

Evaluation of Steel Bridge Details for Susceptibility to Constraint-Induced Fracture

FINAL REPORT

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FOREWORD

The performance of steel bridges in the U.S. continues to inform engineers and owners of design methods and construction techniques that work well and those that do not. Over several decades engineers have developed and implemented a variety of details to make welded structural connections. In certain situations, some of these details may produce a high degree of constraint that can increase their susceptibility to constraint-induced fracture, a sudden and brittle form of failure. As a bridge engineer, it is important to understand the causes and contributors to this behavior for two reasons: (1) identifying and avoiding or mitigating details susceptible to constraint-induced fracture is an important aspect of bridge design, in-service inspection, and potential retrofit, and (2) incorrectly assuming certain details are inherently problematic could lead to unnecessary or ineffective design policies or details, and costly repairs or retrofits.

This report provides a discussion of the fundamental principles of ductile behavior of steel structures, including stress triaxiality, constraint, and the three conditions generally necessary for a detail to be susceptible to constraint-induced fracture. Also included is a simple procedure by which an engineer can evaluate details for susceptibility to constraint-induced fracture, along with suggestions for how to mitigate those conditions. Finally, practical examples are provided to illustrate the procedure for common steel bridge details. This report will benefit owner-agency and consultant engineers involved in the design, fabrication, in-service inspection, and retrofit of steel bridges. Understanding the findings presented in this report will help engineers avoid potentially problematic details during design, and owners make better-informed decisions about the need to retrofit existing bridges.

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Office of Infrastructure
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16. Abstract
This report explains how to evaluate steel bridge details for susceptibility to constraint-induced fracture. The report begins with a review of fundamental principles of ductile behavior of steel structures and the effects of constraint and stress triaxiality. A brief history of constraint-induced fractures of steel bridges in the United States and a review of published research, policies, and practices is also provided. The report then presents a possible method for evaluating a steel detail for the presence the three conditions associated with elevated susceptibility to constraint-induced fracture: high tensile stresses (including residual stress effects), a high degree of constraint, and planar discontinuities approximately perpendicular to the primary flow of tensile stresses. Next, a series of commonly used steel bridge details are evaluated to illustrate the procedure and to provide a baseline library of evaluations. Redesign, inspection, retrofit, and repair options for problematic details are briefly discussed. The report also presents general design details and construction considerations and possible future research topics.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ABBREVIATIONS AND SYMBOLS

AASHTO	American Association of State Highway and Transportation Officials
API	American Petroleum Institute
AREMA	American Railway Engineering and Maintenance-of-Way Association
ASTM	ASTM International
AWS	American Welding Society
BCS	AASHTO LRFD Bridge Construction Specifications
BDS	AASHTO LRFD Bridge Design Specifications
BIRM	Bridge Inspector's Reference Manual
Bridge Welding Code	AASHTO/AWS Bridge Welding Code (D1.5M/D1.5)
CIF	constraint-induced fracture
CJP	complete joint penetration
CVN	Charpy V-notch
DOT	Department of Transportation
FCM	fracture-critical member
FCP	fracture control plan
FEM	finite element method
FHWA	Federal Highway Administration
ksi	kilopounds per square inch
LRFD	Load and Resistance Factor Design
NBIS	National Bridge Inspection Standards
NHI	National Highway Institute
NSBA	National Steel Bridge Alliance
OBS	Office of Bridges and Structures
PJP	partial joint penetration

CHAPTER 1 - EXECUTIVE SUMMARY

Historically, reports of significant problems associated with details featuring intersecting welds in steel bridges have been rare. However, there have been several notable cases involving constraint-induced fracture (CIF). CIF is a particular concern since it can occur in a brittle fashion, suddenly and without warning (different from other types of problems such as corrosion or fatigue crack growth, for example). CIF generally occurs in details that feature a high degree of constraint (leading to a high level of stress triaxiality), in combination with high levels of tensile stress (including residual stresses) and a notch-like or crack-like planar discontinuity approximately perpendicular to the primary flow of tensile stress. Details subjected to a high degree of constraint often feature the intersection of two or three welded structural steel elements. The distinction between "intersecting welds" and "constraint resulting from the intersection of welded structural elements" is important.

Bridges featuring certain types of details with intersecting welded steel elements may be subjected to an increased susceptibility to CIF. In extreme cases, details with high degrees of triaxial constraint and crack-like or notch-like planar discontinuities have experienced sudden, severe fractures, resulting in bridge closures and emergency repairs. There have been several cases of CIF in bridges in the United States, including most notably the Hoan Bridge fracture in Wisconsin on December 13, 2000. In the case of the Hoan Bridge, CIF occurred after the bridge had been in service for over 25 years and resulted in the nearly full-depth fracture of two of the three main girders in one of the approach spans. This prompted an emergency closure of the bridge, which carries six lanes of interstate highway traffic.

Since the Hoan Bridge fracture, research has improved the general understanding of CIF. To provide a better understanding among designers and bridge owners of constraint, CIF, and proper detailing of steel bridges with welded elements, the Federal Highway Administration (FHWA) sponsored the creation of this report. The report is based on a review of current research and practices and the input of a panel of steel-bridge industry experts, including academic researchers, bridge design engineers, steel bridge fabricators, and bridge owners.

The findings in this report are:

- Steel bridge details featuring intersecting welds are not necessarily at elevated susceptibility to CIF.

- Three conditions typically contribute to elevated susceptibility of steel bridge details to CIF: a high net tensile stress, a high degree of constraint, and a planar discontinuity approximately perpendicular to the primary flow of tensile stress.

- Evaluating details with respect to criteria rooted in a technical understanding of CIF can help bridge owners identify details that are candidates for redesign and retrofit.

- Retrofitting and redesigning details with intersecting welds without proper understanding of CIF can lead owners to undertake design and/or retrofit strategies that may result in poorer, not better, performance.

- The bridge community may benefit from:

    - Clarification of the term intersecting welds and the development and use of different terms to describe problematic details.

  - Clarification of the influence of intersecting welds on the behavior and performance of steel bridges.

  - Clarification of the difference between details with intersecting welds and details that are subject to an elevated susceptibility to CIF.

  - Clarification of the minimum width for constraint-relief gaps, including consideration of anticipated fracture and fatigue performance.

  - Education regarding the relative effectiveness of constraint-relief gaps along with other measures that can reduce susceptibility to CIF.

- The minimum width of the constraint-relief gaps (i.e., the gaps between weld toes and/or ends) currently prescribed in the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)) and other practice documents is based on a limited analytical study that considered only ½-inch-thick webs and only 0-inch and ¼-inch gaps between weld toes and/or ends.

- Regarding specific details:

  - Connecting lateral connection plates (lateral bracing gusset plates) directly to the girder web provides indirect, inefficient load paths, and in some situations can result in elevated susceptibility to CIF. Lateral bracing gusset plates can instead be connected directly to the girder flanges.

  - There are unresolved concerns about the degree of stress triaxiality and susceptibility to CIF associated with large, thick bearing stiffeners provided at interior supports (negative moment regions) of multi-span continuous steel girder bridges – a plausible explanation of reported fractures at these locations is available, but might benefit from a more comprehensive research study.

  - The implementation of seal weld detailing for transverse stiffeners, transverse connection plates, and bearing stiffeners offers a potential for improved corrosion protection.

  - Implementation of such sealing weld detailing for coped stiffeners would benefit from a more thorough study of the appropriate size of constraint-relief gaps.

  - Implementation of such sealing weld detailing for non-coped stiffeners (featuring continuous welding to attach transverse stiffeners, transverse connection plates, or bearing stiffeners to girder flanges and webs) would benefit from more thorough study of the susceptibility to CIF of this type of detailing and from study of the maximum permissible gaps between the corners of the stiffener and the flange-to-web welds and of the welding details that would be used for such a connection.

The report describes a suggested evaluation procedure that considers the presence of the three conditions associated with elevated susceptibility to CIF. Several commonly used steel bridge details are presented to demonstrate the suggested procedure.

CHAPTER 2 - BACKGROUND, OUTLINE, AND PURPOSE

2.1 BACKGROUND

Historically, reports of significant problems associated with details featuring intersecting welds in steel bridges have been rare. However, there have been several notable cases involving CIF (see Section 4.1). These problems have generally been associated with details that feature a high degree of constraint (leading to a high level of stress triaxiality), in combination with high levels of tensile stress (including residual stresses) and a notch-like or crack-like planar discontinuity approximately perpendicular to the primary flow of tensile stress. The constraint observed in these situations has often been associated with the intersection of two or three intersecting welded structural steel elements. However, the findings of this report suggest that the presence of intersecting welds, in itself, is not necessarily indicative of susceptibility to CIF.

One goal of this report is to provide an understanding of the factors that could lead to elevated susceptibility to CIF in details. Another goal is to provide a suggested method to evaluate susceptibility to CIF, facilitating identification of details that might need redesign or retrofit.

The report is based on a review of current research and practices and the input of a panel of steel-bridge industry experts, including academic researchers, bridge design engineers, steel bridge fabricators, and bridge owners.

2.2 OUTLINE

Chapter 2 briefly describes the background of this report, a short discussion of its purpose and scope, and various terms used in it. Chapter 3 presents basic concepts of stress triaxiality, constraint, and increased susceptibility to constraint-induced fracture (CIF). Chapter 4 presents a brief synthesis of available research and current practices. This includes a history of CIF of steel bridges in North America and the response of the bridge community to those incidents, a summary of the literature review conducted, and summaries of published research and practice.

Chapter 5 presents a general, nonregulatory procedure for evaluating steel bridge details for an increased susceptibility to CIF, a discussion of fatigue versus fracture, a discussion of concerns about details with intersecting welds, and a discussion of conditions where intersecting welds can be designed and used effectively. Following this, a series of examples of commonly used steel bridge details are assessed using the suggested CIF evaluation procedure. Then, Chapter 6 discusses measures that can be taken to mitigate an elevated susceptibility to CIF.

Chapter 7 summarizes the report findings. Chapter 8 presents design, detailing, and construction considerations, lists potential future research topics, and outlines the conclusions.

Appendix A presents a literature review. Appendix B summarizes current owner-agency policies and practices. Finally, Appendix C discusses stiffener seal-welding mock-up fabrication trials.

2.3 PURPOSE AND SCOPE

This report intends to explain and clarify steel bridge detailing concerns associated with intersecting welds versus triaxially intersecting welded steel elements, and to provide a better understanding of constraint, CIF, and proper detailing of steel bridges to avoid an elevated susceptibility to CIF.

The report focuses on steel-girder highway bridges, but has some relevance to steel arch or truss bridges, steel railway bridges, and other transportation structures constructed of structural steel since the fundamental principles presented herein are broadly applicable.

The scope of the report is further limited to the existing body of knowledge in the steel industry. No new research was performed for this report. The current state of knowledge was synthesized from the following sources:

- a literature review of previously published research;
- a literature review of various owner-agency and industry practice documents;
- a review of current practices; and
- a full-day meeting of a panel of steel-bridge industry experts representing academic researchers, bridge design engineers, steel bridge fabricators, and bridge owners.

This report provides the following:

- a synthesis of current literature, research, and practices;
- explanation of the current state of knowledge related to the performance of steel bridge details that may feature intersecting welds and intersecting welded elements;
- a general procedure for quantifying the susceptibility of steel bridge details to CIF; and
- examples of application of the CIF susceptibility procedure.

To better understand the concept of CIF, it is helpful to first understand some basic concepts of stress triaxiality. Having a clear understanding of stress triaxiality assists in developing a better understanding of triaxial constraint, which in turn leads to a better understanding of CIF. To this end, later sections include reviews of these fundamental concepts.

The report also explores the behavior of details with intersecting welds, illustrates examples where they may be used successfully, and discusses their potential susceptibility to CIF.

The report also identifies areas where the current state of knowledge may be lacking and potential research topics that could advance the state of knowledge in these areas.

2.4 TERMINOLOGY

2.4.1 Intersecting Welds

The American Welding Society (AWS) specifications such as the voluntary AWS A3.0 (AWS, 2010), the voluntary AWS D1.1 (AWS, 2015), and the binding AASHTO/AWS D1.5

(AASHTO/AWS, 2015) (23 CFR 625.4(d)(1)(vii)) do not describe "intersecting welds," but a variety of descriptions of the term "intersecting welds" are presented in other bridge design and bridge inspection practice documents. These descriptions are typically provided in the context of classification of "problematic details," "details susceptible to fatigue," or "details susceptible to fracture."

Perhaps the most common description takes a form similar to this: "welds that run through each other, overlap, touch, or have a gap between their toes of less than ¼ inch" (Ryan et al., 2010). However, this description can be misunderstood when used in the context of evaluating whether a given detail may or may not be problematic. The inclusion of the measurement of "a gap between their toes of less than ¼ inch" implies there is a measurable criterion for characterizing whether a detail has "intersecting welds." Such a criterion, on its own, typically is insufficient for evaluating the susceptibility of a detail to CIF.

Such descriptions might lead an engineer to believe that details with welds that run through each other, overlap, or touch are problematic, and that the introduction of a gap between weld toes of at least ¼ inch should alleviate the situation. However, consider the example of the intersection of flange-to-web fillet welds with a complete joint penetration (CJP) groove weld in a butt joint for a flange or web shop splice. Such a detail would fall under the above-cited description of "intersecting welds." Yet, these types of details have been used extensively in steel bridge fabrication without concerns or reported problems.

To more clearly separate "intersecting welds" from "details subject to an elevated susceptibility to CIF," it would be helpful to consider the term "intersecting welds" as only identifying a condition where welds run through each other, overlap, or touch. The term "constraint-relief gaps" (i.e., the "gap between [weld] toes," or "web gap"), including their measurement and their effect on performance, can then be differentiated from the term "intersecting welds" and instead used as part of a more comprehensive evaluation of details for susceptibility to CIF.

To more explicitly identify the geometry of these types of details, the following terminology is used in this report:

> ***Intersecting welds:*** Welds that run across each other, overlap, or touch.

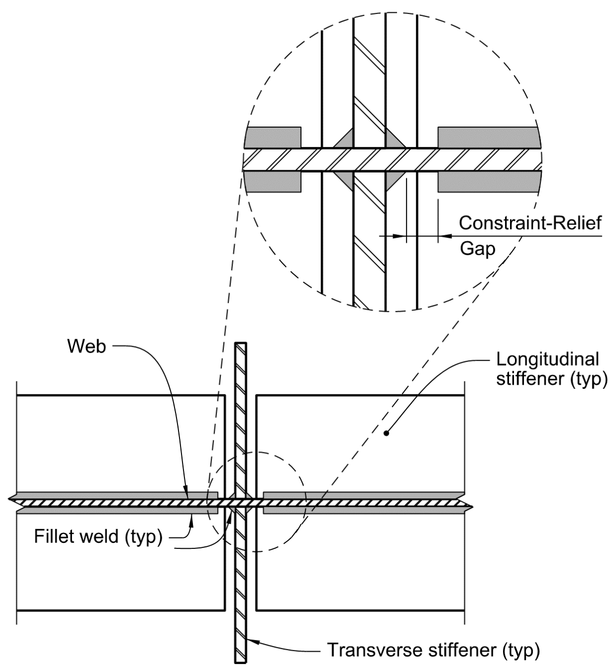
The presence of intersecting welds in and of themselves does not necessarily represent the presence of a problematic detail.

2.4.2 Constraint-Relief Gaps

In previous literature related to CIF, the words "web gap" and "gap between weld toes" were used to denote gaps provided in one element welded to and constraining another element; these gaps are intended to provide relief from triaxial constraint in the constrained element, enabling that element to yield. However, these descriptive terms, which are not binding under FHWA regulations, can be the subject of various interpretations, which might lead to confusion.

A common historical example of this type of constraint-relieving gap is an interruption in a longitudinal stiffener welded to a girder web at the intersection with a vertical stiffener welded to the same web, where the gap in the stiffener is measured at the web (hence the historical term “web gap”). See Figure 1 for an illustration; the dimension in the figure denoted as the “constraint-relief gap” (a term described later in this report) is the “web gap.” Note that the type of detailing shown in this figure can potentially exhibit elevated susceptibility to CIF (as explained later in this report), but may be found in older structures. This so-called “web gap” provides the web with relief from triaxial constraint. However, the term “web gap” has been described as confusing by some, and historically different dimensions have been used.

Furthermore, a more general term would be useful since providing these types of gaps may be beneficial in details other than longitudinal web stiffeners. For the purposes of this report, the term “constraint-relief gap” is used. A constraint-relief gap is an interruption in a welded structural element to provide some measure of relief from triaxial-constraint induced by that element on an attached element. To properly provide relief from triaxial constraint, the gap provided in a constraint-relief gap should be a “clear” gap; as such, it has traditionally been measured as the gap between the toes and/or ends of the welds connecting the constraining element(s) to the constrained element.



Source: FHWA

Figure 1. Illustration. Plan view of girder web with attached transverse and longitudinal stiffeners.

To more explicitly identify the geometry of these types of details, the following terminology is used in this report:

> ***Constraint-relief gap:*** An interruption, of sufficient size, provided in a welded structural element, or its connection to a constrained element, to provide localized relief from constraint induced by that element on a constrained element to which it is attached, so that local yielding can occur.

The dimension describing the size of a constraint-relief gap is measured between the toes and/or ends of the welds attaching the constraining element to the connected, constrained element.

2.4.3 List of Terms

This section provides a list of terms used in this report. Unless otherwise specified, the following terms are not binding under FHWA regulations.

Bearing stiffener: An angle or angles, or a plate or plates, attached to a web of a beam or girder to distribute a bearing reaction or a concentrated load into the web over the height of the stiffeners.

Constraint-induced fracture (CIF): “A type of fracture attributed to local constraint conditions in steel under tension, which may occur at details of certain geometries.” (Russo et al., 2016)

Constraint-relief gap: An interruption, of sufficient size, provided in a welded structural element, or its connection to a constrained element, to provide relief from constraint induced by that element on a constrained element to which it is attached, so that local yielding can occur.

Clip: See *cope*.

Cope: A cutout in a structural steel member to avoid physical conflict with part of another element. Also known as a snipe or clip.

Crack: A fracture-type discontinuity characterized by a sharp tip and high ratio of length and width to opening displacement.

Crack-like Geometry: A geometric condition in a steel structure featuring a discontinuity in an element, in which the discontinuity has very sharp tips that would be expected to introduce very significant stress concentrations.

Fracture: A partial or total severing of a continuous steel element under the action of force, particularly a tensile force, without prior yielding or deformation.

Intersecting welds: Welds that run across each other, overlap, or touch.

Lateral connection plate: A plate used to interconnect lateral bracing members for attachment to a flexural member (such as a girder).

Longitudinal web stiffener: A stiffener, oriented in a direction at least approximately parallel to the primary flow of axial or flexural stress, attached to a component plate of a member to provide additional local and overall compressive resistance of that component.

Notch-like Geometry: A geometric condition in a steel structure featuring a discontinuity in an element, in which the discontinuity may not have very sharp tips, but in which the discontinuity is nonetheless relatively narrow and the tips would still be expected to introduce significant stress concentrations.

Planar Discontinuity: A geometric condition in a steel structure taking the form of a plane of discontinuity in an otherwise continuous structural steel element, typically featuring a crack-like or notch-like geometry. See also *crack-like geometry* and *notch-like geometry*.

Snipe: See *cope*.

Stiffener: A member, usually an angle or plate, attached to a plate or web of a beam or girder to distribute load, to transfer shear, or to prevent buckling of the member to which it is attached.

Stress Triaxiality: "The ratio of the state of stress a material undergoes to the stress that contributes to yielding" (Schafer, 2000).

Transverse connection plate: A vertical stiffener attached to a beam or girder to which a cross-frame, diaphragm, floor beam, or stringer is connected.

Transverse stiffener: A stiffener attached to a component plate approximately perpendicular to the longitudinal axis of the member to provide additional shear or axial compressive resistance.

Web gap: A particular type of constraint-relief gap, specifically in an element attached to, and otherwise constraining, the web of a flexurally or axially loaded steel member.

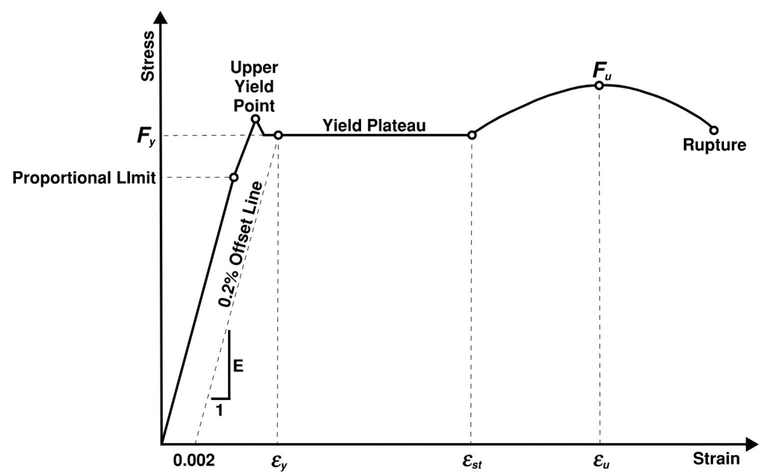
CHAPTER 3 - STRESS TRIAXIALITY, CONSTRAINT, AND SUSCEPTIBILITY TO CIF

3.1 FUNDAMENTAL PRINCIPLES OF DUCTILE BEHAVIOR OF STEEL STRUCTURES AND THE EFFECTS OF CONSTRAINT AND STRESS TRIAXIALITY

While it has often been said that steel is an inherently ductile material, that ductile nature can be compromised if a structure is detailed in manner that inhibits the typical stress-strain behavior of the material. Clarification of this concept is instructive in understanding the nature and causes of CIF.

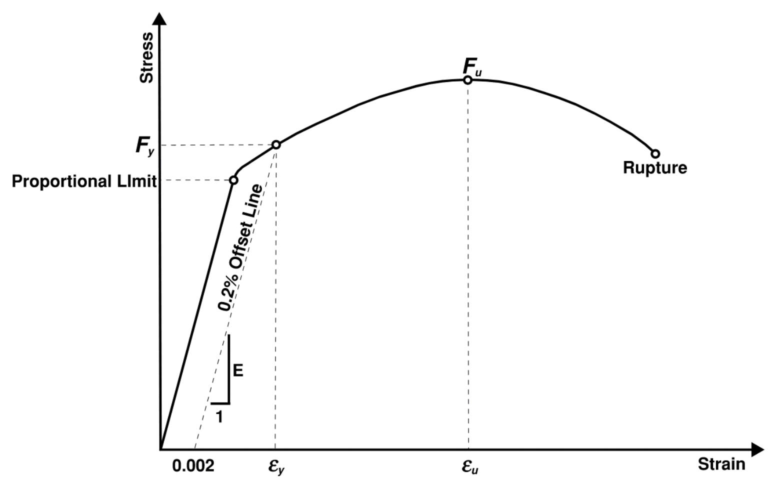
The basis for most statements about the inherent ductility of steel is the nature and shape of the basic stress-strain curve of the material, as established by uniaxially loaded tensile specimens. The stress-strain curve for steels generally exhibits a region of significant plastic deformation prior to rupture or fracture. Bridge steels with a minimum specified yield stress of 70 kilopounds per square inch (ksi) or less (typically 36, 50, and 70 ksi) generally exhibit a defined yield plateau (see Figure 2 for a stress-strain curve generally representative of this type of behavior). The stress-strain curve for Grade HPS 100W bridge steel (which has a yield stress of 100 ksi), does not display a clearly defined yield plateau (see Figure 3 for a stress-strain curve generally representative of this type of behavior), but does exhibit significant plastic deformation prior to rupture.

Figure 2 and Figure 3 show that significant plastic deformation may occur under loading between the uniaxial yield stress, F_y, and the ultimate tensile strength of the material, F_u, with further deformation occurring prior to rupture. This plastic behavior generally results in significant structural deformation prior to reaching the ultimate tensile strength of the material, providing warning of an impending failure. This is generally characterized as “ductile behavior;” that is, the material displays significant ductility prior to failure.



Source: FHWA

Figure 2. Graph. Engineering tensile stress versus strain curve for structural steel with a defined yield plateau.



Source: FHWA

Figure 3. Graph. Engineering stress versus strain curve for structural steels without a defined yield plateau.

This type of ductile behavior of steel materials depends on a variety of presumptions, including the application of uniaxial loading at a slow loading rate and the absence of significant residual stresses and stress concentrations. Furthermore, unaxial loading, by its nature, involves the application of stress in only one direction. If the configuration and loading of the structure result in the application of stresses in more than one direction, a biaxial or triaxial state of stress would exist. In that case, the material would exhibit different behavior. In particular, in the case of a triaxial state of stress, the material can be prevented from plastically deforming (yielding). In such cases, the material can be subjected to a tensile stress equal to the rupture stress without having yielded. The presence of residual stresses and/or stress concentrations, especially extremely high stress concentrations resulting from notch-like or crack-like planar discontinuities, can also produce highly localized tensile stresses, exacerbating the situation. The result can be a sudden, brittle failure by fracture. This type of failure is commonly called CIF.

3.1.1 Illustrations of Ductility via Mohr's Circle of Stress

To better understand this concept, it is helpful to review Mohr's circle of stress and the basic concepts of ductility.

It has long been known that to achieve plastic deformation (yielding) of metal materials, the materials have to be able to experience shear stresses and the ability to deform along shear planes. For example, Gensamer (1941) stated, "This is an important concept and needs to be emphasized: no shear stress, no plastic deformation or flow." At a more fundamental level Bruneau et al. (1998) explain, "Steel is a polycrystalline material, that, when loaded beyond its elastic limit, develops slip planes at 45 degrees. These visible yield lines, also known as Lüder lines, are a consequence of the development of slip planes within the material as yielding develops." In other words, shear stresses are associated with yielding. Conversely, if the development of shear stresses is somehow prevented, then yielding cannot occur and the failure mode changes to rupture without any prior measurable ductility.

Implicit in these statements is that the metallic element is free from triaxial constraint, so as to allow the development of the shear stresses essential for yielding. The underlying concepts associated with this statement can be illustrated via Mohr's circle of stress, which is used below to illustrate the effects of constraint on the behavior of a steel element subjected to an axial tension stress.

Consider a typical steel tension test coupon, as shown in Figure 4.

Source: FHWA

Figure 4. Photo. Typical steel tensile test coupon.

When subjected to a uniaxial tension stress, with the orthogonal stresses equal to zero, Mohr's circle of stress for an element stressed like the test coupon shown in Figure 4 can be drawn as shown in Figure 5. The stress in the x-direction, σ_x, is the applied uniaxial tension stress. The stress in the y-direction, σ_y, is zero, since there is no applied orthogonal stress or constraint. There is also a shear stress occurring in the material, $\tau_{x\text{-}y}$, as is demonstrated when the statics of a discrete element in the test coupon are evaluated.

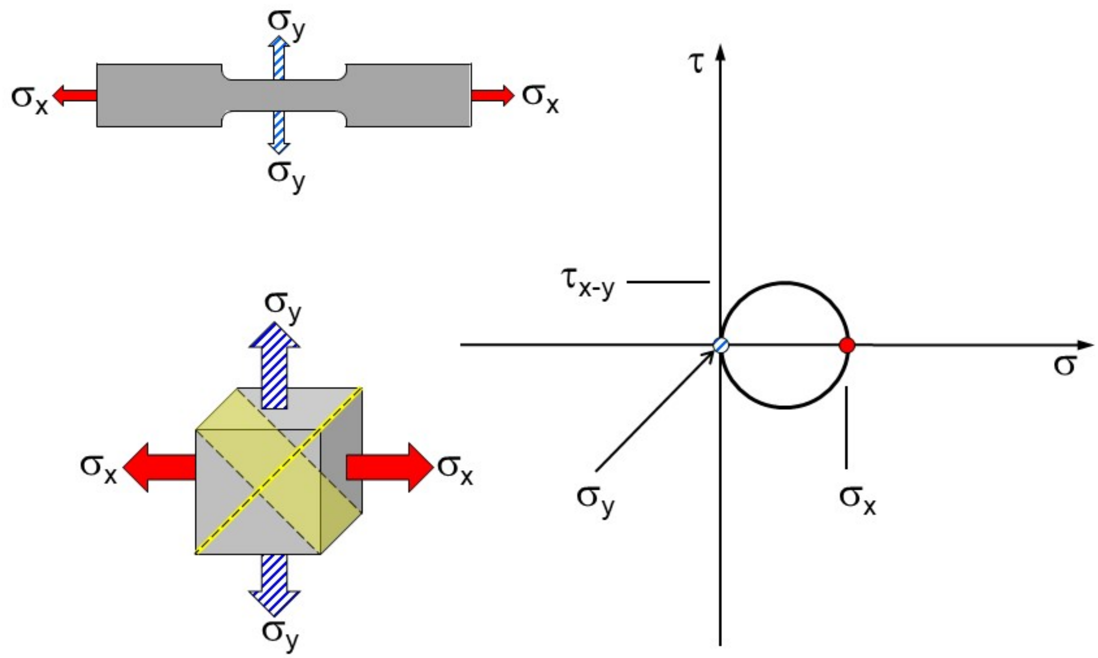


Figure 5. Graph. Mohr's circle of stress for a uniaxial test coupon with stress in the x-direction and zero stress in y-direction (modified by the authors; labels added to stress arrows and graph axes).

A similar Mohr's circle of stress can be drawn considering the stresses in the x- and z-directions. The stress in the x-direction, σ_x, is the applied uniaxial tension stress. The stress in the z-direction, σ_z, is zero, since there is no applied orthogonal stress or constraint. There is also a shear stress in the material, $\tau_{x\text{-}z}$, as is demonstrated when the statics of a discrete element in the test coupon are evaluated. See Figure 6.

