induced fracture (CIF)	1968	No	No	No	No	No
Delaware River Truss (Pa.)	Misdrilled holes filled with weld	1954	No	No	No	No	No

words, those in which a failure has not occurred. The majority of these bridges were put into service before the profession considered the fatigue or fracture limit states in design, prior to mandatory Charpy V-notch (CVN) requirements, and prior to the introduction of much more stringent shop welding procedures. These bridges, which by far exceed the number in which individual problems have been observed, have been carrying traffic safely for decades. As such, this again emphasizes that the overall historical performance of steel bridges with FCMs has been excellent, despite a few isolated failures. Also as stated, when failures have occurred, the outcome has rarely been catastrophic.

PERFORMANCE OF THE "MODERN" STEEL BRIDGE

Prior to discussing how modern steel bridges have performed over the years, one must first define what is meant by the term *modern* steel bridge. In the context of this paper, the author refers to modern steel bridges as those that were designed and fabricated after certain criteria were in place. In the context of FCM, those built after this date will typically possess the following characteristics:

1. Meet modern CVN requirements.
2. Be fabricated to the AASHTO/AWS D1.5 FCP.
3. Be designed using the nominal stress range approach for fatigue.
4. Are unlikely to possess details susceptible to distortion-induced fatigue cracking, which is responsible for most cracking observed in highway bridges.

In general, bridges built after about 1985 meet all of the above criteria, while none of the bridges listed in Table 1 do. More recent work related to CIF would add a fifth criterion to the list. Around 2012, requirements were added to the AASHTO LRFD Bridge Design Specifications to prevent the use of details susceptible to this form of fracture. Fortunately, most if not all girder bridges in the United States that possess CIF susceptible details on FCM designed prior to this date have been retrofit in order to prevent this form of fracture. Thus, this fifth criterion is met by most welded FCM.

History has shown that steel bridges that meet these criteria are extremely unlikely to be susceptible to brittle fracture. In fact, the author has not been able to identify any FCM that have met the preceding criteria in which a brittle fracture has occurred. Thus, the improvements made in the design and fabrication of FCM have resulted in steel members that are highly reliable in terms of the fatigue and fracture limit state.

Despite these major improvements, there has been no relief regarding the 24-month hands-on in-service inspection of bridges that contain FCM. As is well known, these inspections consume considerable resources and place risk on both the inspectors and the public. What this means is that an FCM built in 1955 is treated identically to one in a bridge built in 2015 when it comes to in-service inspection. Obviously, this makes little sense. If a member is deemed to be an FCM, then it shall be inspected every 24 months regardless of any other criteria, period. Shorter intervals are sometimes introduced due to concerns over the condition of the member, but again, these are arbitrary. In some ways, it is not surprising that the inspection interval has not changed as improvements have been made because the current maximum 24-month interval for FCM inspection was not based on engineering. In other words, improvements in fabrication, material, design, and so forth cannot easily be used to justify a change in the interval because these were not the criteria used to set the original interval in the first place.

The one-size-fits-all approach may also have been somewhat reasonable nearly 50 years ago when the National Bridge Inspection Standards (NBIS) were introduced because the average age of many of the bridges in the United States was much younger. Consider that since the U.S. Interstate system was initiated in 1956, most bridges were still relatively new in the early to mid-1970s. Hence, inspections that were calendar based could be justified in a way that is similar to what is done in the health care industry. For example, younger individuals visit the doctor for brief routine checkups at calendar-based intervals of, say, every 12 months or longer. However, for older individuals, say, 70 years or older, routine checkups are often much more frequent, and in most cases, there are specific health issues that require special attention and time. In fact, the suggestion that an 18-year-old should require or *be required* to receive the same health care as a 70-year-old illustrates the error in the current approaches to bridge inspection.

Another very important issue that is commonly overlooked is related to the fact that other countries also have bridges that contain members that would be classified as FCM in the United States. However, international scanning tours for bridge management and fabrication have noted that Europe does not have special policies for FCM with regard to how such members are inspected in service (Connor et al., 2005; Verma, 2003). While other countries inspect their inventory, they do not impose additional and arbitrary inspection criteria on such members. Interestingly, the failure rate of these members is no greater than that observed in the United States from the data that can be found (Verma, 2003). In other words, it appears that the extra in-service inspection efforts mandated by law in the United States for FCM have not resulted in any significant improvement in reliability in modern steel bridges.

