Implementation Guide for IRM Evaluation of Mechanically-Fastened Built-Up Steel Members

Jason B. Lloyd, PE, PhD
National Steel Bridge Alliance

Matthew H. Hebdon, PE, PhD
Virginia Tech

Robert J. Connor, PE, PhD
Purdue University

DECEMBER 2020

Smarter.
Stronger.
Steel.

Table of Contents

List of Figures

List of Tables

1 INTRODUCTION

This implementation guide is intended to provide an overview of background information, requirements for built-up members found in the AASHTO *Guide Specifications for Internal Redundancy of Mechanically-fastened Built-up Steel Member* (hereafter referred to as Guide Spec.), and example evaluations to help illustrate implementation. This document supplements the IRM (internally redundant member) Evaluator spreadsheets produced by the National Steel Bridge Alliance (NSBA), which are spreadsheets developed to facilitate the calculations involved in the evaluation of internal redundancy. The IRM Evaluator spreadsheets can be downloaded for free in the Design Resources section on the NSBA website, **aisc.org/steelbridges**.

1.1 Concepts of Redundancy

AASHTO (2017) defines a fracture-critical member (FCM) as a, "Component in tension whose failure is expected to result in the collapse of the bridge or the inability of the bridge to perform its function." In other words, an FCM is considered non-redundant. It must not have the ability to redistribute load around the failed member, or component, in order to continue to perform its function. A system redundant member is one whose failure does not result in the failure of the bridge. Simply put, it is a member that the bridge can safely operate without for some amount of time at a given load; or in other words at a required level of reliability. An internally redundant member is one whose failure of a single component does not result in complete failure of the member. In other words, it is a member that in the faulted condition can continue to carry service loads for a given amount of time. Redundancy in a bridge can exist in multiple forms, such as Load Path Redundancy, Structural Redundancy, and Internal Member Redundancy. These three forms of redundancy are defined as follows:

- *Load Path Redundancy* is when the bridge has multiple main supporting members, such as girders or trusses, meaning more than two such members.
- *Structural Redundancy* is provided by continuity of main members over interior supports or by other three-dimensional mechanisms born from secondary members providing lateral load redistribution.
- *Internal Member Redundancy* refers to a built-up member detailed using mechanical fasteners, such as bolts or rivets, which limit fracture propagation across the entire member cross section. This characteristic of mechanically-fastened built-up members has been termed, Cross-Boundary Fracture Resistance (CBFR) (Lloyd, 2018).

Complete member fracture is not a plausible failure mode for internally redundancy members because they are comprised of separate isolated components designed and detailed such that should any one of the components fail, the overall member still possesses sufficient strength to safely carry dead load and some portion of live load. This is has been referred to as "fail-safe" design or "damage tolerant design." Damage Tolerant Design (DTD) uses design approaches to create a structure that can sustain defects safely until repair can be made. It is based on the assumption that flaws exist in any structure and that these flaws will propagate over time before they can be detected. DTD is coupled with a maintenance program that will result in detection and repair of the damage before it reduces the capacity of the structure to an unacceptable limit. Such limits could refer to strength and fatigue, for example. These strategies are routinely employed in oil, aircraft and ship structures. Identifying internally redundant members (IRMs) and estimating rational inspection intervals for them is a DTD approach to the asset management of existing fastener built-up steel axial members within the nation's bridge inventory.

1.2 History of Fracture Critical Member Inspection Requirements

Currently, the inspection period for bridges containing FCMs in the United States is mandated to be a maximum of twenty-four months (23 CFR §650.311, 2017). This inspection frequency was first defined in the *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* in December 1995 (FHWA, 1995). The Federal-Aid Highway Act of 1968 (Pub. L. 90-495, 82 Stat. 815) originated a requirement for the Secretary of Transportation to establish the National Bridge Inspection Standards (NBIS) to ensure the safety of the nation's bridges. This legislation followed the well-documented collapse of the Point Pleasant Bridge in 1967. Later, the Federal-Aid Highway Act of 1970 (Pub. L. 91-605, 84 Stat. 1713) limited the NBIS to bridges on the Federal-Aid Highway System. However, the Surface Transportation Assistance Act of 1978 (Publ. L. 95-599, 92 Stat. 2689) extended the NBIS requirements to all bridges greater than 20 feet on public roads. Finally, the Surface Transportation and Uniform Relocation Assistance Act of 1987 (Pub. L. 100-17, 101 Stat. 132) expanded the scope of bridge inspection programs to include arms-length, or hands-on, inspection procedures for fracture critical members.

1.3 Historical Performance of Internally Redundant Members

In-service failures can be edifying for industry, showing areas where further understanding is required to design and build more resilient structures. There is only one known case of a bridge in service that has experienced failure of a single component of a built-up axially-loaded FCM that can be found in the literature; it is the Hastings Tied Arch Bridge discussed below. While there is certainly anecdotal evidence that is sometimes cited, none appear to be documented and reported in detail. This could mean that they rarely occur, or that they simply are not usually documented. Load shedding to adjacent components during fatigue cracking of built-up components observed by Hebdon et al. (2015) may also help explain why it could be rare. The lack of documented cases could also suggest that mechanically-fastened built-up members are performing better than industry tends to recognize. Connor et al. (2005) reported that since 1960, no bridge with built-up members classified as a fracture critical member is known to have failed due to the fracture of a single component propagating a fracture to an adjacent component.

1.3.1. Hastings Bridge, Minnesota

Two separate arms-length inspections revealed two partial member fractures on the Hastings Bridge in Minnesota in 1997 and 1998. The bridge was a tied-arch through truss bridge built in 1949 with riveted built-up members (see Figure 1-1). Both fractures occurred in the same plate of the same tie girder. While the first fracture arrested in adjacent rivet holes, the second fracture propagated through the entire tie girder web plate (see Figure 1-2). Initiating at a tack weld used to improve fabrication adjacent to a floor beam gusset plate, the second fracture ran the entire length of the web plate. Investigations later determined that a single plate used in the fabrication of the tie girder erroneously had extremely low toughness allowing each of these fractures to occur (Niemann, 1999). The internal member redundancy of the built-up tie girder prevented the fracture from propagating into adjacent components.

1.3.2. Milton-Madison Bridge, Indiana & Kentucky

A destructive field test was performed on the Milton-Madison Bridge prior to demolition and replacement. The original bridge was built in 1921 to carry US-421 across the Ohio River connecting Milton, Ky., with Madison, Ind. The bridge was comprised of 19 spans, including several deck truss approach spans on the Madison side (north end) and multiple through-truss main spans and approach spans on the Milton side (south end) (Diggelmann et al., 2013).

In April 2012, researchers at Purdue University instrumented the first deck truss approach span on the Indiana side of the bridge, seen in Figure 1-3. The center three panels of the span were then loaded with 145 kips of sand (approximately 2/3 of the original design live load). The center of the bottom chord was rigged with explosives and severed in two stages. The first blast severed one of the two built-up channels that made up the tension chord, as shown shortly after the explosion in Figure 1-4. The channel was severed by the blast about six feet from the gusset plate connection. Cutting just one (the interior) of the two built-up channels was performed to observe the member level redundancy of the built-up tension chord. The second blast minutes later severed the rest of the fracture critical tension chord. The span successfully redistributed loads, maintaining stability, with deflections that were unperceivable to the human eye.

1.4 Introduction to Probability of Detection

Probability of detection (POD) refers to the likelihood that an unknown defect can be identified by inspectors. This can relate to surface, as well as internal defects. Preliminary results for on-going POD studies of common steel bridge details being carried out at Purdue University suggest that visual detection of small surface breaking fatigue cracks may not be reliable. The implication of unreliable POD is that defects intended to be found may often be missed during routine and arm's length

Fig. 1-1. Hastings Bridge, Minnesota (Niemann, 1999).

Fig. 1-2. Fractured tie girder plate, Hastings Bridge (Niemann, 1999).

Fig. 1-3. Approach span of Milton Madison Bridge tested for redundancy.

Fig. 1-4. Partially severed FCM, Milton Madison Bridge approach span.

inspections. Efficacy of the inspection can be improved when the intent shifts to a reliably detectable defect that can be safely tolerated by the member until the time that it can be identified by inspectors and programmed for repair.

1.5 Summary of Research

Experimental research was conducted at Purdue University testing seven full-scale specimens in order to establish if mechanically-fastened built-up steel members subjected to axial loads were fracture critical or not. The experimental research concluded that due to cross-boundary fracture resistance (CBFR) of the mechanically-fastened built-up members, they are not fracture critical. Cross-boundary fracture resistance is defined as the capacity to resist complete cross-sectional fracture by arresting a running fracture at the boundary between components. Results from the full-scale fracture tests showed that this type of member is able to resist the running fracture propagation across the entire cross section, enabling arrest of the fracture and redistribution of the applied loads. Tests were performed at temperatures that placed the steel into single digit foot pound levels of Charpy V-notch impact values (representative of material toughness), demonstrating that CBFR is independent of toughness. This means that historic steels, which were produced before the fracture control plan (FCP) required a specified toughness, will also possess CBFR (Lloyd, 2018).

In addition to the experimental work, several simplified solutions were developed through a comprehensive finite element based parametric study that quantified the local and global behaviors of the built-up members following failure of a single component (Hebdon et al, 2015; Lloyd, 2018). The simplified solutions are briefly discussed in Section 2.3.

2 OVERVIEW OF THE INTERNAL REDUNDANCY EVALUATION METHOD

The Guide Spec. is broken down into several groups of similar types of built-up members; flexural, multi-component axial, and two-component axial. Flexural members includes girders and beams with at least one tension cover plate. Multi-component axial members are those composed of multiple plates and angles to form boxes and channels. Fundamentally, this means that if any one of the components failed, there would remain a minimum of two more components to carry load. Two-component axial members are just as they sound, being comprised of only two components, such as two rolled channels. Load redistribution within these groups of built-up members is characteristically different, and therefore they are differentiated in the Guide Spec.

Within the axial member section of the Guide Spec, it refers to "interior" and "exterior" plates, such as is illustrated in Figure 2-1. This becomes important for evaluation of multi-component axial members because the stress amplification factors for bending will differ based on the position of the hypothetically failed plate within the cross section of the member. Interior plates distribute their load into adjacent components on both sides of itself, effectively offsetting any localized bending amplifications that might occur during failure of exterior plates. Hence, for interior plate scenarios the bending factor, Ξ_B, is set equal to 1.0. This can be important when considering, for example, a thicker interior plate where larger loads are redistributed upon failure compared to a smaller exterior plate. It may be difficult to judge which case will govern the hypothetical faulted condition fatigue life calculation because although the larger interior plate would result in larger redistributed load and thereby a higher stress range, the failure of an exterior plate would cause localized bending stress amplifications that could result in larger stress ranges in adjacent components. However, if the exterior plate has the largest gross area or an area equal to other plates, then the hypothetical case where the exterior plate fails will always govern the fatigue stress calculation.

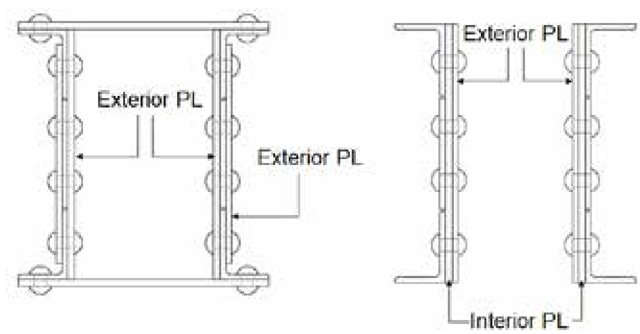


Fig. 2-1. Illustration showing examples of "interior" and "exterior" web plates.

The general requirements of the Guide Spec. are established to screen existing members to ensure that only lower risk members in good condition and with positive remaining fatigue life are allowed to be evaluated. The general requirements are also set up to help guide the design of new internally redundant members such that proportions of adjacent components are favorable toward strength and fatigue limit states of the faulted state evaluation. Two "states" are mentioned in the Guide Spec. The first is the "unfaulted" state referring to the condition of the member as-is, with no failed components. The other is the "faulted" state referring to the hypothetical conditions of the member in which the evaluating engineer assumes the failure of a single component within the member and performs the necessary checks against strength and fatigue. *It is emphasized that the "faulted" state is a hypothetical state assumed only for the purposes of evaluating internal redundancy. The Guide Spec. is not intended to justify leaving members with known failed components in place for long periods of time. These known faulted members should be repaired at the earliest possible time.*

2.1 General Requirements

General requirements of the Guide Spec. ensure that the member does not lack strength in the unfaulted state, meaning that the bridge cannot be load posted due to a strength deficiency. This generally would be a result of corrosion leading to section loss, but may be caused by other problems. Each member must also possess positive fatigue life in the unfaulted state, which is calculated using fatigue Category D for rivets and B for bolts. Additionally, recalculation of fatigue life using Mean Life, as permitted by the *Manual for Bridge Evaluation* (MBE) Article 7.2.7.2.1 is prohibited for the purpose of evaluating the member for internal redundancy to better ensure a very low likelihood of fatigue cracking is present. Using inputs from the user, the IRM Evaluator will calculate the fatigue life in the unfaulted state, as well as the faulted state.

2.2 Redundancy II Load Combination

National Cooperative Highway Research Program (NCHRP) Report 883 details research conducted at Purdue University wherein principles of reliability used in current design specifications, such as for LRFD load combinations, were used to develop new load combinations for the purpose of analyzing steel bridge members traditionally classified as fracture critical to determine if they meet the requirements of a System Redundant Member (SRM). The resulting load combinations are called Redundancy I and Redundancy II. As part of the research, the redundancy load cases were presented to and ratified by AASHTO Committee T-5 Loads and Load Distribution. Redundancy I characterizes the instant when failure of a primary steel tension member occurs and does not apply to the evaluation of built-up members for internal redundancy. Redundancy II was developed to characterize a prolonged period of service between the failure event and discovery of the failure (Connor et al., 2018). Redundancy II was adopted into the *Guide Specifications for Internal Redundancy of Mechanically-fastened Built-up Steel Members*.

Redundancy II requires the application of HL-93 live load design truck with a dynamic load allowance equal to 15%. Two sets of load factors were developed, one assigned to bridges built to the current Fracture Control Plan (FCP), per requirements for fracture critical member fabrication standardized in the AASHTO/AWS D1.5 *Bridge Welding Code*. The second set of load factors was developed for all other bridges, not built to the FCP standards, where likelihood of fracture occurring may be higher than for bridges fabricated to modern FCP standards. The IRM Evaluator spreadsheets automatically select the load factors based on user input of the year the bridge was built. Anything built after 1978 is assumed to have been built to the modern FCP according to the AASHTO/AWS D1.5 *Bridge Welding Code*. However, there may be many bridges that were built after 1978 during transitional years wherein states were transitioning to specifying the requirements of the modern FCP. Hence, the spreadsheets also have an FCP override selection. Thus, if the evaluating engineer knows that the bridge was built after 1978 and also knows that it was *not* built to the FCP, the user may select "Override" on the drop down menu and the spreadsheets will automatically update the load factors appropriate for bridges not build to the FCP. Figure 2-2 shows a screen shot of the FCP override option located under "Load and Resistance Factors (GS 1.7)".

2.3 Simplified Solutions

Extensive non-linear finite element-based parametric studies were performed to understand the load redistribution behavior of built-up steel members. After-fracture net and gross section stress equations were developed. These are simply the load divided by the respective after-fracture area multiplied by amplification factors accounting for localized bending and shear lag effects, for axial members. These equations are provided in article 2.2.1 of the Guide Spec. There are two primary failure scenarios that must be considered for multi-component axial members; failure of a web or flange plate, or failure of a connection angle. Due to the relatively large gross area of plates that can result in considerable load redistribution following failure, the plate failure scenario will typically govern the after-fracture stress calculations. However, there are certain circumstances where failure of the connection angle could produce the greatest after-fracture fatigue stress and thereby the shortest fatigue life. Guidance on when to check each potential failure scenario is included in the Guide Spec. articles 2.2.1 and 2.2.1.1. For flexural members, the after-fracture net and gross stress equations are simply the moment divided by the respective after-fracture section moduli multiplied by a cover plate adjustment factor accounting for shear lag effects. These equations are provided in articles 2.1.2 and 2.1.3 of the Guide Spec. The primary failure scenario is assumed fracture of the outer-most tension cover plate.

2.3.1 Axial Members: Failure of Web or Flange Plate

The IRM Multi-Component Evaluator automatically calculates the factored net and gross section stresses for the unfaulted and faulted states for each failure scenario, assuming individual cases where each component is assumed to fail. The user must manually enter the controlling net area for each component under the column titled, "Net (Str.)," for all components having staggered fastener patterns (using the $s^2/4g$ rule explained in AASHTO LRFD *Bridge Design Specifications* Section 6.8.3). Figure 2-3 shows an example where the user has input the net areas into the required blue cells. When staggered holes are present in the member, the spreadsheet will use the values under "Net (Str.)" to make the net section fracture and gross section yield strength checks. The spreadsheet uses values automatically calculated and tabulated under "Net (Fat.)" for all live load stress range calculations for fatigue life prediction. When fastener holes are not staggered, the spreadsheet uses the same net areas for strength and fatigue calculations.