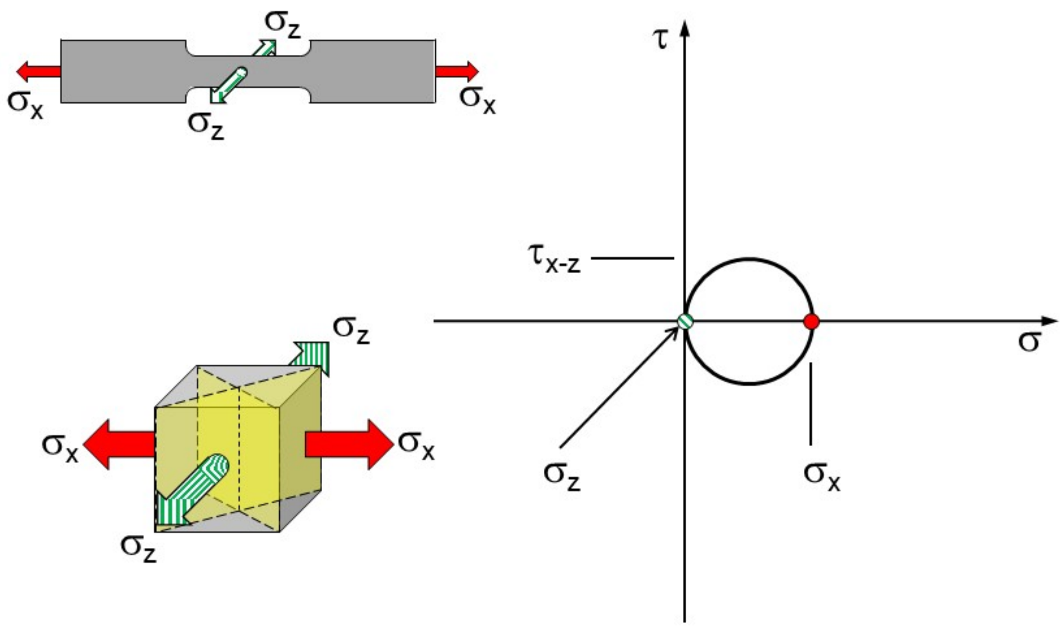


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Figure 6. Graph. Mohr's circle of stress for a uniaxial test coupon with stress in the x-direction and zero stress in the z-direction (modified by the authors; labels added to stress arrows and graph axes).

Finally, a Mohr's circle of stress can be drawn for the case of the y- and z-direction stresses. See Figure 7. In this case, since the principal stresses, σ_y and σ_z, are both zero, $\tau_{x\text{-}z}$, is also zero and the Mohr's circle of stress is just a dot.

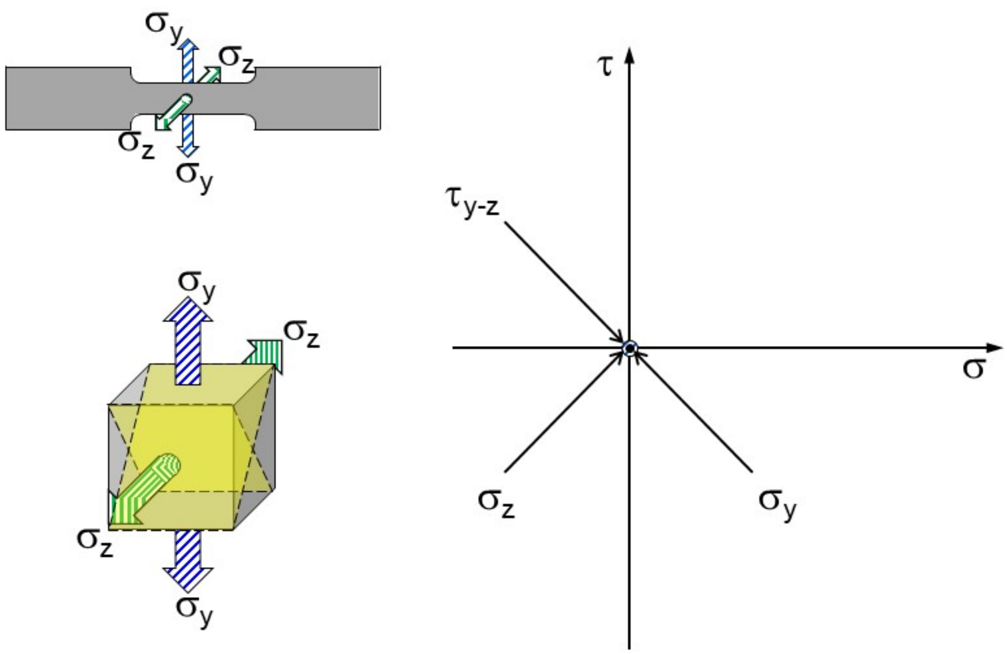

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Figure 7. Graph. Mohr's circle of stress for a uniaxial test coupon with stress in the y- and z-directions (modified by the authors; labels added to stress arrows and graph axes).

All three Mohr's circles of stress (the x- and y-directions, the x- and z-directions, and the y- and z-directions) can be drawn together, noting that σ_y and σ_z are both still zero. Shear is present on two sets of shear planes ($\tau_{x\text{-}y}$, and $\tau_{x\text{-}z}$). See Figure 8.

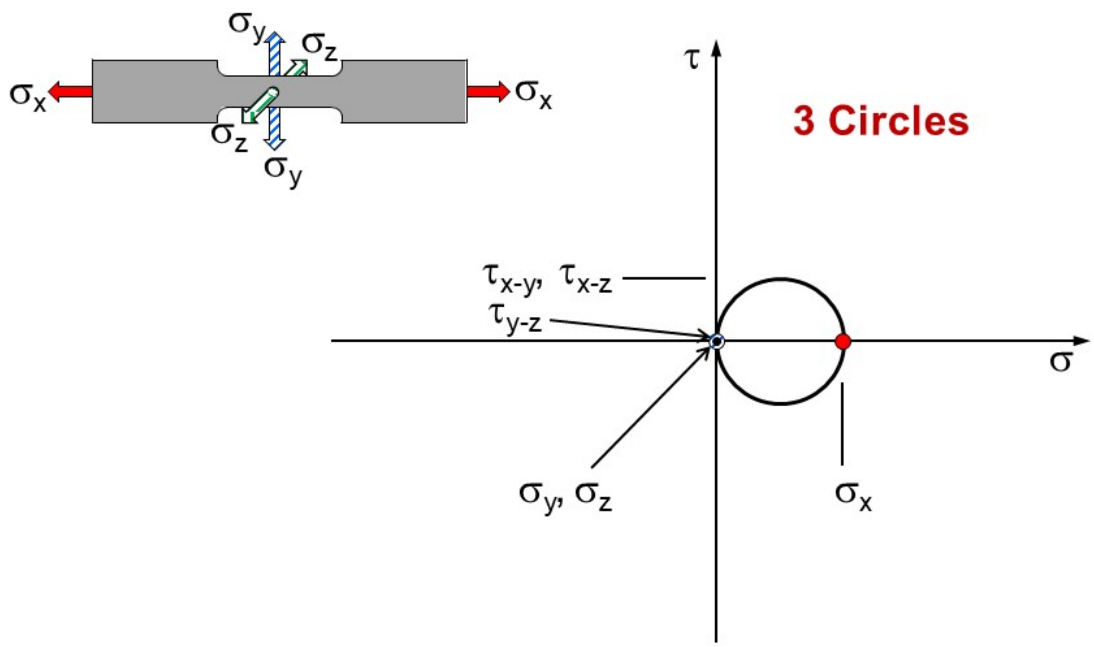


Figure 8. Graph. The three Mohr's circles of stress for a uniaxial test coupon with stress in the x-direction and zero stress in the y- and z-directions (modified by the authors; labels added to stress arrows and graph axes).

With this as a basis, it can be seen that as the uniaxial tension stress, σ_x, is increased from σ_{x1} to σ_{x2}, while σ_y and σ_z are both still zero, the associated shear stresses, $\tau_{x\text{-}y}$ and $\tau_{x\text{-}z}$, also increase proportionally. See Figure 9.

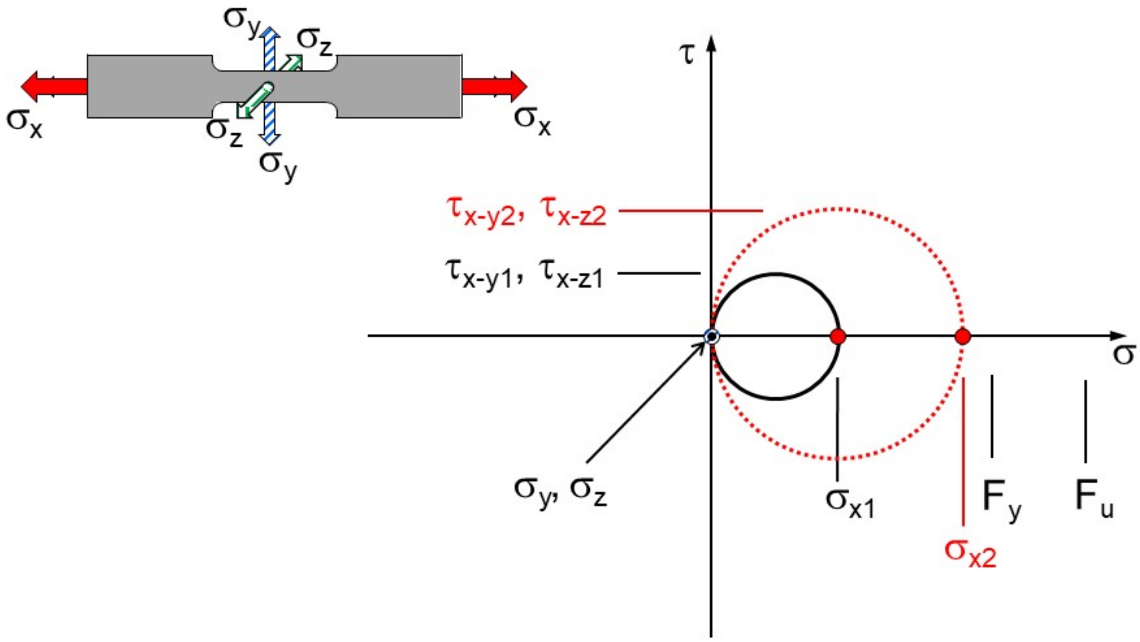


Figure 9. Graph. The three Mohr's circles of stress for a uniaxial test coupon, with higher uniaxial stress (modified by the authors; circle colors changed and labels added to stress arrows and graph axes).

3.1.1.1 Illustrations of Ductile Behavior

When the uniaxial stress, σ_x, is increased to a level greater than the uniaxial yield stress of the material, F_y, shear deformations occur in association with plastic axial deformation. The shear strength of the material in this case can be identified as the shear stress associated with the uniaxial yield stress. This is the “critical shear stress” – the shear stress associated with initiation of slip along the shear plane. The critical shear stress is the shear stress occurring in a uniaxial tension test when loaded to the tension yield stress. In the case of an applied uniaxial stress, this shear plane is oriented 45 degrees from the direction of the applied uniaxial stress. See Figure 10.

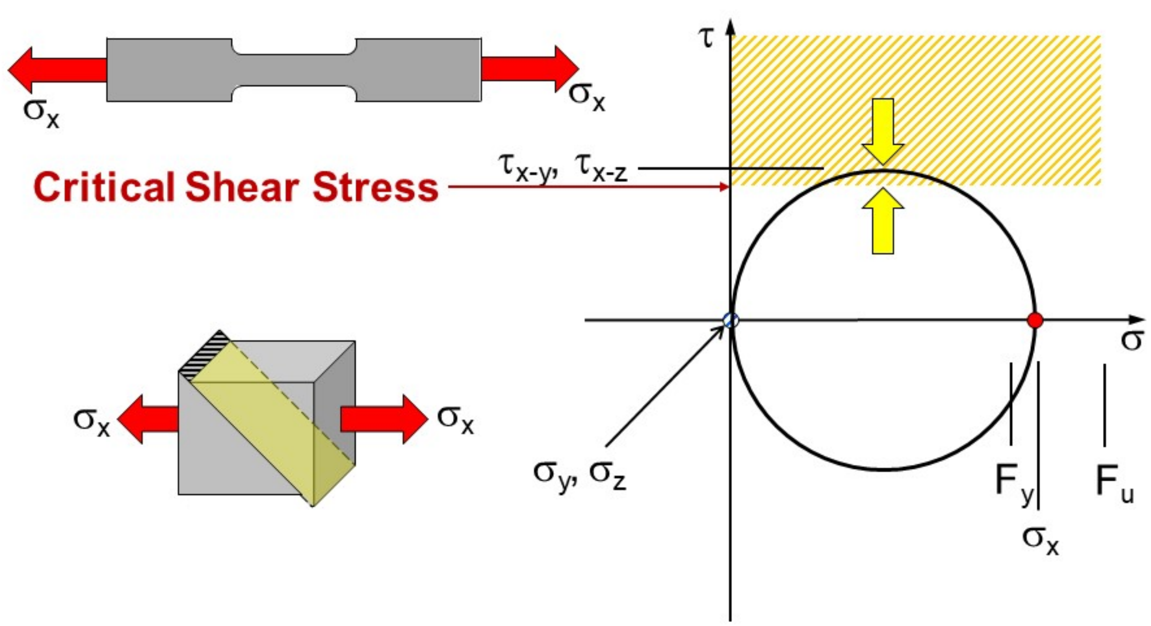


Figure 10. Graph. The three Mohr’s circles of stress for a uniaxial test coupon, showing the shear strength of the material as related to the uniaxial yield stress (modified by the authors; labels added to stress arrows and graph axes).

Now consider a similar test coupon, with an axial tension stress, σ_{x1}, and with an orthogonal compressive stress applied in the y-direction, σ_{y1}. Note in this case that the z-direction stress, σ_z, is zero (but is shown in the figure for completeness). It can be demonstrated that the application of this orthogonal compressive stress in combination with the axial tension stress results in greater shear stress than a uniaxially loaded specimen. See Figure 11.

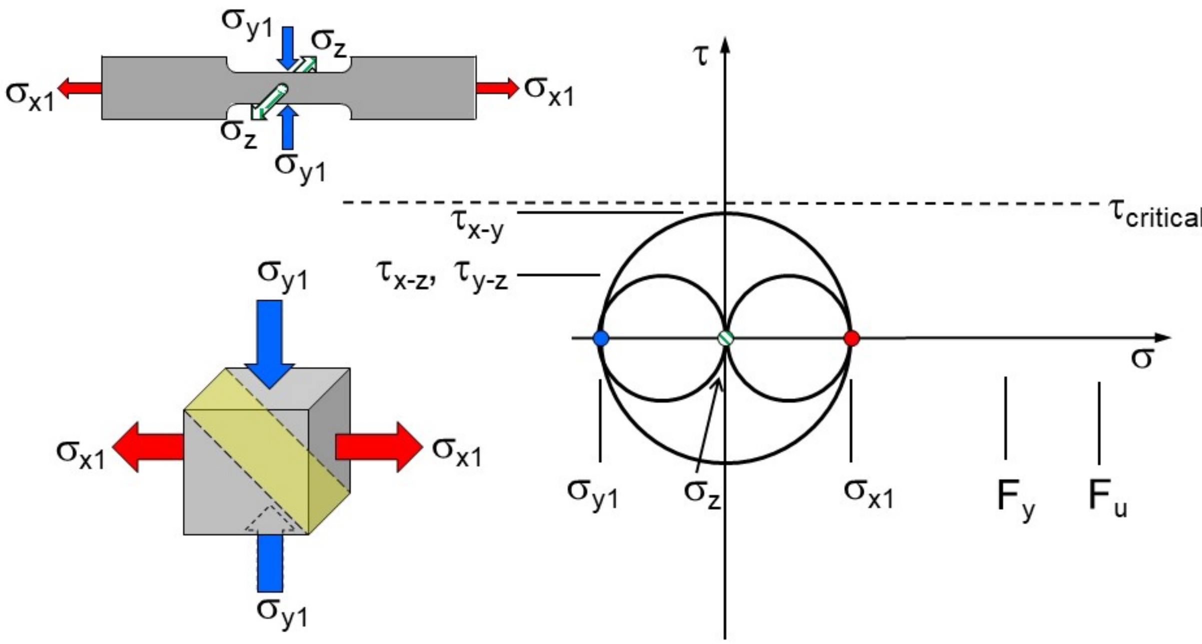


Figure 11. Graph. The three Mohr's circles of stress for a test coupon with axial tensile and orthogonal compression (modified by the authors; labels added to stress arrows and graph axes).

If the axial tension stress increases from σ_{x1} to σ_{x2}, and the orthogonal compression stress increases from σ_{y1} to σ_{y2}, the corresponding shear stresses also increase. The shear stress could exceed the critical value and the material could yield at an applied axial stress value less than that measured in the material's uniaxial tension test. See Figure 12.

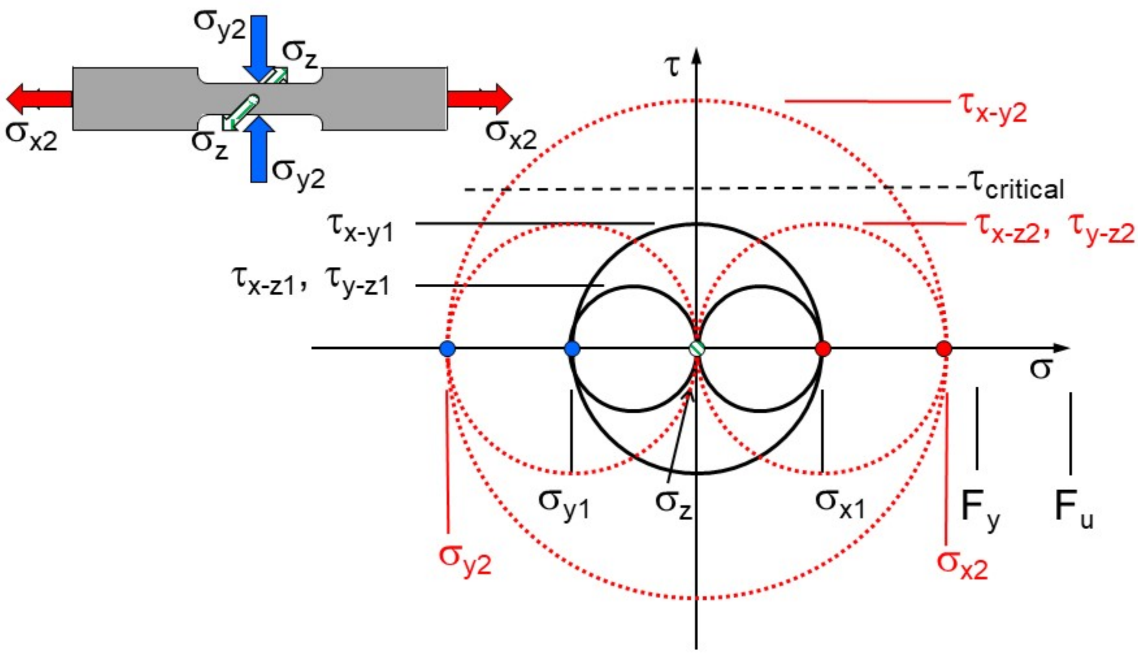


Figure 12. Graph. The three Mohr's circles of stress for a test coupon with increased axial tensile and orthogonal compression (modified by the authors; circle colors changed and labels added to stress arrows and graph axes).

If the applied axial tension stress is increased further, from σ_{x2} to σ_{x3}, and the orthogonal compression stress is also increased, from σ_{y2} to σ_{y3}, the shear stresses correspondingly increase. In fact, the applied axial tension stress can exceed the uniaxial yield stress of the material, but the deformations associated with yielding already started at an applied axial stress less than the uniaxial yield stress as previously discussed and shown in Figure 12. See Figure 13.

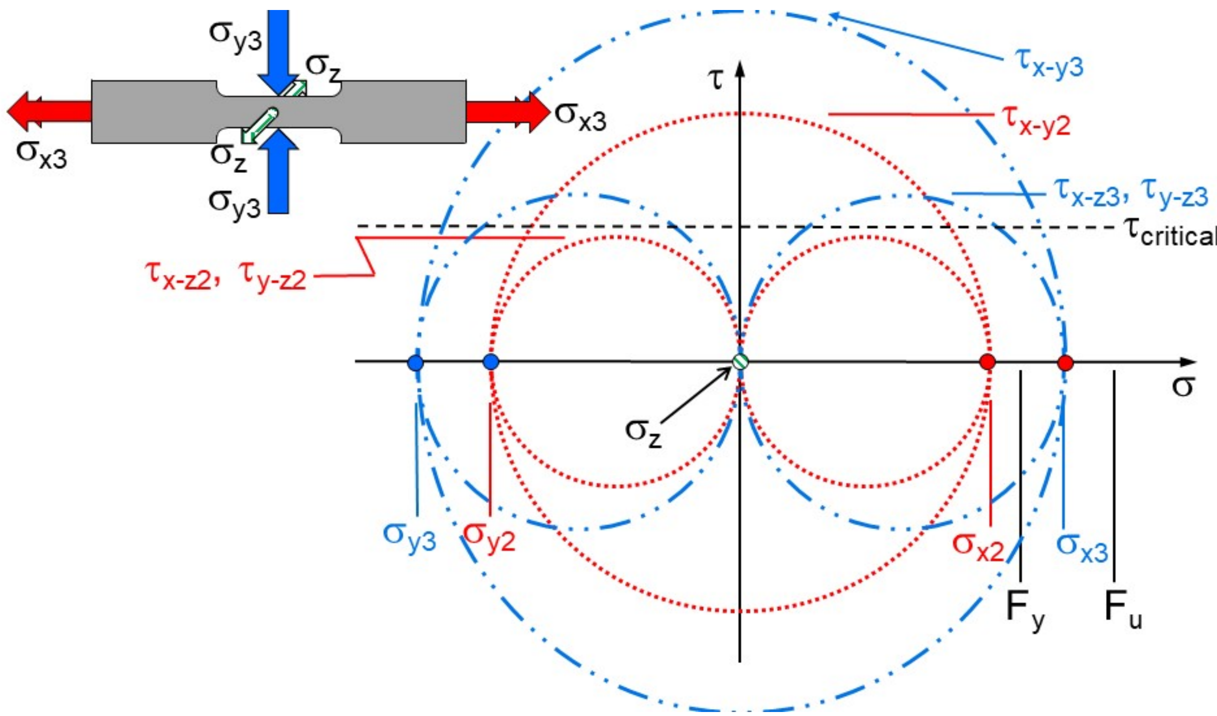


Figure 13. Graph. The three Mohr's circles of stress for a test coupon with further increased axial tensile and orthogonal compression (modified by the authors; circle colors changed and labels added to stress arrows and graph axes).

3.1.1.2 Illustrations of Non-Ductile Behavior

Consider a case where equal tension stresses are applied in all three orthogonal directions (i.e., the x-, y-, and z-directions), $\sigma_{x1} = \sigma_{y1} = \sigma_{z1}$, a hydrostatic state of tension. Following the principles of statics and Mohr's circle of stress, the resulting three Mohr's circles of stress converge to a single dot and the associated shear stresses are zero. See Figure 14. To place this in the context of a real-world situation, consider an element subjected to a tension stress, such as the portion of the web near the tension flange of a steel plate girder subjected to major-axis bending. In such a situation, the x-direction stress would be the major-axis bending stress in web. Now imagine that a vertical stiffener is welded to the web, restraining the web locally in the vertical direction. Assume the vertical stiffener prevents the web from contracting vertically if the x-direction stress in the web exceeds the yield stress; the vertical stiffener represents a vertical constraint on the web and generates a y-direction tension stress when the web tries to yield. Next, also imagine that a longitudinally oriented lateral bracing gusset plate is also welded to the web at the same location as the vertical stiffener. Assume the web, vertical stiffener, and gusset plate are all welded to each other without constraint-relief gaps, that lateral bracing members are attached to the gusset plate, and that cross-frame members are attached to the vertical stiffener. The gusset plate prevents through-thickness yielding of the web; the gusset plate represents a horizontal constraint on the web and generates a z-direction tension stress when the web tries to yield. In conceptual terms, this is similar to the Hoan Bridge detail that suffered from CIF (see Section 4.1.1).

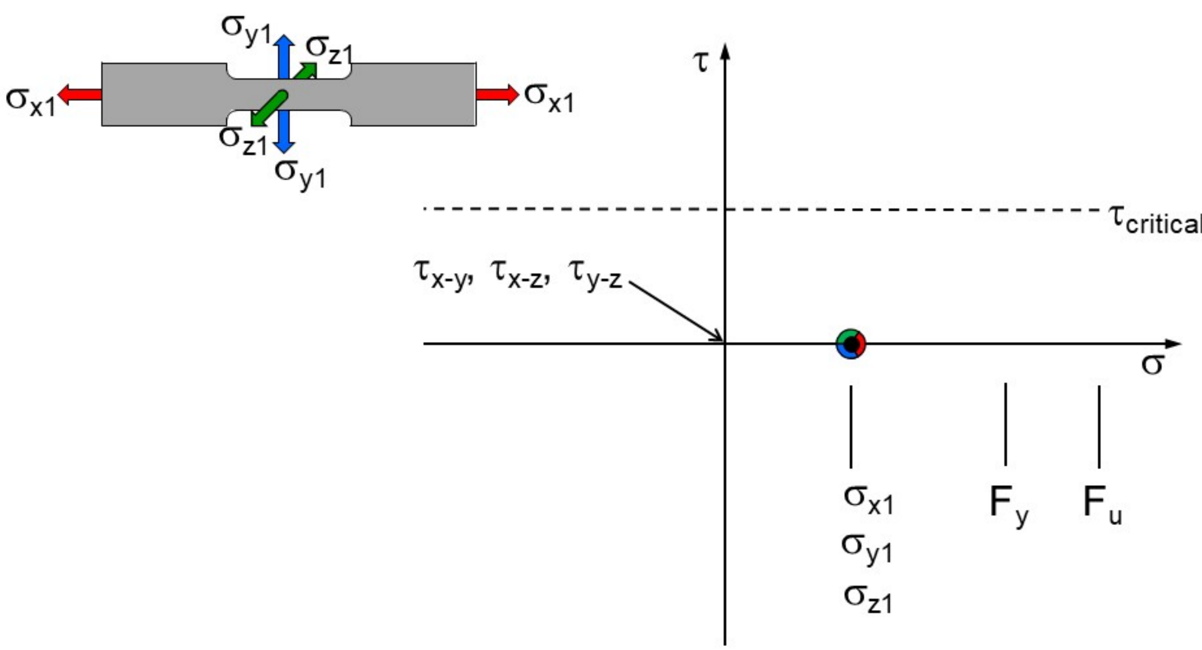


Figure 14. Graph. The three Mohr's circles of stress for a test coupon with equal tensile stresses in all three orthogonal directions (modified by the authors; labels added to stress arrows and graph axes).

If the tension stresses applied in all three orthogonal directions (i.e., the x-, y-, and z-directions) are each increased from σ_{x1}, σ_{y1}, and σ_{z1} to the material's uniaxial yield stress, $\sigma_{x2} = \sigma_{y2} = \sigma_{z2} = F_y$, the resulting Mohr's circle of stress is a still a single dot and the associated shear stresses are still zero. Since there are no shear stresses, there is no deformation (and thus no ductility), even though the axial stress is equal to the uniaxial yield stress. See Figure 15.

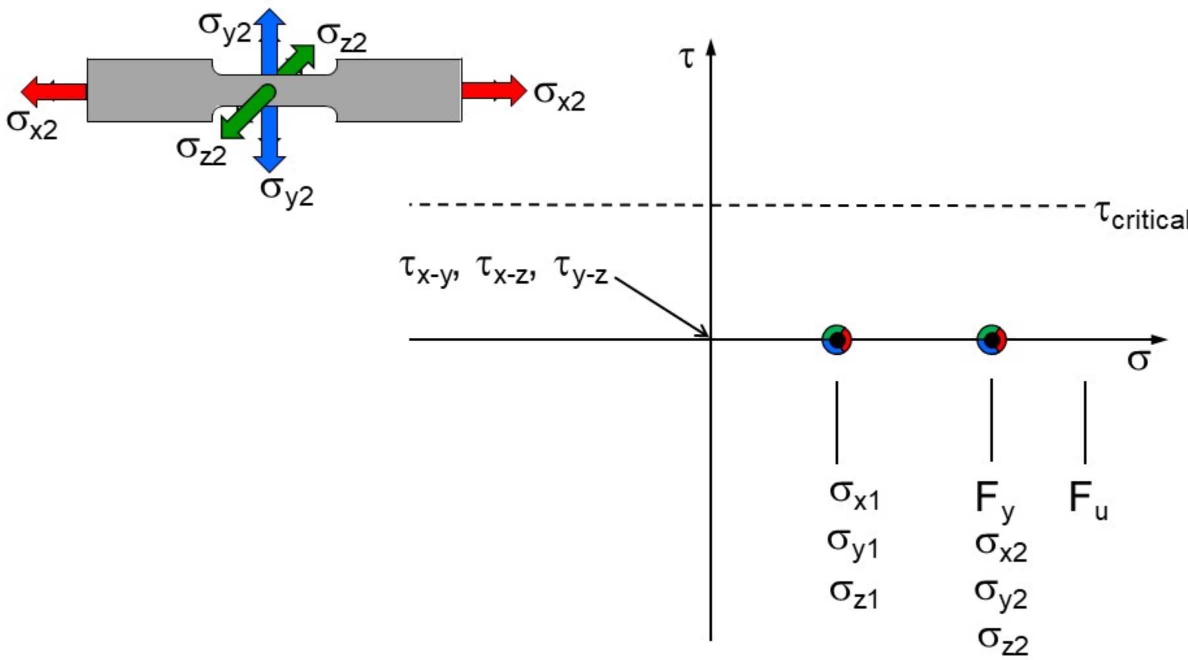


Figure 15. Graph. The three Mohr's circles of stress for a test coupon with tensile stresses in all three orthogonal directions all equal to the material's uniaxial yield stress (modified by the authors; labels added to stress arrows and graph axes).