Finally, when performing any type of inspection, one must consider the concept of probability of detection (POD). POD studies are used to determine the probability of detecting a defect in a specified component under the inspection conditions and procedures provided or commonly used. In the case of highway bridges, this is almost always performed through a visual inspection. POD is typically expressed as a function of a quantifiable target parameter associated with the given flaw (e.g., length). While the target parameter is the single most influential factor in determining the probability of detection, it is also a function of many other physical and operational factors, including the material, geometry, flaw type, nondestructive testing (NDT) method, testing conditions, as well as the inspector and his or her certification, education, and experience (Georgio, 2006).

Probability of detection data can be analyzed as discrete data, where the response is binary (either a hit or a miss), or as continuous data, which is a signal response (tracking how close the noted size of the defect is to the actual size). Hit/miss data produces qualitative information indicating whether a flaw is present or absent.

To investigate the likelihood of detecting a fatigue crack in a steel member, a large-scale POD study was performed at Purdue University (Snyder et al., 2015). This study, believed to be the first statistically significant study of its kind focused on fatigue cracks in bridge members, revealed that the 50-50 crack length (i.e., that length with a 50% probability of an inspector hitting or missing) is on the order of 1 in. using visual inspection. Further, to achieve a POD of 90% requires cracks to be in the range of 5 in. in length. Other studies have shown similar results in terms of the overall low POD associated with visual inspection (Moore et al., 2001; Washer et al., 2014). This is not to criticize the inspection community but is simply pointing out that with limited inspection budgets, coupled with the time constraints placed on inspectors, quality will suffer. This suggests that the inspections being performed may have limited value regarding the detection of a crack that could lead to brittle fracture because it is clear that many cracks are missed. Again, the inference is that the excellent service record of modern FCM is primarily based on the improvements that have greatly decreased the likelihood of cracking and is not due to in-service inspections finding such cracks before they reach some critical size.

DEVELOPMENT OF AN INTEGRATED FRACTURE CONTROL PLAN

Considering the preceding data, it would seem that existing state-of-the-practice engineering concepts could be used to develop a strategy toward inspecting members traditionally classified as FCMs that are based on quantitative engineering principles rather than simply relying upon calendar-based approaches. Ideally, the approach would link the capability of the inspector, the strength or performance of the member, and the interval of the inspection. This would result in what is referred to as an *integrated fracture control plan*. In such an approach, the desired reliability is achieved, and deficiencies in one area (e.g., the inability to reliably find small fatigue cracks) is made up through the design of the member and/or inspection interval. Interestingly, this approach is commonly utilized in other industries, such as oil and gas and aerospace, and is a proven and effective strategy. A few examples of how an integrated fracture control plan can be achieved using different, but rational strategies follow.

Through Internal Redundancy

Mechanically fastened built-up steel members have long been perceived to be highly resistant to complete and sudden catastrophic failure due to brittle fracture. In fact, the AASHTO *Manual for Bridge Evaluation* states in the commentary for Article 7.2.1 that for evaluation of riveted members and connections, Category C, rather than Category D (which is used for design as it represents first cracking) is appropriate since "Category C more accurately represents cracking that has propagated to a critical size. This increase in fatigue life for evaluation purposes is appropriate due to the redundancy of riveted members" (AASHTO, 2016) Thus, AASHTO recognizes that the redundancy within an individual member built-up from multiple components, which provides mechanical separation of elements, can limit crack propagation across the entire member cross-section as compared to an all welded member.

While this type of redundancy was typically known to be present through anecdotal evidence, no guidance existed on how to establish if *adequate and reliable* internal redundancy is present in a given mechanically fastened built-up steel member. Due to a lack of experimental evidence or existing standards, the Federal Highway Administration (FHWA) was not able to recognize mechanically fastened built-up members as having adequate redundancy to alter the inspection rigor from that of a fracture critical inspection associated with, say, a welded member (Lwin, 2012).

Transportation Pooled Fund Project TPF-5(253), "Evaluation of Member Level Redundancy in Built-up Steel Members," was conceived and completed to address all of the critical issues related to performing a "credible" analysis to identify built-up members that have adequate internal redundancy to resist complete failure of the cross-section should one component suddenly fail (Hebdon et al., 2017a, 2017b; Lloyd, 2018). The resistance to such a failure mode is referred to as cross-boundary fracture resistance (CBFR). Through full-scale experimental testing performed in the research, it was shown that brittle fracture in one component does not propagate into the adjacent component if certain conditions are met. The member is then checked to establish if there is sufficient strength in the faulted condition under prescribed load combinations. The experimental work was furthered through fatigue testing to establish how long a member in the faulted state could survive in this damaged condition (i.e., with one component completed failed). Using this information, conservative estimates could then be made to set an appropriate inspection interval and scope to ensure a second component will not fail prior to the next inspection. The research showed that members meeting specific proportion and condition requirements can be designated as internally redundant members (IRM) that do not need to be subjected to the traditional arms-length inspection associated with FCMs. The research was recently incorporated into the AASHTO *Guide Specifications for Internal Redundancy of Mechanically-fastened Built-up Steel Members*, which was approved by the AASHTO Subcommittee on Bridges and Structures in June 2018 (AASHTO, 2018a). These specifications are applicable to both new and old steel bridge structures.