While the shear lag factor was derived during finite element parametric studies as a constant value for all cases, the bending factor was found to be correlated to the number of plates within the member. The correlation was such that increasing the number of plates reduced the bending amplification. This is intuitive since by increasing the number of plates within a given cross-sectional area, the impact of any one of those plates failing is reduced. Hence, preferred detailing for new designs would be to fabricate with the largest number of plates feasible. Also, bending effects were found to be localized near the failure plane, meaning that gusset connections would not experience increased demands resulting from a failed component within multi-component members. Over short distances, typically within just a couple or three fastener spaces to each side of the failure plane, the redistribution of load has re-equilibrated and the full cross section of the member was again engaged in carrying load. This behavior resulted in localized bending effects centered on the failure location, which were resolved within the member without forming moment demands at the connections. The small moments generated were caused by the load being transferred out of the broken component creating an eccentricity relative to the component centroids, rather than the mem-

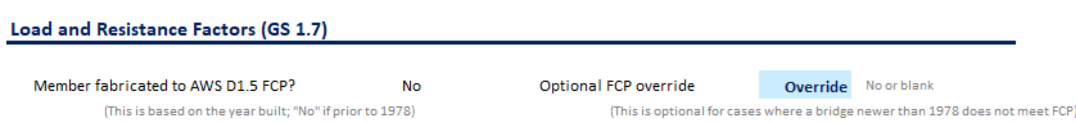


Fig. 2-2. Example showing the optional FCP override selection.

ber centroids. This concept may become clearer when considering the effects resulting from the position of the failed plate discussed in the beginning of Section 2, where it is explained that when a broken plate sheds load into adjacent components on both sides of it (an "interior" plate), the bending effect is effectively cancelled resulting in no stress amplification from bending. The bottom line is that this aspect of the analysis has been streamlined to a factor that inherently accounts for these affects and is simply a function of the number of plates, making the analysis all the more convenient for the evaluating engineer.

The multi-component bending stress amplification factor, Ξ_B, was developed using half of the member cross-section. The derived equations were found to work equally well with box sections with top and bottom web plates when the plate count was made on half of the cross-section. In other words, each of the web plates would be counted as ½ plate. A few examples are provided in Figure 2-4. Dashed lines indicate the portion of a generic member that contributes to the plate count. Full-depth as well as partial-depth plates are counted. Angle legs, lacing bars, and batten plates are not counted. Web plates with hand holes are counted. For example, the plate count, *N*, for Figure 2-4(c) is 4. This is attained by counting ½ for each of the top and bottom web plates (or 1 together), 1 for each of the two full-depth flange plates and 1 for the partial-depth flange plate that fills between the angle legs.

Component	Unfaulted Areas		
	Gross	Net (Fat.)	Net (Str.)
	A_g (in²)	A_n (in²)	A_n (in²)
PL-2f	0.00	[illegible]	
PL-1f	34.13	30.32	28.7
PL-0	34.13	30.32	28.7
Angles	16.82	15.66	14.8
Channel	0.00	[illegible]	
PL-1p	16.25	14.49	13.76
PL-2p	27.50	24.28	23.06
[illegible]	[illegible]	[illegible]	
WP-top	0.00	[illegible]	
[illegible]	[illegible]	[illegible]	
Half Symmetry Total	**128.82**	**115.05**	
Full Member Total	**257.64**	**230.10**	
Half Symmetry Total			**109.02**
Full Member Total			**218.04**

Manual entry of net areas for staggered hole patterns using $s^2/4g$ rule (AASHTO LRFD 6.8.3)

Fig. 2-3. Example showing manual entry of net areas for staggered fastener hole patterns.

Calculation of the after-fracture net and gross section stresses are performed using Guide Spec. Equations 2.2.1-1 and 2.2.1-2, respectively. Stress amplification factors for bending effects, Ξ_B, and shear lag effects, Ξ_{VL}, are provided in Table 2.2.1-1. The IRM Multi-Component Evaluator will automatically perform these calculations based on the user inputs of the member cross section, including counting the number of plates within the half-symmetry of the built-up member.

2.3.2 Axial Members: Failure of Connection Angle

There are rare cases when the connection angle failure scenario can control the after-fracture live load stress range, which is required to be checked in the Guide Spec. Article 2.2.1.1. The first case is when a built-up channel is comprised of only one web plate and the two angles attached to it have a combined gross area greater than or equal to 1.3 times the gross area of the web plate. This would be rather unusual proportions for typical built-up member construction. The second case is for any built-up I-section comprised of a single continuous flange plate and four connection angles. This is illustrated as Case 4 in Table 2.2.1-1 where it shows that the bending amplification factor is set equal to 1.0. The calculation of after-fracture live load stress range for the case of a connection angle failure has been nicknamed the "40-over-fastener" rule in the Guide Spec. due to the fact that finite element parametric studies showed that the maximum after-fracture net section stress could be reasonably estimated by adding the original net section stress (prior to hypothetical failure) to an after-fracture stress found by dividing 40% of the load in the assumed failed angle by the product of the thickness of the flange plate and fastener edge distance (taken from the center of the first fastener hole to the edge of the plate, perpendicular to the direction of primary force). Hence, 40% of the angle's load is distributed into the first fastener's edge distance.

The IRM Multi-Component Evaluator automatically performs these checks based on two user inputs. The first is a Yes-or-No question at the beginning of the sheet asking if the member is a built-up I-section. If the answer is "Yes," the sheet will perform the 40-over-fastener rule calculation. The second is based on angle and plate sizes of built-up channels and is only performed if the half-symmetry built-up channel only has one flange plate *and* meets the gross area criterion set forth in the Guide Spec.

The 40-over-fastener rule is only used to check the after-fracture net section life load stress range used for estimating remaining fatigue life of these details. It is not used for calculating after-fracture net or gross section stresses for the purpose of strength checks.

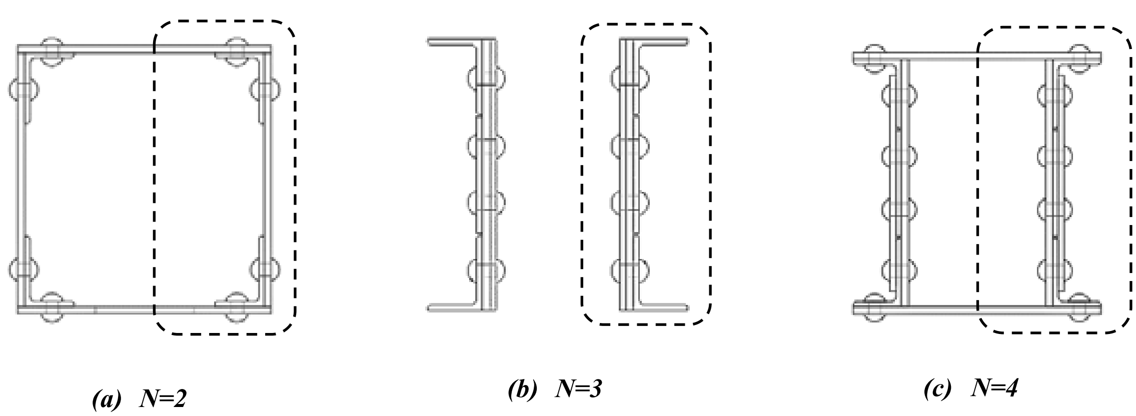


Fig. 2-4. Examples of plate counts, *N*, for bending factor, Ξ_B.

2.3.3 Flexural Members: Failure of Exterior Tension Cover Plate

The cover plate adjustment factor was found to be correlated to the number of plates within the member in the unfaulted state. The correlation was such that increasing the number of tension carrying cover plates in the cross section, increases the local stress amplification in the plate adjacent to the severed plate.

The IRM Flexural Evaluator automatically calculates the factored net and gross section stresses for the unfaulted and faulted states, assuming that the exterior (outermost) tension cover plate is the fractured component. This assumption is the most likely scenario for flexural members because the outermost cover plate would be subjected to the highest live load stress range, and therefore most likely to fatigue. Hence, the cover plate adjustment factor was developed based on the assumed failure of the exterior cover plate. However, it would be conservative to apply it to the case when an interior cover plate is assumed to have severed.

Calculation of the after-fracture net and gross section stresses are performed using Guide Spec. Equations 2.1.2-1 and 2.1.2-2, respectively, for noncomposite members. Equations 2.1.3-1 and 2.1.3-2 are used for composite sections. Local stress amplification resulting from shear lag effect is accounted for with the cover plate adjustment factor, β_{AF}, which is calculated using Guide Spec Equations 2.1.2-3 & 2.1.2-4. Note that the spreadsheet automatically uses the noncomposite section properties when evaluating a member in negative moment, even when the section is defined as composite by the user. Several warning notes appear on the sheet next to stress results when these inputs are selected. Thus, the deck reinforcement is conservatively ignored.

2.4 Fatigue Life Calculation

The IRM Evaluator spreadsheets automatically calculate the fatigue life for unfaulted and faulted states based on the method contained in the *Manual for Bridge Evaluation* (MBE), Section 7. They perform this calculation for the component failure scenario that results in the largest factored after-fracture stress range for axial members, and for the failure of the exterior tension cover plate for flexural members. The fatigue evaluation method found within the MBE was updated in 2018 to better account for the traffic growth rate over the life of the bridge. The IRM Evaluator spreadsheets use the current method when calculating the fatigue life.

There are two general cases that can result, Case I and Case II. Case I classifies members found to have infinite fatigue life in the unfaulted state. In addition, members found to have finite fatigue greater than 25 years are also classified as Case I members; See Guide Spec. Articles 2.5.3 and C2.5.3 for further information. Case I members are broken down into two sub-cases, namely Case Ia and Case Ib. Case Ia is for members having infinite fatigue life in the faulted state, or a finite life greater than 25 years. Case Ib is for members found to have finite fatigue life in the faulted state that is less than 25 years. Case II classifies members determined to have finite fatigue life in the unfaulted state. The IRM Evaluator spreadsheets automatically determine the Case Type for the unfaulted and faulted state. The results are listed within the "Fatigue Life Estimate" portion of the spreadsheets. The Special Inspection interval is calculated based on the estimated fatigue life in the faulted state, which is discussed in the following section.

2.5 Special Inspection Interval

Bridge inspection interval and intensity are regulated by the Code of Federal Regulations (CFR) where it states that routine inspections, as well as inspections of fracture critical members, are required at regular intervals not to exceed twenty-four months. These two types of inspection share a common maximum interval, but differ in intensity. The inspection of fracture critical members is more in-depth, requiring "hands-on," which is defined as "inspection within arms length of the component." However, if a mechanically-fastened built-up steel member is found to meet the criteria of the Guide Spec., then the FCM classification can be removed and it can be re-classified as an IRM. The inspection interval for an IRM is not explicitly defined in the CFR, however, per the Guide Spec., a "special inspection" can be used to establish the interval and intensity of inspections of IRMs. Special Inspections are defined in the CFR as "an inspection scheduled at the discretion of the bridge owner, used to monitor a particular known or suspected deficiency" (23 CFR § 650.305). Within the context of the Guide Spec., inspection of IRMs needs to occur at the frequency determined by the Guide Spec. and at a depth sufficient to reliably detect severed or fractured components. In order for an owner to meet the objectives of the special inspection of IRMs, it is understood that special access equipment may be required for some bridges, while for others it may be possible to detect broken components from the ground using normal visual inspection techniques. It is not intended that the special inspection become the only inspection performed on IRMs. Routine inspections would still be mandated by law and is considered good practice to inspect the bridge for other forms of damage and degradation, such as corrosion, deck spalling, etc. The intent is that the special inspection would replace the fracture critical inspection, set at a maximum interval defined by the Guide Spec., which would be fit into rotations with the routine inspections. For example, if the IRMs on a truss bridge were found to require a special inspection every 6 years, then the routine inspection could be performed every two years and then on the third interval at the six-year mark, the routine inspection would also include a special inspection of the IRMs. At this point the IRMs would be reevaluated based on outcomes of the inspection. For IRMs that possess infinite fatigue life in the unfaulted state, assuming no new damage is detected during the special inspection and loads have not changed, the special inspection interval would remain at 6 years. For IRMs that possess finite fatigue life in the unfaulted state, assuming no new damage is detected during the special inspection and loads have not changed, the special inspection interval would take into account the previous six years of fatigue life consumed between special inspection when estimating the new special inspection interval. This can be done using the spreadsheets by simply changing the input for *current year* and the new Special Inspection interval will be calculated.

Two tables are provided in the Guide Spec., one for Case I members (Table 3.1-1) and one for Case II members (Table 3.1-2). The maximum interval permitted in any case is ten years, while shorter intervals can be applied at the discretion of the owner. The maximum special inspection interval is found by dividing the fatigue life in the faulted condition by two, then rounding to the nearest even number. This is done to apply a very conservative interval as compared to the estimated fatigue life, putting it back to an even number so that it synchronizes with routine inspection rotations.

3 EXAMPLE INTERNAL REDUNDANCY EVALUATION: MULTI-COMPONENT AXIAL MEMBER

The following example is provided to illustrate implementation of the Guide Spec. in evaluating existing members and using the IRM Multi-Component Evaluator available for free download from the NSBA design resources webpage. A brief background for the bridge is provided, as well as detailed inputs and outputs from the IRM Multi-Component Evaluator. Two detailed calculations are provided, one that meets all IRM criteria of the Guide Spec. and one that does not. Results for the remaining members on the span will be summarized for final conclusions.

3.1 IRM Multi-Component Evaluator Overview

The following provides a basic introduction while offering some helpful insights into how to use the IRM Multi-Component Evaluator. If the user desires to use field-measured effective stress ranges to improve the fatigue life calculations, see Section 3.1.2.1.

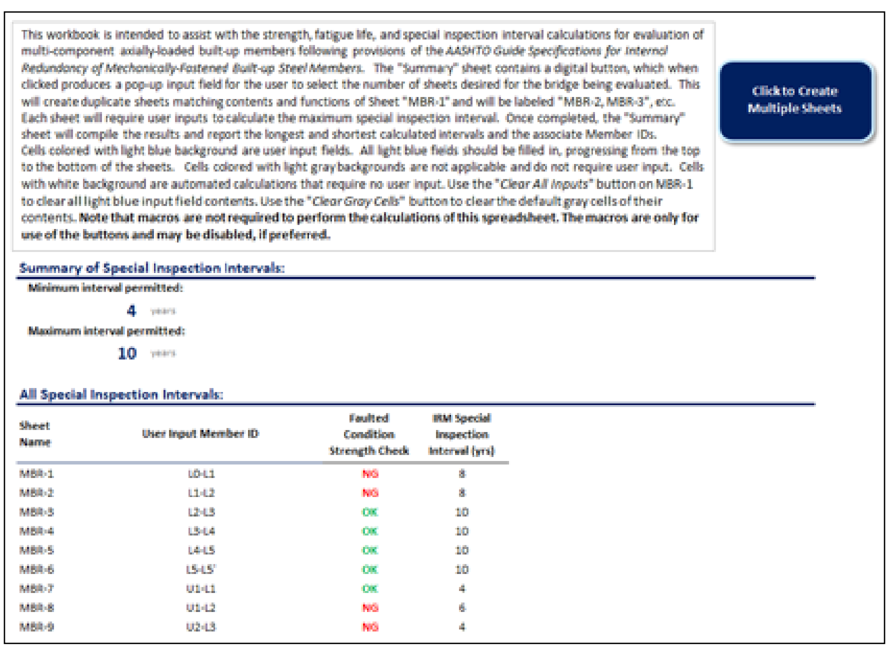
3.1.1 Summary Sheet

Figure 3-1 shows a screen shot of the Summary page taken from the IRM Multi-Component Evaluator. The Summary sheet is used to generate the number of evaluation sheets that will be required for the bridge being evaluated. It then combines the results synopsizing the strength check and special inspection interval results for each member. The Summary sheet lists the members using the names of the worksheet tabs, as well as the unique member identifiers entered into each respective worksheet by the user. Strength limit state results are reported using "NG" in bold red font for not meeting the faulted state strength limit requirements and "OK" in bold green font for passing the faulted state strength provisions of the Guide Spec.. These same results are compiled at the bottom of each of the individual worksheets, as well.

When the user opens the IRM Multi-Component Evaluator for the first time, there will be a single evaluation sheet called, "MBR-1." If the user needs to evaluate more than one multi-component member within the same bridge and wishes to store all of the evaluations in a single Excel file, then by clicking on the blue button entitled, "Click to Create Multiple Sheets" seen in Figure 3-1, a popup window will appear (shown in Figure 3-2). The user may enter the number of members desired for evaluation. Upon clicking "OK," the workbook will automatically generate exact duplicates of MBR-1, naming them numerically, MBR-2, MBR-3, and so on until reaching the number entered into the pop-up window. For example, Figure 3-2 shows the user has input "12," which will result in duplicates of MBR-1 numbered up to and including MBR-12.

User Tip: If several members will have similar components or component sizes, or other user inputs, it will save the user time to partially fill in MBR-1 with the information that is the same between members, before duplicating the sheet. Once duplicated, all information entered in MBR-1 will be propagated into the duplicated sheets and wouldn't need to be re-entered by the user.

The spreadsheet uses a macro in the background to run the sheet duplication function. If the user finds that the macro is not working on their computer, the user may simply right click on the "MBR-1" tab and select "Move or Copy," then check the box next to "Create a copy" at the bottom of the pop-up window to generate a single duplicate sheet. The user will then need to rename the duplicate sheet "MBR-2" so that the Summary page can recognize the new sheet and provide the summary information from it. See Section 3.1.2 on renaming the sheets to something other than "MBR-#." Note that the Summary sheet is preset to synopsize the results for up to 40 members. Should the user need to include more than 40 member evaluations within the same Excel file, then drag the columns to populate additional cells beyond 40. Keep in mind that this will go outside the printing area pre-defined in the sheet, so if the user desires to print a hard copy, the user should also adjust the printable area to include those members beyond the preset 40. Also note that the macro is not required for the spreadsheet to perform the calculations. Disabling the macros will still result in a functional spreadsheet that can be used. This macro simply provides a convenient way to produce multiple duplicates of the "MBR-#" sheet.