The orthogonal tension stresses could be increased from the material's uniaxial yield stress, $\sigma_{x2} = \sigma_{y2} = \sigma_{z2} = F_y$, to the ultimate tensile strength of the material, $\sigma_{x3} = \sigma_{y3} = \sigma_{z3} = F_u$, and the associated shear stresses would still be zero. Since there are no shear stresses, there would be no slip along the shear planes, and thus no deformation. In other words, although the test coupon has been stressed beyond the material's uniaxial yield stress, it still has not experienced any plastic deformation; the test coupon could be at the point of rupture and still not yet exhibit any plastic deformation. In a case like this, the fracture would be sudden and brittle. See Figure 16.

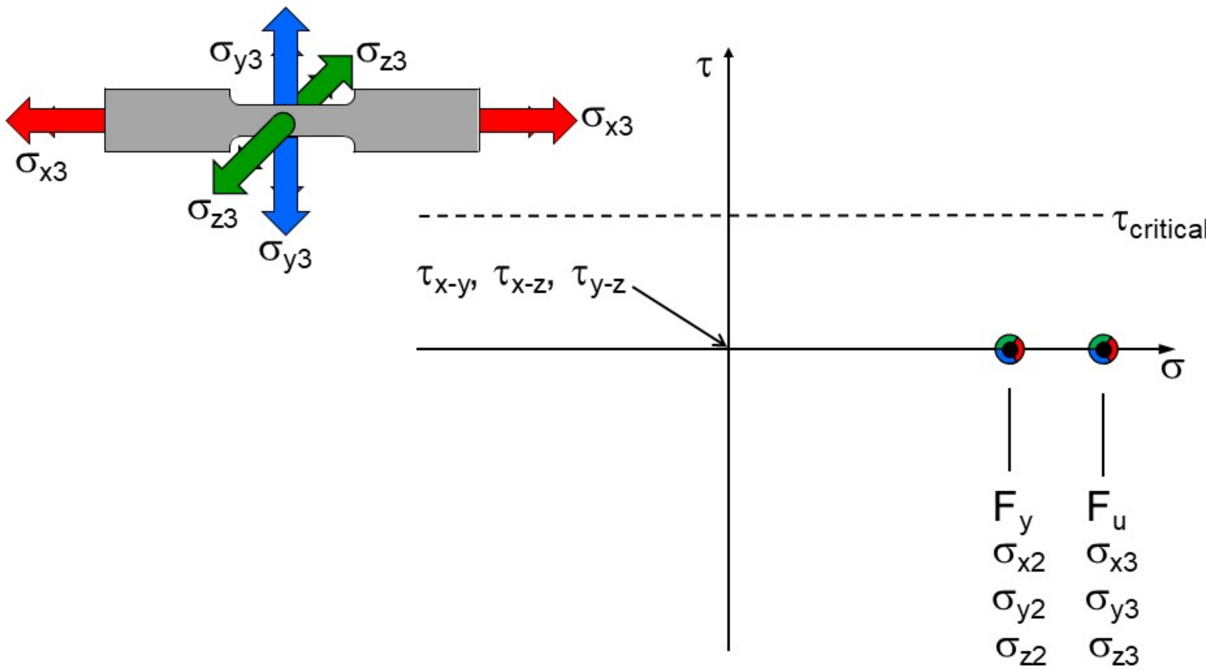


Figure 16. Graph. The three Mohr's circles of stress for a test coupon with tensile stresses in all three orthogonal directions all equal to the material's ultimate tensile strength (modified by the authors; labels added to stress arrows and graph axes).

Figure 16 represents an extreme situation, where the three orthogonal stresses are all increased simultaneously and uniformly to the material's ultimate tensile strength. But similar behavior can occur in less severe cases. Consider a case where the axial tension stress in the x-direction is equal to the material's uniaxial yield stress, with orthogonal tension stresses less than the material's uniaxial yield stress in the y- and z-directions. It can be seen that shear stresses, $\tau_{x\text{-}y}$ and $\tau_{x\text{-}z}$, exist, but that they are of lesser magnitude than they would be if the orthogonal tension stresses, σ_y and σ_z, were zero. In this situation, the x-direction tension stress is equal to the material's uniaxial yield stress, but since the shear stresses are less than the critical shear strength, the test coupon would not exhibit plastic deformation. See Figure 17.

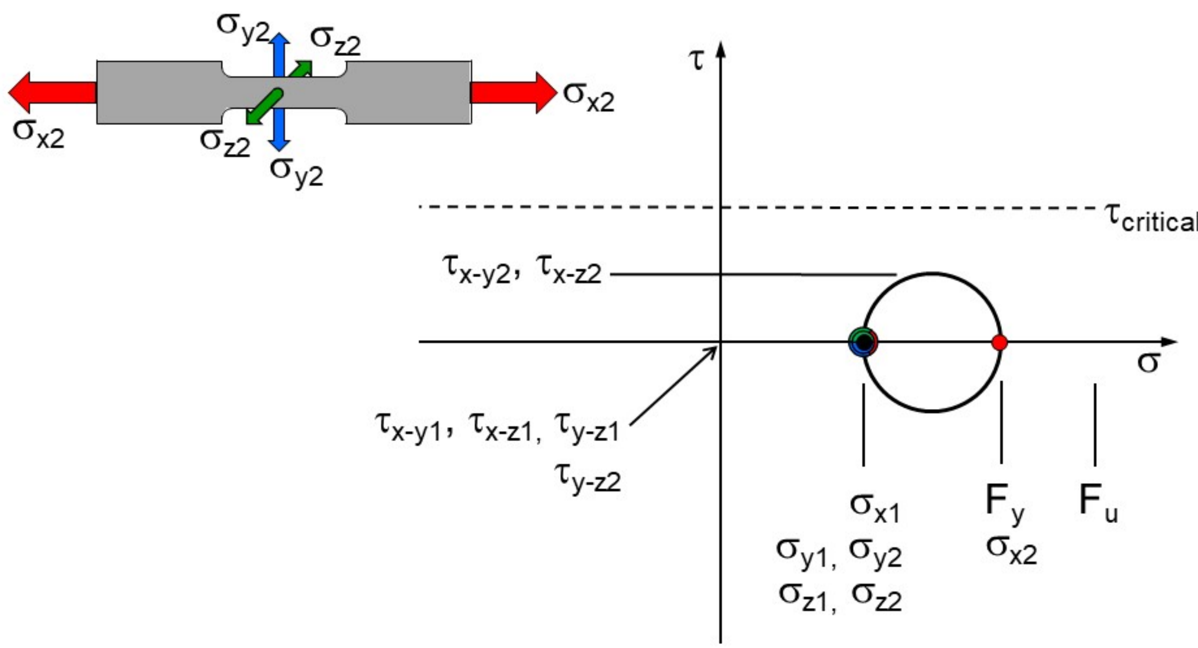


Figure 17. Graph. The three Mohr's circles of stress for a test coupon with tensile stresses in all three orthogonal directions, with x-direction stress equal to uniaxial yield stress (modified by the authors; labels added to stress arrows and graph axes).

If the orthogonal tension stresses in the y- and z-directions stay the same, but the tension stress in the x-direction is increased to the material's ultimate tensile strength, the associated shear stresses would proportionally increase. But depending on the specific magnitudes of the various stresses, it is entirely possible that the shear stresses could still be less than the critical shear stress, and thus, there would still not be any plastic deformation. This means that although the test coupon had been stressed beyond the material's uniaxial yield stress, it still had not experienced any plastic deformation; in fact, if the test coupon were at the point of rupture, it still might not exhibit plastic deformation. In a case such as this, fracture would be sudden and brittle. See Figure 18.

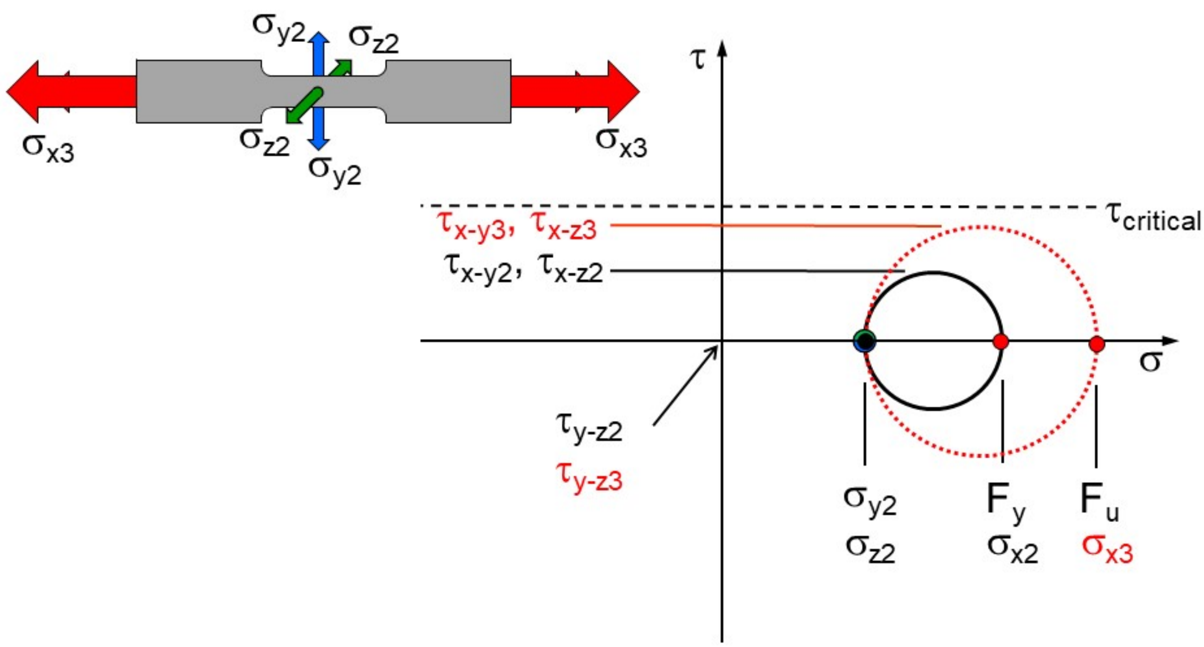


Figure 18. Graph. The three Mohr's circles of stress for a test coupon with tensile stresses in all three orthogonal directions, with x-direction stress equal to ultimate strength (modified by the authors; labels added to stress arrows and graph axes).

These illustrations demonstrate the inherent connection between shear stresses and deformations. This demonstrates that while steel is a material that can exhibit ductility, such behavior is not guaranteed under all circumstances. Furthermore, these illustrations show the link between constraint and fracture. Specifically, when a structural steel element is subjected to triaxial constraint, it can be loaded to a level of stress greater than its uniaxial yield strength and undergo fracture without first experiencing plastic deformation. Figure 16 and Figure 18 illustrate cases where a detail subjected to triaxial constraint could be subject to an elevated susceptibility to CIF.

A more desirable outcome would be for yielding to occur prior to fracture. When a material yields locally, the stress is redistributed to adjacent material and the stress in the yielded section does not immediately continue to elevate to the rupture strength of the material. In addition, in many cases, the plastic deformation associated with yielding is visible and provides an indication of a problem prior to fracture.

Other factors contribute to the ability of a steel structural element to demonstrate ductile or brittle behavior. For example, the inherent toughness of the steel material (i.e., the ability of the material to absorb energy and deform plastically without fracture) affects the material's ductility – tougher steel is more resistant to fracture. Similarly, the temperature of the steel also affects its toughness – the colder the temperatures, the lower the toughness of the material and the less resistant the material becomes to fracture. In older steel bridges, lower toughness steel may naturally be more susceptible to fracture, particularly in low-temperature conditions.

The effects of material toughness and service temperature on the ductility of steel bridges are well-known and have largely been addressed by owners with regard to how they treat older existing bridges and with regard to the design and fabrication of new bridges. However, CIF has occurred in bridges fabricated from steels with good toughness, and has occurred under warm temperature conditions. Good toughness and warm temperatures do not eliminate susceptibility to CIF.

3.2 QUANTIFICATION OF STRESS TRIAXIALITY

Section 3.1 illustrated the fundamental concepts and principles associated with stress triaxiality and the effects of constraint on the behavior of structural steel elements. With these concepts in mind, it is theoretically possible to quantify stress triaxiality and relate those quantifications to performance criteria. Schafer et al. (2000) provides a discussion of triaxiality and presents the following equations that can be used to calculate so-called "triaxiality factors." These factors can be used to evaluate triaxiality in relation to the potential for CIF.

$$T_1 = \frac{\sigma_{hydrostatic}}{\sigma_{eff}}$$

$$T_2 = \frac{\sigma_1}{\sigma_{eff}} = \frac{\sigma_{\max}}{\sigma_{eff}}$$

Where:

$$\sigma_{hydrostatic} = \frac{\sigma_1 + \sigma_2 + \sigma_3}{3}$$

$$\sigma_{eff} = \sqrt{\frac{1}{2}}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_1 - \sigma_3)^2}$$

$$\sigma_1, \sigma_2, \sigma_3 = \textit{principal stresses in the x, y and z direction, respectively}$$

σ_{eff} is also known as the "von Mises Stress."

The T_2 triaxiality factor in particular would be helpful in evaluating the susceptibility of a detail to CIF under a given stress condition. A T_2 factor equal to or greater than the ratio of a material's ultimate tensile strength to its uniaxial yield strength would indicate a situation where the material may fracture at an applied stress equal to or lower than the stress at which the material would otherwise be expected to yield. For example, consider a typical bridge steel such as a Grade 50 steel with a uniaxial yield stress of 50 ksi and an ultimate tensile strength of 65 ksi; the ratio of the material's ultimate tensile strength to its uniaxial yield strength would be approximately 1.3. For a given structural element fabricated with this material, a T_2 factor of 1.3 or greater would indicate a situation where the material may fracture at an applied stress equal to or lower than the stress at which the material would otherwise be expected to yield. With this understanding in hand, a quantitative evaluation of the susceptibility of a detail to CIF can be performed, at least theoretically.

Performing such evaluations on a routine basis is not practical, efficient, or necessary in most cases. Evaluation of the magnitude of the orthogonal stresses can be challenging as these stresses are generally not directly calculated in routine bridge analysis. Also, consideration of residual stresses, stress concentration effects, and tributary area resisting the applied force in an evaluation of stress triaxiality would be involved; but residual stresses are difficult to predict in an accurate manner since they are significantly affected by factors typically beyond the control of the engineer. Furthermore, quantification of stress concentration effects and effective tributary areas involves performing highly refined analyses that are beyond the scope of routine design work.

An alternate approach, particularly for new designs, is to use details that are not subject to triaxial constraint and do not include notch-like or crack-like planar discontinuities approximately perpendicular to the primary flow of tensile stress. Following this practice effectively eliminates one or more of the three conditions associated with elevated susceptibility to CIF, which are discussed in Section 3.3.

Similar concepts can be applied to the retrofitting of existing bridges; retrofits can be designed that mitigate triaxial constraint and notch-like or crack-like planes of discontinuity. However, undertaking a retrofit of an existing bridge typically costs more than using a better detail during design of a new bridge. The decision to undertake a retrofit can be based on an evaluation of the three conditions associated with elevated susceptibility to CIF fracture, which are discussed in Section 3.3.

3.3 THE THREE CONDITIONS CONTRIBUTING TO ELEVATED SUSCEPTIBILITY TO CIF

The existence of constraint alone, even triaxial constraint, in a given welded steel detail does not necessarily equate to an elevated susceptibility to CIF.

Connor and Lloyd (2017) describe three conditions that contribute to the susceptibility of a detail to CIF:

1. "There must be an elevated level of tensile residual stress locked into the local area. While the dominating contribution is residual stresses from welding, other factors

contribute to a lesser degree, such as dead load and erection stress. As is well documented, residual stresses due to welding can easily reach the yield strength of the base metal.

2. "The joint must be highly constrained, resulting in a three-dimensional state of stress that prevents plastic flow, as would [otherwise] occur in a simple uniaxial stress state.

3. "Localized area of stress concentration that intensifies dead load and live load stress level."

Any one of these conditions, taken to extreme limits, could lead to adverse performance or even failure of a structural steel element. However, under normal circumstances, any one of these conditions acting alone, or even any two acting together, likely would not lead to an elevated susceptibility to CIF. Instead, it is the occurrence of all three conditions acting together that typically contributes to an elevated susceptibility. The three conditions are discussed below in the order in which they are most likely to occur in typical steel girder bridges.

Section 5.1 discusses how to apply an understanding of these conditions as part of a screening process to evaluate details for an elevated susceptibility to CIF. Sections 5.5 to 5.8 review several common steel bridge details in the context of evaluating them for susceptibility to CIF.

3.3.1 A High Level of Tensile Stress

Condition 1, as described by Connor and Lloyd (2017) is, "There must be an elevated level of tensile residual stress locked into the local area." During the consensus meeting discussions related to this report (see Section 4.5), this was sometimes discussed in terms of details subject to a high degree of residual and/or net tension stress.

Fracture of steel occurs under conditions of net tensile stress. Structural steel elements subjected to applied tensile forces are typically sized such that the applied tensile stresses are limited to acceptable design values. However, virtually all structural steel elements in a steel girder bridge also exhibit some magnitude and distribution of residual stresses. As noted by Wright (2015), "The process of rolling steel products naturally introduces internal resistance stresses due to the plastic deformation and differential cooling effects during their production." In addition, Connor and Lloyd (2017) noted, "While the dominating contribution is residual stresses from welding, other factors contribute to a lesser degree, such as dead load and erection stress." The magnitude of residual stresses is a function of many variables, including the geometry of the structural element or assembly of elements, how the materials were fabricated, and how the various components were assembled. Several of these items are beyond the control of the engineer, and are essentially unavoidable in the fabrication of a steel structure. Therefore, it is reasonable to assume that any given structural steel element would have some level of residual stress.

The distribution of residual stresses across a given plate or component includes both tensile and compressive stresses. While the resultants of these regions of tensile and compressive residual stresses over an entire cross-section are always in static equilibrium (i.e., the sum of the resultant tensile and compressive forces equals zero), the local peak tensile stresses can potentially be significant. If the given element is a member or component subjected to a tensile stress or stress reversal, the presence of residual stresses tends to exacerbate the effects of the applied tensile stresses in at least some locations.

In some cases, the residual stresses in limited, localized regions of the cross-section of the element, particularly areas that are influenced by the effects of welding, could potentially be as high as the yield stress of the material. Connor and Lloyd (2017) noted, "As is well documented, residual stresses due to welding can easily reach the yield strength of the base metal." Many factors affect this, including weld size, weld length, restraint of the welded elements, sequence of welding, etc.

The magnitude of residual stresses in a given element can theoretically be determined, but as noted by Wright (2015), "Determining the exact distribution and magnitude of residual stress in fabricated members is a very complicated subject that depends on the shape geometry, processing, and the sequence of fabrication operations. It is possible to measure residual stresses through destructive sectioning and hole drilling techniques and through non-destructive X-ray diffraction and neutron diffraction techniques. However, these techniques are impractical except in a research environment."

Therefore, the engineer is left with the problem of how to evaluate the effects of residual stresses. In most normal cases, the magnitude of residual tensile stresses alone in a given, in-service structure are less than the magnitude that produces cracking, fracture, or yielding – if the residual stresses were high enough to cause such issues, those issues would typically have been identified during fabrication or at some point during construction, and some kind of action would have been taken. However, when the structure is constructed and placed in service, the effects of dead load, live load, and other service stresses are also applied. The residual stresses may be additive to, or relieving of, the in-service applied stresses.

When evaluating details for susceptibility to CIF, the effects of residual stresses should be considered. In doing so, it is important to understand that the conditions leading to the initiation of CIF can be highly localized. A highly localized occurrence of a high degree of constraint (discussed in Section 3.3.2), combined with a highly localized planar discontinuity approximately perpendicular to the primary flow of tensile stress (discussed in Section 3.3.3), and then combined with a highly localized occurrence of high tensile stress, can lead to CIF. With this understanding, it can be seen that a concentrated occurrence of residual tensile stresses, when combined with applied, in-service tensile stresses, could result in a localized occurrence of tensile stress equal to or greater than the yield stress of the material and increased susceptibility to CIF.

In summary, the following conclusion can be reached regarding residual stresses:

- It is reasonable to assume a given structural steel element could be subject to residual stresses,
- the magnitude and distribution of residual stresses are difficult to reliably predict, and
- the conditions leading to the initiation of CIF can be highly localized, so

- it is conservative and prudent to assume that tensile residual stresses of high magnitude could occur anywhere in a structural steel element.

Consequently, if the sum of all applied stresses (i.e., the sum of dead load, live load, and other applied stresses) at a given location in a member or component is always tensile, or is even only sometimes tensile (a stress reversal situation), it would be prudent and conservative to assume that the residual stresses at the point of interest are also tensile and thus additive. Further, it would be prudent and conservative to assume that the magnitude of those additive residual tensile stresses, at the point of interest, is high.

Therefore, for the purposes of evaluating details for susceptibility to CIF, Condition 1 can be assumed to be met in most routine cases.

3.3.2 A High Degree of Triaxial Constraint

Condition 2, as described by Connor and Lloyd (2017) is, "The joint must be highly constrained, resulting in a three-dimensional state of stress that prevents plastic flow, as would [otherwise] occur in a simple uniaxial stress state." During consensus meeting discussions (see Section 4.5), this was sometimes referred to as a high degree of triaxial constraint, indicating situations where the local area is highly constrained by various attached structural elements.

The impact of constraint on the ability of a steel element to yield and deform in a ductile manner is illustrated in Section 3.1. A high degree of constraint, particularly triaxial constraint, impedes the ability of the steel to yield and deform plastically. As a result, as demonstrated in Sections 3.1 and 3.2, when a detail is subjected to a high degree of constraint, the tensile stresses in the steel can reach or exceed the ultimate strength of the material without the steel having yielded. At that point, failure in the form of brittle fracture would be sudden, without any plastic deformation and without warning.

Given the basic assumption of the presence of tensile residual stresses as discussed in Section 3.3.1, it follows that even if the applied design tensile stress in a given detail may be relatively low, such tensile stress could be exacerbated by the presence of residual tensile stresses, potentially as high as the uniaxial yield stress of the material. A detail subject to both a high degree of constraint and an applied tensile stress could potentially experience tensile stresses in excess of the material's uniaxial yield stress without experiencing yielding (i.e., without the critical shear stress being exceeded and thus without undergoing plastic deformation).

A theoretical method for quantifying the effects of stress triaxiality and constraint is presented in Section 3.2. However, in practice, the identification of conditions representing a high level of constraint involves consideration of a wide variety of geometric and structural parameters. The degree to which these parameters may contribute to constraint and stress triaxiality can be difficult to quantify.

A high degree of constraint can be avoided by following good detailing practices. These detailing practices are simple and well described in the literature, including in the AASHTO bridge design specifications (BDS) (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)) and the non-binding FHWA/National Highway Institute (NHI) *Design and Evaluation of Steel Bridges for Fatigue and Fracture – Reference Manual* (Russo et al., 2016), among other sources, and are discussed further in Chapter 5.

3.3.3 A Planar Discontinuity Approximately Perpendicular to the Primary Flow of Tensile Stress

Condition 3, as described by Connor and Lloyd (2017) is, "Localized area of stress concentration that intensifies dead load and live load stress level. The presence of discontinuities within the weld, as well as certain geometry of the connection can both act as discontinuities that interrupt stress flow and cause concentrations." During the consensus meeting (see Section 4.5) , this was sometimes referred to as a crack-like geometry.

"Cracks" (as described in Section 2.4.3) are defects that are typically considered unacceptable, and that have historically been repairable in some manner. True cracks have very sharp tips and introduce the potential for very high stress concentrations. These stress fields and the material's behavior under them can be characterized by the field of fracture mechanics. The terms "crack-like" and "notch-like" are used in this report to describe certain geometric conditions of welded connections that can result in very high-stress gradients that are similar in nature to the idealized fracture mechanics view of cracks and notches; however, these types of geometries may not fully exemplify a true crack or notch. For the purposes of characterizing susceptibility to CIF later in this report (particularly in Chapter 5) the term "planar discontinuity" is used to encompass "crack-like" or "notch-like" planes of discontinuity.

These types of discontinuities are particularly problematic when they occur in a plane that is perpendicular (or approximately so) to the primary flow of tensile stress. In these situations, the planar discontinuity can act as the initiation point of the fracture event. The presence of a planar discontinuity featuring a "crack-like" or "notch-like" geometry and oriented approximately perpendicular to the primary flow of tensile stress introduces two critical characteristics:

- a stress concentration; and
- a crack initiator.

If these characteristics exist in a detail that is already subjected to a high level of tensile stress (potentially including additive residual tensile stresses) and a high degree of constraint (such that the material in the detail is unable to yield), it is easy to see how a sudden brittle fracture could occur.

When the plane of discontinuity is parallel (or nearly so) to the primary flow of tensile stress, the effects generally do not produce stress concentrations; since the plane of discontinuity is parallel to the primary flow of tensile stress, it does not interrupt that flow. Additionally in this situation, the tensile stress is not acting in a direction that would further "open" or enlarge the discontinuity; as a result, the discontinuity would not serve as a fracture initiator.

Practical examples of various types and forms of planar discontinuities are discussed and illustrated in Chapter 5.

CHAPTER 4 - REVIEW OF ENGINEERING PRACTICES

4.1 EXAMPLES OF CONSTRAINT-INDUCED FRACTURES OF STEEL BRIDGES

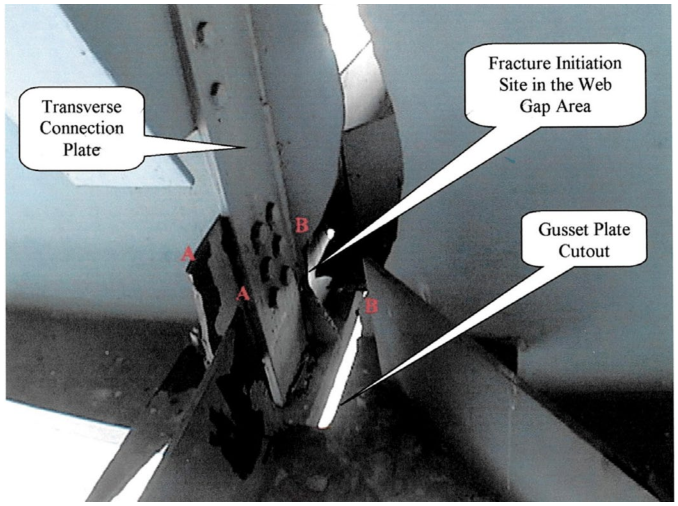
A number of bridges have suffered from serious fractures. Four well-documented cases are discussed below. In addition, there have been anecdotal reports of other bridges experiencing fractures that may be due to CIF. Some situations involve so-called "Hoan-like details," while in other cases the details are, at least visually, different.

4.1.1 Hoan Bridge

The fracture of the Hoan Bridge carrying Northbound I-794 over the Milwaukee River in Milwaukee, WI, is perhaps the most influential instance of a severe fracture occurring in a steel-girder bridge. The Hoan Bridge was opened to traffic in 1974. On December 13, 2000, it was discovered that all three girders in one of the southern approach spans of the bridge had fractured; the interior girder and the east exterior girder had experienced full-depth fractures, while the west exterior girder had experienced several 3-foot deep fractures. A full forensic investigation was undertaken immediately, and within 7 months, by July 10, 2001, the FHWA issued a technical memorandum (Cooper, 2001) about the fracture, including findings, conclusions, and possible actions for other bridges.

Attachment A in the Cooper memorandum states that the cause of the fractures was excessive triaxial constraint with very small (1/8 inch) "web gaps" (see Section 2.4.2 for this report's usage of the terms "web gap" and "constraint-relief gap"). The apparent point of initiation of the fracture was a small gap between a welded lower lateral bracing system connection (gusset) plate and a welded cross-frame connection plate where these various elements framed into the girder web (see Figure 19, Figure 20, and Figure 21). Attachment A also noted that low temperatures at the time did not cause the initiation of fracture, but reduced the ability of the structure to arrest dynamic crack growth. One of the "significant findings" (in the words of the memorandum) highlighted in Attachment A is particularly instructive:

> A narrow gap between the gusset plate and the transverse connection/stiffener plate created a local triaxial constraint condition and increased the stiffness in the web gap region at the fracture initiation site. This constraint prevented yielding and redistribution of the local stress concentrations occurring in this region. As a result, the local stress state in the web gap was forced well beyond the yield strength of the material. Under triaxial constraint, the apparent fracture toughness of the material is reduced and brittle fracture can occur under service conditions where ductile behavior is normally expected.