It is also important to note that the primary objective of the in-service special inspections of an IRM is to detect fully severed components and *not* to find very small fatigue cracks emanating from any one of thousands of fastener holes. Conceptually, this is illustrated in Figure 1. (It is noted that the crack must extend beyond the rivet head in order to be detected in the first place. Hence, it is implied in the current inspection strategies that the member can at least tolerate cracks on the order of a few tenths of an inch before

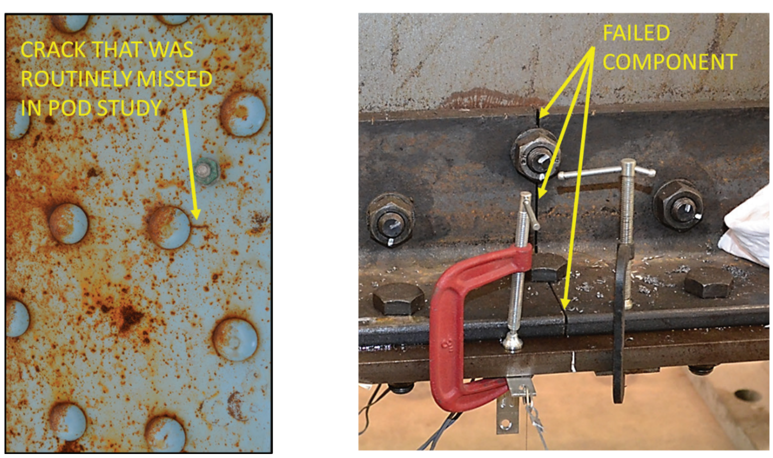

Fig. 1. Photographs of two different cracks in built-up members contrasting the differences in cracks that need to be detected in a traditional FCM inspection vs. those associated with an IRM.

they are detected.) The POD study cited earlier showed that it is unrealistic to assume that small cracks can be identified with a high level of reliability using traditional visual inspection techniques. In contrast, the likelihood of detecting a severed component in a built-up member, on the order of 10 in. long or more, is obviously much higher. Using the experimental data from the member testing and the data from the POD study, an integrated fracture control plan was developed for IRM as follows:

- Through rational evaluation procedures, the member can be shown to be capable of tolerating the assumed damage (i.e., an entirely failed component).
- The inspection interval is based on experimental data derived from fatigue testing of damaged girders with a safety factor incorporated in order to set a rational inspection interval based on the time required to fail a second component.
- The damage is large enough to be detected with a high level of reliability. For example, rather than trying to find a ½-in. crack emanating from any one of thousands of rivets, the inspector need only find an entirely broken component. As stated, the POD for such damage is high.

In this way, the fracture control plan is "integrated" in that the inspection interval and capability of the inspector is linked to the tolerance of the member as a function of time. The entire methodology is based on quantitative engineering and not simply a calendar-based approach. The converse is what is being done today; inspectors look for cracks that they are unlikely to find and that are not likely to be critical in the first place at an interval that is entirely arbitrary. For example, there is no calculation that shows the 24-month interval is the appropriate duration to ensure cracks are reliably found before they become critical. Obviously, the integrated FCP is a major departure from the current calendar-based approach to setting inspection intervals.

A subtle, but very important difference in the preceding approach is that the damage that is deemed critical is assumed to have occurred (i.e., fracture of a component is assumed to have occurred immediately following the previous inspection). In other words, inspectors are not randomly looking for small fatigue cracks or other damage that may not be critical, which could be located almost anywhere on the member. Rather they are searching for a specific form of damage that has been shown to be of importance based on engineering; hence, they can focus on looking for what is important.

Through System Analysis

Another form of redundancy can be exploited through the use of advanced analytical tools. In such an approach, analysis is performed in which entire FCM are assumed to have failed and the structure analyzed to establish the consequence of the failure. By definition, failure of an FCM is generally presumed to probably result in collapse of the structure or a portion thereof. However, as shown in Table 1, and based on historical experience, in the rare instances an FCM has failed, catastrophic collapse did not result. Thus, what if it could be shown that even in such a faulted condition, the structure possesses sufficient reserve capacity that it can carry some appropriate level of live load? To develop and codify the procedures to evaluate the redundancy of bridges with members traditionally designated as FCMs through 3D system analysis, NCHRP Project 12-87a was conducted (Connor et al., 2018).