This workbook is intended to assist with the strength, fatigue life, and special inspection interval calculations for evaluation of multi-component axially-loaded built-up members following provisions of the *AASHTO Guide Specifications for Internal Redundancy of Mechanically-Fastened Built-up Steel Members*. The "Summary" sheet contains a digital button, which when clicked produces a pop-up input field for the user to select the number of sheets desired for the bridge being evaluated. This will create duplicate sheets matching contents and functions of Sheet "MBR-1" and will be labeled "MBR-2, MBR-3", etc. Each sheet will require user inputs to calculate the maximum special inspection interval. Once completed, the "Summary" sheet will compile the results and report the longest and shortest calculated intervals and the associate Member IDs. Cells colored with light blue background are user input fields. All light blue fields should be filled in, progressing from the top to the bottom of the sheets. Cells colored with light gray backgrounds are not applicable and do not require user input. Cells with white background are automated calculations that require no user input. Use the "*Clear All Inputs*" button on MBR-1 to clear all light blue input field contents. Use the "*Clear Gray Cells*" button to clear the default gray cells of their contents. **Note that macros are not required to perform the calculations of this spreadsheet. The macros are only for use of the buttons and may be disabled, if preferred.**

Click to Create Multiple Sheets

Summary of Special Inspection Intervals:

Minimum interval permitted:

4 years

Maximum interval permitted:

10 years

All Special Inspection Intervals:

Sheet Name	User Input Member ID	Faulted Condition Strength Check	IRM Special Inspection Interval (yrs)
MBR-1	L0-L1	NG	8
MBR-2	L1-L2	NG	8
MBR-3	L2-L3	OK	10
MBR-4	L3-L4	OK	10
MBR-5	L4-L5	OK	10
MBR-6	L5-L5'	OK	10
MBR-7	U1-L1	OK	4
MBR-8	U1-L2	NG	6
MBR-9	U2-L3	NG	4

Fig. 3-1. Screenshot showing the summary sheet within the IRM Multi-Component Evaluator.

3.1.2 MBR-# Sheets

The member sheets must be named "MBR-#," such as MBR-1, in order for the Summary sheet to find and synopsize the results. If the user desires to name the tabs differently for organizational purposes, the user will be required to unprotect the Summary sheet and modify the "Sheet Name" column to match the tab labeling scheme in order to retain the automated summary functionality.

The MBR-# sheets are the core of the IRM Multi-Component Evaluator where the user will input all parameters for the member being evaluated and where all area calculations, faulted condition strength checks, and fatigue life estimates are performed and compared against requirements of the IRM Guide Specifications. There are cells with three different background colors, blue, gray, or white. Blue cells are user input cells requiring some kind of answer or numerical value for the sheet to perform its functions. Ideally the blue cells are filled out progressing from the top to the bottom of the worksheet. This is because depending on the input for some blue cells, various cells below may turn from blue to gray, or vice versa. Gray cells are cells that do not require user input. Be advised that the gray cells are still functional, so if the user inputs values into gray cells it may affect calculations. Cells with white background are automated calculation fields that will provide programmed results based on blue cell inputs.

The MBR-# sheets also have two "clear" buttons. One is titled, "Clear All Inputs", which when clicked will remove the input values for all gray and blue cells (with the exception of the angle type). The other is titled, "Clear Gray Cells", which when clicked will remove the input values for all default gray cells. Default gray cells are those cells that are gray prior to filling out any of the sheet. Both buttons use a macro to function. Note that the macro is not required for the spreadsheet to perform the calculations. Disabling the macros will disable the "clear" buttons, but still result in a functional spreadsheet that can be used. This macro simply provides a convenient way to clear out inputs prior to initiating a new evaluation in the "MBR-#" sheets.

Each blue cell also contains a short message helping the user to better understand what is needed or in what format to input data into the respective cell. By simply clicking inside the blue input cells, the yellow message window will appear. By clicking into a different cell, the message window will disappear again. An example of the yellow message windows is shown in Figure 3-3 where it shows a reference to the Guide Spec. and additional guidance on what is needed to be entered into the blue input cell. In addition, some input cells will contain a dropdown menu. The menu may be used to click an option, or the same options may be manually typed into the cell. If an unacceptable or incorrect value is entered, the cell will provide a warning message, such as that shown in Figure 3-4. Simply click on "Cancel" and re-enter an acceptable value or choose one of the dropdown menu options.

Finally, there are a series of yes-no questions at the top of the MBR-# sheets. These questions are intended to help the user ensure that some screening criteria set forth in the Guide Spec. are checked before proceeding with the evaluation. One Guide Spec. screening criterion that is not explicitly asked at the top of the MBR-# sheet, however, is if there is positive remaining fatigue life in the unfaulted state for existing structures. It is required to possess positive fatigue life in the unfaulted state in order to meet the provisions of the Guide Spec. (See Guide Spec. Article 1.4). If this is known before evaluation begins, then the user is able to screen the member, if necessary. However, if the user is unsure what the remaining fatigue life is in the unfaulted state, the MBR-# sheet will perform this calculation based on user-input loads, at which point it can be checked and screened, if necessary.

3.1.2.1 Field-Measured Effective Live Load Stress Range

If field testing is performed to determine the effective live load stress range, rather than using calculated loads to do so, the user is pointed to Guide Spec. Article C2.5 wherein it provides guidance on how to back calculate an effective axial load by taking the field-measured effective stress range and multiplying by the net area of the member.

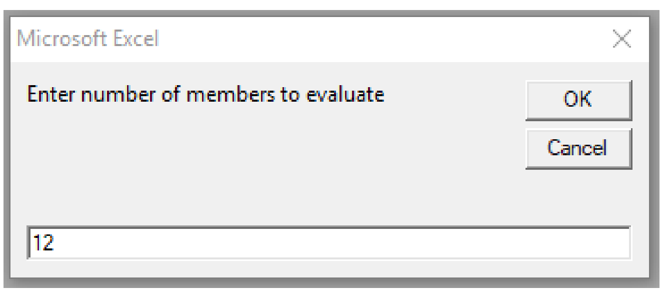


Fig. 3-2. Pop-up window that appears to enter the number of members to be evaluated.

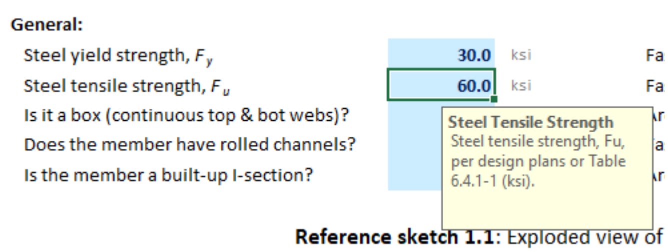


Fig. 3-3. Example showing yellow message windows that appear when clicking on input cells

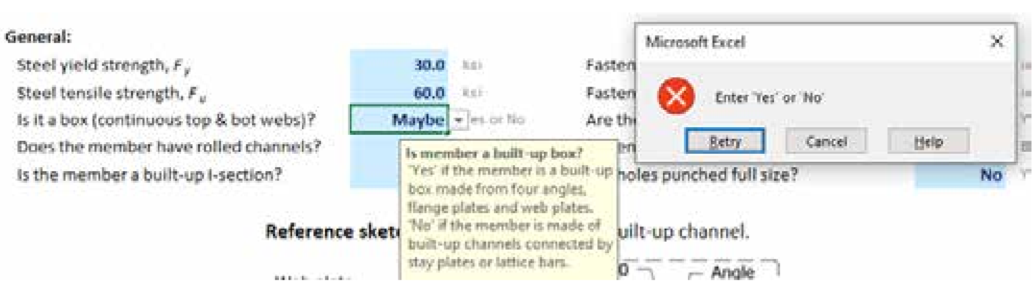


Fig. 3-4. Example warning message for incorrect value entered.

3.1.2.2 Reference Sketches

Four reference sketches are provided within the MBR-# worksheets to help guide the user in the nomenclature used to identify components and define their geometry. Three reference sketches show three types of multi-component built-up axial members; built-up channel from angles and plates (which may or may not have web plates), built-up channel from rolled angles and additional flange plates, and a built-up I-section (the web plate is a continuous plate, no lattice bars or intermittent battens). Geometry inputs are based on one half of the symmetric member for member types matching some variation of those depicted in Reference Sketch 1.1 and 1.2. Geometry inputs are based on the entire member for member types matching a variation of Reference Sketch 1.3. Nomenclature for the plates includes the letters "PL" for plate, followed by a dash and then a number and letter, either "f" or "p." The "f" stands for full depth, such as is typical for flange plates set to the inside of the connection angles. The "p" stands for partial depth, such as is typical for flange plates set to the outside of the connection angles. All member types will have a plate PL-0. If a member is similar to that depicted in Reference Sketch 1.2, except that it is only comprised of two rolled channels (no additional flange plates), then this member cannot be evaluated using the IRM Multi-Component Evaluator. That type of member must be evaluated using the IRM Two-Channel Evaluator, which is currently under development and pending final proposed guide specifications additions by AASHTO Committee T-14.

Fig. 3-5. Photo of the original bridge built in 1917 (City of Portland Archives, 1917).

Note that the worksheet does not evaluate the hypothetical case wherein plate PL-1p fails. During parametric studies conducted as part of the research used to develop the Guide Spec., there was never a case in which failure of plate PL-1p controlled the strength or fatigue limit states, unless that plate was made to be much thicker than other plates within the member. This is simply due to the fact that this plate fits between the connection angle legs reducing its depth, and thereby reducing the amount of load it carries and the effect it has on the member if it were to fail.

The fourth reference sketch is shows a cross section and elevation view of a built-up I-section that has a staggered fastener hole pattern along the top connection angle and a non-staggered fastener spacing along the bottom. This sketch is intended to illustrate how to measure (or calculate) the first fastener hole dimension required in the spreadsheet. For the case of staggered holes, the dimension is taken perpendicular to primary stress from the edge of the plate to the center of the first fastener hole. If the member is not symmetric about the longitudinal axis, then the user should input the smallest of these dimensions.

3.2 Background for Example Bridge

The Pacific Highway Interstate Bridge carrying I-5 traffic across the Columbia River between Portland, Ore., and Vancouver, Wash., includes two nearly identical structures. The current northbound structure is the original bridge built in 1917, as shown in Figure 3-5 (view looking southbound). The bridge is 3,500 feet long with 14 original spans (16 spans currently), including a 275-ft. lift span. Although it could not be confirmed in the available design drawings, based on the year it was built the original bridge was most likely constructed from A7 steel. However, for the purposes of this example, values taken from AASHTO *Manual for Bridge Evaluation* Table 6A.6.2.1-1 will be used.

The original elevation view of the 1917 structure is shown in Figure 3-6. It was riveted built-up construction using $7/8$-inch diameter rivets placed in reamed rivet holes ($15/16$-inch diameter). In 1958 the southbound structure was opened to traffic. The more modern structure was very similar to the original; however, it was built from A36 steel using high-strength bolted built-up construction. In addition, during construction of the southbound structure, the original (northbound) structure was modified to include a rise and fall in the elevation to allow for improved marine traffic. This involved, among other improvements, replacing Spans 5 and 6 with a single through-truss span that is approximately 529 feet long, replacing span 14 with two plate girder approach spans, and

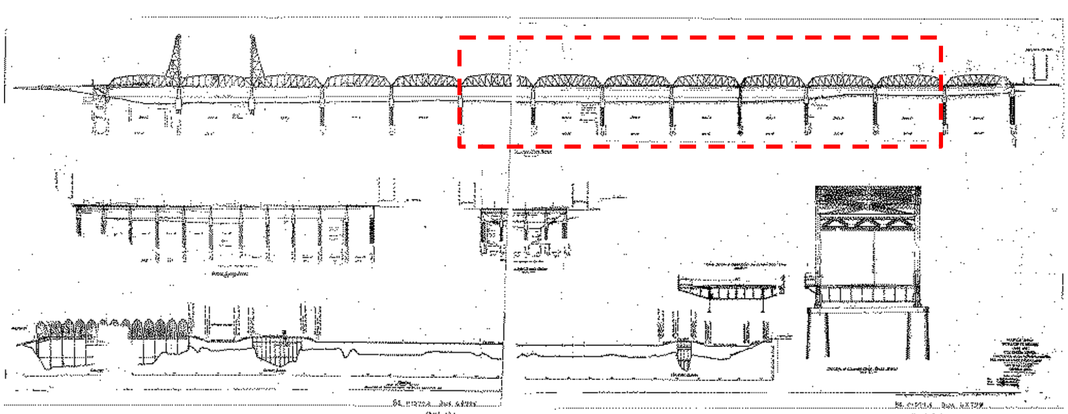


Fig. 3-6. Original design drawings for the 1917 spans (obtained from Oregon DOT).

increasing the height of several piers while retaining seven out of ten of the original 261-ft through-truss spans.

Figure 3-6 shows the seven spans from the original structure were retained during the modification in the mid-1950s inside a dashed red line. These spans are each of identical length and member sizes, making them ideal candidates for quick evaluation using "families" of identical members and loads. Figure 3-7 shows the original design drawings for the half symmetry of the seven spans from which member sizes were obtained for this example. Details for the member components will be provided throughout the example, as needed. Only members that carry a net tensile live load that exceeds any compressive dead load are required to be evaluated. These have been highlighted in Figure 3-7. Yellow highlights indicate a member that requires evaluation, which will be included in the current example. Red highlights indicate a member that requires evaluation; however, due to ongoing proposed additions to the Guide Spec. that have not yet been finalized for these member types (two-channel members), they cannot yet be evaluated in this example. Once the Guide Spec. is updated to include angle-only and two-channel members, the example below will be updated accordingly with all red highlighted members also being evaluated.

Additionally, due to the symmetry of the through-trusses, and having confirmed that member conditions are similar among all members within this span and for all seven like-spans, only a quarter of the tensile load carrying members will need to be evaluated. This is because member geometry and loading are identical for each respective member within the seven spans. If there were member conditions or retrofits and repairs, etc., that had changed strength or fatigue resistance of any of the members to be included in the evaluations, these would need to be individually evaluated rather than as "families" of similar members.

3.3 Load, Member Geometry, and ADTT Data

The bridge was built in 1917 before the AASHTO/AWS D1.5 Fracture Control Plan (FCP). This will affect the load factors required to be used, as seen in the Guide Spec. Table 1.7.1-1. The yield and tensile strength of the materials are not known. These could be tested, if desired. For this example, AASHTO *Manual for Bridge Evaluation* Table 6A.6.2.1-1 will be used, where it states that for year of construction between 1905 and 1936, use 30 ksi and 60 ksi for the minimum yield and tensile capacity, respectively.

Table 3-1. **Member Loads**

Member ID	PDC (kip)	PDW (kip)	P_{LL+IM} (kip)	P_{FAT+IM} (kip)
L0-L1	400	0	156	39
L1-L2	400	0	156	39
L2-L3	640	0	247	62
L3-L4	788	0	302	76
L4-L5	876	0	334	84
L5-L5′	900	0	340	85
U1-L1	90	0	107	27
U1-L2	412	0	173	43
U2-L3	277	0	138	35

The bridge carries three lanes of traffic with an estimated ADT of 60,000 vehicles per day. Applying factors taken from AASHTO LRFD Table 3.6.1.4.2-1 and C3.6.1.4.2-1, this yields an estimated $(ADTT)_{SL}$ of 7,200 trucks per day (60,000 × 0.15 × 0.80). According to AASHTO *LRFD Bridge Design Specifications* commentary C3.6.1.4.2, research indicates that ADT for a single lane is physically limited to 20,000 vehicles per day. This suggests that the bridge may have reached its physical ADT limit. Hence, $(ADTT)_{LIMIT}$ will be taken as 7,200 trucks per day, as well, with an expected annual $(ADTT)_{SL}$ growth rate, g, of 0.0% *(Note: This must be entered into the MBR-# worksheet, cell F77, as something slightly higher than zero, e.g. 0.00001, in order for the cell function to not yield a division by zero error, #DIV/0!)*

Table 3-1 lists the design loads for each of the members to be evaluated. The loads were taken from current load rating calculations, which is preferred for evaluation, including any special permit vehicle load cases that would be necessary for the structure. Table 3-2 summarizes the component sizes for each of the members, as well as the gross and net areas for each component and the member half-symmetry areas. All areas are for the unfaulted state of the member; they are automatically calculated by the spreadsheet, but are provided in the table for reference. Net area calculations take into account the number of fastener holes within a single cross section, as entered into the worksheet by the user. An example of this is demonstrated below for mem-

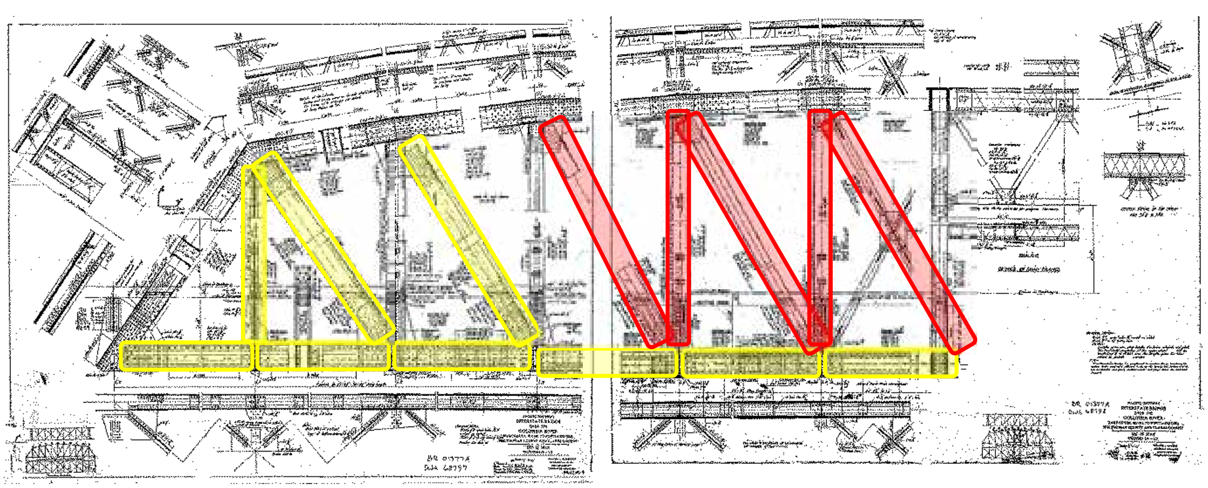


Fig. 3-7. Original design drawings for the identical spans (obtained from Oregon DOT).