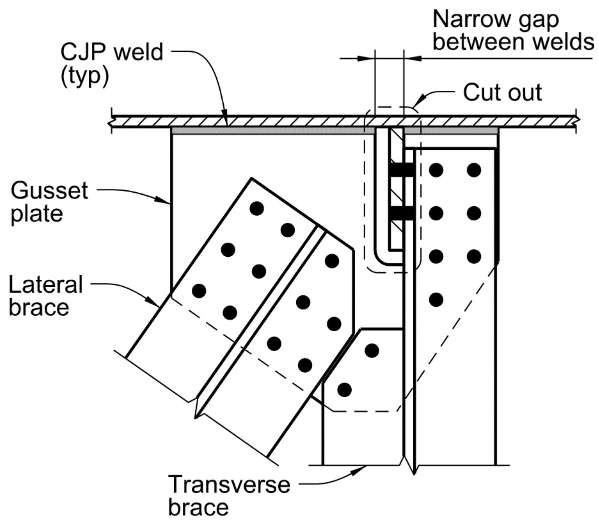


Source: FHWA

Figure 19. Photo. Hoan Bridge fracture initiation site.

Source: FHWA

Figure 20. Photo. Hoan Bridge fracture initiation site.



Source: FHWA

Figure 21. Illustration. Plan view of Hoan Bridge bracing node connection detail, fracture initiation site.

Subsequent assessments of the Hoan Bridge fracture, studies of similar fractures in other bridges, and other related research and investigations, largely supported this conclusion (e.g., Fisher et al. 2001; Wright et al., 2003). The cause of the Hoan Bridge fracture was CIF originating in details with high-stress triaxiality, which resulted from:

- a high level of constraint, provided by the various attachments locally constraining the ability of the web to yield;
- high levels of tensile stress associated with residual stresses induced by welding of the various attachments to the web; and
- crack-like geometry, specifically where the so-called "web gap" (a constraint-relief gap) between the lateral bracing connection plate (the "gusset plate" in Figure 19) and the cross-frame connection plate (the "transverse connection plate" in Figure 19) was very narrow.

The steel was found to exhibit reasonable toughness with no evidence of fatigue cracking prior to the CIF event.

The horizontally oriented lateral bracing connection plate was detailed with a "slot" or "cutout" so that it could fit around the vertically oriented cross-frame connection plate. As a result, the lateral bracing connection plate, which was a longitudinal attachment to the web, was effectively interrupted. This resulted in discontinuities in the longitudinal element, interrupting the flow of longitudinal stresses. The gap between the lateral bracing connection plate and the cross-frame connection plate was very small; the toes or ends of the welds of these various attachments were very close, were touching, or were intersecting at various instances of this detail along the bridge. In these situations, the gap represented a "crack-like" planar discontinuity approximately perpendicular to the primary flow of tensile stress, producing a very high stress concentration.

In addition, the lateral bracing introduced out-of-plane stress and constraint in the web.

Attachment B in the Cooper memorandum illustrates details that are susceptible to CIF, using the Hoan Bridge details as examples. Attachment B indicates that CIF is a concern in elements subjected to net tension. Attachment B also states that the rate of crack growth is an indicator of the nature of the underlying cause of the cracking, stating that fast crack growth suggests fracture, whereas slow crack growth suggests fatigue as the underlying cause.

Attachment A in the Cooper memorandum states, "There was no evidence of fatigue cracking prior to fracture initiation." Attachment A also states:

> Inspection reports indicate that web cracks were found in other locations of the bridge as early as 1995. The cracks were thought to be fatigue cracks and retrofit actions were taken based on this assumption. The forensic investigation has determined that these prior cracks were fractures similar to the ones resulting in failure. However, all prior web cracks arrested at the flange and didn't trigger the chain reaction failure.

4.1.2 US 422 Bridge

On May 20, 2003, a fracture was identified during a routine inspection of a bridge carrying US 422 over the Schuylkill River in Pottstown, PA, while the bridge was undergoing retrofits to lateral gusset plate connections. The bridge was designed and built in 1965. The fracture occurred in one of the two main girders in the bridge cross-section and initiated at the intersection of the web with a welded lateral bracing connection plate and a welded cross-frame connection plate, approximately 3 inches above the bottom (tension) flange (see Figure 22 and Figure 23). The fracture extended 6 inches above the lateral bracing connection plate. The fracture was discussed by Connor et al. (2007), who concluded that the cause of the fracture was excessive triaxial constraint and poor weld quality. Fracture occurred at two separate locations at the lateral connection plate nearly simultaneously. Fracture occurred even though the material met AASHTO Zone 2 fracture-critical toughness criteria for Grade 36 steel (Connor et al., 2007). There was no evidence of an initiating welding discontinuity or fatigue crack extension prior to the fracture.

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Figure 22. Photo. Fractured bottom flange of girder and arrested web fracture in the US 422 bridge. View of web on opposite side of lateral gusset connection.

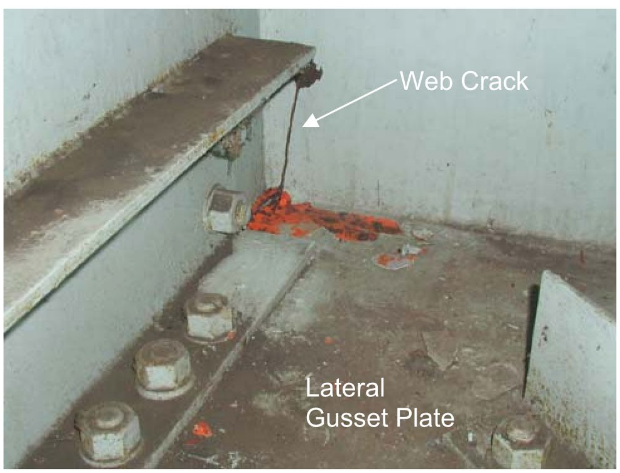

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Figure 23. Photo. View of lateral gusset plate connection and fracture at the weld intersection of the gusset and vertical stiffener plates to the web in the US 422 bridge

4.1.3 Diefenbaker Bridge

On August 29, 2011, a major fracture was discovered in one of the two main girders of the Diefenbaker Bridge, a seven-span, 1,000-foot long, steel-girder bridge in Prince Albert, Canada. The fracture was discussed by Ellis et al. (2013). The fracture extended from the bottom flange through nearly the entire height of the girder web. The fracture was located at the intersection of the girder web with welded connections to the bridge's lower lateral bracing system and a cross-frame connection plate (see Figure 24). The girders were continuous welded steel plate girders, with welded stiffeners and gusset plates. The rolled shapes used for bracing members and floor beams were bolted to the girders or gusset plates. "The horizontal gusset plate was welded to the girder web on each side of the vertical stiffener and to the vertical stiffener. The horizontal gusset-to-web welds intersected the stiffener-to-web welds" (Ellis et al., 2013). Ellis et al.'s investigation and testing determined that the cause of the fracture was not related to temperature or material properties, but rather to CIF, similar to the brittle fractures of the Hoan Bridge in Wisconsin and the US 422 Bridge over the Schuylkill River in Pennsylvania. Ellis et al. stated, "As an aside, since the fracture occurred on a warm August day, temperature was also not a factor."

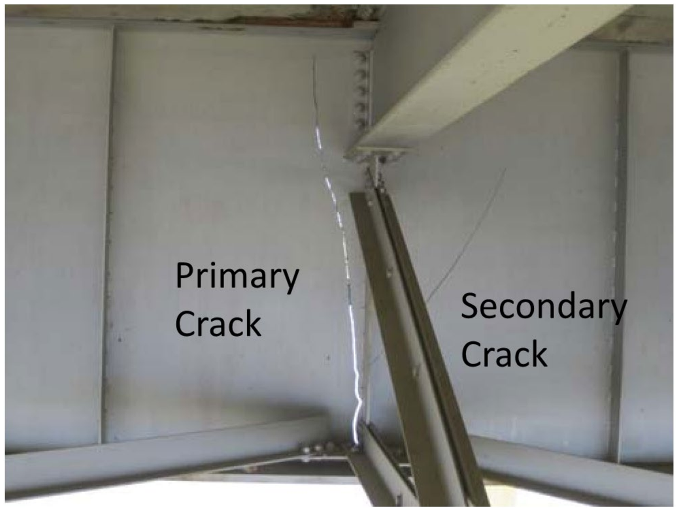


Figure 24. Photo. Primary and secondary cracks in web of the Diefenbaker Bridge.

4.1.4 I-64 Blue River Bridge

In May 1994, maintenance crews from the Indiana Department of Transportation (DOT) discovered a fracture of the bridge carrying I-64 over the Blue River and Blue Road in Harrison County, IN. The fracture occurred in the bottom flange of the exterior girder and extended up 69 inches through the 70-inch depth of the web. The bridge was built in 1974. The fracture occurred sometime after an inspection of the bridge in the fall of 1993 and is believed to have occurred during a period of extremely cold temperatures in January 1994 (Bowman, 2002). The roadway carried by the bridge was being supported by the remaining three girders in the bridge cross-section.

A report by Bowman (2002) documents the investigation of

> …the brittle fracture that occurred on the I-64 Blue River Bridge in Harrison County of southern Indiana. The fracture occurred in the middle span of a three-span structure at a location where both a lateral diaphragm and a horizontal bracing member framed into a vertical and horizontal plate, respectively. The study involved experimental studies to evaluate the material performance and behavior of the bridge steel and analytical studies to assess the fracture resistance and susceptibility to brittle fracture. Suggestions for retrofit and repair of similar bridge details were formulated to decrease the fracture susceptibility from distortion related fatigue cracking.

The Bowman report concluded that "…the brittle fracture initiated in the girder web near the intersection of a vertical connection plate and a horizontal gusset plate. Moreover, it is believed that the crack initiated as a fatigue crack in the web gap region immediately adjacent to the weld toe of the web-to-vertical stiffener weld." (See Section 2.4.2 for this report's usage of the terms "web gap" and "constraint-relief gap.") The report further states:

> … four factors are believed to have elevated the stresses in the gusset-to-stiffener connection welds: lack of positive attachment between the horizontal gusset plate and the vertical diaphragm, a small lateral gap distance between the toes of the horizontal and vertical fillet welds, loose bolts in the horizontal bracing to gusset plate connection, and impact forces introduced into the web via the horizontal bracing members.

As illustrated in Figure 25, Figure 26, and Figure 27, the details in the Blue River Bridge were similar to the Hoan Bridge details, with small but significant differences. Notably, the Blue River Bridge's lateral bracing connection plates were welded to the cross-frame connection plates, but the welds do not intersect the welds connecting the lateral bracing connection plates to the web, resulting in a discontinuity in the longitudinal attachment (the lateral bracing connection plate). Furthermore, the cross-frame connection plates were not welded to the tension flange of the girder as is now specified in the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)), and the lateral gusset plate was not attached to the cross-frames (referred to as the "vertical diaphragms" in the Bowman report). Both conditions contributed to distortion-induced fatigue and out-of-plane bending of the web in the vicinity of the intersection of the lateral gusset and cross-frame connection plates with the web.

Figure 25. Photo. View of fracture in the outside plate girder of the I-64 Blue River Bridge.

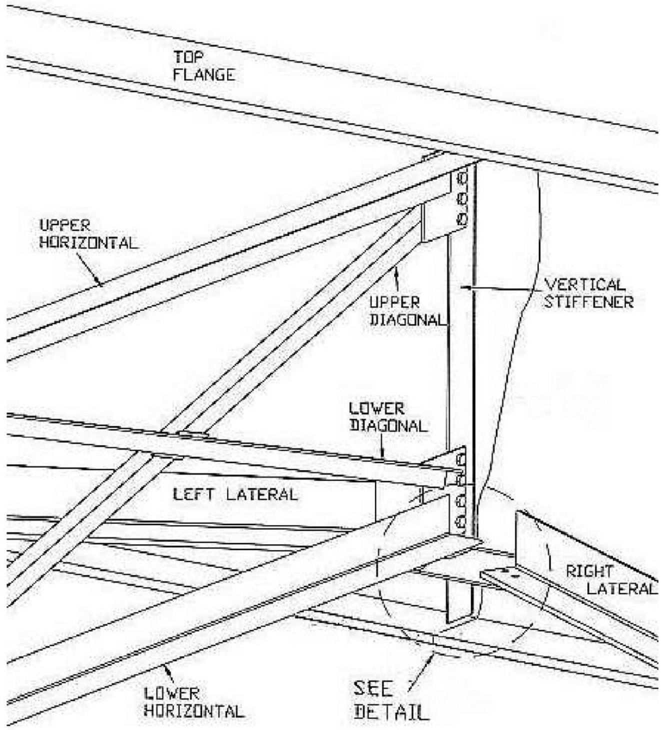


Figure 26. Illustration. Sketch of connection detail at the brittle fracture location in the I-64 Blue River Bridge.

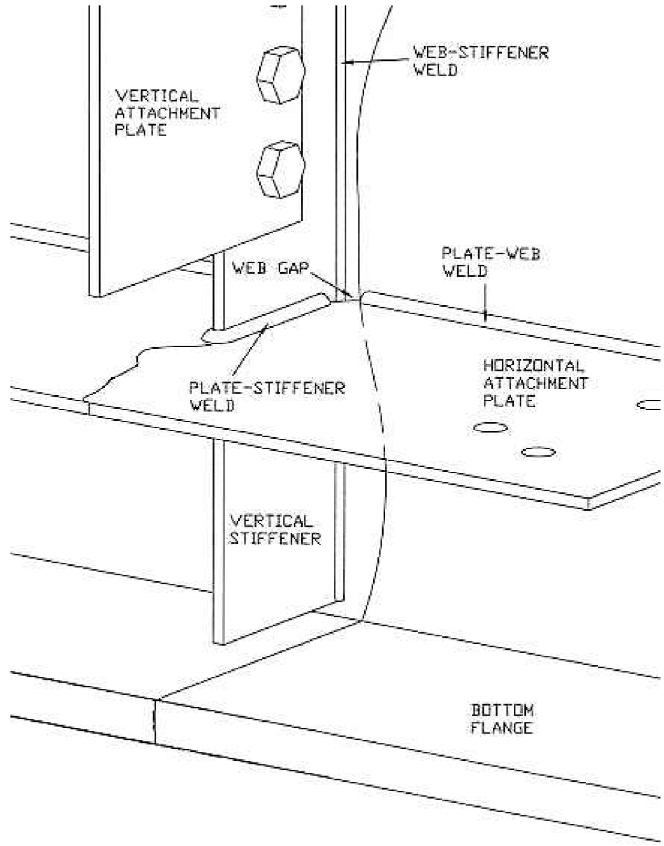


Figure 27. Illustration. Close-up view of crack position relative to the vertical stiffener and the horizontal attachment plate in the I-64 Blue River Bridge.

4.2 RESPONSES TO CONSTRAINT-INDUCED FRACTURE

The most significant and influential case of CIF in a steel bridge is the December 2000 Hoan Bridge fracture discussed in Section 4.1.1. The event received national media attention and, even though there were no injuries or vehicular accidents, was a significant incident in terms of disruptions to local traffic. Another noteworthy legacy of the Hoan Bridge fracture is the impact it had on bridge engineering practice in the United States. The findings published in association with the investigations into the Hoan Bridge fracture have had a widespread and long-lasting effect on bridge design, bridge inspection, and bridge maintenance and repair policies.

The initial and most influential Hoan Bridge fracture report was the memorandum published by the FHWA, authored by Cooper (2001) and discussed in detail in Section 4.1.1. This memorandum presented an explanation that the Hoan Bridge fracture was caused by CIF. In discussing possible actions for identifying similar details in other bridges, Attachment B of the Cooper memorandum cited “touching welds” and “intersecting welds” as indicators of constraint-relief gaps that are too small. The memorandum states:

> Structures that are known to have narrow web gaps in tension zones should be inspected closely with a hands-on visual inspection. This can be accomplished by review of inspection reports as long as there is sufficient photographic documentation to assess the gap area. If *intersecting or touching welds* [italics added] are identified or suspected, steps should be taken to further evaluate the connection and consider possible retrofit options. In cases where there is a clear gap between the two welds, the susceptibility to constraint-induced fracture is lower. Retrofits are probably not required unless inspection reveals fatigue cracking in the gap area.

The intent of the memorandum was to present technical criteria related to the identification of narrow constraint-relief gaps in tension zones. But the discussion of "touching welds" and "intersecting welds" as indicators of undersized constraint-relief gaps could lead to confusion. Some bridge designers and/or bridge owners could read it as a message to treat details with intersecting welds as problematic in general.

The results can be seen in a number of practice documents and actions taken by various owner-agencies in design and fabrication practices as shown in Section 4.4. Some of these actions could place undue burdens on new designs, may be ineffective, or may result in design details with inferior performance.

Other practice documents present options for the repair or retrofit of details that may be susceptible to CIF. For example, Connor and Lloyd (2017) provide a presentation of details that may be susceptible to CIF and suggest mitigation actions. The report describes two types of triaxially intersecting welded element details that are typically of concern.

First, for so-called "Hoan-like details" – where welded lateral connection plates intersect with the web and web stiffeners without sufficient constraint-relief gaps to relieve the development of triaxial constraint – the report introduces three repair/retrofit strategies:

- **Lateral Connection Plate Cope Retrofit** - The lateral connection plate cope retrofit creates a sufficiently sized constraint-relief gap (minimum of ¼ inch of the web exposed) to eliminate the localized constraint of the web plate. See Figure 28.

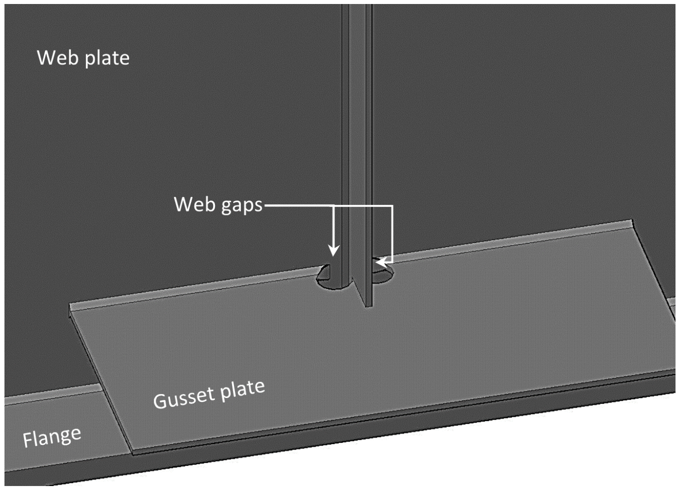


Figure 28. Illustration. Lateral connection plate cope retrofit (secondary members not shown for clarity).

- **Web Plate Isolation Holes Retrofit** - The web plate isolation hole retrofit installs a mechanism to arrest a fracture immediately after it initiates, isolating the web plate and flanges from further fracture propagation. See Figure 29**.**

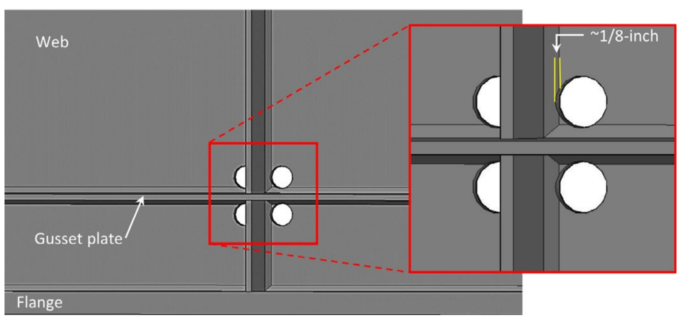


Figure 29. Illustration. Web plate isolation holes retrofit for CIF details.

- **Ball End Mill Retrofit** - The ball end mill retrofit mitigates fracture at CIF details by removing the constraint and reducing the stress concentrations at the intersection of the vertical and horizontal welds. See Figure 30.

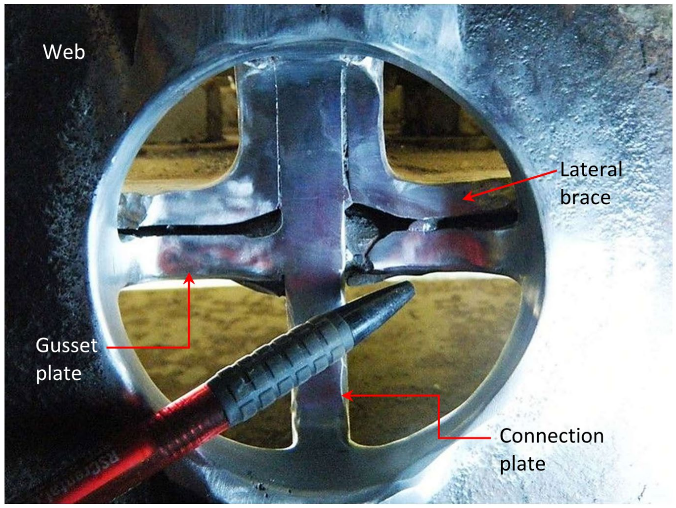


Figure 30. Illustration. Ball end mill retrofit detail.

Second, for details where webs, web transverse stiffeners, and longitudinal stiffeners intersect, similar isolation holes and ball end mill retrofits are discussed. In addition, a stiffener coping retrofit (which involves cutting back the longitudinal stiffener from the transverse stiffener/connection plate) is also discussed.

In addition, Russo et al. (2016) provides a discussion of the design and evaluation of steel girder bridges for fatigue and fracture, including a discussion of CIF and its phenomenon in steel structures with illustrations of details that should and should not be used with respect to CIF.

4.3 CONCERNS ABOUT INTERSECTING WELDS

As noted in Section 4.2, the discussion of "touching welds" and "intersecting welds" as indicators of undersized constraint-relief gaps could lead to confusion. The following discussion provides some clarification.

Details that feature high degrees of triaxial constraint and crack-like or notch-like planar discontinuities approximately perpendicular to the primary flow of tensile stress, where the constraint-relief gaps are so small that the weld toes or ends touch, or "intersect," are potentially at elevated susceptibility to CIF. Such details warrant investigation, evaluation, and potential retrofit (in existing bridges) or redesign (in new bridge designs). Several owner-agencies have published documents that help identify and address these situations (see Section 4.4).

However, many other commonly used steel bridge details featuring intersecting welds are not susceptible to CIF.

Detailing efforts aimed at avoiding intersecting welds could lead to poorer performance in service. During the Consensus Meeting (see Section 4.5), it was discussed that one owner-agency had, on occasion, provided holes in girder webs at a location where the flange-to-web fillet welds crossed the intersection of co-located web and flange shop splices (i.e., co-located CJP groove welds in butt joints). This was done to avoid the intersection of the three sets of welds. The detail is not particularly susceptible to CIF, but is defined in AASHTO (2017a) (23 CFR 625.4(d)(1)(v)) as a Category D fatigue detail, whereas if the hole had not been drilled, the fatigue categorization would have been Category B.

Similarly, some features of details with intersecting welds, in certain applications, can present concerns not related to CIF. These concerns are often associated with difficulty in avoiding welding imperfections in details with complicated geometry. For example, where three orthogonally oriented elements meet and the connecting welds all run into a corner at least one weld will invariably be interrupted. If the geometry is complex or clearances are limited, it may be difficult to achieve a quality weld pass all the way into the corner. In other cases, the intersecting welds may be impossible to make without introducing a weld access hole, which may represent a poor fatigue detail.

4.4 DOCUMENT REVIEW

4.4.1 Literature Review

The Contractor reviewed 48 documents as part of the literature review. This does not include the review of various owner-agency documents discussed in Section 4.4.2.

The literature review included four design and/or construction specifications:

- the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v));
- the AASHTO *LRFD Bridge Construction Specifications (*BCS*)* (AASHTO, 2017b) (23 CFR 625.4(d)(1)(iv));
- the AASHTO/AWS *Bridge Welding Code* (AASHTO/AWS, 2015) (23 CFR 625.4(d)(1)(vii)); and
- the non-binding AASHTO *Guide Specifications for Fracture Critical Non-redundant Steel Bridge Members* (AASHTO, 1978).

Each of the four are directly related to steel bridge design and construction.

The literature review also included 14 design practice documents, 11 of which address the design of new steel bridges or maintenance actions, repairs, and retrofits of existing steel bridges. The remaining three are used in the offshore platform structures industry and include:

- the American Petroleum Institute (API) *Recommended Practice 2A-WSD, Planning, Designing, and Constructing Fixed Offshore Platforms-Working Stress Design* (API, 2014);
- the *Review of Current Inspection Practices for Topsides Structural Components* (Bucknall, 2000); and
- the *Comparison of No. 47 Shipbuilding and Repair Quality Standard (1996, Rev. 1 1999, Rev. 2 Dec. 2004) with Japan Shipbuilding Quality Standard (JSQS, 1004, Appendix*

0144) and Production Standard of the German Shipbuilding Industry (FS, 2003) (Germanischer Lloyd, Unknown Date).

In addition, the literature review included two inspection practice documents, specifically the Participant Manual from the NHI/FHWA fracture critical bridge inspection course (Ryan et al., 2010) and the NHI/FHWA *Bridge Inspector's Reference Manual* (Ryan et al., 2012).

The literature review also included nine failure investigation reports that described various investigations related to the fractures experienced in a number of bridges. Several of those reports are discussed in Section 4.1.

The literature review also included 14 research reports on various fatigue and fracture topics.

Finally, the literature review included five articles that appeared in either journals or books featuring collections of technical papers. Four of these articles are general articles on various fatigue and/or fracture topics, and one is specific to the analysis of the Hoan Bridge fracture.

The complete literature review is summarized in Appendix A. Section 4.4.1 provides a summary of key findings from the literature review.

4.4.1.1 Effects of Triaxial Constraint

A limited amount of research exists on triaxial constraint effects. Excessive triaxial constraint can lead to CIF.

The role of stress triaxiality as it influences ductile versus brittle behavior has long been known. Schafer et al. (2000) cited work by Gensamer (1941) in which a classical model was presented to explain the role of triaxiality. A fundamental understanding of Mohr's circle of stress, and the importance of the ability to develop shear stresses as an associated prerequisite for ductile behavior, helps to illustrate the implications of excessive triaxial constraint. Schafer suggested using two "triaxiality factors" to quantify the degree of triaxiality. Schafer showed that stress levels can reach 150 percent of the nominal uniaxial yield stress without actually yielding the material. This level of stress is greater than the typical ultimate strength of the material (130 percent to 140 percent of the nominal yield stress). In situations of high triaxial constraint, stress levels can reach the rupture strength of the material and a fracture can potentially occur without inelastic straining of the material. More detailed discussions of stress triaxiality are provided in Sections 3.1 and 3.2. The three conditions associated with an elevated susceptibility to CIF under normal circumstances are discussed in Section 3.3.

Intersecting welds, where two or three welds intersect, are not necessarily prone to CIF (see Sections 4.1 and 4.2). The parameters that contribute to an elevated susceptibility to CIF include a notch-like or crack-like plane of discontinuity approximately perpendicular to the primary flow of tensile stress, the presence of high tensile stress, and excessive triaxial constraint, which can potentially be developed in welded details featuring intersecting elements.

The AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)) and practice documents such as the FHWA/NHI *Design and Evaluation of Steel Bridges for Fatigue and Fracture – Reference Manual* (Russo et al., 2016), address a variety of details associated with intersecting or nearly intersecting welds. Discussion of these details focuses on situations that may be susceptible to triaxial constraint, such as where vertical web stiffeners may interface with longitudinal webs stiffeners or lateral bracing connection plates.

The susceptibility to CIF of a detail that features welds that run through each other, overlap, or touch, is affected by a number of factors, particularly the presence of the three conditions that contribute to an elevated susceptibility to CIF, as enumerated by Connor et al. (2007), and discussed more extensively in Section 3.3:

1. a sufficiently high net tensile stress, including consideration of residual stresses;
2. a high degree of constraint, preventing local yielding; and
3. a crack-like or notch-like geometry.

As an example, consider the case of the intersection of flange-to-web fillet welds with a CJP groove weld in a butt joint used to accomplish the shop splice of a flange in a welded steel plate girder. As indicated previously, this detail clearly features "intersecting welds," but is not subject to an elevated susceptibility to CIF, and there are no reported cases of fractures associated with the use of this type of detail. An evaluation of the three factors listed above helps explain the nature of this detail and why it is, in and of itself, not problematic; neither a high degree of constraint nor a planar discontinuity approximately perpendicular to the primary flow of tensile stress are present in this detail. Further discussion of this type of evaluation, including illustrative examples, is provided in Chapter 5.

4.4.1.2 Detailing

Design specifications such as the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)) and non-binding practice documents such as the FHWA/NHI *Design and Evaluation of Steel Bridges for Fatigue and Fracture – Reference Manual* (Russo et al., 2016), discuss a variety of details associated with intersecting or nearly intersecting welds. Discussion of these details focuses on situations that may be susceptible to triaxial constraint, such as situations where vertical web stiffeners may interface with longitudinal webs stiffeners or lateral bracing connection plates. These discussions are generally helpful; however, in some cases conflicting minimum web gap dimensions are presented.

As is documented in other sections of this report, details with intersecting welds, in and of themselves, are not necessarily problematic. Several factors affect the susceptibility to CIF of a detail that features welds that run through each other, overlap, or touch. In particular the presence of the three conditions listed below contribute to an elevated susceptibility to CIF, as enumerated by Connor et al. (2007), and discussed more extensively in Section 3.3 of this report:

1. a sufficiently high net tensile stress, including consideration of residual stresses;
2. a high degree of constraint, preventing local yielding; and
3. a crack-like or notch-like geometry.