While guidance on the required level of analysis is essential (e.g., nonlinear vs. linear analysis, etc.), other very important criteria must be established in order to ensure uniform evaluation procedures. For example, some critical questions arise, such as:

- What is an acceptable target reliability for a damaged structure?
- What level of live load should a faulted structure be capable of carrying?
- How should live load be positioned on the bridge?
- What criteria (service and strength) should be used to define failure?
- What is the exposure period during which the bridge should be assumed to be in the faulted state, and how does this tie into future inspection needs?

The NCHRP 12-87a project attempted to answer all of these questions. The research resulted in the AASHTO *Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members*, which was approved by the AASHTO Subcommittee on Bridges and Structures in June 2018 (AASHTO, 2018b). This *Guide Specification* provides owners and engineers with a robust benchmarked analysis methodology that can be used to increase the effectiveness and efficiency of inspection in steel bridges with members traditionally classified as an FCM. These specifications are also applicable to both new and old steel bridge structures.

One significant aspect of the *Guide Specifications* is the development of new load combinations specifically intended for evaluating the performance of a steel bridge in which an FCM is assumed to be completely failed. The reliability principles used in current design and evaluation specifications were utilized to develop load combinations that capture uncertainty in load and resistance. Two new load combinations, referred to as Redundancy I and Redundancy II, were developed. Redundancy I characterizes the instant when a primary steel tension member fails, in which the dynamic amplification of load is considered. As has been discussed, most bridges in which a brittle fracture of an FCM has occurred carried live load for some extended period of time,—in some cases, up to a few months. This observation resulted in the development of the Redundancy II load combination. This load combination characterizes the loading during an extended period of service between the occurrence of the failure and the discovery of the failure.

Other strength and serviceability requirements were also developed. If the bridge is able to satisfy these strength and serviceability requirements when subjected to the redundancy load combinations after the failure of a member previously designated as an FCM, such a member can be redesignated as a system redundant member (SRM). An SRM must still be fabricated to AASHTO/AWS D1.5, clause 12, but does not need to be subjected to hands-on inspections every 24 months.

While the level of analytical effort is admittedly considerable in some cases, one must also consider the objective of the analysis. Specifically, an engineer who performs such an evaluation is attempting to establish that if a primary tension member were to fail, the bridge either is or is not capable of (1) surviving the event while subjected to some level of live load and dynamic amplification and (2) carrying the traveling public for some extended but undefined interval. One must also recognize that for bridges satisfying the requirements of the evaluation, future hands-on inspections will not be required by law and, hence, may not be performed for quite some time. Clearly, this is a weighty responsibility and an area of bridge engineering in which the profession has virtually no experience. Thus, it is not an unreasonable request that the engineer perform some rather rigorous analysis.

However, as with any methodology, experience is gained with time, and the author believes the same will eventually be true regarding this type of system analysis. As more and more structure types are analyzed using the *Guide Specifications* described herein, the industry as a whole will begin to identify structural configurations that inherently possess considerable reserve strength, even in a severely faulted state (i.e., with one girder or tension member completely failed). Such appears to be the case with continuous-span twin tub girder bridges. Ongoing work suggests that such bridge types can be very robust when certain basic design and detailing criteria are utilized (e.g., curvature limits, span limits, required details, etc.). For example, full-depth full-width diaphragms between girders and the use of shear studs that extend above the bottom layer of deck reinforcement provide significant load redistribution capabilities when one of the tubs is assumed to have completely failed at some critical cross section. The author believes that, very soon, the industry will be able to develop simple guidelines that can be used to "prequalify" the girders of certain continuous twin tub-girder bridges as SRM and eliminate the need for complex 3D FEA.