Table 3-2. **Component Sizes and Half-Symmetry Areas for the Unfaulted State**

Member ID	Angles Area (in.²)	PL-0 Area (in.²)	PL-1f Area (in.²)	PL-1p Area (in.²)	Half Member Area (in.²)
	Gross | Net	Gross | Net	Gross | Net	Gross | Net	Gross | Net
L0-L1	4×3½×½″	24×7⁄16″	–	–	–
	7.0 | 6.53	10.5 | 9.68	–	–	17.5 | 16.21
L1-L2	4×3½×½″	24×7⁄16″	–	–	–
	7.0 | 6.53	10.5 | 9.68	–	–	17.5 | 16.21
L2-L3	4×3½×½″	24×7⁄16″	24×7⁄16″	15½×7⁄16″	–
	7.0 | 6.53	10.5 | 9.68	10.5 | 9.68	6.78 | 5.96	34.78 | 31.85
L3-L4	4×3½×½″	24×7⁄16″	24×7⁄16″	15½×7⁄16″	–
	7.0 | 6.53	10.5 | 9.68	10.5 | 9.68	6.78 | 5.96	34.78 | 31.85
L4-L5	4×3½×½″	24×7⁄16″	24×7⁄16″	15½×⅝″	–
	7.0 | 6.53	10.5 | 9.68	10.5 | 9.68	9.69 | 8.52	37.69 | 34.41
L5-L5′	4×3½×½″	24×7⁄16″	24×7⁄16″	15½×⅝″	–
	7.0 | 6.53	10.5 | 9.68	10.5 | 9.68	9.69 | 8.52	37.69 | 34.41
U1-L1	4×3×⅜″	14×5⁄16″	–	–	–
	4.98 | 7.63	4.38 | 3.79	–	–	9.36 | 8.42
U1-L2	4×4×7⁄16″	24×½″	–	–	–
	6.60 | 5.78	12.0 | 11.06	–	–	18.60 | 17.25
U2-L3	3½×3½×⅜″	20×7⁄16″	–	–	–
	5.0 | 4.30	8.75 | 7.93	–	–	13.75 | 12.58

ber L0-L1. Because the failures are hypothetical, the engineer must consider the worst case scenario for resulting net areas. Therefore, the user must consider which cross section would have the greatest number of fastener holes, yielding the smallest net area, which should then be entered into the worksheet.

Figure 3-8 shows two members whose evaluation will be details for this example and results for the remainder of the members listed in Table 3-2 are summarized below. The first member to be looked at in detail is member L0-L1. This member is shown on the left in Figure 3-8 where it can be seen that there are two flange plates, four connection angles, and stay plates intermittently spaced along the longitudinal axis (shown in light gray, which are not to be confused as flange plates). The second member to be detailed is U1-L1, shown on the right in the figure. This member is a built-up I-section hanger made from a single flange plate and four connection angles.

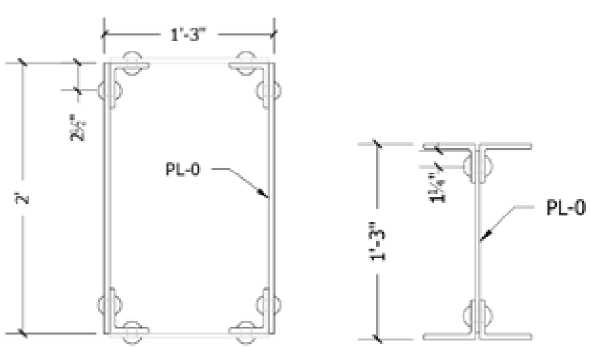


Fig. 3-8. Cross sections for members L0-L1 (left; showing stay plates) and U1-L1 (right).

3.4 Evaluation of Member L0-L1

Figure 3-9 shows a screenshot taken from the evaluation worksheet for member L0-L1 of the example bridge. The user entered that the member does not have continuous web plates (meaning that it is not a built-up box member), which resulted in the web plate dimension data entry cells turning gray, meaning no data entry is required. In addition, the user has entered that there is one full-depth flange plate and no partial-depth flange plates. Thus, only two blue data entry cells are required to be filled out, namely the plate depth and plate thickness for component PL-0. These are equal to 24 inches and 7⁄16 inches, matching information provided in Table 3-2. Figure 3-9 also shows the dimensions input for first fastener hole locations. Fastener hole data required for the web/flange plate includes the number of holes within a single cross section, as well as the distance from the top (or bottom) edge of the plate to the middle of the fastener hole. This information is used in the case that a 40-over-fastener rule calculation is required (for more information about the 40-over-fastener rule, see Section 2.3.2).

Figure 3-10 shows a screenshot of the data entered for the connection angle sizes and fastener holes for the connection angles of member L0-L1. The figure also shows fields for rolled channels, which have been grayed out due to user answering that the member contains no rolled channels. Connection angle sizes can be selected from the dropdown menu that appears when the user clicks into the blue cell under “Angle Dimensions.” Gross area and angle thickness properties are automatically brought in based on the user selection. Fastener hole data required for the angles is simply the number of holes per leg per cross section. The engineer should consider locations where a stay, or batten, plate is tied into the horizontal angle leg such that the smallest net area of the angle is captured in the calculation. Typically fasteners are staggered to avoid reducing the net area, but this is not always the case. For member L0-L1, stay plates were positioned such that the three rivets connecting them to the horizontal angle legs were staggered with the rivets in the vertical angle leg, thus "0" is entered into the blue cell for "Holes in Horz Angle Leg, $N_{holes.hz.agl}$" as shown in Figure 3-10.

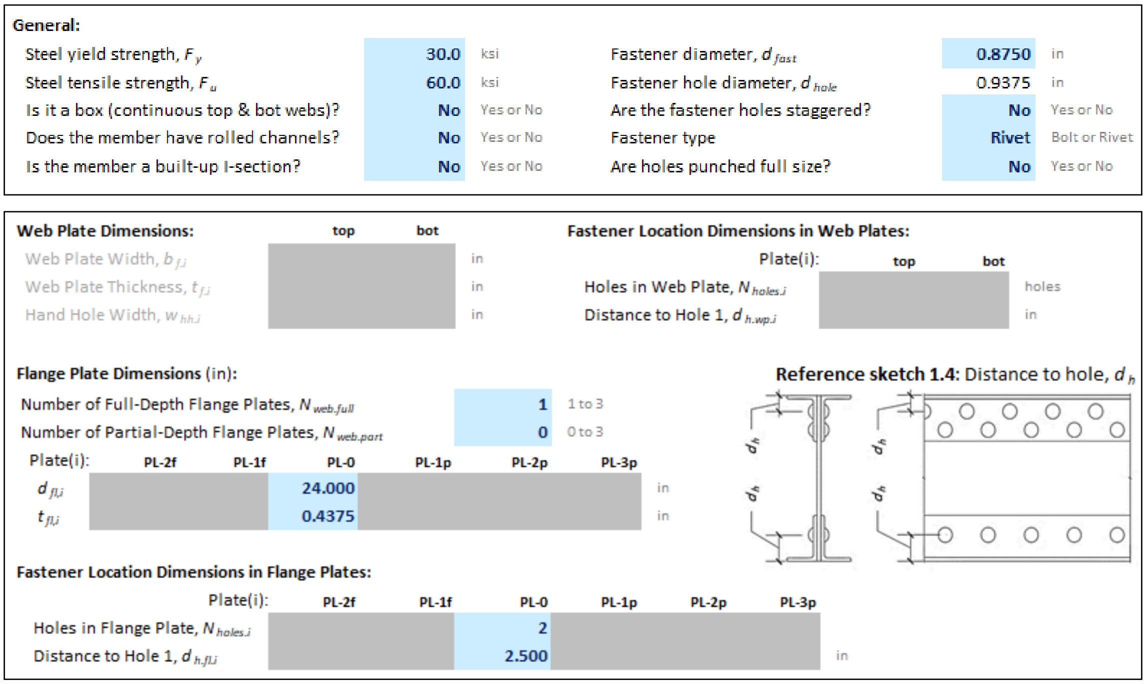


Fig. 3-9. Screenshot showing entered values for the example bridge, member L0-L1.

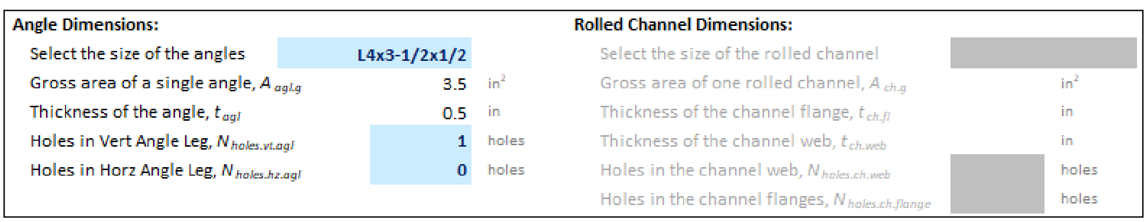


Fig. 3-10. Screenshot showing entered values for the angle sizes and fastener holes for member L0-L1.

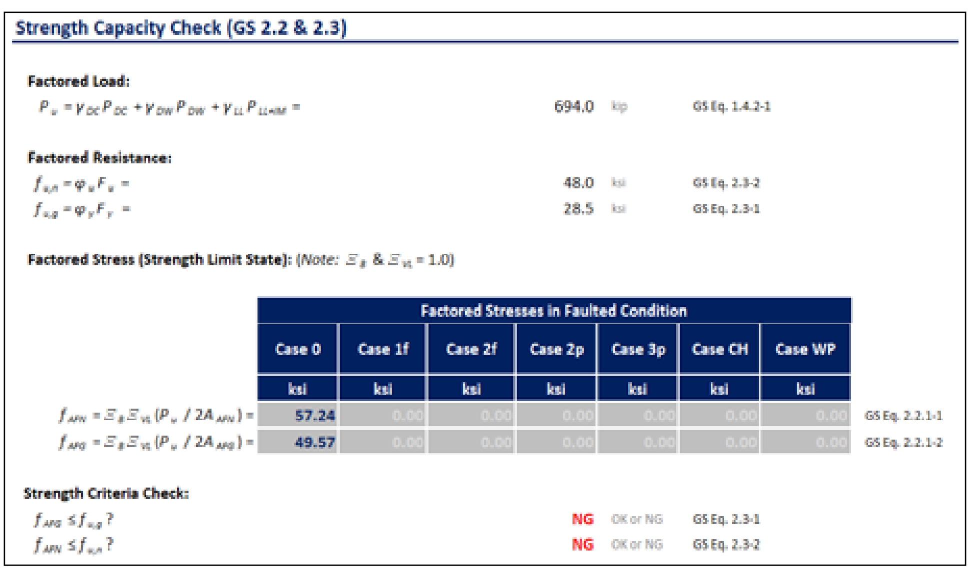


Fig. 3-11. Screenshot showing results of strength checks for member L0-L1.

3.4.1 Member L0-L1 Area Calculations and Strength Checks

Gross and net area calculations are automatically performed once all member geometric data is entered. Areas of the fasteners holes are also broken out and presented to the user. The MBR-# worksheet calculates "Half Symmetry Total" areas as well as the "Full Member Total" areas. Half symmetry net and gross areas are used for the strength and fatigue limit calculations and the full member totals are provided as an optional check against any existing known areas if desired, such as what might exist on original design drawings.

Stress amplification factors are automatically calculated for the user, as well as a check to determine if the 40-over-fastener rule is required for the member. Next, the axial load is factored and compared to the factored resistance of the member in the faulted condition. This is done for both net section fracture and gross section yield for each possible failure scenario. Results for these calculations are tabulated within the worksheet. Calculations for member L0-L1 are shown in Figure 3-11. The Redundancy II factored load was 694 kips. Keeping in mind that for the strength checks, the stress amplification factors are made equal to 1.0, the table in Figure 3-11 shows the possible failure scenario, Case 0, or failure of the web plate, PL-0. As can be seen in the figure, the member does not pass the net section fracture check. It also fails the gross section yield check. It is possible that material testing in the field could result in higher yield and tensile strengths that might help a member pass strength requirements. For the case where a member is very close to passing, it may be worth the cost and effort to perform field testing. However, in this case, due the small size of the connection angles, the yield strength would need to be nearly 50 ksi before passing the gross section yield limit state, which is most likely not the actual material yield strength. If a member is close to passing the strength checks and the engineer believes that material testing could help it to pass, it would be up to the owner/engineer to decide if pursuing the more accurate material property data is worth the cost and effort. If other members on the same bridge do meet all provisions of the Guide Spec. and can be reclassified as IRMs, then it would be recommended to obtain the actual material properties and improve accuracy of the calculations.

Thus, for the example bridge member L0-L1, "NG" is shown in bold red font at the bottom of Figure 3-11 indicating that the member does not pass faulted condition gross section yield and net section fracture. This means that member L0-L1 does not meet provisions of the Guide Specifications and therefore cannot be reclassified as an IRM.

3.5 Evaluation of Member U1-L1

The next member evaluation to be considered in detail in this example is for the hanger U1-L1. The cross-section for this member is sketched on the right side of Figure 3-8. It is a built-up I-section composed of a single continuous web plate and four

General:

Steel yield strength, F_y	30.0	ksi	Fastener diameter, d_{fast}	0.8750	in
Steel tensile strength, F_u	60.0	ksi	Fastener hole diameter, d_{hole}	0.9375	in
Is it a box (continuous top & bot webs)?	No	Yes or No	Are the fastener holes staggered?	No	Yes or No
Does the member have rolled channels?	No	Yes or No	Fastener type	Rivet	Bolt or Rivet
Is the member a built-up I-section?	Yes	Yes or No	Are holes punched full size?	No	Yes or No

Web Plate Dimensions:

	top	bot	
Web Plate Width, $b_{f.i}$			in
Web Plate Thickness, $t_{f.i}$			in
Hand Hole Width, $w_{hh.i}$			in

Fastener Location Dimensions in Web Plates:

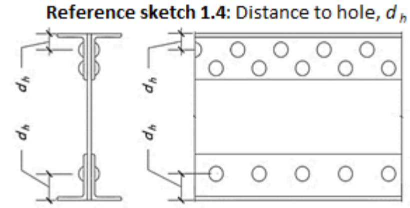
Plate(i):	top	bot	
Holes in Web Plate, $N_{holes.i}$			holes
Distance to Hole 1, $d_{h.wp.i}$			in

Flange Plate Dimensions (in):

Number of Full-Depth Flange Plates, $N_{web.full}$	1	1 to 3
Number of Partial-Depth Flange Plates, $N_{web.part}$		0 to 3

Plate(i):	PL-2f	PL-1f	PL-0	PL-1p	PL-2p	PL-3p	
$d_{fl.i}$			14.000				in
$t_{fl.i}$			0.3125				in

Reference sketch 1.4: Distance to hole, d_h



Fastener Location Dimensions in Flange Plates:

Plate(i):	PL-2f	PL-1f	PL-0	PL-1p	PL-2p	PL-3p	
Holes in Flange Plate, $N_{holes.i}$			2				
Distance to Hole 1, $d_{h.fl.i}$			1.250				in

Fig. 3-12. Screenshot showing entered values for the example bridge, member U1-L1.

Angle Dimensions:

Select the size of the angles	L4x3x3/8	
Gross area of a single angle, $A_{agl.g}$	2.49	in^2
Thickness of the angle, t_{agl}	0.375	in
Holes in Vert Angle Leg, $N_{holes.vt.agl}$	1	holes
Holes in Horz Angle Leg, $N_{holes.hz.agl}$	0	holes

Rolled Channel Dimensions:

Select the size of the rolled channel		
Gross area of one rolled channel, $A_{ch.g}$		in^2
Thickness of the channel flange, $t_{ch.fl}$		in
Thickness of the channel web, $t_{ch.web}$		in
Holes in the channel web, $N_{holes.ch.web}$		holes
Holes in the channel flanges, $N_{holes.ch.flange}$		holes

Fig. 3-13. Screenshot showing entered values for the angle sizes and fastener holes for member U1-L1.

connection angles. Initial questions and web plate dimensions are shown in Figure 3-12. Notice that the question at the upper right corner of the figure asks if the member is a built-up I-section. The purpose of this question is to determine if the 40-over-fastener rule must be enforced. A single full-depth web plate has been entered with zero partial-depth plates. Dimensions for the web plate are 14-inch depth by 5⁄16-inch thickness.

Rivet hole locations have been defined for the plate, as can be seen in Figure 3-12. There are two holes within the same cross-section of the plate and the distance from the top (or bottom) edge of the plate to the center of the rivet hole is 1¼," which is shown in the sketch in Figure 3-8. Figure 3-13 shows that the connection angles are 4×3×5⁄8" angles, which have been selected in the dropdown menu. And once again, the connection angles have a single rivet hole at each cross section. The rolled channel dimensions are grayed out due to the fact that the user answered "No" to the question at the top of the worksheet regarding rolled channels in the member. In the case where a member does possess rolled channels and cover plates (or additional flange plates), the user would answer "Yes" and these cells would become light blue for data entry. A dropdown menu is provided in this case, to select the rolled channel section, similar to that provided for the angles.