As an example, consider the case of the intersection of flange-to-web fillet welds with a CJP groove weld in a butt joint used to accomplish the shop splice of a flange in a welded steel plate girder. As indicated previously, this detail clearly features "intersecting welds," but is not subject to an elevated susceptibility to CIF, and there are no reported cases of fractures associated with the use of this type of detail. An evaluation of the three factors listed above helps explain the nature of this detail and why it is, in and of itself, not problematic; neither a high degree of constraint nor a planar discontinuity approximately perpendicular to the primary flow of tensile stress are present in this detail. Further discussion of this type of evaluation, including illustrative examples, is provided in Chapter 5.

4.4.1.3 Constraint-Relief Gaps

Given an understanding that triaxial constraint contributes to an elevated susceptibility to CIF, many of the design and retrofit suggestions in the literature focus on providing a constraint-relief gap of at least some minimum width. The width of a constraint-relief gap is measured as the distance between the toes and/or ends of the welds attaching an interrupted constraining element to a constrained element. There are many examples of constraint-relief gaps, such as the gaps provided in discontinuous vertical stiffeners, discontinuous longitudinal stiffeners, discontinuous fitted lateral bracing connection plates, etc., that may be attached to, and contributing to the constraint of, the webs of steel girders. By providing a sufficiently wide constraint-relief gap, local triaxial constraint of the web can be relieved, allowing the web to yield when subjected to tension in the primary stress direction and thus avoiding sudden brittle fracture.

Throughout much of the literature, reference is made to a paper by Mahmoud et al. (2005) in which it was concluded that: "… [a] slight increase in the web gap size (¼ inch) will result in smaller triaxial stresses and less potential for fracture." This ¼-inch minimum "web gap" (measured between weld toes and/or ends) suggestion appears to be the foundational data point for most design and retrofit suggestions related to details that may otherwise be subject to high degrees of triaxial constraint. For example, Connor et al. (2007), Connor and Lloyd (2017), Delong and Bowman (2010), Fish et al. (2015), Ryan et al. (2010), and Ryan et al. (2012), all refer to the ¼-inch minimum constraint-relief gap dimension as a critical threshold. Though not required by FHWA regulations, these various authors state, in one form or another, that constraint-relief gaps of less than ¼ inch (measured between the weld toes and/or ends) are considered problematic and new designs or retrofits should maintain constraint-relief gaps of ¼ inch or greater.

Earlier work by Pass et al. (1983) also references the importance of the constraint-relief gap dimension, in this case in the context of the fatigue behavior and performance of a steel girder with intersecting longitudinal and transverse web stiffeners. The authors stressed the importance of the gap size with regard to the stress concentrations in the web in this region.

Meanwhile, other practice documents and design specifications suggest different constraint-relief gap dimensions. Article 6.6.1.2.4 of the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)), includes the following language:

> Welded structures shall be detailed to avoid conditions that create highly constrained joints and crack-like geometric discontinuities that are susceptible to constraint-induced fracture… If a gap is specified between the weld toes at the joint under consideration, the gap shall not be less than 0.5 [inch].

Table 6.6.1.2.4-1 provides sketches of "Details to Avoid Conditions Susceptible to Constraint-Induced Fracture at the Intersection of Longitudinal Stiffeners and Vertical Stiffeners Welded to the Web." Table 6.6.1.2.4-2 provides sketches of "Details to Avoid Conditions Susceptible to Constraint-Induced Fracture at the Intersection of Lateral Connection Plates and Vertical Stiffeners Welded to the Web." In both tables, a footnote states: "If a gap is specified between the weld toes, the recommended minimum distance between the weld toes is 0.75 [inch] but shall not be less than 0.5 [inch]. Larger gaps are also acceptable."

At the same time, the non-binding Reference Manual for FHWA/NHI *Design and Evaluation of Steel Bridges for Fatigue and Fracture – Reference Manual* (Russo et al., 2016), provides a suggestion to use a wider constraint-relief gap, and directly quotes language from the same article of the previous 7th Edition of the AASHTO BDS, which is different from Article 6.6.1.2.4 of the AASHTO BDS, 8th Edition (23 CFR 625.4(d)(1)(v)):

> To the extent practical, welded structures shall be detailed to avoid conditions that create highly constrained joints and crack-like geometric discontinuities that are susceptible to constraint-induced fracture. Welds that are parallel to the primary stress but interrupted by intersecting members shall be detailed to allow a minimum gap of 1 inch between weld toes.

These differences in minimum constraint-relief gap dimensions are a function of the evolving nature of industry publications related to CIF; the ¼-inch minimum constraint-relief gap dimension was originally suggested by Mahmoud et al. (2005). Later suggestions presenting bigger gap dimensions in new designs appear to have been intended to avoid asking inspectors to measure tiny ¼-inch gaps between weld toes and/or ends, and take advantage of the ability of the designer to easily specify a larger gap without adding cost or complexity to new designs. It has been anecdotally reported that at one point in the development of the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)), 1 inch was suggested as the minimum width, but it was determined such a large physical stiffener gap would lead to other adverse design consequences. The published provisions take the form of a range: a minimum web gap of ¾ inch is used, but the absolute minimum gap allowed is ½ inch for new designs. Meanwhile, for in-service inspection, most practice documents such as the *Bridge Inspector's Reference Manual* (BIRM) (Ryan, 2010) present ¼ inch as the minimum gap for identifying potential issues, typically checked by an inspector using a ¼-inch-thick piece of stock material.

However, during the consensus meeting (see Section 4.5) participants said that the ¼-inch minimum constraint-relief gap criteria is based on fairly limited analytical research and may not be conservative in all situations. Possible future research into constraint-relief gap size, which does not and will not reflect requirements under FHWA regulations, is discussed in Section 8.2.1. Before such research is completed, currently published suggestions by Connor et al. (2007), Connor and Lloyd (2017), Delong and Bowman (2010), Fish et al. (2015), Ryan et al. (2010), and Ryan et al. (2012), agree on providing at least a ¼-inch constraint-relief gap for evaluation of existing structures and generally discuss larger gaps for new design and for retrofits of existing bridges.

Note that certain details can be susceptible to "distortion-induced fatigue," where excessive out-of-plane flexing of the web might lead to fatigue cracking. A full discussion of detailing to avoid distortion-induced fatigue is beyond the scope of this report; discussion is provided in AASHTO BDS (2017a) (23 CFR 625.4(d)(1)(v)).

4.4.2 Owner-Agency Practices Related to Intersecting Weld Details

Several published owner-agency documents were reviewed for practices related to intersecting weld details. A summary of key findings is provided below. A more complete summary of the reviewed documents is provided in Appendix B.

4.4.2.1 Repair, Retrofit, or Modification of Details in Existing Bridges

The Wisconsin DOT Bridge Design Manual (Wisconsin Department of Transportation, 2018) discusses retrofitting connections that are "too rigid," which is described as areas where excessive restraint of movement introduces the potential for high stresses. The cited example is lateral bracing connection plate details. As a retrofit, the manual states:

> The solution is to create spaces [constraint-relief gaps] large enough (approximately 1/4 inch or more) for more material to flex thus reducing the concentration of stress. For gusset connection plates, provide a larger gap than 1/4 inch and no intersecting welds. For existing conditions, it may be necessary to drill holes at high stress concentrations.

4.4.2.2 Design Details in New Steel Bridge Designs

Several owner-agencies have published practices related to the use of intersecting welds in new steel bridge designs. The practices varied. A summary is provided below:

- A number of owner-agency design practices and construction specifications, including the Alabama DOT *Structure Design Manual* (Alabama Department of Transportation, 2017), the Colorado DOT *Standard Specifications for Road and Bridge Construction* (Colorado Department of Transportation, 2017), the Florida DOT *Structures Detailing Manual* (Florida Department of Transportation, 2019), the Georgia DOT *Bridge and Structures Design Manual* (Georgia Department of Transportation, 2018), the Montana DOT *Structures Manual* (Montana Department of Transportation, 2002), the South Carolina DOT *Bridge Design Manual* (South Carolina Department of Transportation, 2006), the Utah DOT *Structure Design and Detailing Manual* (Utah Department of Transportation, 2017), and the Vermont DOT *VTrans Structures Design Manual* (Vermont Department of Transportation, 2010), explicitly state that intersecting welds are either prohibited or are to be avoided, without further explanation, clarification, or illustration.

- The Pennsylvania DOT *Design Manual, Part 4* (2015) prohibits the use of intersecting welds and uses a minimum clearance of 1 ½ inch from the end of a fillet weld as a representative means of avoiding intersecting welds.

- The Iowa DOT does not explicitly prohibit the use of intersecting welds, but published a electronic memorandum (Iowa Department of Transportation, 2011) with an attached white paper (Iowa Department of Transportation, 2010) that provides qualitative discussion of triaxial stress conditions and how they contribute to the potential for brittle fracture, and then suggests that existing details that have less than a ¼ inch minimum "web gap" (constraint-relief gap) between weld toes should be retrofitted by coring 2- or 3-inch diameter holes through the web on both sides of the stiffener.

- The Kansas DOT *Design Manual, Volume III – Bridge Section, U.S. Customary Units* (2016) provides figures that show a way to detail the conjunction of longitudinal and transverse (vertical) stiffeners to avoid intersecting weld details.

- The New York State DOT *Bridge Manual* (2017) provides instructions that transverse stiffeners be placed on the opposite side of the web from longitudinal stiffeners.

- The Missouri DOT *Engineering Policy Guide* (2019) provides example details that show minimum "web gaps" (constraint-relief gaps), including a 1-inch minimum "web gap," where longitudinal stiffeners are interrupted at transverse stiffeners, and a 3-inch minimum "web gap," where lateral bracing connection plates are coped around transverse stiffeners.

4.4.2.3 Fabrication of New Steel Bridges

The Wisconsin DOT *Structure Inspection Manual, Part 1 – Administration, Chapter 4 – Fundamentals of Structure Inspection* (2017), Section 1.4.3.3, "Identification of Critical Details," discusses intersecting welds as a "fabrication flaw due to welding." This same manual also cites "groove welds between intersecting longitudinal stiffeners and members" as a fatigue-prone detail, which should be closely inspected.

Also, the Colorado DOT *Standard Specifications for Road and Bridge Construction* (2017), Section 509.20 (i) state that "Intersecting fillet welds will not be allowed."

4.4.2.4 Fabrication Inspection of New Steel Bridges

Among the owner-agency documents reviewed, no practices or construction specification provisions related to the fabrication inspection of intersecting welds in new steel bridges were found. However, the Wisconsin DOT *Structure Inspection Manual, Part 1 – Administration, Chapter 3 – Types of Bridge Inspections and Assessments* (2018b), Section 1.3.4, "In-Depth Inspection," subsection 1.3.4.2, "Purpose," discusses intersecting welds as a "fabrication flaw due to welding" and a "fatigue-prone detail" that could eventually be subject to fatigue cracking and which could potentially "propagate to a size where it may trigger a fracture in a structural member." The associated list of other "fabrication flaws due to welding" included items such as incomplete fusion, slag inclusions, porosities, blow holes, undercuts, and craters, among other items.

4.4.2.5 National Bridge Inspection Standards (NBIS) Inspection of Existing Steel Bridges

The BIRM (Ryan et al., 2012), includes discussion of intersecting welds and triaxial constraint in steel bridges. The BIRM provides discussions of both topics, including an explanation of the implications of triaxial constraint as a potential source of fracture at low stress levels. The BIRM further continues in the same section to describe "Intersecting Welds" as "welds that run through each other, overlap, touch, or have a gap between their toes of less than ¼ inch (see Figure

6.4.48).” The BIRM usage of the term “intersecting welds” includes the intersection of a flange or web butt splice with the flange-to-web fillet welds, but does not differentiate these from other details that include intersecting welds. The BIRM’s Figure 6.4.48 shows a case that is akin to a triaxial constraint situation, and states:

> This problematic detail allows for alternate, unanticipated stress paths that may act as stress risers, leading to crack initiation. Intersecting welds are not fatigue related or material dependent and may consequently occur under low stress levels in a ductile material with good toughness properties. Additionally, intersecting welds may leave large residual stresses after welding, leading to possible cracking and reduced fatigue strength. Welds are terminated short of the intersection by at least ¼ inch to avoid intersecting welds. In most cases, it is desirable to allow the longitudinal weld (parallel with the applied stress) to be continuous. This avoids a Category E type detail at the weld termination if it is interrupted. The end termination of a transverse weld does not directly affect its fatigue strength and is classified as Category C’ for plates.

Among the owner-agency documents reviewed, several published practices or provisions related to the in-service NBIS inspection of steel bridges with intersecting welds were found.

The Pennsylvania DOT *Bridge Safety Inspection Manual* (Pennsylvania Department of Transportation, 2010), Section 2.4.9.2, “Intersecting Welds,” includes discussion of the inspection of intersecting welds, which are discussed as “… welds that run through each other, overlap, touch, or have a gap between their toes of less than ¼ inch. The intersecting welds of the web-splice-to-flange or flange-splice-to-web are not of concern here. Three-dimensional details with intersecting welds are the critical intersecting welds.” The discussion lists examples of critical details, actions to take if such welds are discovered, and commentary about the potential failures associated with these types of details.

The Montana DOT *Bridge Inspection Manual* (Montana Department of Transportation, 2015), recognizes intersecting weld details as susceptible to fatigue and fracture.

The Alabama DOT *Bridge Inspection Manual* (Alabama Department of Transportation, 2014), Chapter 12, “Fracture Critical Members and Fracture Critical Bridges,” instructs inspectors to focus on key areas, particularly areas where stress concentrations may exist; among other details and conditions, the manual mentions a number of details that feature intersecting welds or details that might exhibit high degrees of triaxial constraint.

4.5 FEDERAL HIGHWAY ADMINISTRATION (FHWA) CONSENSUS MEETING

A consensus meeting was held in the Washington, DC, FHWA offices on December 4, 2018. The participants included academic researchers, practicing bridge designers, bridge fabricators, and representatives of owner-agencies. The discussions included:

- a review of current research and practices;
- a review of fundamental behavior, including discussion of ductility, constraint, and triaxiality;

- CIF;
- fatigue;
- fabrication; and
- in-service inspection.

The consensus meeting served to confirm a number of items related to the current state of knowledge of CIF and related topics, expose several gaps in current knowledge, and present several potential improvements to current practices.

CHAPTER 5 - EVALUATING DETAILS FOR SUSCEPTIBILITY TO CIF

5.1 GENERAL CIF EVALUATION PROCEDURE

As described in Section 3.3, three conditions contribute to an elevated susceptibility to CIF, specifically:

1. a sufficiently high net tensile stress, including consideration of residual stresses;
2. a high degree of constraint, preventing local yielding; and
3. a planar discontinuity approximately perpendicular to the primary flow of tensile stress.

Following is a discussion of each condition in detail:

As discussed in Section 3.3.1, it is reasonable to assume that the first condition, *a sufficiently high net tensile stress, including consideration of residual stresses*, is present in any and all members or components subjected to a tensile stress or stress reversal. As previously discussed, virtually all structural steel members are subject to some level of residual stress. Residual stresses include both regions of tensile and compressive stresses, which are always in static equilibrium (i.e., the sum of the resultant tensile and compressive forces equals zero), and the magnitude of residual tensile stresses can potentially exceed the uniaxial yield stress of the material. Theoretically, residual stresses can be quantified, but it is impractical to try to do so outside of the academic research environment. So, for the purposes of evaluating a given detail for susceptibility to CIF, a high level of tensile stress can be assumed to exist whenever that given element is subjected to a net applied tensile stress or stress reversal.

The second condition, *a high degree of constraint, preventing local yielding*, is a function of the specific geometry of a given detail. Most structural steel elements in transportation structures are typically relatively thin, such as girder web plates, stiffeners, diaphragms, cross-frame members, and the like. Relatively thin steel elements, on their own, are not subject to a high degree of constraint and typically can yield when stressed to their yield stress. However, when several such elements are assembled together as they typically are in a steel bridge, there can be many locations where one or more elements constrain other elements. The Mohr's circle illustrations presented in Section 3.1 show that a steel element deforms when a uniaxial tensile stress equal to the yield stress of the material is applied, but that yielding of the material is prevented if orthogonal tensile stresses are introduced, restraining shear deformations. For example, the "Hoan Bridge Detail" discussed in Section 4.1.1 featured the intersection of three welded structural steel plates (the girder web, vertically oriented connection plates, and longitudinally oriented gusset plates), with attached structural elements (bracing members). The girder web was severely constrained and could not yield locally. This condition, combined with a sufficiently high net tensile stress (including consideration of residual stresses) and a crack-like planar discontinuity approximately perpendicular to the primary flow of tensile stress (discussed below), led to elevated susceptibility to CIF.

Any structural steel detail can be evaluated to determine whether it may be subject to a high degree of constraint. As discussed in Section 3.2, the degree of stress triaxiality can be quantified, but trying to do so in a design environment is impractical due to the difficulty associated with quantifying the magnitude of residual stresses and the degree of constraint provided by various attached elements. However, the evaluation need not be quantitative - a qualitative evaluation is technically sufficient. If a given structural steel detail is configured such that the various elements may provide constraint, relatively simple steps can typically be taken to reconfigure the detail such that sufficient relief is provided to allow for local yielding.

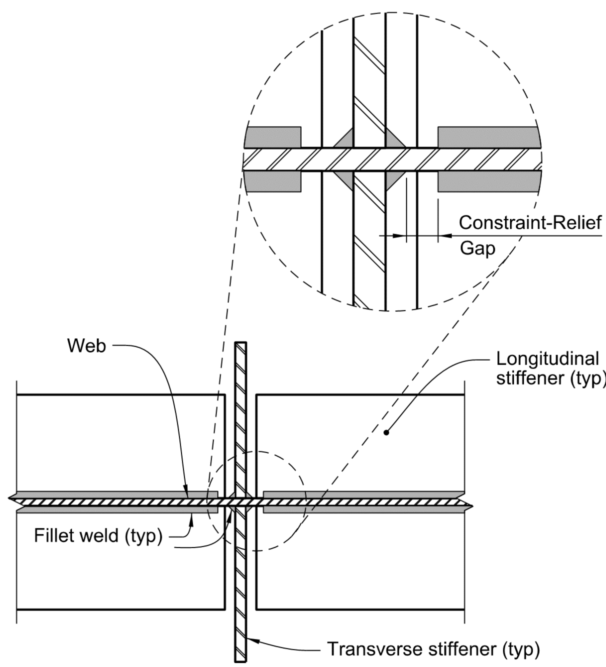
The third condition, *a planar discontinuity approximately perpendicular to the primary flow of tensile stress*, is similarly a function of the specific geometry of a given detail. This type of condition, specifically when it exhibits a “crack-like” or “notch-like” geometry, provides both a stress concentration and a crack initiator. The crack-like or notch-like geometry can arise from any number of sources, some macro, some micro, including narrow gaps in longitudinal attachments, weld discontinuities or imperfections, or similar items. The key is that a plane of discontinuity approximately perpendicular to the primary flow of tensile stress represents a potential problem, whereas a plane of discontinuity parallel to the primary flow of tensile stress does not.

To understand the concept of a planar discontinuity, consider first the term “plane.” For the purposes of this report, a “plane” is defined as “a surface in which if any two points are chosen a straight line joining them lies wholly in that surface” (Merriam-Webster, 2021). An ideal plane has two measurable dimensions, while the third dimension measures as zero; in other words, a plane would have a measurable width and length, but no thickness. For the purposes of evaluating susceptibility to CIF, a “planar discontinuity” is a discontinuity in a structure that takes the form of a plane. Theoretically, a planar discontinuity might have zero “thickness” (zero gap between the discontinuous structural elements), but planar discontinuities generally have some measurable thickness (some measurable gap between the discontinuous structural elements).

A classic example of a planar discontinuity approximately perpendicular to the primary flow of tensile stress would be the interruption of a longitudinal web stiffener attached to a girder web in a region where the web is subjected to tension or stress reversal. In some existing structures, such a discontinuity might occur at a location where a longitudinal web stiffener is interrupted to avoid conflict with a transverse web stiffener. The “plane” associated with the planar discontinuity is the plane formed by the end of the longitudinal stiffener; this plane has a width (the width of the longitudinal stiffener) and a height (the thickness of the longitudinal stiffener), and is oriented approximately perpendicular to the longitudinal axis of the girder, and thus perpendicular to the primary flow of tensile stress along the length of the girder in the tension flange, web, and longitudinal stiffener.

If such a planar discontinuity is very “thin” (i.e., if there is a very small gap between the discontinuous structural elements, in this case between the end of the longitudinal web stiffener and the face of the transverse web stiffener), it might represent a crack-like or notch-like feature. The presence of such a crack-like or notch-like planar discontinuity approximately perpendicular to the primary flow of tensile stress would contribute to an elevated susceptibility to CIF; the tension would act to open the discontinuity further and there would be stress concentrations at the end of the discontinuity. However, if the gap between the discontinuous structural elements is wide, it might represent an adequately sized “constraint-relief gap,” which would reduce susceptibility to CIF. See Section 2.4.2 for detailed discussion of constraint-relief gaps; in this case the constraint-relief gap would be the gap between the ends of the longitudinal stiffener-to-web fillet welds and the toes of the transverse stiffener-to-web fillet welds.

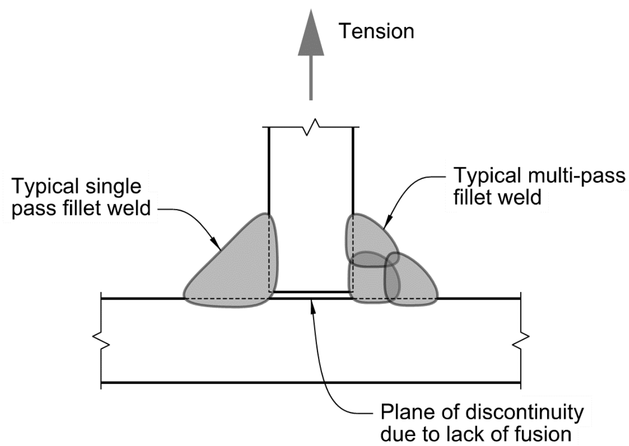
For example, consider the detail shown in Figure 31. This figure shows a plan view of a steel girder web with transverse web stiffeners (vertical stiffeners) and longitudinal web stiffeners. In this case, the longitudinal web stiffeners are interrupted at the transverse web stiffeners, with a small gap between the ends of the longitudinal web stiffeners and the transverse web stiffeners. The gaps represent discontinuities in the longitudinal web stiffeners; longitudinal stress in the longitudinal web stiffeners cannot flow across the gap and instead transitions into the web at the gaps. The gaps thus represent a plane of discontinuity approximately perpendicular to the primary flow of stress, with stress concentrations at the points where the ends of the longitudinal stiffeners are attached to the girder web. For the purposes of this discussion, imagine that the web and the longitudinal web stiffeners are subjected to tension or stress reversal. If the gaps are sufficiently wide, there may be sufficient "web gap" (constraint-relief gap) distance to allow the web to yield prior to fracture. However, if the gaps are narrow, then the combination of the transverse and longitudinal web stiffeners act to prevent local through-thickness yielding of the web. Thus, this type of detailing could be found to be susceptible to CIF. This type of detailing may exist in older bridges.



Source: FHWA

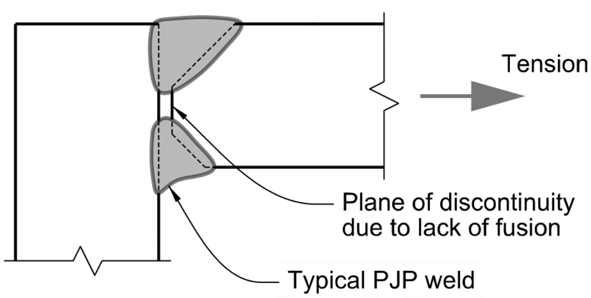
Figure 31. Illustration. Plan view of girder web with attached transverse and longitudinal stiffeners.

Other examples of a planar discontinuity are less obvious. For example, the naturally occurring plane of unfused steel between back-to-back fillet welds, or in a partial joint penetration (PJP) weld, in a T-joint or a corner joint would represent a plane of discontinuity. See Figure 32 and Figure 33. These are not welding defects or imperfections. The welds shown in these figures meet applicable design and specification criteria, but by design, they are not intended to be full-penetration welds and so they naturally have some discontinuity. If the plane of this discontinuity is oriented perpendicular, or nearly perpendicular, to the primary flow of tension in the connection, that tension acts to try to open the discontinuity further, with stress concentrations at the ends of the discontinuity. Since the ends of discontinuity feature a narrow or sharp, "crack-like," geometry, they can serve as crack initiators.



Source: FHWA

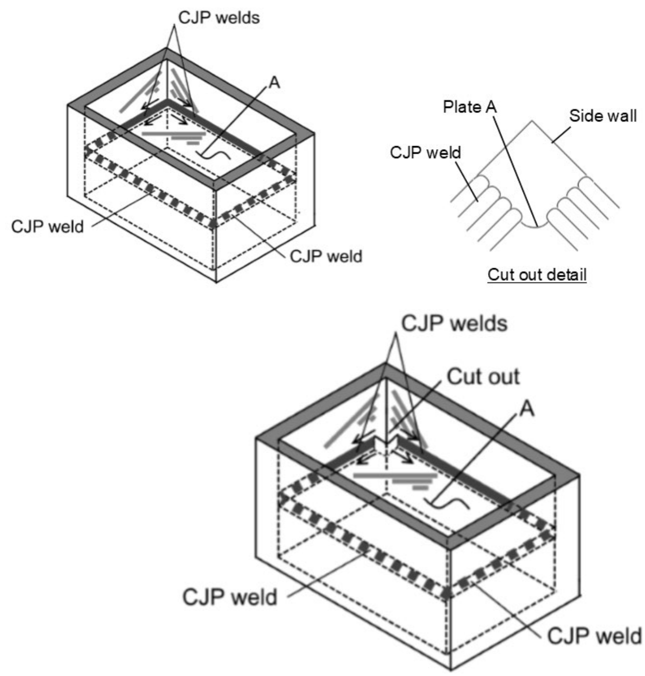
Figure 32. Illustration. Fillet welded T-joint subjected to tension.



Source: FHWA

Figure 33. Illustration. Partial joint penetration (PJP) corner joint subjected to tension.

Generally, elevated susceptibility to CIF is associated with the presence of all three conditions. An elevated susceptibility to CIF is possible when only two of the conditions exist, but these cases are relatively uncommon in most typical steel bridge structures. Consider the case of a highly constrained box weldment, such as that shown in Figure 34. Shown in the upper left portion of the figure is a four-sided box member with an interior plate at mid-depth. The four side plates are welded to each other with CJP welds and the interior plate is also welded to the four side plates with CJP welds. No constraint-relief gaps or other interruptions of the connections are provided. Such a detail would be subject to high residual stresses, resulting from the heating and cooling associated with the CJP welds, and would also be subject to a high level of constraint. Such a detail would be subject to an elevated susceptibility to CIF even if no externally applied tensile loading was present. This type of detail would also be subject to greater susceptibility to hydrogen-assisted cracking due to the high level of residual stresses. In fact, a weldment such as this could potentially crack or fully fracture during fabrication. This is an extreme case; typically constraint-relief gaps (the "cut out" details shown in the upper right and lower portions of Figure 34) are provided, both to help relieve constraint and also to facilitate weld quality.



Source: FHWA

Figure 34. Illustration. Highly constrained box weldment (Medlock et al., 2019).

It is likewise easy to imagine other scenarios where it may appear that only two of the three conditions exist, but the detail might still be subject to an elevated susceptibility to CIF. However, in most situations found in normal design practice, elevated susceptibility to CIF is associated with the presence of all three conditions.

With this understanding of the three conditions in mind, virtually any structural steel detail can be easily evaluated for susceptibility to CIF. A summary is provided below. Later in this report, the CIF evaluation procedure is summarized using a "scorecard" format. Numerical scoring values can be used to assign relative weights to conditions that can contribute to CIF. These weights are subjective; the scoring values used by the authors are presented below.

Condition 1: A sufficiently high net tensile stress, including consideration of residual stresses. As noted in Section 3.3.1, it is reasonable to assume that this condition is present in any and all members or components subjected to a tensile stress or stress reversal. For the purposes of completing the "CIF Evaluation Scorecard," the following scoring values could be used:

- **Cases where the area of interest is subjected to a net applied tensile stress or stress reversal: Score = 1.0**

- **Cases where the area of interest is subjected to a net applied compressive stress under any and all conditions: Score = 0.0**

Condition 2: A high degree of constraint, preventing local yielding. Details that feature the intersection of multiple welded steel elements, generally in a roughly orthogonal configuration, may be indicative of this condition. For example, a detail featuring the intersection of a web plate, a vertical stiffener, and a longitudinal stiffener or other longitudinal attachment, may be subject to a high degree of constraint. Consider the potential for some part of the detailing to offer relief to the constraint, such as the presence of an appropriately detailed and sized constraint-relief gap in the constraining element. For the purposes of completing the "CIF Evaluation Scorecard," the following scoring values could be used:

- **Details featuring a high degree of triaxial constraint** (e.g., details that feature the intersection of three or more welded steel elements, generally in a roughly orthogonal configuration): **Score = 1.0**

- **Details featuring a moderate degree of biaxial constraint** (e.g., details that feature the intersection of two welded steel elements, generally in a roughly orthogonal configuration): **Score = 0.5**

- **Details featuring a low degree of constrain**t (e.g., details that feature no intersecting welded steel elements): **Score = 0.0**

Condition 3: A planar discontinuity approximately perpendicular to the primary flow of tensile stress. In some cases, a planar discontinuity with crack-like or notch-like geometry may be easily recognized, but other cases may involve a more thoughtful evaluation. For example, it may be easy to identify a constraint-relief gap of insufficient width. But more subtle conditions may exist, such as "hidden" planes of discontinuity associated with incomplete fusion in welded connections. Such conditions may be "intentional" (e.g., lack of joint penetration in a T-joint made with fillet welds or a partial joint penetration weld) or "unintentional" (e.g., incomplete fusion in a difficult-to-accomplish complete joint penetration weld). The orientation of the plane of discontinuity is also important; a plane of discontinuity parallel to the primary flow of tension stress is generally not a concern, but a plane of discontinuity approximately perpendicular to the primary flow of tension stress is potentially problematic.