Through Design and Material Selection

Another, possibly more forward-thinking approach to addressing the concerns associated with FCM is related to reducing the likelihood of the fracture to an extremely low level. As has been stated, for bridges fabricated after the introduction of the AASHTO/AWS FCP nearly 40 years ago, there have been no brittle fractures observed in the field. However, one must also recognize that in the past 40 years, there have been major improvements in design, understanding of fatigue and fracture, and fabrication. Further, with the introduction of HPS grades, modern steels can be economically produced that possess toughness levels much higher than the current specified minimums. Thus, it does not seem out of the question to begin to treat failure due to fracture like any other failure mode, such as ultimate strength or buckling. As discussed, bridge engineers do not become overly concerned with nonredundant compression members and identify them as BCM on plans. Such members are not required to meet more stringent out-of-straightness tolerances during fabrication, nor are they treated differently during design by using lower nominal compressive resistance than that specified. Special measurements are not required for such members during in-service inspections to establish if they have deviated from the as-built out-of-straightness or if corrosion has resulted in some minor change of the cross-section. While failure of a compression member is likely more critical than a tension member, engineers are not overly concerned with possible failure of these members when they are deemed non-redundant. The reason is simple: The bridge engineering community as a whole believes the results of many years of experimental research and analysis. There is a considerable body of work that documents the effects of geometric imperfections, residual stresses, fabrication tolerances, etc., on the behavior of compression members (Ziemian, 2010). This work has resulted in conservative approaches to economical design of such members that are also very reliable.

While isolated failures of tension members have occurred in the past, the bridge engineering community is very reluctant to accept the decades of research and advancements that have been made in the understanding of fracture mechanics, the availability of steels with superior toughness, and advances in fabrication methods and NDT. Other industries, however, have not taken the same view. For example, the aircraft industry routinely designs components using state-of-the-practice fitness-for-service (FFS) principles in which in-service inspection strategies, inspection intervals, material selection, and design are all linked to ensure a target reliability against failure/fracture. Again, that industry is utilizing an integrated approach to fracture control that has resulted in a high level of reliability associated with air travel. The author believes that these methods are easily extended to bridge structures and, with relatively little effort, could be used to show that the fracture limit state can be treated like any other failure mode, as is routinely done for ultimate strength, buckling, etc.

As an example, consider a truss bridge in which all the connections are bolted and the lowest fatigue detail is Category B. Suppose also that the steel possesses toughness much greater than the current minimums required by ASTM A709, possibly as high as 100 ft-lb at the lowest anticipated service temperature (ASTM, 2017). Finally, assumed that during design, the members were all sized to ensure infinite fatigue life. Using basic FFS principles, it could be shown that if one were to assume some reasonable initial defect, sudden brittle fracture is less likely to occur than failure due to some other limit state, such as strength. Further, one could design the member such that the critical defect is of a size that could be reliably found during inspection. With reasonable estimates of the in-service stress range and number of cycles applied per day, one could estimate the time needed for a small initial defect to become critical. During design, it could be ensured that the time needed for such a defect to become critical is 20 to 30 years, or even more. If one were to apply a safety factor of 2, hands-on inspection would be required only every 10 to 15 years. These inspections would be based on engineering principles because the scope, depth, and interval would be based on FFS and not the calendar. These approaches are not new and have been well-vetted in other industries. Thus, it seems that applying them to certain bridge structures during design could be done to more effectively manage the inspection needs of the inventory and result in lower life-cycle costs.

SUMMARY

Over the past 40 years, the steel bridge industry has seen many improvements and changes in materials, analytical tools, design methods, and fabrication. However, the fundamental assumptions regarding the likelihood of failure and the associated consequences specifically associated with FCM have not kept up with these advancements. For example, there are basically two distinct families of FCM: those built to the modern FCP and those that are not. However, these are both treated the same regarding in-service inspection. Further, welded FCM are treated the same as those that are built up from multiple components mechanically fastened together, again with no delineation between bolted or riveted members. Finally, regardless of the overall configuration of the structure, failure of an FCM is presumed to probably result in collapse of a portion of or the entire bridge. As a result, the overall risk associated with an FCM is always high in that it is assumed they will very likely fail sometime during the life of the structure, and the consequence of the failure is high.

This paper has presented a number of significant advancements that have been made in recent decades that can be used to more rationally treat members traditionally classified as FCM and lower the perceived and actual risk associated with such members. Despite these advances and the overall excellent service record of members classified as FCM, the profession still holds an unsubstantiated perception associated with their performance. The reality is that steps that have been taken to reduce the likelihood of fracture, such as the introduction of the AASHTO/AWS FCP, have been shown to be highly effective. Other strategies can be used to show the consequence associated with a partial or complete member fracture, though highly unlikely, is also low. Lowering the likelihood of failure and/or consequence will reduce the risk associated with FCM. Using state-of-the-practice approaches—such as system analysis or exploiting internal redundancy combined with the advancements in design, fabrication, and material—the risk associated with brittle fracture can be as low or lower than failure due to other limit states. Further, explicitly considering the fracture limit state in such a way allows for the development of an integrated fracture control plan in which the inspection interval, scope, and capability of the inspector are linked to the tolerance of the member or structure as a function of time. Moving forward with an integrated fracture control plan would allow a more rational treatment of these members that is based on engineering principles rather than perception and feeling.

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