3.5.1 Member U1-L1 Area Calculations and Strength Checks

Calculations for member U1-L1 are shown in Figure 3-14. The Redundancy II factored load was 264 kips. Once again, keeping in mind that for the strength checks, the stress amplification factors are made equal to 1.0, the table in Figure 3-14 shows the only possible failure scenario for strength, Case 0, which is failure of the web plate, PL-0. As can be seen in the figure, the member passes both gross and net section checks.

Thus, for the example bridge member U1-L1, two bold green "Ok's" are shown at the bottom of Figure 3-14 indicating that the member does pass faulted condition strength requirements of the Guide Spec. This means that member U1-L1 may continue to fatigue life calculations to possibly become reclassified as an IRM and set the minimum special inspection interval.

3.5.2 Member U1-L1 Fatigue Life in the Faulted State

Figure 3-15 shows a screenshot taken from the evaluation worksheet for member U1-L1 of the example bridge. Notice that the stress amplification factors have been calculated, as well as the number of plates, N_{AX}. For this member, the bending factor, Ξ_B, is equal to 1.0 since it is a built-up I-section (See Guide Spec. Table 2.2.1-1, Case 4). The figure also shows the check for the 40-over-fastener rule, where the response "Yes" indicates to the user that this check is required for the member and will be taken into account when computing the fatigue life in the faulted state.

The MBR-# worksheet performs the Fatigue I (infinite life) and Fatigue II (finite life) calculations, each in the unfaulted condition as well as in the faulted state, calculating the largest resulting live load stress range from all possible failure scenarios. The worksheet uses the fatigue life calculation method that was adopted into the AASHTO *Manual for Bridge Evaluation (MBE)*

Strength Capacity Check (GS 2.2 & 2.3)

Factored Load:

$P_u = \gamma_{DC}P_{DC} + \gamma_{DW}P_{DW} + \gamma_{LL}P_{LL+IM} =$ 264.0 kip GS Eq. 1.4.2-1

Factored Resistance:

$f_{u,n} = \varphi_u F_u =$ 48.0 ksi GS Eq. 2.3-2

$f_{u,g} = \varphi_y F_y =$ 28.5 ksi GS Eq. 2.3-1

Factored Stress (Strength Limit State): (*Note:* Ξ_B & Ξ_{VL} = 1.0)

	Factored Stresses in Faulted Condition							
	Case 0	Case 1f	Case 2f	Case 2p	Case 3p	Case CH	Case WP	
	ksi	ksi	ksi	ksi	ksi	ksi	ksi	
$f_{AFN} = \Xi_B \Xi_{VL}(P_u / 2A_{AFN}) =$	15.43	0.00	0.00	0.00	0.00	0.00	0.00	GS Eq. 2.2.1-1
$f_{AFG} = \Xi_B \Xi_{VL}(P_u / 2A_{AFG}) =$	13.25	0.00	0.00	0.00	0.00	0.00	0.00	GS Eq. 2.2.1-2

Strength Criteria Check:

$f_{AFG} \le f_{u,g}$? **OK** OK or NG GS Eq. 2.3-1

$f_{AFN} \le f_{u,n}$? **OK** OK or NG GS Eq. 2.3-2

Fig. 3-14. Screenshot showing results of strength checks for member U1-L1.

Stress Amplification Factors (GS 2.2)

Stress Amplifiers (GS Table 2.2.1-1):

Shear Lag Factor, Ξ_{VL}	1.20
Number of web plates, N_{web}	1
Number of flangeplates, N_{fl}	0
Number of total plates, N_{AX}	1
Bending Factor, Ξ_B	1.00

***40-over-fastener Rule* Required?** (GS 2.2.1.1): Yes

Fig. 3-15. Screenshot showing the automatically computed amplification factors.

by AASHTO in 2018. Hence, the user is required to enter an expected annual $(ADTT)_{SL}$ growth rate, g. As mentioned above, if the desired growth rate is zero, the user must enter some number just larger than zero, such as 0.00001, in order to avoid the division by zero error in Excel. The user is also required to enter the resistance factor, R_R, as defined in the *Manual for Bridge Evaluation* Table 7.2.5.2-1. A dropdown menu of options is provided for convenience or the same numbers can be manually typed into the cell. For member U1-L1, the unfaulted state effective stress range was found to be 2.28 ksi. The maximum stress range, $(\Delta f)_{max}$, used for the infinite life check was 5.01 ksi (i.e., $\gamma_{FATI}/\gamma_{FATII}(\Delta f)_{eff} = 1.75/0.8 \times 2.28 = 2.2 \times 2.28$ ksi).

Fatigue categories for riveted and bolted details have been defined in the Guide Spec. Table 2.5-2 for the unfaulted and faulted states. In the unfaulted state, a riveted detail is classified as AASHTO Category D with a constant amplitude fatigue threshold (CAFT) equal to 7 ksi. Since 7 ksi is greater than 5.01 ksi, this member possesses infinite fatigue life in the unfaulted state (Case I member, see Guide Spec. Article 2.5). Recall that positive remaining fatigue life in the unfaulted state is a screening criterion for IRMs. Thus, member U1-L1 meets this criterion and may continue on in the evaluation process.

Fatigue life in the faulted state is checked next. Figure 3-16 shows a screenshot from the worksheet for member U1-L1 where it can be seen that the amplified stress range resulting from failure of the web plate, Case 0, is approximately 3.79 ksi. Recall that the 40-over-fastener rule is also required for this member. Therefore, below the table it can be seen that the fatigue + dynamic allowance load in a single angle was found to be 4.7 kips, and using equation 2.2.1.1-1 from Guide Spec. Article 2.2.1.1, this results in a stress range of about 6.99 ksi. Since 6.99 ksi is greater than 3.79 ksi, the controlling failure scenario is failure of a single connection angle.

At this point the worksheet calculates the fatigue life in the faulted state for the controlling scenario. For member U1-L1, the faulted state effective stress range was found to be about 7.27 ksi (6.99 ksi × R_p; where R_p is defined in *MBE* Eq. 7.2.2.2.1-1 and is calculated automatically in the MBR-# worksheet). The maximum stress range, $(\Delta f)_{max}$, used for the infinite life check was about 15.99 ksi (i.e., $\gamma_{FATI}/\gamma_{FATII}(\Delta f)_{eff} = 1.75/0.8 = 2.2 \times 7.27$ ksi). In the faulted condition, a riveted detail is classified as AASHTO Category C with a constant amplitude fatigue threshold (CAFT) equal to 10 ksi. Since 10 ksi is less than 15.99 ksi, this member is in the finite life regime while in the faulted condition (Case Ib member, see Guide Spec. Article 2.5).

Effective Stress Range & Max Stress Range in Faulted State:

	Factored Stresses in Faulted Condition							
	Case 0	Case 1f	Case 2f	Case 2p	Case 3p	Case CH	Case WP	
	ksi	ksi	ksi	ksi	ksi	ksi	ksi	
$f_{AFN} = \Xi_B \Xi_{VL}(P_{FAT+IM} / 2A_{AFN}) =$	3.79	0.00	0.00	0.00	0.00	0.00	0.00	GS Eq. 2.2.1-1

40-over-fastener Rule (GS 2.2.1.1):

Load in an angle, $P_{FAT+IM-L}$	4.69	kip
$f_{AFN} = P_{FAT+IM} / A_N + 0.4P_{FAT+IM-L} / l_f t_p =$	6.99	ksi

Fig. 3-16. Screenshot showing the amplified stress ranges in the faulted state for member U1-L.

Fatigue Life Estimate in the Faulted State:

Fatigue Category	C		GS Table 2.5-2
Detail Category Constant	44.00	$\times 10^8$ ksi^3	
Constant Amplitude Fatigue Limit, $(\Delta F)_{TH}$	10.00	ksi	
Infinite Fatigue Life Check (Faulted State):			
Is $(\Delta f)_{max} \le (\Delta f)_{TH}$?	No	Yes or No	MBE Eq. 7.2.4-1
Finite Fatigue Life Estimate (Faulted State):			
Initially available stress cycles, N_{ov}	1.15E+07	cycles	MBE Eq. 7.2.5.1-2
No. of consumed stress cycles, N_1	0	cycles	
$N_{ov} > N_1$?	Yes	Yes or No	MBE Eq. 7.2.5.1-1
Remaining fatigue life, Y_{REM}	4.4	Years	MBE Eq. 7.2.5.1-4
Future ADTT for single lane, $ADTT_{FUTURE}$	7,200	trucks/day	MBE Eq. 7.2.5.1-6
$(ADTT_{SL})_{FUTURE} \le (ADTT_{SL})_{LIMIT}$?	No	Yes or No	MBE Eq. 7.2.5.1-5
$(Y_{ADTT})_{LIMIT} =$	0.00	Years	MBE Eq. 7.2.5.1-7
Remaining fatigue life, Y_{REM}	4.36	Years	MBE Eq. 7.2.5.1-8
Total faulted state remaining fatigue life, Y_{REM}	**4.4**	Years	MBE 7.2.5.1
Case I or II for unfaulted state?	**I(b)**	I(a), I(b), II	GS 2.5

Fig. 3-17. Screenshot showing results of faulted condition fatigue life calculations for member U1-L1.

Figure 3-17 shows a screenshot from the U1-L1 worksheet where fatigue life calculations are made for the faulted condition of the member. It can be seen that the worksheet has defined the fatigue category as AASHTO Category C and that it does not meet AASHTO requirements for infinite life. Next it computes the available number of constant amplitude fatigue cycles at the effective stress range, N_{av}, which was equal to 1.15×10^7 cycles. The number of consumed cycles is equal to zero because this is the number of consumed fatigue cycles in the faulted condition, which has not happened yet. This yields a remaining fatigue life of about 4.4 years in the faulted condition.

The final fatigue calculation is to perform a linear sum of accumulated fatigue damage for the member, including any fatigue damage accumulated in the unfaulted state. Guide Spec. Article 2.5.3 contains provisions for this calculation. Results for member U1-L1 are shown in Figure 3-18. In this case no fatigue damage is accumulated because the member possesses infinite fatigue life in the unfaulted state. This is why Y_u = "∞," resulting in the total fatigue life available to simply be the fatigue life of the member in the faulted condition, or 4.4 years.

Member U1-L1 passed all screening criteria, exceeded strength limit requirements, possessed positive fatigue life in the unfaulted state, and positive fatigue life in the faulted state. Thus, member U1-L1 has met all requirements of the Guide Specifications and qualifies to be re-designated as an Internally Redundant Member. The final step is to calculate the maximum special inspection interval for the member. This is contained at the bottom of the MBR-# worksheet under "Summary of Results."

The "Summary of Results" area of the worksheet (shown in Figure 3-19) holds the most important values from the evaluation in one place for a quick reference. It also contains the maximum special inspection interval for the member. Guide Spec. Tables 3.1-1 and 3.1-2 provide guidance on determining the special inspection intervals. Member U1-L1 was found to be a Case Ib member, having infinite fatigue life in the unfaulted state and finite fatigue life in the faulted state. Thus, Table 3.1-1 was used.

The total remaining fatigue life, N_f, was computed as 4.4 years. Being less than 20 years, the maximum permitted interval becomes the larger of 2 years or $0.5N_f$, where the Guide Spec. allows for the result of $0.5N_f$ to be rounded up to the nearest next even-year interval. The user is referred to Guide Spec. Article C3.1 for more information about rounding up to the next even-year interval.

3.6 Summary of Results for Example Bridge

Nine members were evaluated for the example bridge, including six tension chord members, one hanger member, and two tension diagonal members. Due to the symmetry of this span and the other six identical spans, results can be extrapolated to member of similar geometry and condition. It was found that any member comprised of only a single plate and two connection angles, such as Member L0-L1 detailed above, did not possess sufficient strength in the faulted condition when it was assumed that the web plate failed. Table 3-3 summarizes the results showing that member L0-L1, L1-L2, U1-L2, and U2-L3 did not pass the requisite strength checks in the faulted condition and therefore could not be reclassified as IRMs. Their respective special inspection intervals were computed, but cannot be implemented because the members are not IRMs.

Total Remaining Fatigue Life:

No. of accumulated years in unfaulted state, NU	102.0	Years	GS 2.5.3
Total finite fatigue life <u>only</u> in unfaulted state, Yu	∞	Years	GS 2.5.3
Total remaining fatigue life <u>only</u> in faulted state, Yf	4.4	Years	GS 2.5.3
Total remaining fatigue life, Nf = Yf(1-Nu/Yu)	**4.4**	Years	GS Eq. 2.5.3-1

Fig. 3-18. Screenshot showing the total remaining fatigue life for member U1-L1.

Summary of Results

Strength check =	**OK**	OK or NG	GS 2.3
Fatigue case =	I(b)	I(a), I(b), II	GS 2.5
Stress range in unfaulted state, Δf_{UFS} =	2.19	ksi	
Controlling stress range in faulted state, Δf_{FS} =	6.99	ksi	
Controlling faulted state remaining fatigue life, Y_{REM}	**4.4**	Years	
Total remaining fatigue life, N_f	**4.4**	Years	GS Eq. 2.5.3-1
Maximum Interval for Special Inspections =	**4.0**	Years	GS 3

Fig. 3-19. Screenshot showing the summary of results for IRM evaluation of member U1-L1.

Table 3-3. **Summary of Results for the Example Bridge IRM Evaluation**

Sheet Name	User Input Member ID	Faulted Condition Strength Check	IRM Special Inspection Interval (yrs)
MBR-1	L0-L1	NG	8
MBR-2	L1-L2	NG	8
MBR-3	L2-L3	OK	10
MBR-4	L3-L4	OK	10
MBR-5	L4-L5	OK	10
MBR-6	L5-L5′	OK	10
MBR-7	U1-L1	OK	4
MBR-8	U1-L2	NG	4
MBR-9	U2-L3	NG	4

Members L2-L3, L3-L4, L4-L5, L5-L5’ and U1-L1 (detailed above) all passed the required strength checks and possessed positive fatigue life in the unfaulted state. All of these members except hanger U1-L1, also possessed infinite fatigue life in the faulted state resulting in a maximum special inspection interval of 10 years. Member U1-L1 was found to have finite fatigue life in the faulted state with a maximum special inspection interval of 4 years. Thus, several of the fracture-critical members on these trusses can be reclassified as IRMs. However, those FCMs that did not qualify to be IRMs would continue to require a 2-year arm’s-length inspection cycle. This is a case where the owner would need to consider the options and decide if it makes sense to break out the FCMs and inspect them every two years and program the special inspections on the IRMs according to their respective intervals. Suspending any unnecessary inspection of IRMs would save time, resources, and reduce lane closures and worker exposure helping to reduce risk. For these reasons an owner may choose to implement the special inspection intervals allowed per provisions of the Guide Spec.

4 EXAMPLE INTERNAL REDUNDANCY EVALUATION: FLEXURAL MEMBER

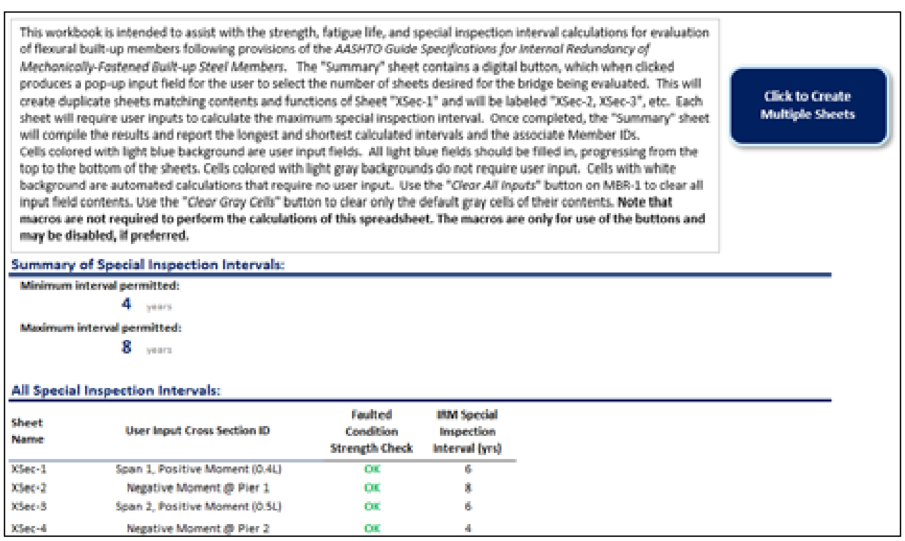
The following example is provided to illustrate implementation of the Guide Spec. in evaluating existing built-up members and using the IRM Flexural Evaluator available for free download from the NSBA design resources webpage. The bridge used in the example is an actual bridge currently in service in the United States. However, details of the bridge and its identity are withheld. The detailed inputs and outputs from the IRM Flexural Evaluator are provided, including screenshots appropriate to illustrate the correct use of the spreadsheet. The bridge contains several duplicate, or symmetric, spans. Thus, four locations are chosen for the example evaluation; maximum positive moment in the end span, negative moment at the first pier, maximum positive moment in the first interior span (matching all other interior spans), and negative moment at the second pier. While inputs for only some of the cross sections are shown for illustrative purposes, results for all cases are summarized at the end.

4.1 IRM Flexural Evaluator Overview

The following provides a basic introduction while offering some helpful insights into how to use the IRM Flexural Evaluator. If the user desires to use field-measured effective stress ranges to improve the fatigue life calculations, see Section 4.1.2.1 of this manual.