In some cases, the discontinuity may clearly be narrow and/or sharp and may obviously represent a "crack-like" condition. In other cases, a more careful examination may be warranted to determine whether the discontinuity is narrow or sharp enough to be considered "crack-like" or wide enough, with blunt enough tips, to be considered a sufficient constraint-relief gap. The key word is "planar," which indicates the discontinuity generally takes the form of a plane, i.e., "a flat surface on which a straight line joining any two points on it would wholly lie." This implies the discontinuity exhibits more of a two-dimensional geometry, rather than a three-dimensional geometry where the discontinuity has noticeable "depth." A discontinuity that has more of a three-dimensional geometry, where all three dimensions are of noticeable size, might be a candidate for consideration as a constraint-relief gap rather than a planar discontinuity.

Furthermore, it is important to consider the orientation of the planar discontinuity; a planar discontinuity parallel to the flow of primary tension stress is generally not a concern, but a planar discontinuity approximately perpendicular to the flow of primary tension stress is generally problematic.

For the purposes of completing the “CIF Evaluation Scorecard,” the following scoring values could be used

- **Details featuring a planar discontinuity approximately PERPENDICULAR to the primary flow of tensile stress: Score = 1.0**
- **Details featuring a planar discontinuity approximately PARALLEL to the primary flow of tensile stress: Score = 0.0**
- **Details NOT featuring a planar discontinuity: Score = 0.0**

The scores for each of the three conditions could then be added to determine the total score for the evaluation. The following criteria could then be used to evaluate the total score.

For evaluation of new designs:

- **Details with a total score of 2.5 or higher: The detail has a HIGH level of susceptibility to CIF.** Actions that can be taken to redesign or reconfigure the detail to reduce the susceptibility to CIF typically include revising the detail so that an interrupted longitudinal element is made continuous, or reconfiguring the design to reduce the level of constraint.
- **Details with a total score of 2.0 or lower: The detail has a LOW level of susceptibility to CIF.** Redesign or reconfiguration of the detail is not indicated.

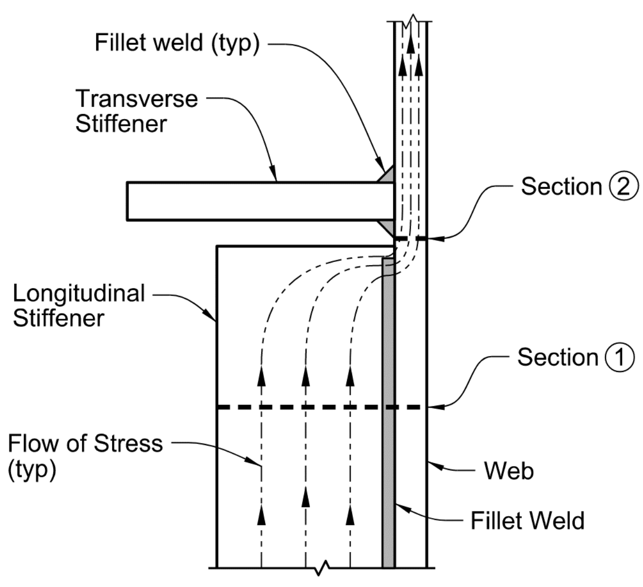
For evaluation of existing structures:

- **Details with a total score of 3.0: The detail has a HIGH level of susceptibility to CIF.** Actions that can be taken to retrofit the detail to reduce the level of susceptibility to CIF can be found in Connor and Lloyd (2017).
- **Details with a total score of 2.5: The detail MAY have a HIGH level of susceptibility to CIF.** Further evaluation of the structure could be undertaken to inform the decision about whether to implement some type of retrofit to reduce the level of susceptibility to CIF. Alternately, a conservative decision could be made, without further evaluation, to implement some type of retrofit to reduce the level of susceptibility to CIF.
- **Details with a total score of 2.0 or lower: The detail has a LOW level of susceptibility to CIF.** Retrofit of the structure is not indicated.

A lower threshold for new designs reflects that the redesign or reconfiguration of a detail in a new design is generally very easy to undertake without incurring increased cost or complexity.

Different criteria may be used for the evaluation of existing structures because retrofits can be costly and/or complicated and may have consequences beyond just structural considerations (e.g., impacts on the traveling public). The "further evaluation" actions in the case of an existing structure with a total score of 2.5 could include items such as more detailed inspections of the structure (perhaps to clarify the presence of crack-like or notch-like planes of discontinuity), refined analysis to more thoroughly understand the stresses in the element of interest, testing to measure the magnitude of residual stresses, etc. In addition, consideration can be given to the potential consequences of CIF if it were to occur. For example, the consequence of CIF in a non-redundant, two-girder, simple-span bridge carrying high volumes of traffic may be more severe than the consequence of CIF in a highly redundant, multi-girder, multiple-span continuous bridge carrying a very low volume of traffic.

To illustrate the CIF evaluation procedure, consider a detail similar to that shown previously in Figure 31. For illustration purposes, this example considers a single transverse web stiffener and a single longitudinal web stiffener, on the same side of the web. It should be emphasized that this type of detailing could be found to be susceptible to CIF as previously explained, and is only shown here for illustrative purposes. Figure 35 shows this scenario and includes a representation of the flow of an assumed longitudinal tension stress from the longitudinal stiffener into the girder web at the end of the longitudinal stiffener.



Source: FHWA

Figure 35. Illustration. Flow of stress at the end of a longitudinal web stiffener.

At "Section 1," at some distance away from the end of the longitudinal stiffener, the flow of stress is relatively uniformly distributed through the full cross-section of the longitudinal stiffener (and the web as well). The stress is tensile, and it can be assumed that there are tensile residual stresses present as well, so that overall it can be assumed that Condition 1, sufficiently high net tensile stress, including consideration of residual stresses, exists. Thus, for this example, the item "Tensile/Residual Stress" would be receive a score of 1.0 (high).

At "Section 1," the presence of the longitudinal stiffener restricts through-thickness yielding of the web. In this instance, the presence of a longitudinal stiffener, but no other constraining elements, represents a case of biaxial constraint. Thus, for this example, the item "Degree of Constraint" would be receive a score of 0.5 (biaxial).

However, at "Section 1," there is no discontinuity that interrupts the flow of stress. Recall the statement above: "the flow of stress is relatively uniformly distributed through the full cross-section of the longitudinal stiffener (and the web as well)." Therefore, Condition 3, a planar discontinuity approximately perpendicular to the primary flow of tensile stress, does not exist. Thus, for this example, the item "Planar Discontinuity" would be receive a score of 0.0 (not present). Lacking this third condition, it is reasonable to assume that, under normal circumstances, there is not an elevated susceptibility to CIF at "Section 1."

The total score for this detail would then be the sum of the individual item scores of 1.0, 0.5 and 0.0, which totals to 1.5, and this detail would be characterized as having a low susceptibility to CIF.

A summary is provided in Table 1:

Table 1. CIF evaluation scorecard for "Section 1" in Figure 35.

ITEM	SCORE
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	0.5 (biaxial)
3. Planar Discontinuity	0.0 (not present)
TOTAL	**1.5 (low susceptibility to CIF)**

From a fatigue standpoint, this would be a category B detail per AASHTO BDS Table 6.6.1.2.3-1, Description 3.1 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

Next, at "Section 2," at the end of the longitudinal stiffener, the flow of stress from the longitudinal stiffener has transitioned into the web. The stress is tensile, and it can be assumed that there are tensile residual stresses present as well, so that overall it can be assumed that Condition 1, sufficiently high net tensile stress, including consideration of residual stresses, exists. Thus, for this example, the item "Tensile/Residual Stress" would be receive a score of 1.0 (high).

At "Section 2," the presence of the longitudinal stiffener and the transverse stiffener restrict through-thickness yielding of the web. In this instance, the presence of both a longitudinal stiffener and an orthogonally oriented transverse web stiffener represents a case of triaxial constraint. As a basic premise for this example, assume that the gap between the ends of the longitudinal stiffener-to-web welds and the toe of the transverse stiffener-to-web weld (the constraint-relief gap) is very small. Without a sufficiently sized constraint-relief gap to allow for through-thickness yielding of the web, it can be assumed that Condition 2, a high degree of constraint, preventing local yielding, exists. Thus, for this example, the item “Degree of Constraint” would be receive a score of 1.0 (triaxial).

Importantly, at “Section 2,” there is a discontinuity that interrupts the flow of stress. The longitudinal stiffener has ended, and the stress formerly carried by that stiffener now suddenly redistributes into the web. A long, gradual transition of the longitudinal stiffener width is not provided, nor is a transition radius provided. The stiffener ends; its width changes from full width to zero width. A severe stress concentration can be expected at this location. Considering this, it can be seen that Condition 3, a planar discontinuity approximately perpendicular to the primary flow of tensile stress, exists. Thus, for this example, the item “Planar Discontinuity” would be receive a score of 1.0 (perpendicular).

The total score for this detail would then be the sum of the individual item scores of 1.0, 1.0, and 1.0. Therefore, the total score would be 3.0 and this detail would be characterized as having a high susceptibility to CIF.

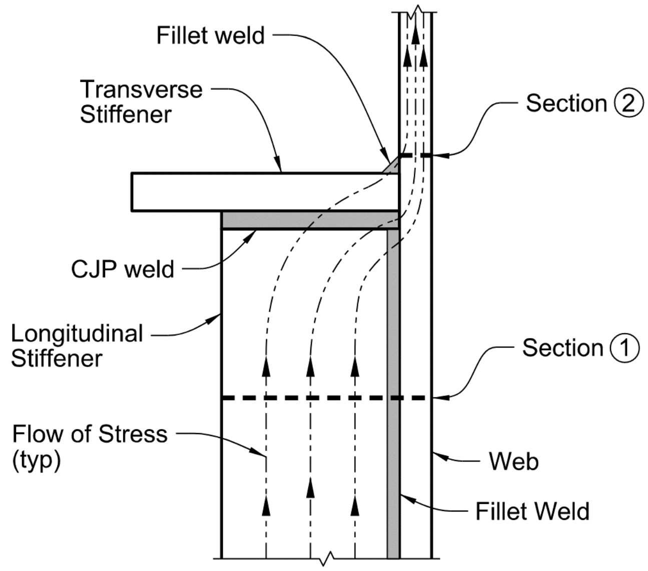
A summary is provided in Table 2:

Table 2. CIF evaluation scorecard for “Section 2” in Figure 35.

ITEM	SCORE
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	1.0 (triaxial)
3. Planar Discontinuity	1.0 (perpendicular)
TOTAL	**3.0 (high susceptibility to CIF)**

From a fatigue standpoint, the performance of this detail has been shown by Pass et al. (1983) and Platten (1980) to be worse than that of an E' detail per AASHTO BDS Table 6.6.1.2.3-1 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

Next, as an academic exercise, consider a modified version of this detail, where the longitudinal stiffener is attached to the transverse stiffener with a CJP weld. Assume the CJP weld is perfectly fabricated and completely free from any discontinuities or other imperfections and has 100 percent fusion. See Figure 36.



Source: FHWA

Figure 36. Illustration. Flow of stress at the end of a longitudinal web stiffener, CJP welded to a transverse stiffener.

At "Section 1" in Figure 36, the conditions are nearly identical to the conditions at "Critical Section 1" in Figure 35. An evaluation of susceptibility to CIF at "Critical Section 1" in Figure 36 may produce the same conclusions; there is low susceptibility to CIF at "Section 1" in Figure 35.

However, at "Section 2" in Figure 36, the conditions are different from those at "Section 2" in Figure 35. There is still tension stress at "Section 2" in Figure 36, so Condition 2, sufficiently high net tensile stress, including consideration of residual stresses, exists. Thus, for this example, the item "Tensile/Residual Stress" would receive a score of 1.0 (high).

"Section 2" in Figure 36 is located just past the transverse stiffener. At this location, a discontinuity interrupts the flow of stress. The longitudinal stiffener is attached to the transverse stiffener, which acts as a de-facto extension of the longitudinal stiffener, but this combined element has ended, and the stress formerly carried by that stiffener redistributes into the web. A long, gradual transition of the longitudinal stiffener width is not provided, nor is a transition radius provided. The stiffener ends; its width changes from full width to zero width. A severe stress concentration can be expected at this location. Considering this, Condition 3, a planar discontinuity approximately perpendicular to the primary flow of tensile stress, exists. Thus, for this example, the item "Planar Discontinuity" would receive a score of 1.0 (perpendicular).

However, at "Section 2" in Figure 36, the degree of constraint is different from that at "Section 2" in Figure 35. Without the longitudinal stiffener, the web is free to experience through-thickness yielding past the transverse stiffener. As a result, Condition 2, a high degree of constraint preventing local yielding, is not present. Thus, for this example, the item "Degree of Constraint" would receive a score of 0.0 (low).

Lacking the condition of constraint, it is reasonable to assume that, under normal circumstances, there is not an elevated susceptibility to CIF at “Critical Section 2” in Figure 36. The total score for this detail would then be the sum of the individual item scores of 1.0, 0.0, and 1.0. Therefore, the total score is 2.0 and this detail would be characterized as having a low susceptibility to CIF.

Note that although there is a low susceptibility to CIF at both “Section 2” and “Section 1” in Figure 36, the detailing represented in this figure would still exhibit very poor fatigue performance, comparable to or worse than an AASHTO Category E or E' detail depending on the size of the longitudinal stiffener (see Pass et al., 1983 and Patten, 1980). Remember, there is still a very severe stress concentration at “Critical Section 1” and there is a significant amount of tensile stress in the long attachment represented by the longitudinal stiffener, which migrates suddenly from that stiffener into the web at the termination of the stiffener-to-web weld.

A summary of the evaluation for “Section 2” in Figure 36 is provided in a scorecard format in Table 3:

Table 3. CIF evaluation scorecard for “Section 2” in Figure 36.

ITEM	SCORE
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	0.0 (low)
3. Planar Discontinuity	1.0 (perpendicular)
TOTAL	**2.0 (low susceptibility to CIF)**

From a fatigue standpoint, the performance of this detail would be expected to be comparable to, or worse than, that of a category E or E' detail (Pass et al., 1980, and Patten, 1983).

5.2 FATIGUE VERSUS CONSTRAINT-INDUCED FRACTURE (CIF)

Some details may not be subject to an elevated susceptibility to CIF, but may exhibit poor fatigue performance. The difference between details that may exhibit poor fatigue performance and those subject to an elevated susceptibility to CIF is noteworthy. CIF represents a sudden, brittle failure mode, providing virtually no warning prior to the fracture event. Fatigue cracking, on the other hand, typically occurs over a longer period, allowing some opportunity for identification of the cracks during periodic in-service bridge inspections.

Neither increased susceptibility to CIF nor poor fatigue performance is a desirable characteristic in a steel bridge detail, but it is important to differentiate the two conditions as they exhibit different performance and may warrant different mitigation approaches. In all cases, both fatigue performance and susceptibility to CIF should be evaluated. Even a detail with low susceptibility to CIF may still exhibit poor fatigue performance, or vice versa.

In the example evaluations of common details (Section 5.3), the fatigue category of each detail, when defined in the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)), is listed to help illustrate these concepts.

5.3 CONCERNS ABOUT DETAILS WITH INTERSECTING WELDS

Proper understanding of CIF provides bridge designers the ability to assess if details featuring intersecting welds are potentially problematic. Intersecting welds are not, in and of themselves, necessarily problematic. Instead, details with high degrees of constraint and crack-like or notch-like planes of discontinuity approximately perpendicular to the primary flow of tensile stress may exhibit elevated susceptibility to CIF.

Examples of commonly used steel bridge details featuring intersecting welds are evaluated in Section 5.5 and it is demonstrated that these details are not subject to an elevated susceptibility to CIF.

This is not to say that details involving the intersection of welds are always free of any concerns. There are some caveats, primarily associated with the potential for the introduction of weld imperfections or discontinuities in highly complex weld details or in weld details that are difficult to fabricate. Such weld imperfections or discontinuities may represent a point of crack initiation or a failure plane, possibly leading to the following:

- greater chance of fatigue cracking in details that are otherwise fatigue-prone; or
- greater susceptibility to CIF in details that otherwise also feature a high degree of triaxial constraint.

As discussed in this report, details with welds that happen to intersect are not necessarily problematic. Details can be evaluated regarding the potential for a high degree of triaxial constraint, and/or for crack-like or notch-like planes of discontinuity approximately perpendicular to the primary flow of tensile stress, to evaluate their level of susceptibility to CIF.

5.4 CONDITIONS OR DETAILS WHERE INTERSECTING WELDS ARE APPROPRIATE

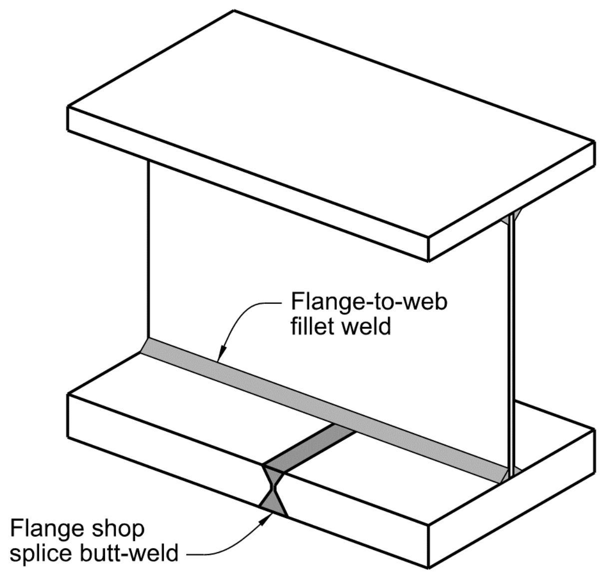
There are many situations where the use of details featuring intersecting welds may be advantageous. For example, many routine details (such as details involving the intersection of flange-to-web fillet welds with flange shop splices accomplished using CJP groove welds in butt joints) offer advantages in terms of efficient structural performance, ease of fabrication, or practicality. In other cases, such as sealing faying surfaces, the intersection of welds is unavoidable, but provides for beneficial corrosion protection; the seal welding of stiffeners is discussed in Sections 5.6.5, 5.6.6, 7.2.1, and 7.2.2. In addition, there may be other, less common or less obvious situations where the use of details with intersecting welds may be beneficial.

5.5 EXAMPLE EVALUATIONS OF COMMON INTERSECTING WELD DETAILS

There are many commonly used steel bridge details that feature intersecting or nearly intersecting welds. Some of the more prevalent are discussed in this section. Each detail is subjected to the evaluation procedure described in Section 5.1, and commentary is provided regarding the detail's potential advantages or disadvantages.

5.5.1 Intersection of Flange-to-Web Welds with Welded Flange Shop Splices

One of the most common bridge details featuring intersecting welds is the intersection of flange-to-web fillet welds with flange shop splices accomplished using CJP groove welds in butt joints. This situation is unavoidable in many bridge designs. The flange shop splice is generally accomplished prior to attaching the flange to the web. While there are clearly intersecting welds in this detail, both a qualitative evaluation and a long history of good performance support that this detail is not subject to an elevated susceptibility to CIF. See Figure 37 and Figure 38.



Source: FHWA

Figure 37. Illustration. Intersection of flange-to-web fillet welds with a CJP groove weld in a butt joint used to accomplish a flange shop splice.

Source: FHWA

Figure 38. Photo. Steel plate girder with flange-to-web welds intersecting CJP groove welds in butt joints used to accomplish flange and web shop splices.

To demonstrate that this detail is not subject to an elevated susceptibility to CIF, evaluate the detail using the procedure described in Section 5.1.

Condition 1: A sufficiently high net tensile stress, including consideration of residual stresses. As noted in Section 3.3.1, it is reasonable to assume that this condition is present in any and all members or components subjected to a tensile stress or stress reversal, due to the presence of potentially high levels of tensile residual stresses. Thus, this detail would receive a score of 1.0 in this category.

Condition 2: A high degree of constraint, preventing local yielding. At their juncture, the flange prevents local through-thickness yielding of the web to some degree, although the constraint is biaxial, not triaxial. Thus, this detail would receive a score of 0.5 in this category.

Condition 3: A planar discontinuity approximately perpendicular to the primary flow of tensile stress. First, look for obvious, immediately visible examples of crack-like or notch-like geometry, such as discrete cut-outs or notches – there are no such features in this detail. Next, consider the welds. The CJP groove welds in butt joints used to fabricate shop splices of flange plates are easily accomplished. These types of shop splice welds are typically performed in the flange and web plates prior to their being welded together into a full plate girder, are accomplished under controlled conditions in a fabrication shop, and are subjected to thorough inspection and testing, which provides a high level of assurance of quality. Consequently, it is reasonable to assume there are no planar discontinuities in the CJP welds used in the butt joints. On the other hand, there are planar discontinuities in the fillet-welded T-joints connecting the flanges to the web (due to intentional lack of joint penetration between the fillet welds), but they are oriented parallel to the primary flow of tensile stress in the flanges and the web. Thus, this detail would receive a score of 0.0 for the planar discontinuities parallel to the primary flow of tensile stress.

A summary of the evaluation is provided in Table 4:

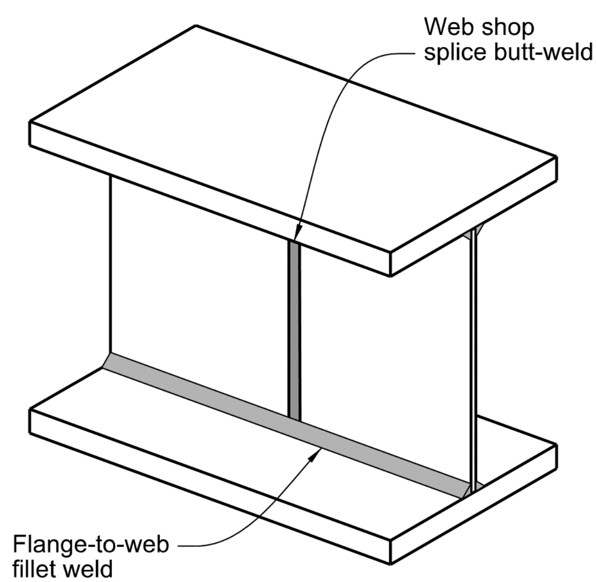
Table 4. CIF evaluation scorecard for the intersection of flange-to-web fillet welds with a CJP groove weld in a butt joint used to accomplish a flange shop splice.

ITEM	SCORE
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	0.5 (biaxial)
3. Planar Discontinuity	0.0 (parallel)
TOTAL	**1.5 (low susceptibility to CIF)**

From a fatigue standpoint, this would be a category B or B' detail per AASHTO BDS Table 6.6.1.2.3-1, Descriptions 3.1 and 5.1 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

5.5.2 Intersection of Flange-to-Web Welds with Welded Web Shop Splices

A similar, related detail involves the intersection of flange-to-web fillet welds with web shop splices accomplished using CJP groove welds in butt joints. This case of intersecting welds is also, for all practical purposes, unavoidable in many bridge designs. The web shop splice is generally accomplished prior to attaching the web to the flange. See Figure 38 and Figure 39.



Source: FHWA

Figure 39. Illustration. Intersection of flange-to-web fillet welds with a CJP groove weld in a butt joint used to accomplish a web shop splice.

Evaluate the detail for the three conditions associated with elevated susceptibility to CIF. The evaluation is very similar to that for the case of flange-to-web welds intersecting a flange shop splice (see Section 5.5.1).

Condition 1: A sufficiently high net tensile stress, including consideration of residual stresses. As noted in Section 3.3.1, it is reasonable to assume that this condition is present in any and all members or components subjected to a tensile stress or stress reversal, due to the presence of potentially high levels of tensile residual stresses. Thus, this detail would receive a score of 1.0 in this category.

Condition 2: A high degree of constraint, preventing local yielding. At their juncture, the flange prevents local through-thickness yielding of the web to some degree, although the constraint is biaxial, not triaxial. Thus, this detail would receive a score of 0.5 in this category.

Condition 3: A planar discontinuity approximately perpendicular to the primary flow of tensile stress. This condition is not present in this type of detail. There is no obvious crack-like or notch-like geometry (no discrete cut-outs or notches). There is also very little chance of a "hidden" plane of discontinuity in the welds. The CJP groove welds in butt joints used to fabricate shop splices of web plates are easily accomplished, since these types of shop splices are typically performed in the flange and web plates prior to their being welded together into a full plate girder. These welds are also subjected to thorough inspection and testing, providing a high level of assurance of quality. There is a possibility of a plane of discontinuity in the T-joint of the flange and the web (due to incomplete fusion between the fillet welds), but such a plane of discontinuity would be oriented parallel to the flow of primary tension stress in the flanges and the web. Thus, this detail would receive a score of 0.0 for the planar discontinuities parallel to the primary flow of tensile stress.

A summary of the evaluation is provided in Table 5:

Table 5. CIF evaluation scorecard for the intersection of flange-to-web fillet welds with a CJP groove weld in a butt joint used to accomplish a web shop splice.

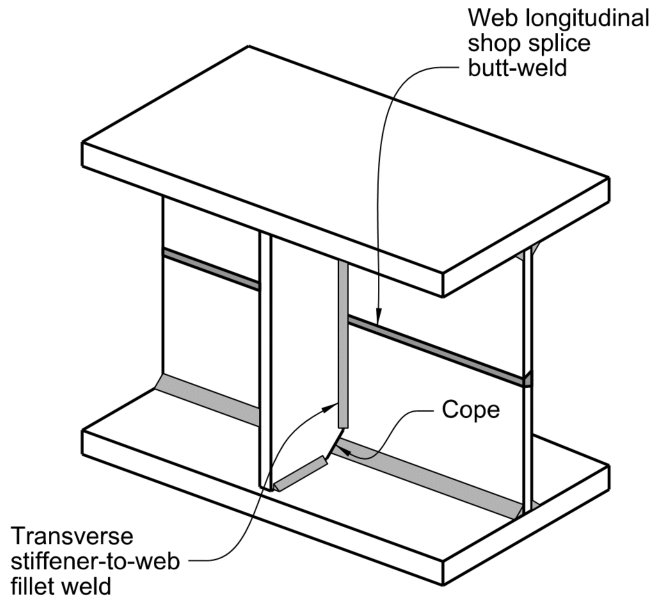
ITEM	SCORE
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	0.5 (moderate)
3. Planar Discontinuity	0.0 (parallel)
TOTAL	**1.5 (low susceptibility to CIF)**

From a fatigue standpoint, this would be a category B or B' detail per AASHTO BDS Table 6.6.1.2.3-1, Descriptions 3.1 and 5.1 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

5.5.3 Intersection of Transverse Stiffener-to-Web Welds with Welded Web Longitudinal Shop Splices

A similar, related detail involves the intersection of transverse (vertical) stiffener-to-web fillet welds with a web longitudinal shop splice accomplished using a CJP groove weld in a butt joint. This detail is perhaps less common since longitudinal shop splices of a web generally occur only in girders with very deep webs, but in such bridges, this detail would be difficult to avoid. See Figure 40.

A related evaluation of CIF at the intersection of the transverse stiffeners, the girder web, and the girder flange is provided in Section 5.6.1.



Source: FHWA

Figure 40. Illustration. Intersection of transverse stiffener-to-web fillet welds with a CJP groove weld in a butt joint used to accomplish a web longitudinal shop splice.

Evaluate the detail for the three conditions associated with elevated susceptibility to CIF.

Condition 1: A sufficiently high net tensile stress, including consideration of residual stresses. As noted in Section 3.3.1, it is reasonable to assume that this condition is present in any and all members or components subjected to a tensile stress or stress reversal, due to the presence of potentially high levels of tensile residual stresses. Thus, this detail would receive a score of 1.0 in this category.

Condition 2: A high degree of constraint, preventing local yielding. At their juncture, the transverse stiffener prevents local through-thickness yielding of the web to some degree, but the constraint is biaxial, not triaxial, and more importantly the constraint only affects a short distance in the direction of the primary flow of tensile stress. The web could easily yield on either side of the stiffener. Thus, this detail would receive a score of 0.0 in this category.

Condition 3: A planar discontinuity approximately perpendicular to the primary flow of tensile stress. This condition is not present in this type of detail. There is no obvious crack-like or notch-like geometry (no discrete cut-outs or notches). There is also very little chance of a "hidden" plane of discontinuity in the welds. The CJP groove welds in butt joints used to fabricate shop splices of web plates are easily accomplished, since these types of shop splices are typically performed in the flange and web plates prior to their being welded together into a full plate girder. These welds are also subjected to thorough inspection and testing, providing a high level of assurance of quality. There is a possibility of a plane of discontinuity in the T-joint of the transverse web stiffener and the web plate (due to incomplete fusion between the fillet welds), but such a plane of discontinuity would be oriented parallel to the flow of primary tension stress in the web. Thus, this detail would receive a score of 0.0 for the planar discontinuities parallel to the primary flow of tensile stress.

A summary of the evaluation is provided in Table 6:

Table 6. CIF evaluation scorecard for the intersection of transverse stiffener-to-web fillet welds with a CJP groove weld in a butt joint used to accomplish a web longitudinal shop splice.