4.1.1 Summary Sheet

Figure 4 1 shows a screen shot of the Summary page taken from the IRM Flexural Evaluator. The Summary sheet is used to generate the number of evaluation sheets that will be required for the bridge being evaluated. It then combines the results synopsizing the strength check and special inspection interval results for each member. The Summary sheet lists the members using the names of the worksheet tabs, as well as the unique member identifiers entered into each respective worksheet by the user. Strength limit state results are reported using “NG” in bold red font for not meeting the faulted state strength limit requirements and “OK” in bold green font for passing the faulted state strength provisions of the Guide Spec. These same results are compiled at the bottom of each of the individual worksheets, as well.



This workbook is intended to assist with the strength, fatigue life, and special inspection interval calculations for evaluation of flexural built-up members following provisions of the *AASHTO Guide Specifications for Internal Redundancy of Mechanically-Fastened Built-up Steel Members*. The "Summary" sheet contains a digital button, which when clicked produces a pop-up input field for the user to select the number of sheets desired for the bridge being evaluated. This will create duplicate sheets matching contents and functions of Sheet "XSec-1" and will be labeled "XSec-2, XSec-3", etc. Each sheet will require user inputs to calculate the maximum special inspection interval. Once completed, the "Summary" sheet will compile the results and report the longest and shortest calculated intervals and the associate Member IDs.
Cells colored with light blue background are user input fields. All light blue fields should be filled in, progressing from the top to the bottom of the sheets. Cells colored with light gray backgrounds do not require user input. Cells with white background are automated calculations that require no user input. Use the "*Clear All Inputs*" button on MBR-1 to clear all input field contents. Use the "*Clear Gray Cells*" button to clear only the default gray cells of their contents. **Note that macros are not required to perform the calculations of this spreadsheet. The macros are only for use of the buttons and may be disabled, if preferred.**

Click to Create Multiple Sheets

Summary of Special Inspection Intervals:

Minimum interval permitted:
4 years

Maximum interval permitted:
8 years

All Special Inspection Intervals:

Sheet Name	User Input Cross Section ID	Faulted Condition Strength Check	IRM Special Inspection Interval (yrs)
XSec-1	Span 1, Positive Moment (0.4L)	OK	6
XSec-2	Negative Moment @ Pier 1	OK	8
XSec-3	Span 2, Positive Moment (0.5L)	OK	6
XSec-4	Negative Moment @ Pier 2	OK	4

Fig. 4-1. Screenshot showing the summary sheet within the IRM Flexural Evaluator.

When the user opens the IRM Flexural Evaluator for the first time, there will be a single evaluation sheet called, “XSec-1”. If the user needs to evaluate more than one member, or more than one cross section within the same member, and wishes to store all of the evaluations in a single Excel file, then by clicking on the blue button entitled, “Click to Create Multiple Sheets” seen in Figure 4-1, a popup window will appear (shown in Figure 4-2). The user may enter the number of cross sections desired for evaluation. Upon clicking “OK”, the workbook will automatically generate exact duplicates of XSec-1, naming them numerically, XSec-2, XSec-3, and so on until reaching the number entered into the pop-up window. For example, Figure 4-2 shows the user has input “12”, which will result in duplicates of XSec-1 numbered up to and including XSec-12.

User Tip: If several members will have similar components or component sizes, or other user inputs, it will save the user time to partially fill in XSec-1 with the information that is the same between members, before duplicating the sheet. Once duplicated, all information entered in XSec-1 will be propagated into the duplicated sheets and wouldn’t need to be re-entered by the user.

The spreadsheet uses a macro in the background to run the sheet duplication function. If the user finds that the macro is not working on their computer, the user may simply right click on the “XSec-1” tab and select “Move or Copy”, then check the box next to “Create a copy” at the bottom of the pop-up window to generate a single duplicate sheet. The user will then need to rename the duplicate sheet “XSec -2” so that the Summary page can recognize the new sheet and provide the summary information from it. See Section 3.1.2 on renaming the sheets to something other than “XSec -#”.

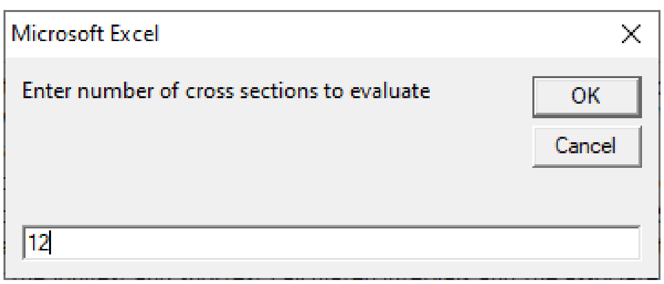


Fig. 4-2. Pop-up window that appears to enter the number of cross sections to be evaluated.

Note that the Summary sheet is preset to synopsize the results for up to 40 cross sections. Should the user need to include more than 40 cross section evaluations within the same Excel file, and then drag the columns to populate additional cells beyond 40. Keep in mind that this will go outside the printing area predefined in the sheet, so if the user desires to print a hard copy or print to PDF for record purposes, the user should also adjust the printable area to include those members beyond the preset 40. Also note that **the macro is not required for the spreadsheet to perform calculations**. Disabling the macros will still result in a functional spreadsheet. The macro simply provides a convenient way to produce multiple duplicates of the “XSec-#” sheet.

4.1.2 XSec-# Sheets

The evaluation sheets must be named “XSec-#”, such as XSec-1, in order for the Summary sheet to find and synopsize the results. If the user desires to name the tabs differently for any reason, the user will be required to unprotect the Summary sheet and modify the “Sheet Name” column to match the tab labeling scheme in order to retain the automated summary functionality.

The XSec-# sheets are the core of the IRM Flexural Evaluator where the user will input all parameters for the cross section being evaluated and where all area calculations, faulted state strength checks, and fatigue life estimates are performed and compared against requirements of the IRM Guide Specifications. There are cells with three different background colors, blue, gray, or white. Blue cells are user input cells requiring some kind of answer or numerical value for the sheet to perform its functions. Ideally the blue cells are filled out progressing from the top to the bottom of the worksheet. This is because depending on the input for some blue cells, various cells below may turn from blue to gray, or vice versa. Gray cells are cells

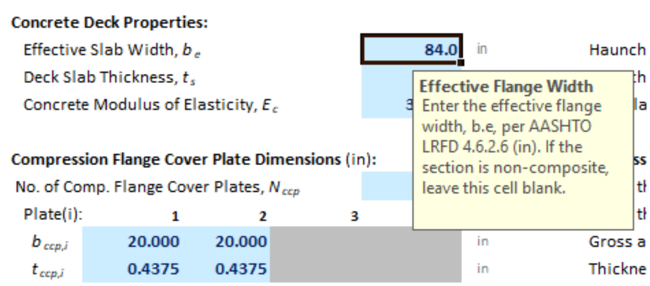


Fig. 4-3. Example showing yellow message windows that appear when clicking on input cells.

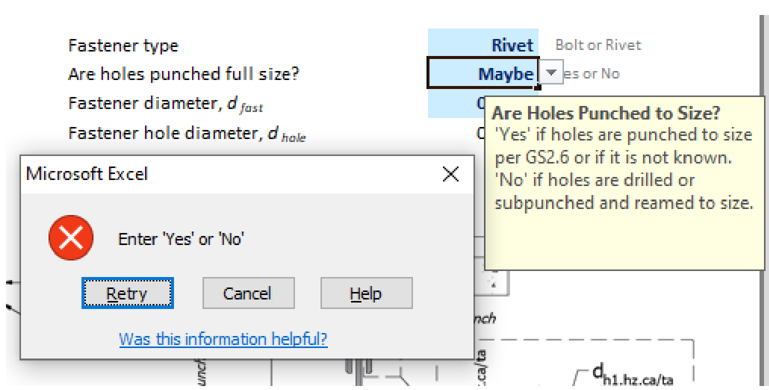


Fig. 4-4. Example warning message for incorrect value entered.

that do not require user input. Be advised that the gray cells are still functional, so if the user inputs values into gray cells it may affect calculations. Cells with white background are automated calculation fields that will provide programmed results based on blue cell inputs.

The XSec-# sheets also have two "clear" buttons. One is titled, "Clear All Inputs", which when clicked will remove the input values for all gray and blue cells (with the exception of the angle type). The other is titled, "Clear Gray Cells", which when clicked will remove the input values for all default gray cells. Default gray cells are those cells that are gray prior to filling out any of the sheet. Both buttons use a macro to perform this function. **The macro is not required for the spreadsheet to perform the calculations.** Disabling the macros will disable the "clear" buttons, but still result in a functional spreadsheet that can be used. This macro simply provides a convenient way to clear out inputs prior to initiating a new evaluation in the "XSec-#" sheets.

Each blue cell also contains a short message helping the user to better understand what is needed or in what format to input data into the respective cell. By simply clicking inside the blue input cells, the yellow message window will appear. By clicking into a different cell, the message window will disappear again. An example of the yellow message windows is shown in Figure 4-3 where it shows a reference to the Guide Spec. and additional guidance on what is needed to be entered into the blue input cell. In addition, some input cells will contain a dropdown menu. The menu may be used to click an option, or the same options may be manually typed into the cell. If an unacceptable or incorrect value is entered, the cell will provide a warning message, such as that shown in Figure 4-4. Simply click on "Cancel" and re-enter an acceptable value or choose one of the dropdown menu options.

Finally, there are a series of yes-no questions at the top of the XSec-# sheets. These questions are intended to help the user ensure that required screening criteria set forth in the Guide Spec. are checked before proceeding with the evaluation. One Guide Spec. screening criterion that is not explicitly asked at the top of the XSec-# sheet, however, is if there is positive remaining fatigue life in the unfaulted state for existing structures. It is required to possess positive fatigue life in the unfaulted state in order to meet the provisions of the Guide Spec. (See Guide Spec. 1.4). If this is known before evaluation begins, then the user is able to screen the member, if necessary. However, if the user is unsure what the remaining fatigue life is in the unfaulted state, the XSec-# sheet will perform this calculation based on user-input loads, at which point it can be checked and screened, if necessary.

4.1.2.1 Field-Measured Effective Live Load Stress Range

If field testing is performed to determine the effective live load stress range, rather than using calculated moments to do so, the user is pointed to Guide Spec. C2.5 wherein it provides guidance on how to apply the measured effective stress. This is done by multiplying the measured effective stress by the unfaulted state section modulus. This results in an effective live load moment that can be input for the M_{LL+IM}.

4.1.2.2 Reference Sketches

Four reference sketches are provided within the XSec-# worksheets to help guide the user in the nomenclature used to identify components and define their geometry. The reference sketches show four similar versions of built-up flexural members. Two of those cross sections are shown in Figure 4-5. *Note that the outer most cover plate is always labeled "PL-1". **It is critical that the user input cover plate geometries according to these schematics** so that correct sectional moduli are computed for both the unfaulted and faulted states.*

When entering the number of cover plates in the cross section into the spreadsheet, the user will select from the provided drop-down menu an option for 1 to 4 cover plates. The Guide Spec does not limit cross sections at 4 cover plates, but the spreadsheet was created to only evaluate up to this number because cross sections with 5 cover plates are extremely rare. The input cells for cover plate dimensions are organized by compression and tension, rather than top and bottom. This is so that the spreadsheet can be used the same way for both positive and negative moment regions of the flexural member. Thus, the user must be cognizant of inputting the correct geometries. For example, for a negative moment region, the user would input the top cover plate geometries for the "Tension Flange Cover Plate Dimensions" inputs.

The fourth reference sketch shows a cross section view of a built-up I-section with the composite deck parameters, distance to holes in the flange angles, the haunch, as well as the out-to-out depth. The out-to-out depth of the member is not necessarily the depth of the web plate (and most often it is not). Typically the web plate was designed to be slightly shallower than the out-to-out depth leaving a small quarter to half inch gap between the edge of the web plate and the first cover plate to facilitate fit-up. Thus, these fields are separate inputs and are accounted for in the section property calculations.

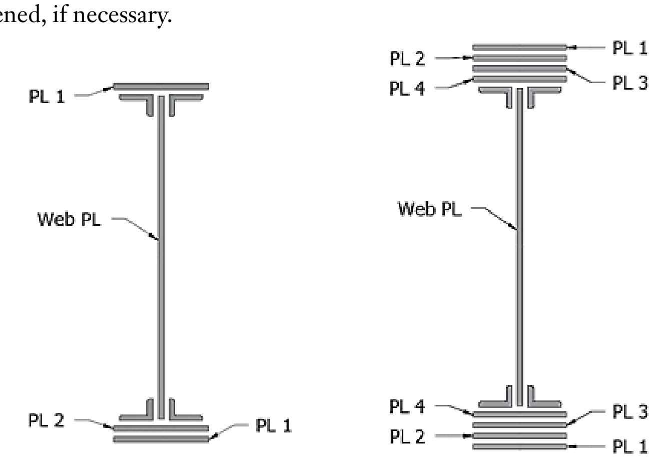


Fig. 4-5. IRM Flexural Evaluator spreadsheet reference sketches illustrating proper cover plate labels.

4.2 Background for Example Bridge

The bridge used in this evaluation example is a twin structure, each with two riveted built-up girders with parabolic haunched webs. A photo of the underside of the bridge is shown in Figure 4-6. The bridge is 2,403 feet long with two end spans of 111'-2⅞" and 14 symmetric interior spans of 155'-9" each. The yield strength of the steel was not explicitly stated in the design drawings. However, it did state that the unit stress of the structural steel was 18 ksi. This most likely means the yield strength was between 33 and 36 ksi. However, because it was not stated the plans, the yield and tensile strengths were taken from AASHTO *Manual for Bridge Evaluation* Table 6A.6.2.1-1, where for a bridge built in 1958 the yield strength is listed as 33 ksi and tensile strength as 66 ksi.

Fig. 4-6. Photo of the underside of the twin, two-girder bridges.

Figure 4-7 shows eight spans of the 16 symmetric spans of the bridge. The first two spans, shown encased within the red dashed line, are the two spans evaluated as part of this example. *Note that the floorbeams are not included in the evaluation example, but depending on spacing may need to be included for an actual evaluation.* Due to the symmetry of the design, these girders make ideal candidates for quicker evaluation using "families" of identical members. This means that sections of the girders having the same dead and live load moments with the same cross-sectional properties, along with the same general condition, can be evaluated together. Any cross sections having different condition (e.g. section loss resulting in loss of capacity, unrepaired impact damage, etc.) or any having been repaired resulting in a different cross section (e.g. built-up plating to restore section), must be evaluated on an individual bases. This method proves advantageous for saving time when evaluating a structure this long.

Details for the member components will be provided throughout the example, as needed. Only members that carry a net tensile live load that exceeds any compressive dead load are required to be evaluated. Typically, for a flexural member this would include locations of maximum live load moment. Figure 4-8 provides the elevation view of the original design drawings where the field splices and haunched webs can be seen. For this example, four cross sections will be evaluated within the first two spans. Stress reversal zones are not evaluated here, although it would be recommended to do so in an actual bridge evaluation. The approximate four locations are shown with solid colored lines at 0.4L of span 1, center line of pier 1, 0.5L of span 2, and center line of pier 2. These four locations represent the cross sections with the largest positive or negative live load moments in the first two spans.

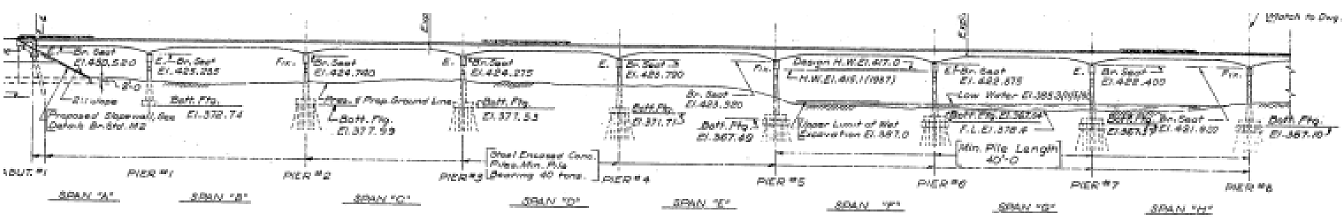


Fig. 4-7. Original design drawings for the 1958 built-up riveted two-girder bridge.

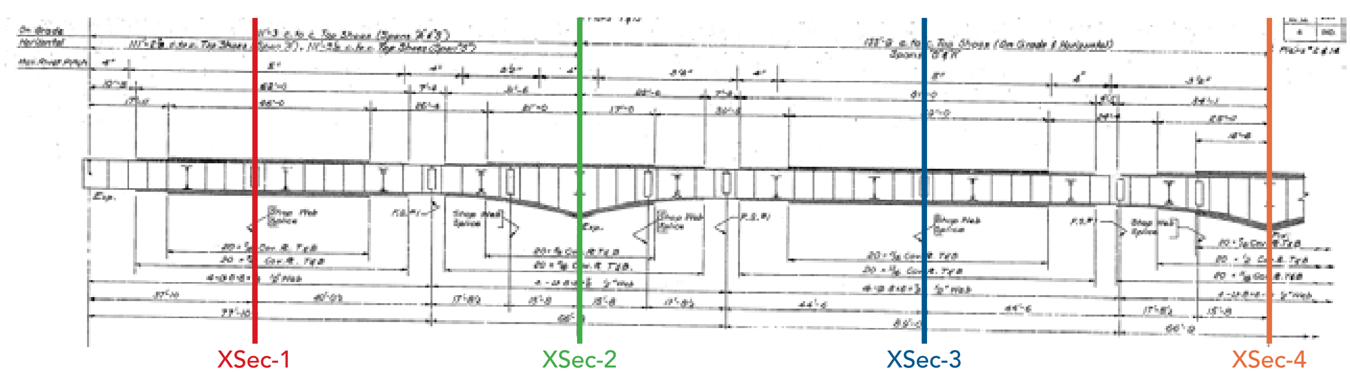


Fig. 4-8. Elevation view of the built-up riveted girders.

Table 4-1. **Moments at the Cross Sections**

X-Section ID	M_{DC1} (kip)	M_{DC1} (kip)	M_{DW} (kip)	M_{LL+IM} (kip)	M_{FAT+IM} (kip)
XSec-1	1118	0	148	3354	1106
XSec-2	3113	0	300	5328	1758
XSec-3	1639	0	199	3824	1268
XSec-4	4508	0	395	6887	2272

Table 4-2. **Section Moduli for Unfaulted and Faulted States**

X-Section ID	Gross Composite Section Modulus (in^3)		Gross Noncomposite Section Modulus (in^3)		Net Composite Section Modulus (in^3)		Net Noncomposite Section Modulus (in^3)	
	Unfaulted	Faulted	Unfaulted	Faulted	Unfaulted	Faulted	Unfaulted	Faulted
XSec-1	3,954.3	3,346.3	3,236.3	2,726.1	3,600.3	3,048.0	2,910.7	2,451.0
XSec-2	13,240.4	12,244.3	6,950.9	5,741.3	12,468.3	11,591.1	6,286.7	5,200.7
XSec-3	4,672.0	3,716.6	3,914.7	3,100.0	4,253.5	3,385.2	3,525.7	2,791.2
XSec-4	13,931.8	13,228.0	7,659.2	6,807.2	13,105.8	12,486.0	6,936.3	6,170.8

4.3 Example 1 – Load, Member Geometry, and ADTT Data

The bridge was built in 1958 before the AASHTO/AWS D1.5 Fracture Control Plan (FCP). This will affect the load factors required to be used, as seen in the Guide Spec. Table 1.7.1-1. The yield and tensile strength of the materials are not provided in the design drawings. These could be tested, if desired. For this example, AASHTO *Manual for Bridge Evaluation* Table 6A.6.2.1-1 will be used, where it states that for year of construction between 1936 and 1963, use 33 ksi and 66 ksi for the minimum yield and tensile capacity, respectively.

Each of the twin bridges carries two lanes of traffic with an estimated ADT of 12,600 vehicles per day. Applying factors taken from AASHTO *LRFD Bridge Design Specifications* Table 3.6.1.4.2-1 and C3.6.1.4.2-1, this yields an estimated $(ADTT)_{SL}$ of 1,606 trucks per day (12,600 × 0.15 × 0.85). According to AASHTO *LRFD Bridge Design Specifications* commentary C3.6.1.4.2, research indicates that ADT for a single lane is physically limited to 20,000 vehicles per day. Hence, $(ADTT)_{LIMIT}$ will be taken as 5,100 trucks per day (20,000 trucks/day/lane × 2 lanes × 0.15 × 0.85). An expected annual $(ADTT)_{SL}$ growth rate, g, of 1% (i.e., 0.010) is used *(Note: If a 0% growth rate is desired, it must be entered into the Xsec-# worksheet, as something slightly higher than zero, e.g. 0.00001, in order for the cell function to not yield a division by zero error, #DIV/0!)*.

Table 4-1 lists the design loads for each of the cross sections to be evaluated. The moments were calculated using the NSBA SIMON LRFD software applying the HL-93 load design loading, as required by the Guide Specifications article 1.7. Table 4 2 summarizes the gross and net section moduli in the faulted and unfaulted states. Recall that the faulted state assumes the outermost cover plate is fractured. All section properties are automatically calculated by the IRM Flexural Evaluator and are provided in this table for convenience to the reader. Net area calculations take into account the number of fastener holes within a single cross section and their position within the member, as entered into the worksheet by the user. For the example bridge, all cross sections have two tension cover plates except for XSec-4, which has 3 tension cover plates. The section moduli are calculated for all composite and non-composite sections in both the faulted and unfaulted states. However, depending on the user inputs (i.e. composite section or not, negative or positive moment region), the sheet will use different section moduli to calculate the flexural stresses for the strength and fatigue load cases. The spreadsheet currently conservatively ignores any contribution of the deck reinforcement in the calculation of fatigue live load stress range. Thus, if the user inputs that the cross section is in a negative moment region, the spreadsheet will automatically use the noncomposite net section modulus and a pop-up note appears, such as is seen in Figure 4-9.

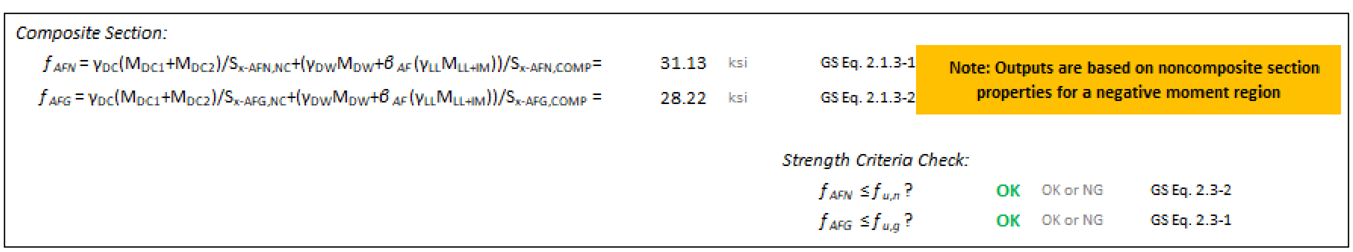


Fig. 4-9. Screenshot of yellow pop-up notifying user that the noncomposite section properties are used.

4.4 Example 1 – Evaluation of Cross Section XSec-1

Figure 4-10 shows a screenshot taken from the evaluation worksheet for cross section XSec-1 (located at the maximum positive moment of span 1 of the example bridge. Under “General”, the user entered that the cross section is composite with the deck and to include the haunch, and that the section is not in a negative moment region. The type of fastener is selected and the diameter of the rivet, specified on the design drawings, was input. The spreadsheet automatically applies the standard hole size based on the fastener diameter. This can be manually changed by the user if desired, by unprotecting the sheet and entering the hole diameter desired. Additionally, Figure 4-10 shows “Concrete Deck Properties” with the input cells turned light blue. This is because the user indicated a composite deck and to include the haunch. In the case that either of these options is not selected, the associate cells will remain gray indicating no input is necessary. This figure also shows the web plate dimensions entered. Notice that the out-to-out depth is greater than the web plate depth. This is often the case in built-up construction where a small gap is designed into the cross section to allow for fit-up between the web plate and the cover plates. The out-to-out depth is defined in the reference sketch contained in the spreadsheet, as the distance from the outer surface of the compression flange angles to the outer surface of the tension flange angles.

Figure 4-11 shows a screenshot of the data entered for the tension flanges, including the cover plates, flange angles, and the fastener holes for the connection angles of XSec-1. Connection angle sizes can be selected from the dropdown menu that appears when the user clicks into the blue cell under “Tension Flange Angle Properties”. Next, the orientation of the angle legs is defined for cases when the angle legs are not equal. Automatic outputs for the angles are then displayed. These are obtained using a “LookUp” function in Excel referencing a hidden sheet called “Shapes”. The shapes are obtained from the latest AISC shape database. Fastener hole data required for the angles is simply the number of holes per leg per cross section. Typically fasteners are staggered to avoid reducing the net area, but this is not always the case. For XSec-1, there was only a single fastener hole on each leg of the angle at a given cross section of the flange angles. This is reflected in the blue cells for holes in vertical and horizontal legs, as shown in Figure 4-11.

General:

Steel yield strength, F_y	33.0	ksi	Fastener type	Rivet	Bolt or Rivet
Steel tensile strength, F_u	66.0	ksi	Are holes punched full size?	No	Yes or No
Is the girder composite with the deck?	Yes	Yes or No	Fastener diameter, d_{fast}	0.8750	in
Include haunch in section properties?	Yes	Yes or No	Fastener hole diameter, d_{hole}	0.9375	in
Is this for a negative moment region?	No	Yes or No	Is compression flange welded to web?	No	Yes or No

Concrete Deck Properties:

Effective Slab Width, b_e	84.0	in	Haunch Width, b_{haunch}	22.0	in
Deck Slab Thickness, t_s	8.0	in	Haunch Thickness, t_{haunch}	3.9	in
Concrete Modulus of Elasticity, E_c	3400.0	ksi	Modular Ratio of the concrete, n	8.5	

Web Plate Dimensions:

Web Plate Depth, D_w	70.00	in	Flange Angles Out-to-out Depth, D_{oto}	70.50	in
Web Thickness, t_w	0.5000	in			

Fig. 4-10. Screenshot showing entered values for the example bridge, cross-section XSec-1.

Tension Flange Cover Plate Dimensions (in):

No. of Tension Flange Cover Plates, N_{tcp}				2	1 to 4
Plate(i):	1	2	3	4	
$b_{tcp,i}$	20.000	20.000			in
$t_{tcp,i}$	0.4375	0.4375			in

Note: Plate 1 is always the outer-most CP, see Ref Sketches

Tension Flange Angle Properties:

Select the size of the angles	L8X8X7/8	
Select the orientation of the long leg	Vert	V or H
Gross area of a single angle, $A_{ta.g}$	13.3	in^2
Thickness of the angle, t_{ta}	0.875	in
Moment of Inertia about horiz. axis, $I_{xx.ta}$	79.7	in^4
Outer Horiz. Leg to N.A., y_{ta}	2.31	in

Tension Flange Angle Fastener Holes:

Holes in Vert Angle Leg, $N_{holes.vt.ta}$	1	holes	Holes in Horz Angle Leg, $N_{holes.hz.ta}$	1	holes
Distance to Hole 1, $d_{h1.vt.ta}$	6.00	in	Distance to Hole 1, $d_{h1.hz.ta}$	6.00	in
Distance to Hole 2, $d_{h2.vt.ta}$		in	Distance to Hole 2, $d_{h2.hz.ta}$		in

Fig. 4-11. Screenshot showing entered values for tension flanges and fastener holes for XSec-1.

4.4.1 XSec-1 Factored Moment and Strength Checks

Gross and net section moduli, along with other cross-sectional properties, are automatically performed once all member geometric data is entered. This is done for both the unfaulted and faulted states. The progression of these calculations is made transparent to the user in the gray fields to the right of the main area of the spreadsheet shown in white. This way the user can verify calculations and make comparisons, as desired. The primary outputs are summarized on the main portion of the spreadsheet for convenience under the Unfaulted Member Section Properties and Faulted Member Section Properties subheadings. The spreadsheet also automatically lists the load factors being applied, based on whether or not the bridge was built according to requirements of the Fracture Control Plan (FCP). AASHTO resistance factors and fatigue category constants are also listed for convenience.

Calculations for XSec-1 strength checks are shown in Figure 4 12. The *Redundancy II* factored moment is 6,502 kip-ft. Keep in mind that the cover plate adjustment factor is made equal to 1.0 for the strength checks. As can be seen in the figure, the member passes the net section fracture check where 26.83 ksi < 52.8 ksi. It also passes the gross section yield check where 24.36 ksi < 31.35 ksi. It is possible that material testing in the field could result in higher yield and tensile strengths that might help a member pass strength requirements. For the case where a member is very close to passing, it may be worth the cost and effort to perform field testing. If a member is close to passing the strength checks and the engineer believes that material testing could help it to pass, it would be up to the owner/engineer to decide if pursuing the more accurate material property data is worth the cost and effort. If other members on the same bridge do meet all provisions of the Guide Spec. and can be reclassified as IRMs, then it would be recommended to obtain the actual material properties and improve accuracy of the calculations.

For XSec-1, because the cross section was input as composite and in a positive moment region, the outputs are displayed under "Composite Section". There it can be seen that two bolded, green "OK"s are displayed indicating that the strength criteria for failure of the outer cover plate is satisfied.

If the strength criteria were not satisfied, "NG" would be shown in bold red font, standing for "No Good". This would mean that cross section does not meet provisions of the Guide Specifications and therefore cannot be reclassified as an IRM.

4.4.2 XSec-1 Fatigue Life in Unfaulted State

Positive remaining fatigue life in the unfaulted state is a screening criterion contained in Section 1.4 of the Guide Specifications. However, as a convenience to the user, the IRM Flexural Evaluator spreadsheet is set up to make this calculation, as well. This way if remaining fatigue life in the unfaulted state is not known, the user can simply input the required moments and cross-sectional geometry and the spreadsheet will make this calculation for them. In the case of negative remaining fatigue life, the MBE allows for truncation of the total life distribution to present bridge age, as discussed in the AASHTO MBE Article C7.2.7.2.3. However, this approach is prohibited for the evaluation of IRMs due to the possibility that inspections could miss existing fatigue cracks and the presence of cracks is undesirable for the purposes of internal redundancy; as is discussed in the Guide Specifications under C1.4. *Note: The owner may consider obtaining field measured effective fatigue stress ranges at the desired cross sections. Field measurement of live load stresses often shows that theoretical calculations of life load moment are conservative due to several factors, such as system load sharing, secondary member contributions, partial fixity, etc. Consider that when calculating fatigue life, the stress range is cubed. Thus, reducing the live load stress range through field measurement will have a significant impact to the estimated remaining fatigue life.*

Strength Capacity Check (GS 2.1 & 2.3)

Redundancy II Factored Moment:					
$M_u = \gamma_{DC}(M_{DC1} + M_{DC2}) + \gamma_{DW}M_{DW} + \gamma_{LL}M_{LL+IM} =$	6,502	k-ft	GS Eq. 1.7.1-1		
Factored Resistance:					
$f_{R,n} = \varphi_u F_u =$	52.80	ksi	GS Eq. 2.3-2		
$f_{R,g} = \varphi_y F_y =$	31.35	ksi	GS Eq. 2.3-1		
Factored Stress in the Faulted State: (*Note:* $\beta_{AF} = 1.0$)					
Noncomposite Section:					
$f_{AFN} = \beta_{AF}(M_u / S_{x\text{-}AFN}) =$	-	ksi	GS Eq. 2.1.2-1		
$f_{AFG} = \beta_{AF}(M_u / S_{x\text{-}AFG}) =$	-	ksi	GS Eq. 2.1.2-2		
			Strength Criteria Check:		
			$f_{AFN} \le f_{R,n}$?	-	OK or NG	GS Eq. 2.3-2
			$f_{AFG} \le f_{R,g}$?	-	OK or NG	GS Eq. 2.3-1
Composite Section:					
$f_{AFN} = \gamma_{DC}(M_{DC1}+M_{DC2})/S_{x\text{-}AFN,NC}+(\gamma_{DW}M_{DW}+\beta_{AF}(\gamma_{LL}M_{LL+IM}))/S_{x\text{-}AFN,COMP} =$	26.83	ksi	GS Eq. 2.1.3-1		
$f_{AFG} = \gamma_{DC}(M_{DC1}+M_{DC2})/S_{x\text{-}AFG,NC}+(\gamma_{DW}M_{DW}+\beta_{AF}(\gamma_{LL}M_{LL+IM}))/S_{x\text{-}AFG,COMP} =$	24.36	ksi	GS Eq. 2.1.3-2		
			Strength Criteria Check:		
			$f_{AFN} \le f_{R,n}$?	**OK**	OK or NG	GS Eq. 2.3-2
			$f_{AFG} \le f_{R,g}$?	**OK**	OK or NG	GS Eq. 2.3-1

Fig. 4-12. Screenshot showing results of strength checks for XSec-1.

The worksheet uses the current fatigue life calculation method found in the AASHTO *Manual for Bridge Evaluation* (MBE). Hence, the user is required to enter an expected annual $(ADTT)_{SL}$ growth rate, g. As mentioned above, if the desired growth rate is zero, the user must enter some number just larger than zero, such as 0.00001, in order to avoid the division by zero error in Excel. The user is also required to enter the resistance factor, R_R, as defined in the *Manual for Bridge Evaluation* Table 7.2.5.2-1. A drop-down menu of options is provided for convenience or the same numbers can be manually typed into the cell. The spreadsheet steps through the fatigue life calculation outputting the major variables, first checking *Fatigue I* (infinite life). For XSec-1, the unfaulted state effective stress range was found to be 3.69 ksi. The maximum stress range, $(\Delta f)_{max}$, used for the infinite life check was 8.17 ksi (i.e., $\gamma_{FATI}/\gamma_{FATII} \times (\Delta f)_{eff} = 1.75/0.8 \times 3.69 \approx 2.2 \times 3.69$ ksi).

Fatigue categories for riveted and bolted details have been defined in the Guide Spec. Table 2.5-2 for the unfaulted and faulted states. These categories are the result of full-scale fatigue testing by Hebodon et al. (2015). In the unfaulted state, a riveted detail is classified as AASHTO Category D with a constant amplitude fatigue threshold (CAFT) equal to 7 ksi. Since 7 ksi is less than 8.17 ksi, this member possesses finite fatigue life in the unfaulted state (*Case II* member, see Guide Spec. Article 2.5). Recall that positive remaining fatigue life in the unfaulted state is a screening criterion for IRMs. Thus, XSec-1 meets this criterion and may continue on in the evaluation process.

The final two rows of the unfaulted fatigue life calculation section report the remaining fatigue life of the cross section and reports the Case type, either *Case I* or *Case II*. This is shown in Figure 4-13 where it can be seen that XSec-1 has 28.5 years of fatigue life in the unfaulted state, classifying it as a *Case II* (which is any case having finite fatigue life in the unfaulted state). A bold green "OK" or bold red "NG" indicates the go-no-go results of this calculation. A cross section not having positive remaining fatigue life does not meet provisions of the Guide Specifications and therefore cannot be reclassified as an IRM.