ITEM	SCORE
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	0.0 (low)
3. Planar Discontinuity	0.0 (parallel)
TOTAL	**1.0 (low susceptibility to CIF)**

From a fatigue standpoint, this would be a category C' detail per AASHTO BDS Table 6.6.1.2.3-1, Descriptions 3.1, 4.1, and 5.1 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

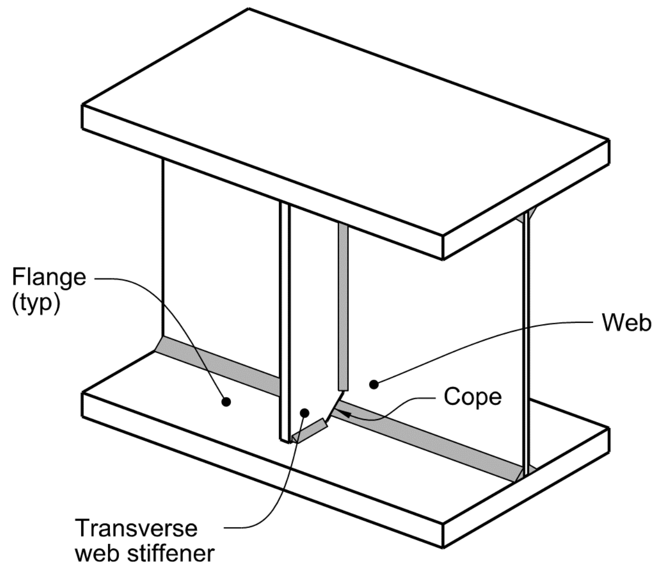
5.6 EXAMPLE EVALUATIONS OF COMMON DETAILS WITH VERTICALLY ORIENTED STIFFENERS

Many commonly used steel bridge details feature vertically oriented stiffeners (such as transverse web stiffeners, transverse connection plates, or bearing stiffeners). Some of the more prevalent are discussed in this section. Each detail is evaluated under the procedure described in Section 5.1, and commentary is provided regarding the detail's potential advantages or disadvantages.

5.6.1 Intersection of a Transverse Web Stiffener with a Girder Web and a Girder Flange

A number of common steel girder bridge details feature the intersection of three welded steel plates configured in a roughly orthogonal arrangement, such as details involving transverse web stiffeners, transverse connection plates, or bearing stiffeners.

Transverse webs stiffeners are provided to increase the shear resistance of the web. These stiffeners are typically welded to the web and to at least one if not both flanges, usually using fillet welds. At locations where the stiffener is welded to both the web and the flange, there exists an instance of the intersection of three orthogonal structural elements (the web, the flange, and the transverse stiffener). Generally, the inside corners of these stiffeners (the corners of the stiffener plates near the intersection of the girder flange and girder web) are coped to clear the continuous girder flange-to-web fillet weld. See Figure 41.



Source: FHWA

Figure 41. Illustration. Intersection of a transverse web stiffener with a girder web and a girder flange.

Evaluate the detail for the three conditions associated with elevated susceptibility to CIF.

Condition 1: A sufficiently high net tensile stress, including consideration of residual stresses. As noted in Section 3.3.1, it is reasonable to assume that this condition is present in any and all members or components subjected to a tensile stress or stress reversal, due to the presence of potentially high levels of tensile residual stresses. Thus, this detail would receive a score of 1.0 in this category.

Condition 2: A high degree of constraint, preventing local yielding. At their juncture, the flange and the transverse web stiffeners prevent local through-thickness yielding of the web. However, a constraint-relief gap is provided by means of the copes of the stiffener; this provides relief of what might otherwise have been triaxial constraint of the web at the location of high tensile stresses in the web and the flange. So at any given position, the constraint would be biaxial. Thus, this detail would receive a score of 0.5 in this category.

Condition 3: A planar discontinuity approximately perpendicular to the primary flow of tensile stress. There is not a planar discontinuity approximately perpendicular to the primary flow of stress in this detail. The planes of discontinuity that might exist if there is incomplete fusion between the fillet welds connecting the transverse web stiffener to the girder flange or the girder web are parallel to the primary flow of tensile stress. Thus, this detail would receive a score of 0.0 for the planar discontinuities parallel to the primary flow of tensile stress.

A summary of the evaluation is provided in Table 7:

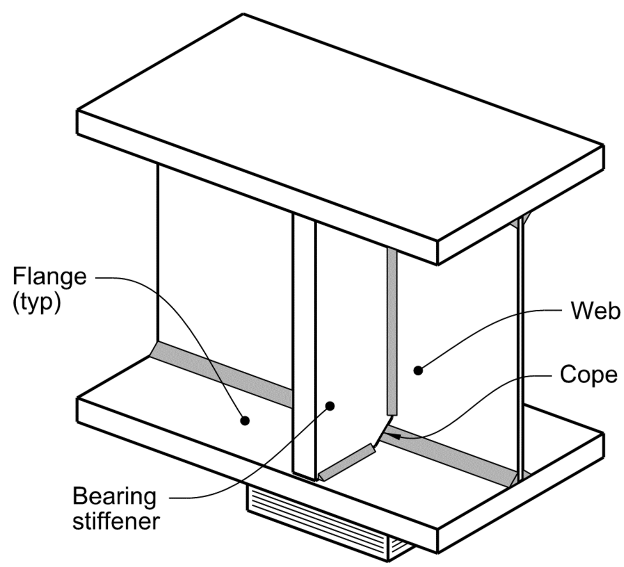
Table 7. CIF evaluation scorecard for the intersection of a transverse web stiffener with a girder web and a girder flange.

ITEM	SCORE
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	0.5 (biaxial)
3. Planar Discontinuity	0.0 (parallel)
TOTAL	**1.5 (low susceptibility to CIF)**

From a fatigue standpoint, this would be a category C' detail per AASHTO BDS Table 6.6.1.2.3-1, Descriptions 3.1 and 4.1 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

5.6.2 Intersection of Bearing Stiffeners with a Girder Web and a Girder Flange

Bearing stiffeners represent a very similar detail to transverse web stiffeners. Again, these stiffeners are typically welded to the web using fillet welds and to at least one if not both flanges, preferably using fillet welds. At locations where the stiffeners are welded to both the web and the flange, there exists an instance of the intersection of three orthogonal structural elements (the web, the flange, and the bearing stiffeners). Generally, the inside corners of these stiffeners (the corners of the stiffener plates near the intersection of the girder flange and girder web) are coped to clear the continuous girder flange-to-web fillet weld. However, bearing stiffeners are often noticeably thicker than transverse web stiffeners, and often also function as connection plates for cross-frames or diaphragms. See Figure 42 (the bearing stiffener on the other side of the web is not visible in this view).



Source: FHWA

Figure 42. Illustration. Intersection of bearing stiffeners with the girder web and a girder flange.

Evaluate the detail for the three conditions associated with elevated susceptibility to CIF.

Condition 1: A sufficiently high net tensile stress, including consideration of residual stresses. As noted in Section 3.3.1, it is reasonable to assume that this condition is present in any and all members or components subjected to a tensile stress or stress reversal, due to the presence of potentially high levels of tensile residual stresses. Thus, this detail would receive a score of 1.0 in this category.

Condition 2: A high degree of constraint, preventing local yielding. At their juncture, the flange and the bearing stiffeners prevent local through-thickness yielding of the web. Bearing stiffeners are often noticeably thicker than transverse web stiffeners, providing more constraint. However, a constraint-relief gap is provided by means of the copes of the stiffeners; this provides relief of what might otherwise have been triaxial constraint of the web at the location of high tensile stresses in the web and the flange. So at any given position, the constraint would be biaxial. Thus, this detail would receive a score of 0.5 in this category.

Condition 3: A planar discontinuity approximately perpendicular to the primary flow of tensile stress. There is not a planar discontinuity approximately perpendicular to the primary flow of stress in this detail. The planes of discontinuity that might exist if there is incomplete fusion between the fillet welds connecting the bearing stiffeners to the girder flange or the girder web are parallel to the primary flow of tensile stress. Thus, this detail would receive a score of 0.0 for the planar discontinuities parallel to the primary flow of tensile stress.

A summary of the evaluation is provided in a scorecard format in Table 8:

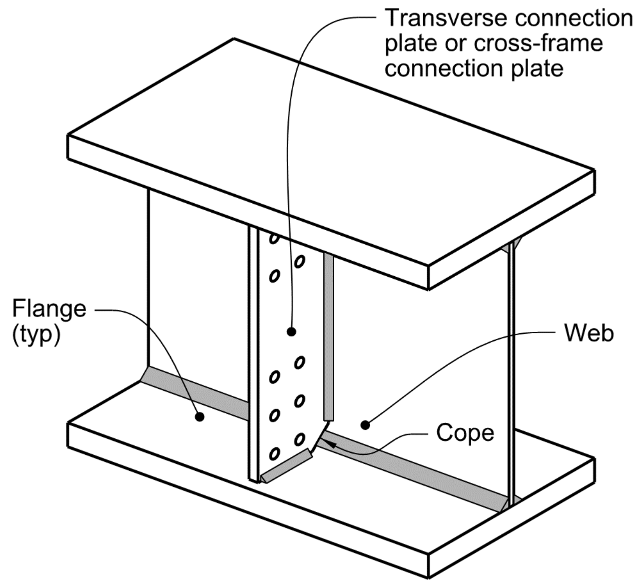
Table 8. CIF evaluation scorecard for intersection of bearing stiffeners with a girder web and a girder flange.

ITEM	**SCORE**
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	0.5 (biaxial)
3. Planar Discontinuity	0.0 (parallel)
TOTAL	**1.5 (low susceptibility to CIF)**

From a fatigue standpoint, this would be a category C' detail per AASHTO BDS Table 6.6.1.2.3-1, Descriptions 3.1 and 4.1 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

5.6.3 Intersection of a Transverse Connection Plate with a Girder Web and a Girder Flange (Welded to Both Flanges)

The third of this group of similar details involves transverse connection plates. A transverse connection plate is a transverse web stiffener or bearing stiffener that also functions to connect a cross-frame or diaphragm to the girder. Transverse connection plates in new designs are designed to be welded to the web and to both flanges per Article 6.10.11.1.1 of the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)). Typically, fillet welds are used for these attachments. Generally, the inside corners of these stiffeners (the corners of the stiffener plates near the intersection of the girder flange and girder web) are coped to clear the girder's flange-to-web fillet weld. See Figure 43.



Source: FHWA

Figure 43. Illustration. Intersection of a transverse connection plate with a girder web and a girder flange (welded to both flanges).

The attachment of a cross-frame to the transverse connection plate can potentially introduce out-of-plane loading on the web. However, the relative stiffness of the transverse connection plate's attachment to the flanges suggests that most of that loading would be distributed to the flanges and not cause significant out-of-plane loading of the web. Article 6.10.11.1.1 of the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)) includes provisions for attachment of transverse connection plates to the girder flanges and webs which minimize the chances of distortion-induced fatigue in the web.

Evaluate the detail for the three conditions associated with elevated susceptibility to CIF.

Condition 1: A sufficiently high net tensile stress, including consideration of residual stresses. As noted in Section 3.3.1, it is reasonable to assume that this condition is present in any and all members or components subjected to a tensile stress or stress reversal, due to the presence of potentially high levels of tensile residual stresses. Thus, this detail would receive a score of 1.0 in this category.

Condition 2: A high degree of constraint, preventing local yielding. At their juncture, the flange and the transverse connection plates prevent local through-thickness yielding of the web. Furthermore, the cross-frame members attached to the transverse connection plate, if loaded in tension, could exacerbate the constraint of the web, although the relative stiffness of the transverse connection plate's attachment to the flanges suggests that a significant portion of that loading would be distributed to the flanges without causing significant out-of-plane loading of the web.A constraint-relief gap is provided by means of the copes of the stiffener; this provides relief of what might otherwise have been triaxial constraint of the web at the location of high tensile stresses in the web and the flange. So at any given position, the constraint would be biaxial. Thus, this detail would receive a score of 0.5 in this category.

This specific example situation assumes the thickness of the transverse connection plate is in the typical range of transverse connection plate thicknesses (i.e., in the range of approximately 5/8 inch to ¾ inch thick). The example shown in Section 5.6.4 features a bearing stiffener with unique connection details demonstrates that if the transverse connection plate is also functioning as a bearing stiffener and is much thicker, the degree of constraint may be greater.

Condition 3: A planar discontinuity approximately perpendicular to the primary flow of tensile stress. In this detail, there are no planar discontinuities approximately perpendicular to the primary flow of stress. Planar discontinuities might exist if there is incomplete fusion in the fillet welds connecting the transverse connection plate to the girder flange or the girder web, but those planes would be parallel to the primary flow of tensile stress. Thus, this detail would receive a score of 0.0 for the planar discontinuities parallel to the primary flow of tensile stress.

A summary of the evaluation is provided in Table 9:

Table 9. CIF evaluation scorecard for the intersection of a transverse connection plate with a girder web and a girder flange (welded to both flanges).

ITEM	SCORE
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	0.5 (biaxial)
3. Planar Discontinuity	0.0 (parallel)
TOTAL	**1.5 (low susceptibility to CIF)**

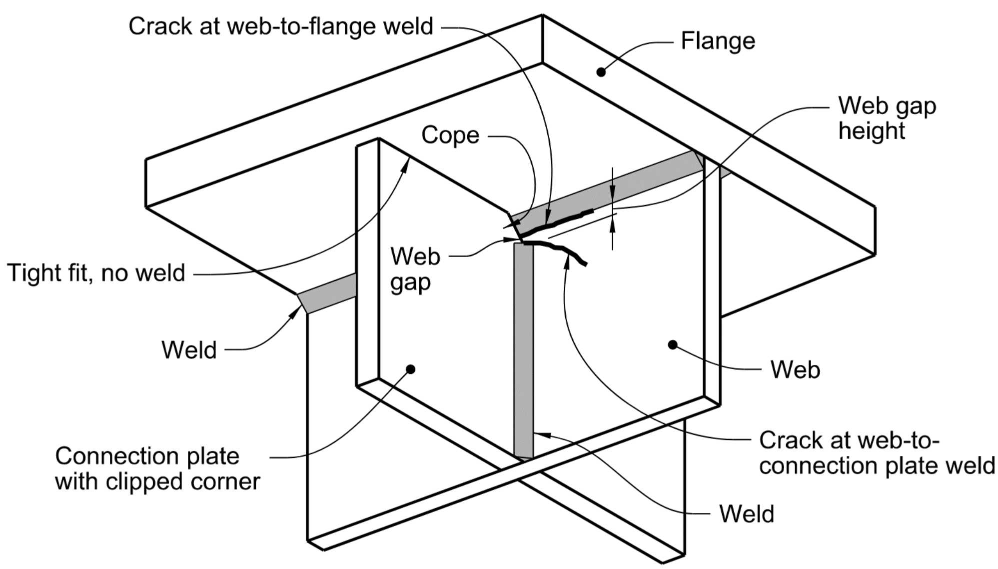
From a fatigue standpoint, this would be a category C' detail per AASHTO BDS Table 6.6.1.2.3-1, Descriptions 3.1 and 4.1 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

5.6.4 Intersection of a Bearing Stiffener (also Functioning as a Transverse Connection Plate) with a Girder Web and a Girder Flange (Welded to the Compression Flange Only)

This detail is essentially the same as the detail discussed in Section 5.6.3, except that in this case the transverse connection plate is welded only to the compression flange, and is not welded to the tension flange. This type of detailing is suspected of contributing to fractures in at least two existing bridges (Fisher et al., 2010, Hodgson et al., 2018). Transverse connection plates in new designs are intended to be welded to the web and both flanges per Article 6.10.11.1.1 of the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)), but in some older designs, a transverse connection plate that was also functioning as a bearing stiffener might not be welded to the tension flange (e.g., the top flange at an interior support of a multiple-span continuous bridge). See Figure 44, which also shows the cracking reported by Hodgson et al. (2018) at a similar detail that experienced an in-service fracture. The orientation of the cracks in this figure suggests the nature of the out-of-plane bending imposed in the web by the cross-frame members attached to the connection plate - the cracks are oriented horizontally, indicating a vertical flow of tensile stress in at least one face of the web.

This represents an unusual situation with a very complicated state of stress in the web. In addition to the web being subjected to tensile stress in the longitudinal direction due to major-axis bending of the girder, it might also be subjected to out-of-plane bending stresses induced by the forces in the cross-frame members connected to the bearing stiffener (which is also functioning as a transverse connection plate), since the stiffener is not welded to the tension flange. These out-of-plane bending stresses would be acting in a vertical direction, with the flow of tensile stress being vertical in one face of the web. This would orient the flow of tensile stress parallel to an "attachment" (i.e., the bearing stiffener), so that the "end" of the attachment (the end of the bearing stiffener not welded to the tension flange) would represent a planar discontinuity approximately perpendicular to the flow of the vertical tensile stress in that face of the web.

If the effective constraint-relief gap (i.e., the "web gap" between the flange-to-web weld and the ends of the stiffener-to-web welds) is narrow, there would be a high degree of triaxial constraint at this same location, resulting in an elevated susceptibility to CIF. In fact, there have been reported cases of CIF occurring in bearing stiffeners with this type of detailing (Hodgson et al., 2018, Fisher and Kaufmann 2010).



Source: FHWA

Figure 44. Illustration. Intersection of a bearing stiffener also functioning as a transverse connection plate with a girder web and a girder flange (welded to the compression flange only).

Evaluate the detail for the three conditions associated with elevated susceptibility to CIF.

Condition 1: A sufficiently high net tensile stress, including consideration of residual stresses. As noted in Section 3.3.1, it is reasonable to assume that this condition is present in any and all members or components subjected to a tensile stress or stress reversal, due to the presence of potentially high levels of tensile residual stresses. Thus, this detail would receive a score of 1.0 in this category.

Condition 2: A high degree of constraint, preventing local yielding. At their juncture, the flange and the bearing stiffeners prevent local through-thickness yielding of the web. Bearing stiffeners are often much thicker than typical transverse web stiffeners or transverse connection plates, increasing the constraint of the web. Furthermore, in this example, there are cross-frame members attached to the bearing stiffeners, so they are also functioning as transverse connection plates. The cross-frame members, when loaded in tension, would impose a larger orthogonal stress on the web (exacerbating the constraint of the web). A constraint-relief gap is provided by means of the copes of the stiffener; if large enough, this constraint-relief gap could provide relief of what would otherwise be triaxial constraint of the web at the location of high tensile stresses in the web and the flange. However, a large gap here would also likely result in an elevated susceptibility to distortion-induced fatigue cracking. For the purposes of this evaluation, assume the gap is small and that thus the degree of constraint is high. Thus, this detail would receive a score of 1.0 in this category.

Condition 3: A planar discontinuity approximately perpendicular to the primary flow of tensile stress. The top termination of the attachment of the bearing stiffener/connection plate to the web represents a planar discontinuity approximately perpendicular to one of the primary flows of tensile stress - in this case, the out-of-plane bending stress induced in the web by the cross-frame member forces. Thus, this detail would receive a score of 1.0 for the planar discontinuities perpendicular to the primary flow of tensile stress.

A summary of the evaluation is provided in Table 10:

Table 10. CIF evaluation scorecard for the intersection of a bearing stiffener / transverse connection plate with a girder web and a girder flange (welded to the compression flange only).

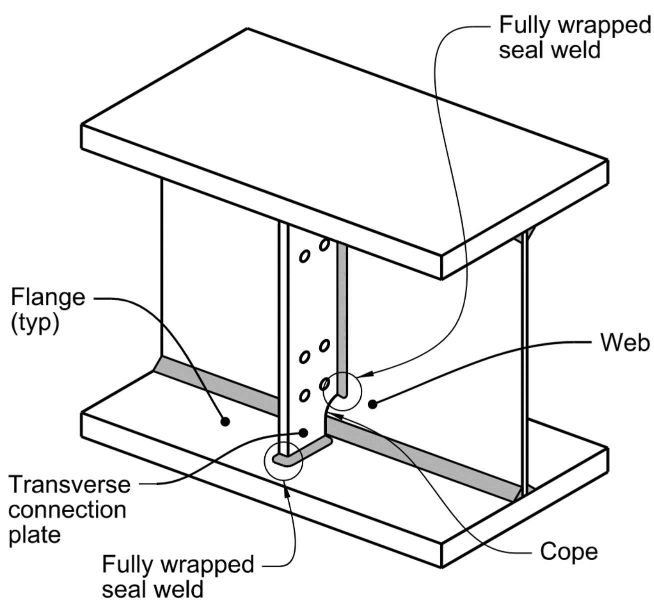
ITEM	SCORE
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	1.0 (triaxial)
3. Planar Discontinuity	1.0 (perpendicular)
TOTAL	**3.0 (high susceptibly to CIF)**

From a fatigue standpoint, there is no comparable detail in AASHTO BDS Table 6.6.1.2.3-1, (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

To reiterate earlier discussion, this type of detailing is suspected of contributing to fractures in at least two existing bridges (Fisher et al., 2010, Hodgson et al., 2018). The provisions of the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)) for stiffeners functioning as transverse connection plates involve welding to both flanges of the girder.

5.6.5 Intersection of a Coped, Seal-Welded Transverse Stiffener, Bearing Stiffener, or Transverse Connection Plate, with a Girder Web and a Girder Flange

Consider a modified version of the typical transverse web stiffener, bearing stiffener, or transverse connection plate (discussed in Sections 5.6.1, 5.6.2, and 5.6.3, respectively). Assume that a cope is provided in the corners of the stiffener to clear the continuous flange-to-web weld, similar to the previously described details. However, in this case, assume the fillet welds wrap around all the free edges of the stiffener as a corrosion protection measure. This type of detailing facilitates sealing the faying surfaces of the stiffener. See Figure 45, which shows a typical transverse connection plate; the welded connections would be similar for bearing stiffeners or a transverse web stiffener. See also Appendix E, which discusses welding mock-up trials of similar details and includes photos. Further study of this detail is suggested in Section 7.2.1.



Source: FHWA

Figure 45. Illustration. Intersection of a coped, seal-welded transverse connection plate with a girder web and a girder flange.

The attachment of a cross-frame to the transverse connection plate could potentially introduce out-of-plane loading on the web. However, the relative stiffness of the transverse connection plate's attachment to the flanges suggests that a significant portion of that loading would be distributed to the flanges without causing significant out-of-plane loading of the web.

Evaluate the detail for the three conditions associated with elevated susceptibility to CIF.

Condition 1: A sufficiently high net tensile stress, including consideration of residual stresses. As noted in Section 3.3.1, it is reasonable to assume that this condition is present in any and all members or components subjected to a tensile stress or stress reversal, due to the presence of potentially high levels of tensile residual stresses. Thus, this detail would receive a score of 1.0 in this category.

Condition 2: A high degree of constraint, preventing local yielding. At their juncture, the flange and the transverse stiffener or connection plate prevent local through-thickness yielding of the web. If the stiffener also functions as a bearing stiffener, it would likely be thicker than a typical transverse web stiffener, increasing the level of constraint. If the stiffener is also functioning as a transverse connection plate, the attached cross-frame members, if loaded in tension, could exacerbate the constraint of the web. However, since a cope is provided in the corner of the stiffener, there is an opportunity to introduce constraint-relief gaps. The size of the constraint-relief gaps would be measured between the toes of the flange-to-web welds and the welds that seal the faying surfaces of the stiffener; if these constraint-relief gaps are too small, they may not provide sufficient relief of the constraint, but if they are adequately sized, they may provide sufficient relief. The size of the stiffener corner copes could also potentially be too large; as explained in C6.10.11.1.1 of the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)), the maximum size of the cope is limited to avoid vertical buckling of the unsupported web. From a scoring standpoint, this detail would receive a score of 0.5 if the constraint-relief gaps are sized sufficiently, or 1.0 if the constraint-relief gaps are not large enough.

Condition 3: A planar discontinuity approximately perpendicular to the primary flow of tensile stress. There is not a planar discontinuity approximately perpendicular to the primary flow of stress in this detail. The planes of discontinuity that might exist if there is incomplete fusion between the fillet welds connecting the stiffener plate to the girder flange or the girder web are parallel to the primary flow of tensile stress. Thus, this detail would receive a score of 0.0 for the planar discontinuities parallel to the primary flow of tensile stress.

A summary of the evaluation is provided in Table 11:

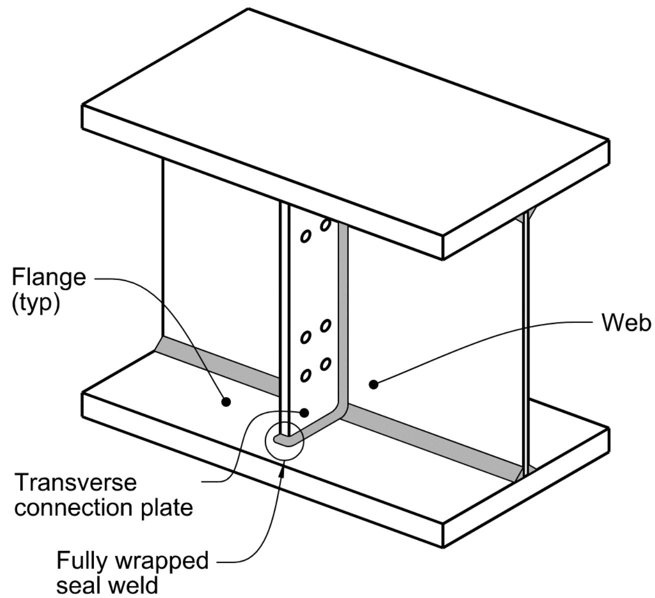
Table 11. CIF evaluation scorecard for the intersection of a coped, seal-welded transverse stiffener, bearing stiffener, or transverse connection plate with a girder web and a girder flange.

ITEM	SCORE
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	0.5 (biaxial) to 1.0 (triaxial) depending on the size of the constraint-relief gaps
3. Planar Discontinuity	0.0 (parallel)
TOTAL	**1.5 to 2.0 (low susceptibility to CIF)**

From a fatigue standpoint, this would be a category C' detail per AASHTO BDS Table 6.6.1.2.3-1, Descriptions 3.1 and 4.1 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

5.6.6 Intersection of a Continuously Seal-welded Transverse Stiffener, Bearing Stiffener, or Transverse Connection Plate, with a Girder Web and a Girder Flange

Consider next a modified version of the typical transverse web stiffener, bearing stiffener, or transverse connection plate (discussed in Sections 5.6.1, 5.6.2, and 5.6.3, respectively). Rather than providing a cope in the corners of the stiffener to clear the flange-to-web weld, instead assume the fillet welds connecting the stiffener to the web and flanges are continuous. Assume the fillet welds wrap around the free edges of the stiffener and continue back into the corner where the stiffener, the web, and the flange intersect. This type of detailing facilitates sealing the faying surfaces of the stiffener as a corrosion protection measure. See Figure 46, which shows a typical transverse web stiffener; the welded connections would be similar for bearing stiffeners or a transverse connection plate. See also Appendix E, which discusses welding mock-up trials of similar details and includes photos.



Source: FHWA

Figure 46. Illustration. Intersection of a continuously seal-welded transverse connection plate with a girder web and a girder flange.

The attachment of a cross-frame to the transverse connection plate could potentially introduce out-of-plane loading on the web. However, the relative stiffness of the transverse connection plate's attachment to the flanges suggests that a significant portion of that loading would be distributed to the flanges without causing significant out-of-plane loading of the web.

This type of continuously welded stiffener/connection plate detail is not known to have been tested or used in steel bridges in the United States. As such, this detail does not yet have a documented record of good performance in bridges. However, this type of detailing has been widely used in the petroleum industry (API, 2014 and Bucknall, 2000) and in Japan (Verma, 2001) with no known reports of problems. Due to the lack of testing in steel bridge applications in the United States, further study of this detail is discussed in Section 8.2.3.

Evaluate the detail for the three conditions associated with elevated susceptibility to CIF.

Condition 1: A sufficiently high net tensile stress, including consideration of residual stresses. As noted in Section 3.3.1, it is reasonable to assume that this condition is present in any and all members or components subjected to a tensile stress or stress reversal, due to the presence of potentially high levels of tensile residual stresses. Thus, this detail would receive a score of 1.0 in this category.

Condition 2: A high degree of constraint, preventing local yielding. At their juncture, the flange and the stiffener plates prevent local through-thickness yielding of the web. If the stiffener also functions as a bearing stiffener, the stiffener would likely be thicker than a typical transverse web stiffener, increasing the level of constraint. If the stiffener is also functioning as a transverse connection plate, the attached cross-frame members, if loaded in tension, could exacerbate the constraint of the web. Since the stiffener is continuously welded into the corner where the stiffener, the web, and the flange intersect, there is no constraint-relief gap, and thus no local relief of the constraint. At the juncture of the stiffener, the web, and the flange, a high degree of triaxial constraint of the web would be expected. Thus, this detail would receive a score of 1.0 in this category.

Condition 3: A planar discontinuity approximately perpendicular to the primary flow of tensile stress. There is not a planar discontinuity approximately perpendicular to the primary flow of stress in this detail, assuming the stiffener has an appropriate controlled fit not only to the girder flanges and web, but also to the flange-to-web welds. The planes of discontinuity that might exist if there is incomplete fusion between the fillet welds connecting the stiffeners to the girder flange or the girder web are parallel to the primary flow of tensile stress. Thus, this detail would receive a score of 0.0 for the planar discontinuities parallel to the primary flow of tensile stress.

A summary of the evaluation is provided in Table 12:

Table 12. CIF evaluation scorecard for the intersection of a continuously seal-welded transverse connection plate with a girder web and a girder flange.