4.4.3 XSec-1 Fatigue Life in the Faulted State

Fatigue life in the faulted state is checked next. As part of the fatigue life calculation in the faulted state, the spreadsheet calculates the cover plate adjustment factor based on the number of tension cover plates input by the user. This factor accounts for localized stress amplification within the vicinity of the assumed fracture. The number of cover plates is reported along with the calculated adjustment factor, as seen in Figure 4-14.

The XSec-# worksheet once again performs the Fatigue I (infinite life) and Fatigue II (finite life) calculations, this time for the faulted state, calculating the live load stress range from failure of the outermost cover plate, *PL*-1. Fatigue categories for riveted and bolted details have been defined in the Guide Spec. Table 2.5-2 for the unfaulted and faulted states. In the faulted state, a riveted detail is classified as AASHTO Category C with a constant amplitude fatigue threshold (CAFT) equal to 10 ksi.

Figure 4-15 shows a screenshot from the worksheet for XSec-1 where it can be seen that the amplified stress range for the composite section resulting from failure of the outer cover plate, *PL*-1, is approximately 5.99 ksi. Therefore, this results in an effective stress range of 6.03 ksi and a max stress range of 13.27 ksi. Since 10 ksi CAFT is less than 13.27 ksi, this cross section has finite fatigue life in the faulted state. Figure 4-16 shows an additional screenshot from the XSec-1 worksheet where fatigue life calculations are made for the faulted state of the cross section. It can be seen that the worksheet has defined the fatigue category as AASHTO Category C and that it does not meet AASHTO requirements for infinite life. Next, it computes the available number of constant amplitude fatigue cycles at the effective stress range of 6.03 ksi, N_{av}, which was equal to 2.00×10^7 cycles. The number of consumed cycles is equal to zero because this is the number of consumed fatigue cycles in the faulted state, which has not yet happened. The future single lane ADTT is computed using the annual growth rate, g. A check is made to determine if the future single lane is less than the single lane ADTT limit. In this case it is, thus, $(Y_{ADTT})_{LIMIT}$ is not computed, reporting "N.A." for not applicable. Finally, the fatigue life calculation yields a remaining fatigue life of about 29.3 years in the faulted state.

Total unfaulted state remaining fatigue life, Y_{REM}	28.5	Years	MBE 7.2.5.1	Unfaulted fatigue life?	OK
Case I or II for unfaulted state?	II	I or II	GS 2.5		

Fig. 4-13. Screenshot of the results for fatigue life in the unfaulted state for XSec-1.

Cover Plate Adjustment Factor (GS 2.1.2 & 2.1.3)

Fatigue Adjustment Factors (GS Eq. 2.1.2-3 & Eq. 2.1.3-3):

CP Adjustment Factor, β_{AF} =	1.375
Number of tension CPs, N_{FL} =	2

Fig. 4-14. Screenshot of the cover plate adjustment factor output.

The final fatigue calculation is to perform a linear sum of accumulated fatigue damage for the member, including any fatigue damage accumulated in the unfaulted state. Guide Spec. Article 2.5.3 contains provisions for this calculation. Results for XSec-1 are shown in Figure 4-17. In this case fatigue damage is accumulated in the unfaulted state, which is taken into account for the total remaining life. If the cross section possessed infinite life in the unfaulted condition, then the result would have been simply the fatigue life calculated in the faulted state, or 29.3 years. In such a scenario, Y_u would be set equal to "∞". However, for XSec-1, this is not the case and the total remaining fatigue life, accounting for both the unfaulted state accumulation of fatigue damage, and the fatigue life in the hypothetical faulted state is equal to just over 9 years.

XSec-1 passed all screening criteria, exceeded strength limit requirements, possessed positive fatigue life in the unfaulted state, and positive fatigue life in the faulted state. Thus, XSec-1 has met all requirements of the Guide Specifications and qualifies to be reclassified as an Internally Redundant Member (IRM). The final step is to calculate the maximum special inspection interval for the cross section. This is contained at the bottom of the XSec-# worksheet under "Summary of Results".

Effective Fatigue Stress Range & Max Fatigue Stress Range in Faulted State:			
Noncomposite Section:			
$f_{AFN} = \beta_{AF}(M_{FAT+IM} / S_{x\text{-}AFN}) =$	-	ksi	GS Eq. 2.1.2-1
Composite Section:			
$f_{AFN} = \beta_{AF}(M_{FAT+IM}) / S_{x\text{-}AFN,COMP} =$	**5.99**	ksi	GS Eq. 2.1.3-1
Faulted state stress range, Δf =	5.99	ksi	MBE 7.2.2
Faulted state effective stress range, $(\Delta f)_{eff}$ =	6.03	ksi	MBE Eq. 7.2.2-1
Faulted state, $(\Delta f)_{max} = (\gamma_{FAT1}/\gamma_{FAT2})(\Delta f)_{eff} \approx 2.2(\Delta f)_{eff}$	13.27	ksi	MBE 7.2.4

Fig. 4-15. Screenshot from spreadsheet showing XSec-1 faulted state live load stress range calculations.

Fatigue Life Estimate in the Faulted State:			
Fatigue Category	C		GS Table 2.5-1
Detail Category Constant	44.00	$\times 10^8$ ksi^3	
Constant Amplitude Fatigue Limit, $(\Delta F)_{TH}$	10.0	ksi	
Infinite Fatigue Life Check (Faulted State):			
Is $(\Delta f)_{max} \leq (\Delta f)_{TH}$?	No	Yes or No	MBE Eq. 7.2.4-1
Finite Fatigue Life Estimate (Faulted State):			
Initially available stress cycles, N_{av}	2.00E+07	cycles	MBE Eq. 7.2.5.1-2
No. of consumed stress cycles, N_1	0	cycles	
$N_{av} > N_1$?	Yes	Yes or No	MBE Eq. 7.2.5.1-1
Remaining fatigue life, Y_{REM}	29.3	Years	MBE Eq. 7.2.5.1-4
Future ADTT for single lane, $ADTT_{FUTURE}$	2,149	trucks/day	MBE Eq. 7.2.5.1-6
$(ADTT_{SL})_{FUTURE} \leq (ADTT_{SL})_{LIMIT}$?	Yes	Yes or No	MBE Eq. 7.2.5.1-5
$(Y_{ADTT})_{LIMIT}$ =	N.A.	Years	MBE Eq. 7.2.5.1-7
Remaining fatigue life, Y_{REM}	N.A.	Years	MBE Eq. 7.2.5.1-8
Total faulted state remaining fatigue life, Y_{REM}	**29.3**	Years	MBE 7.2.5.1
Case I or II for faulted state?	**II**	I(a), I(b), II	GS 2.5
Total Remaining Fatigue Life:			
No. of accumulated years in unfaulted state, N_U	62.0	Years	GS 2.5.3
Total remaining fatigue life only in **unfaulted** state, Y_u	90.5	Years	GS 2.5.3
Total remaining fatigue life only in **faulted** state, Y_f	29.3	Years	GS 2.5.3
Total remaining fatigue life, $N_f = Y_f(1-N_u/Y_u)$	**9.2**	Years	GS Eq. 2.5.3-1

Fig. 4-16. Screenshot showing results of faulted state fatigue life calculations for XSec-1.

The "Summary of Results" area of the worksheet (shown in Figure 4-18) holds the most important values from the evaluation in one place for a quick reference. It also contains the maximum special inspection interval for the cross section (or member) being evaluated. Guide Spec. Tables 3.1-1 and 3.1-2 provide guidance on determining the special inspection intervals. XSec-1 was found to be a *Case II* member, having finite fatigue life in the unfaulted state and finite fatigue life in the faulted state. Thus, Table 3.1-2 was used.

The total remaining fatigue life, N_f, was computed as 9.2 years. Being less than 20 years and greater than 5, the maximum permitted interval is calculated as $0.5N_f$, where the Guide Spec. allows for the result of $0.5N_f$ to be rounded up to the nearest next even-year interval. The user is referred to Guide Spec. C3.1 for more information about rounding up to the next even-year interval.

Table 4-3. **Summary of Results for the Example Bridge IRM Evaluation**

Sheet Name	User Input Member ID	Faulted State Strength Check	IRM Special Inspection Interval (yrs)
XSec-1	Span 1, Positive Moment (0.4L)	OK	6
XSec-2	Negative Moment @ Pier 1	OK	8
XSec-3	Span 2, Positive Moment (0.5L)	OK	6
XSec-4	Negative Moment @ Pier 2	OK	4

4.5 Discussion of Results and Potential Implementation

Four cross sections were evaluated for the example bridge girder. Due to the symmetry of the spans, results can be extrapolated to cross sections of similar geometry and condition. It was found that all cross sections met provisions of the Guide Specifications resulting in Special Inspection intervals ranging from 4 to 8 years. Table 4-3 summarizes the results with the respective special inspection intervals. Keep in mind that these four cross sections represent the same continuous girder. They were selected as locations of highest live load, making them the most critical cross sections to evaluate. For continuous construction, stress reversal zones would also be recommended for evaluation.

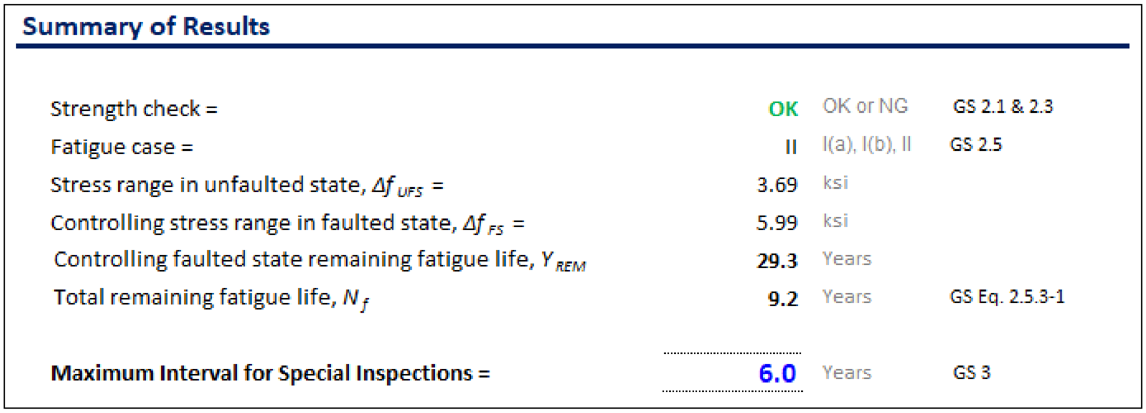
All cross sections passed the IRM evaluation requirements. Thus, the fracture-critical member could be reclassified as an IRM. In the hypothetical case that a continuous girder has some cross sections that pass the Guide Spec provisions and some that don't, it would fall upon the owner to decide how they wish to handle this. It could be that the continuous girder is divided into sections of positive and negative moment, such as is shown in Figure 4-19. Each section could be treated as a separate member with one or more cross sections evaluated for each member. Suggested cross sections to evaluate include those with maximum (or minimum for negative moment regions) live load moments, or those having abrupt section modulus changes, such as at the termination of a cover plate. (Only a single cross section was evaluated for each member in the example above to keep the example simple and brief.) Another possible solution, similar to the first, is to subdivide the girder some distance from the center line of the piers so that reversals zones are clearly included in a member, rather than falling on a dividing point. Consider that members need not be evaluated for internal redundancy at mechanically fastened splices (See Guide Specifications 1.4), which will often be located at a stress reversal position on the continuous girder.

As is shown in this example, each cross section met the provisions of the IRM Guide Specifications. However, the analyses resulted in different special inspection intervals. This is a case where the owner would need to consider the options and decide if it makes sense to break out the sections of the girder and perform the special inspection for them on their different schedules or perhaps choose the section with the shortest inspection interval and perform the special inspection interval for the entire girder on that schedule such that the other special inspection interval requirements are exceeded. Suspending any unnecessary inspection of IRMs could save time, resources, and reduce lane closures and worker exposure helping to reduce risk. This aspect of implementation is not prescribed in the Guide Specifications and therefore falls upon the discretion of the owner. There is ample flexibility built into the Code of Federal Regulations for programming and conducting these inspections; keep in mind the best use of resources and aspects of safety that are intended to benefit from the reliability of member-level redundancy.

Total Remaining Fatigue Life:

No. of accumulated years in unfaulted state, N_u	62.0	Years	GS 2.5.3
Total remaining fatigue life only in **unfaulted** state, Y_u	90.5	Years	GS 2.5.3
Total remaining fatigue life only in **faulted** state, Y_f	29.3	Years	GS 2.5.3
Total remaining fatigue life, $N_f = Y_f(1\text{-}N_u/Y_u)$	9.2	Years	GS Eq. 2.5.3-1

Fig. 4-17. Screenshot showing the total remaining fatigue life for XSec-1.



Summary of Results

Strength check =	OK	OK or NG	GS 2.1 & 2.3
Fatigue case =	II	I(a), I(b), II	GS 2.5
Stress range in unfaulted state, Δf_{UFS} =	3.69	ksi	
Controlling stress range in faulted state, Δf_{FS} =	5.99	ksi	
Controlling faulted state remaining fatigue life, Y_{REM}	29.3	Years	
Total remaining fatigue life, N_f	9.2	Years	GS Eq. 2.5.3-1
Maximum Interval for Special Inspections =	6.0	Years	GS 3

Fig. 4-18. Screenshot showing the summary of results for IRM evaluation of XSec-1.

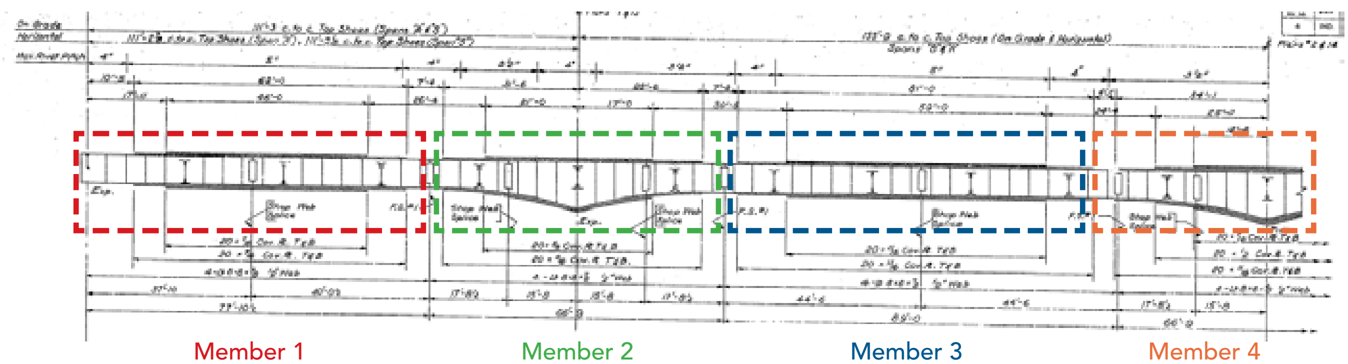


Fig. 4-19. Example of how to subdivide continuous girder for evaluation and inspection of IRMs.

5 ACKNOWLEDGMENTS

The author wishes to thank all who have contributed to this implementation guide. Specifically, Alex Lim with the Oregon Department of Transportation was extremely helpful in providing information on the bridge used to develop the axial evaluation example.

6 REFERENCES

AASHTO (2017). *AASHTO LRFD Bridge Design Specifications. Transportation* (7th Ed.). Washington D.C.

Cha, H., Lyrenmann, L., Connor, R. J., and Varma, A. H. (2014). "Experimental and Numerical Evaluation of the Postfracture Redundancy of a Simple Span Truss Bridge." *Journal of Bridge Engineering*. American Society of Civil Engineers. DOI: 10.1061/(ASCE)BE.1943-5592.0000622.

City of Portland Archives (1917). Pacific Highway Interstate Bridge [online]. Available from: https://vintageportland.wordpress.com/2012/09/17/pacific-highway-interstate-bridge-1917/. [Accessed January 2019].

Conner, R. J., Bonachera Martin, F. J., Korkmaz, C., Varma, A. H., and Lai, Z. (2018). *NCHRP Report 883: Fracture-Critical System Analysis for Steel Bridges*. Transportation Research Board, National Research Council, Washington, D.C.

Connor, R. J., Dexter, R., & Mahmoud, H. (2005). *NCHRP Synthesis 354: Inspection and Management of Bridges with Fracture-Critical Details* (p. 84). Washington D.C.

FHWA. (1995). *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*. Report No. FHWA-PD-96-001.

Hebdon, M. H., Korkmaz, C., Martín, F. J., & Connor, R. J. (2015). Member-level redundancy of built-up steel girders subjected to flexure. West Lafayette, IN: Purdue University. https://doi.org/10.5703/1288284316728

Judy, S. (2013). "FDOT Scrambles to Make Repairs to Mathews Bridge" [online]. *Engineering News Record*. Available from: https://www.enr.com/articles/12001-fdot-scrambles-to-make-repairs-to-mathews-bridge. [Accessed June 2018].

Lloyd, J. B. (2018). *Internal Redundancy of Mechanically-Fastened Steel Built-up Axially-Loaded Members*. West Lafayette, Ind.: Purdue University. Dissertations.

Niemann, T. L. (1999). Hastings Bridge Fracture Investigation. In *The International Bridge Conference*. Pittsburgh, Pa.

Rice, J. (2017). "Large Fracture in Steel Truss Closes Deleware River Bridge" [online]. *Engineering News Record*. Available from: https://www.enr.com/blogs/11-mid-atlantic-monitor/post/41308-large-fracture-closes-delaware-river-bridge. [Accessed June 2018].

Smarter. Stronger. Steel.

National Steel Bridge Alliance
312.670.2400 | www.aisc.org

B010-20