ITEM	SCORE
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	1.0 (triaxial)
3. Planar Discontinuity	0.0 (parallel) assuming the stiffener has an appropriate controlled fit not only to the girder flanges and web, but also to the flange-to-web welds
TOTAL	**2.0 (low susceptibility to CIF)**

From a fatigue standpoint, this would be a category C' detail per AASHTO BDS Table 6.6.1.2.3-1, Descriptions 3.1 and 4.1 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

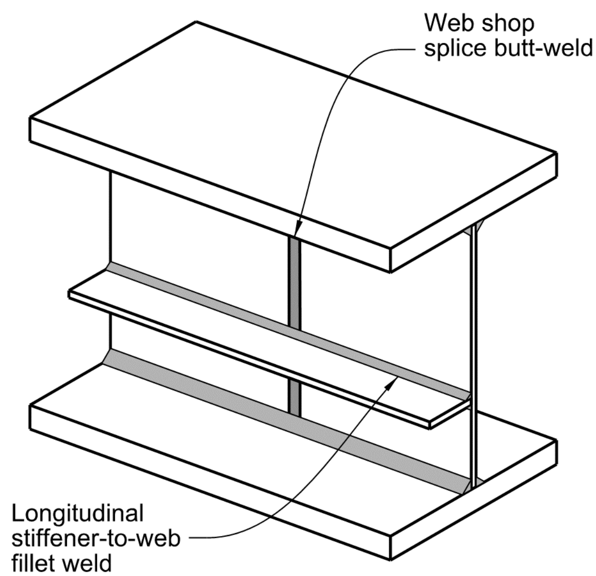
Key for successful implementation of this type of detailing would be providing an appropriate controlled fit of the stiffener not only to the girder web and flange, but also to the flange-to-web welds. An excessively large gap in this region might result in the temptation to fill the gap with excess weld metal. Such practices could lead to increased opportunities to introduce welding discontinuities and imperfections that could manifest themselves as crack-like or notch-like planes of discontinuity approximately perpendicular to the primary flow of tensile stress, one of the three conditions associated with elevated susceptibility to CIF.

5.7 EXAMPLE EVALUATIONS OF COMMON DETAILS WITH LONGITUDINALLY ORIENTED STIFFENERS OR ATTACHMENTS

A number of commonly used steel bridge details feature longitudinally oriented stiffeners or attachments (such as longitudinal web stiffeners or lateral connection plates). Some of the more prevalent are discussed in this section. Each detail is subjected to the evaluation procedure described in Section 5.1, and commentary is provided regarding the detail's potential advantages or disadvantages.

5.7.1 Intersection of a Longitudinal Web Stiffener with a Web Shop Splice

A common steel bridge detail involves the intersection of a longitudinal web stiffener (attached to the web with fillet welds) intersecting a web shop splice (accomplished using a CJP groove weld in a butt joint). The use of longitudinal web stiffeners is generally limited to girders with very deep webs, but in such bridges, this detail would be difficult to avoid. See Figure 47.



Source: FHWA

Figure 47. Illustration. Intersection of a fillet-welded longitudinal web stiffener with a web shop splice accomplished using a CJP groove weld in a butt joint.

Evaluate the detail for the three conditions associated with elevated susceptibility to CIF.

Condition 1: A sufficiently high net tensile stress, including consideration of residual stresses. As noted in Section 3.3.1, it is reasonable to assume that this condition is present in any and all members or components subjected to a tensile stress or stress reversal, due to the presence of potentially high levels of tensile residual stresses. Thus, this detail would receive a score of 1.0 in this category.

Condition 2: A high degree of constraint, preventing local yielding. At their juncture, the longitudinal stiffener prevents local through-thickness yielding of the web to some degree, but the constraint is biaxial, not triaxial. Thus, this detail would receive a score of 0.5 in this category.

Condition 3: A planar discontinuity approximately perpendicular to the primary flow of tensile stress. This condition is not present in this type of detail. There is no obvious crack-like or notch-like geometry (no discrete cut-outs or notches). There is also very little chance of a "hidden" plane of discontinuity in the web shop splice. The CJP groove welds in butt joints used to accomplish shop splices of webs are easily accomplished, since these types of shop splices are typically performed in the flange and web plates prior to their being welded together into a full plate girder. These welds are also subjected to thorough inspection and testing, providing a high level of assurance of quality. There is a possibility of a plane of discontinuity in the T-joint of the longitudinal stiffener and the web due to incomplete fusion between the fillet welds, but such a plane of discontinuity would be oriented parallel to the flow of primary tension stress in the longitudinal stiffener and the girder. Thus, this detail would receive a score of 0.0 for the planar discontinuities parallel to the primary flow of tensile stress.

A summary of the evaluation is provided in Table 13:

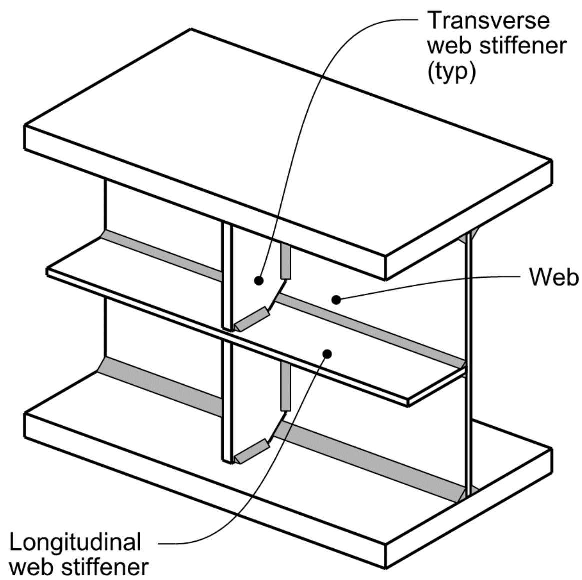
Table 13. CIF evaluation scorecard for the intersection of a fillet-welded longitudinal web stiffener with a web shop splice accomplished using a CJP groove weld in a butt joint.

ITEM	SCORE
1. Tensile/Residual Stress	1.0 (low)
2. Degree of Constraint	0.5 (biaxial)
3. Planar Discontinuity	0.0 (parallel)
TOTAL	**1.5 (low susceptibility to CIF)**

From a fatigue standpoint, this would be a category B detail (in the location away from the longitudinal stiffener termination) per AASHTO BDS Table 6.6.1.2.3-1, Descriptions 3.1 and 5.1 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

5.7.2 Intersection of a Continuous Longitudinal Web Stiffener with a Girder Web and a Discontinuous Transverse Web Stiffener

Longitudinal web stiffeners often intersect both the girder web and also transverse web stiffeners, transverse connection plates, or bearing stiffeners. Various details have been used at the points of intersection of the three orthogonal structural elements. In this case, the longitudinal web stiffener is continuous and the transverse web stiffener is interrupted or discontinuous. This is the preferred detailing for this situation in general, and certainly for cases where the intersection is subjected to tension or stress reversal. See Figure 48, which shows detailing similar to that presented in Table 6.6.1.2.4-1 of the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).



Source: FHWA

Figure 48. Illustration. Intersection of a continuous longitudinal web stiffener with a girder web and a discontinuous transverse web stiffener.

Evaluate the detail for the three conditions associated with elevated susceptibility to CIF.

Condition 1: A sufficiently high net tensile stress, including consideration of residual stresses. As noted in Section 3.3.1, it is reasonable to assume that this condition is present in any and all members or components subjected to a tensile stress or stress reversal, due to the presence of potentially high levels of tensile residual stresses. Thus, this detail would receive a score of 1.0 in this category.

Condition 2: A high degree of constraint, preventing local yielding. At their juncture, the longitudinal web stiffener and the transverse web stiffener prevent local through-thickness yielding of the web. However, a constraint-relief gap is provided by means of the copes of the stiffener; this provides relief of what might otherwise have been triaxial constraint of the web at the location of high tensile stresses in the web and the flange. For this example, it is assumed that the constraint-relief gaps are adequately sized, so at any given position the constraint is only biaxial. Thus, this detail would receive a score of 0.5 in this category. However, if the constraint-relief gaps were not large enough, this detail would receive a score of 1.0 in this category.

Condition 3: A planar discontinuity approximately perpendicular to the primary flow of tensile stress. There is no planar discontinuity approximately perpendicular to the primary flow of stress in this detail. This is a critical concept; since the longitudinal web stiffener is continuous (as are the girder web and flanges) and the transverse web stiffener is interrupted, there is no discontinuity in the primary flow of tensile stress in the members loaded in tension (i.e., the longitudinal stiffeners, the girder web, and the girder flanges). Thus, this detail would receive a score of 0.0 for the planar discontinuities parallel to the primary flow of tensile stress.

A summary of the evaluation is provided in Table 14:

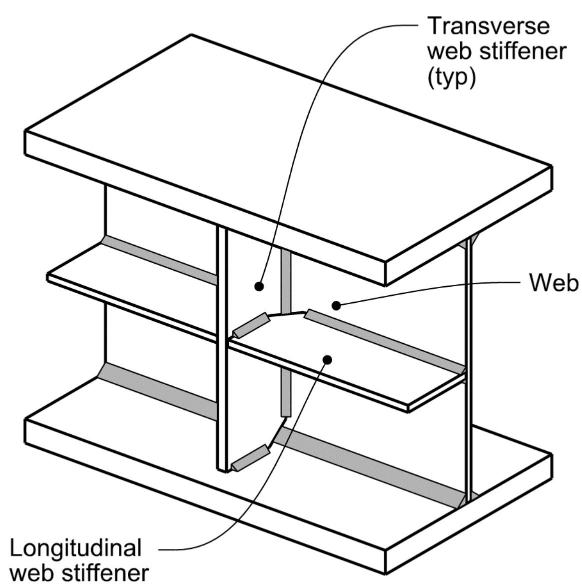
Table 14. CIF evaluation scorecard for the intersection of a continuous longitudinal web stiffener with the girder web and a discontinuous transverse web stiffener.

ITEM	SCORE
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	0.5 (biaxial) to 1.0 (high) depending on the width of the constraint-relief gaps
3. Planar Discontinuity	0.0 (parallel)
TOTAL	**1.5 to 2.0 (low susceptibility to CIF)**

From a fatigue standpoint, this would be a category C' detail per AASHTO BDS Table 6.6.1.2.3-1, Descriptions 3.1 and 4.1 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

5.7.3 Coped and Welded Intersection of a Discontinuous Longitudinal Web Stiffener with a Girder Web and a Continuous Transverse Web Stiffener

Consider next a modified version of the case illustrated in Section 5.7.2, a case where the longitudinal web stiffeners are interrupted or discontinuous and the transverse web stiffeners are continuous. In this case, also assume that the longitudinal web stiffeners are connected to the transverse web stiffeners with fillet welds. For new designs, Table 6.6.1.2.4-1 of the AASHTO BDS (2017a) (23 CFR 625.4(d)(1)(v)) only permits this type of detailing for cases where the intersection is always subjected to compression, and only at bearing stiffeners. However, this type of detailing may be found in existing structures. See Figure 49. For the purposes of this evaluation, assume that this detail is in an existing structure and is subjected to tension or stress reversal.



Source: FHWA

Figure 49. Illustration. Coped and welded intersection of a discontinuous longitudinal web stiffener with a girder web and a continuous transverse web stiffener.

Evaluate the detail for the three conditions associated with elevated susceptibility to CIF. Again, for the purposes of this evaluation, assume that this detail is in an existing structure and is subjected to tension or stress reversal.

Condition 1: A sufficiently high net tensile stress, including consideration of residual stresses. As noted in Section 3.3.1, it is reasonable to assume that this condition is present in any and all members or components subjected to a tensile stress or stress reversal, due to the presence of potentially high levels of tensile residual stresses. Thus, this detail would receive a score of 1.0 in this category.

Condition 2: A high degree of constraint, preventing local yielding. At their juncture, the longitudinal web stiffener and the transverse web stiffener prevent local through-thickness yielding of the web. Constraint-relief gaps are provided by means of the copes of the stiffener; this provides some relief of what might otherwise have been triaxial constraint of the web at the location of high tensile stresses in the web and the flange. The width of these gaps (which would be related to the size of the cope and the sizes and detailing of the welds attaching the longitudinal web stiffeners and the transverse web stiffeners to the girder webs) are a critical parameter. If the gaps at any given position, measured between the weld toes or ends, are sufficiently wide enough to permit through-thickness yielding of the web the constraint would only be biaxial, and the degree of constraint being imposed would not be severe. But if gaps are too narrow, such that they do not provide sufficient relief of the constraint, the gaps could act more like a crack-like or notch-like discontinuity than constraint-relief gaps. Thus, this detail would receive a score of 0.5 in this category if the constraint-relief gaps are sufficiently sized, but would receive a score of 1.0 if the constraint-relief gaps were not large enough.

Condition 3: A planar discontinuity approximately perpendicular to the primary flow of tensile stress. In this case, it is likely that a planar discontinuity approximately perpendicular to the primary flow of tensile stress exists. The fillet welds connecting the longitudinal web stiffener to the transverse web stiffener are likely subject to incomplete fusion, creating a planar discontinuity parallel to the transverse web stiffener. Such a plane would be approximately perpendicular to the primary flow of tensile stress in the longitudinal stiffener. Furthermore, as mentioned above, if the constraint-relief gap is narrow, it may act more like a crack-like or notch-like discontinuity than a constraint-relief gap. Thus, this detail would receive a score of 1.0 for the planar discontinuities perpendicular to the primary flow of tensile stress.

A summary of the evaluation is provided in Table 15:

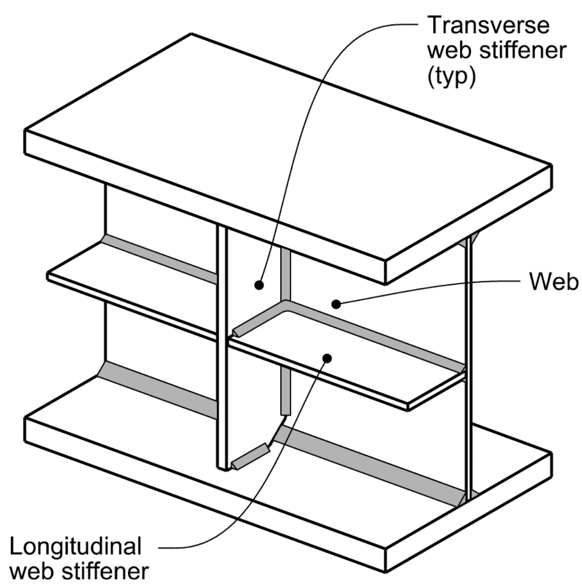
Table 15. CIF evaluation scorecard for a coped and welded intersection of a discontinuous longitudinal web stiffener with a girder web and a continuous transverse web stiffener.

ITEM	SCORE
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	0.5 (biaxial) to 1.0 (triaxial) depending on the width of the constraint-relief gaps
3. Planar Discontinuity	1.0 (perpendicular)
TOTAL	**2.5 to 3.0 (high susceptibility to CIF)**

If this detail occurred in an existing structure and had adequately sized constraint-relief gaps, its score of 2.5 would result in a categorization of "may have high susceptibility to CIF," and further evaluation of the structure could be undertaken to inform the decision about whether to implement some type of retrofit to reduce the level of susceptibility to CIF. Alternately, a conservative decision could be made to implement a retrofit without further evaluation. From a fatigue standpoint, this would be a category C detail per AASHTO BDS Table 6.6.1.2.3-1, Descriptions 3.1, 4.1, and 5.4 as adjusted by Eq. 6.6.1.2.5-4 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

5.7.4 Continuously Welded Intersection of a Discontinuous Longitudinal Web Stiffener with a Girder Web and a Continuous Transverse Web Stiffener

Consider next another modified version of the case illustrated in Section 5.7.2, where the longitudinal web stiffeners are interrupted or discontinuous and the transverse web stiffeners are continuous. In this case, also assume that the longitudinal web stiffeners are connected to the transverse web stiffeners with continuous fillet welds. For new designs, Table 6.6.1.2.4-1 of the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)) only permits this type of detailing in cases where the intersection is always subjected to compression; e.g., at intersections with bearing stiffeners. See Figure 50. This type of detailing could exhibit elevated susceptibility to CIF if subjected to tension or stress reversal (per the evaluation below). This type of detailing may be subjected to such conditions in existing structures. For the purposes of this report, assume that this detail is in an existing structure and is subjected to tension or stress reversal.



Source: FHWA

Figure 50. Illustration. Continuously welded intersection of a discontinuous longitudinal web stiffener with a girder web and a continuous transverse web stiffener.

Evaluate the detail for the three conditions associated with elevated susceptibility to CIF. Remember, for the purposes of this evaluation, assume that this detail is in an existing structure and is subjected to tension or stress reversal.

Condition 1: A sufficiently high net tensile stress, including consideration of residual stresses. As noted in Section 3.3.1, it is reasonable to assume that this condition is present in any and all members or components subjected to a tensile stress or stress reversal, due to the presence of potentially high levels of tensile residual stresses. Thus, this detail would receive a score of 1.0 in this category.

Condition 2: A high degree of constraint, preventing local yielding. At their juncture, the longitudinal web stiffener and the transverse web stiffener prevent local through-thickness yielding of the web. No constraint-relief gaps are provided, so a high degree of triaxial constraint would be expected. Thus, this detail would receive a score of 1.0 in this category.

Condition 3: A planar discontinuity approximately perpendicular to the primary flow of tensile stress. In this case, it is likely that a planar discontinuity approximately perpendicular to the primary flow of tensile stress exists. The fillet welds connecting the longitudinal web stiffener to the transverse web stiffener are likely subject to incomplete fusion, creating a planar discontinuity parallel to the transverse web stiffener. Such a plane would be approximately perpendicular to the primary flow of tensile stress in the longitudinal stiffener. Thus, this detail would receive a score of 1.0 for the planar discontinuities perpendicular to the primary flow of tensile stress.

A summary of the evaluation is provided in Table 16:

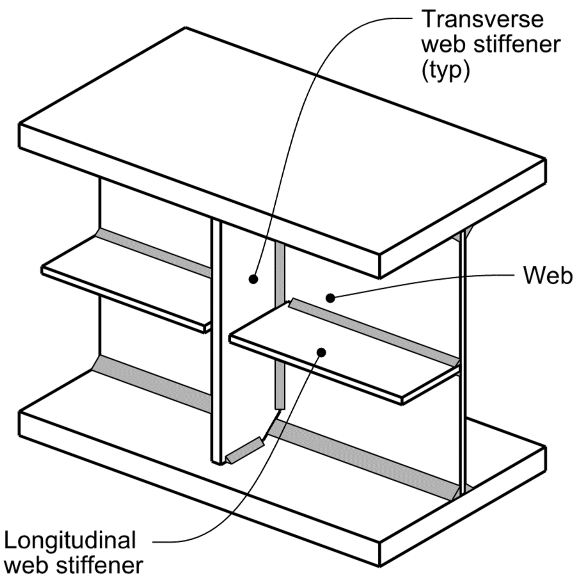
Table 16. CIF evaluation scorecard for a continuously welded intersection of a discontinuous longitudinal web stiffener with a girder web and a continuous transverse web stiffener.

ITEM	SCORE
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	1.0 (triaxial)
3. Planar Discontinuity	1.0 (perpendicular)
TOTAL	**3.0 (high susceptibility to CIF)**

From a fatigue standpoint, this would be a category C' detail per AASHTO BDS Table 6.6.1.2.3-1, Descriptions 3.1, 4.1, and 5.4, as adjusted by Eq 6.6.1.2.5-4 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

5.7.5 Gapped Intersection of a Discontinuous Longitudinal Web Stiffener with a Girder Web and a Continuous Transverse Web Stiffener

Next consider a further modified version of the case illustrated in Sections 5.7.3 and 5.7.4, again a case where the longitudinal web stiffener is interrupted or discontinuous and the transverse web stiffener is continuous. However, in this case, assume that the longitudinal web stiffeners are not connected to the transverse web stiffeners, but instead that there are gaps between the ends of the longitudinal web stiffeners and the transverse web stiffeners. Further assume that these gaps are narrow, say less than ¼ inch wide. See Figure 51. This type of detailing could exhibit elevated susceptibility to CIF if subjected to tension or stress reversal (per the evaluation below), but may be subjected to such conditions in existing structures. For the purposes of this evaluation, assume that this detail is in an existing structure and is subjected to tension or stress reversal; in some older designs the longitudinal stiffener was extended into the tension region of the web where it did not contribute to the stability of web.



Source: FHWA

Figure 51. Illustration. Gapped intersection of a discontinuous longitudinal web stiffener with a girder web and a continuous transverse web stiffener.

Evaluate the detail for the three conditions associated with elevated susceptibility to CIF. Remember, for the purposes of this evaluation, assume that this detail is in an existing structure and is subjected to tension or stress reversal.

Condition 1: A sufficiently high net tensile stress, including consideration of residual stresses. As noted in Section 3.3.1, it is reasonable to assume that this condition is present in any and all members or components subjected to a tensile stress or stress reversal, due to the presence of potentially high levels of tensile residual stresses. Thus, this detail would receive a score of 1.0 in this category.

Condition 2: A high degree of constraint, preventing local yielding. At their juncture, the longitudinal web stiffener and the transverse web stiffener prevent local through-thickness yielding of the web. The critical parameter in this detail is the width of the constraint-relief gaps (the gaps between the ends of the longitudinal web stiffeners and the transverse web stiffeners), as measured between the weld toes or ends. For this example, it is being assumed that the gaps, measured between the weld toes or ends, are not sufficiently wide enough to permit through-thickness yielding of the web. Consequently, the gaps actually act more like a crack-like or notch-like discontinuity than constraint-relief gaps. Thus, this detail would receive a score of 1.0.

Condition 3: A planar discontinuity approximately perpendicular to the primary flow of tensile stress. In this case, the narrow gaps (assumed to be ¼ inch wide) between the ends of the longitudinal web stiffeners and the transverse web stiffeners definitely represent crack-like or notch-like planes of discontinuity approximately perpendicular to the primary flow of tensile stress in the longitudinal web stiffeners and the web. Thus, this detail would receive a score of 1.0 for the planar discontinuities perpendicular to the primary flow of tensile stress.

A summary of the evaluation is provided in Table 17:

Table 17. CIF evaluation scorecard for a gapped intersection of a discontinuous longitudinal web stiffener with a girder web and a continuous transverse web stiffener.

ITEM	SCORE
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	1.0 (triaxial)
3. Planar Discontinuity	1.0 (perpendicular)
TOTAL	**3.0 (high susceptibility to CIF)**

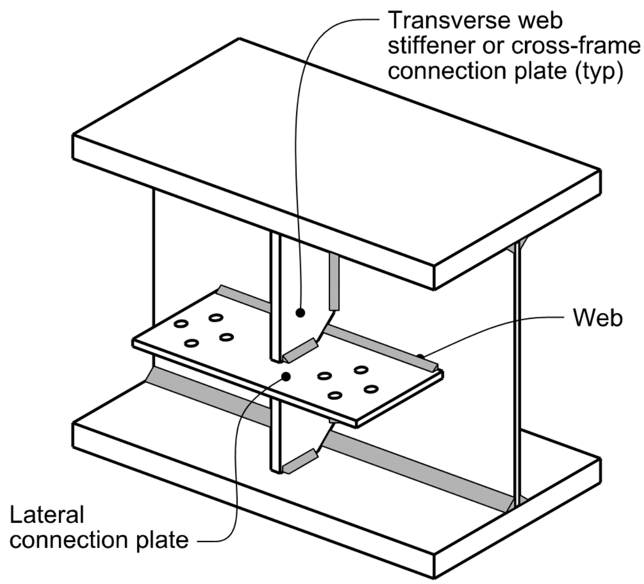
From a fatigue standpoint, the performance of this detail has been shown by Pass et al. (1983) and Platten (1980) to be worse than that of an E' detail per AASHTO BDS Table 6.6.1.2.3-1 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

This detail is conceptually very similar to the “Hoan Bridge detail” (see Section 4.1.1), and would be subject to an elevated susceptibility to CIF unless sufficiently wide constraint-relief gaps were provided.

Furthermore, it is noteworthy that this type of detailing would exhibit extremely poor fatigue performance and the terminations of the longitudinal stiffener-to-web fillet welds in this case would be classified as Category E or E' details per the provisions of Table 6.6.1.2.3-1 of the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

5.7.6 Intersection of a Continuous Lateral Connection Plate with a Girder Web and a Discontinuous Transverse Web Stiffener

Lateral connection plates (i.e., the horizontally oriented gusset plates used to connect lateral bracing systems to the girders) are sometimes located in positions where they intersect transverse web stiffeners, transverse connection plates, or bearing stiffeners. These situations occur largely in older structures, where the lateral connection plates would frame into the girder web at some distance away from the flanges. For example, Figure 52 shows detailing presented in Table 6.6.1.2.4-2 of the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)). This type of detailing provides inefficient, indirect load paths. Modern designs typically use details where the lateral bracing frames directly into the girder flanges or into lateral connection plates that are attached to the flanges rather than into the girder webs, providing a more direct load path.



Source: FHWA

Figure 52. Illustration. Intersection of a continuous lateral connection plate with the girder web and a discontinuous transverse web stiffener.

For the purposes of evaluating susceptibility to CIF, this case is very similar to the case of the intersection of a continuous longitudinal web stiffener with an interrupted or discontinuous transverse web stiffener (described in Section 5.7.2). The main difference is that the lateral connection plate has lateral bracing members attached to it. Those lateral bracing members, if loaded in tension, could impose a larger orthogonal stress on the web (exacerbating the constraint of the web).

Evaluate the detail for the three conditions associated with elevated susceptibility to CIF.

Condition 1: A sufficiently high net tensile stress, including consideration of residual stresses. As noted in Section 3.3.1, it is reasonable to assume that this condition is present in any and all members or components subjected to a tensile stress or stress reversal due, to the presence of potentially high levels of tensile residual stresses. Thus, this detail would receive a score of 1.0 in this category.

Condition 2: A high degree of constraint, preventing local yielding. At their juncture, the lateral connection plate and the transverse web stiffener prevent local through-thickness yielding of the web. Furthermore, the lateral bracing members attached to the lateral connection plate, if loaded in tension, could impose a larger orthogonal stress on the web (exacerbating the constraint of the web). However, a constraint-relief gap is provided by means of the copes of the stiffener; this provides relief of what might otherwise have been triaxial constraint of the web at the location of high tensile stresses in the web and the flange. The key parameter here is the size of the constraint-relief gaps. If the gaps, measured between the weld toes or ends, are sufficiently wide enough to permit through-thickness yielding of the web, at any given position the constraint would only be biaxial, and the degree of constraint being imposed would not be severe. But if gaps are too narrow, such that they do not provide sufficient relief of the constraint, the gaps actually act more like a crack-like or notch-like discontinuity than constraint-relief gaps. Thus, this detail would receive a score of 0.5 in this category if the constraint-relief gaps are sufficiently sized, but would receive a score of 1.0 if the constraint-relief gaps were not large enough.

Condition 3: A planar discontinuity approximately perpendicular to the primary flow of tensile stress. There is no planar discontinuity approximately perpendicular to the primary flow of stress in this detail. This is a critical concept; since the lateral connection plate is continuous (as are the girder web and flanges) and the transverse web stiffener is interrupted or discontinuous, there is no discontinuity in the primary flow of tensile stress in the members loaded in tension (i.e., the longitudinal stiffeners, the girder web, and the girder flanges). Thus, this detail would receive a score of 0.0 for the planar discontinuities parallel to the primary flow of tensile stress.

A summary of the evaluation is provided in Table 18:

Table 18. CIF evaluation scorecard for the intersection of a continuous lateral connection plate with a girder web and a discontinuous transverse web stiffener.

ITEM	**SCORE**
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	0.5 (biaxial) to 1.0 (triaxial) depending on the width of the constraint-relief gaps
3. Planar Discontinuity	0.0 (parallel)
TOTAL	**1.5 to 2.0 (low susceptibility to CIF)**

From a fatigue standpoint, this would be a category C' detail at the transverse stiffener per AASHTO BDS Table 6.6.1.2.3-1, Descriptions 3.1 and 4.1 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)), a category E detail at the terminations of the lateral connection plate per AASHTO BDS Table 6.6.1.2.3-1, Descriptions 6.1 (AASHTO, 2017a), and a category C detail in the attachment of the lateral connection plate to the web per AASHTO BDS Table 6.6.1.2.3-1, Descriptions 6.4 and 5.4 as adjusted by Eq. 6.6.1.2.5-4 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

This particular case could be modified by reconfiguring the detail to have continuous transverse web stiffeners, with discontinuous lateral connection plates that are notched to fit around the transverse web stiffeners. The lateral connection plate would be fillet-welded to the transverse web stiffener (similar to the case illustrated in Section 5.7.2). The evaluation of the resulting detail would conclude that it was subject to a high susceptibility to CIF if the detail is subjected to net tension or stress reversal, similar to the conclusion for the detail illustrated in Section 5.7.2). The fatigue categorization of this detail would be fairly complicated. The connection of the transverse stiffener to the flange would be considered a Category C' fatigue detail per the provisions of Condition 4.1 of Table 6.6.1.2.3-1 of the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)). Without a transition radius in the connection plate, the terminations of the fillet welds attaching the lateral connection plate to the web would be considered Category E details per Description 6.1. For evaluating fatigue of the lateral connection plate itself, the detail would be considered a Category C as adjusted by Eq. 6.6.1.2.5-4, per Description 6.4, which refers back to Description 5.4. This detail is presented in Table 6.6.1.2.4-2 of the AASHTO BDS

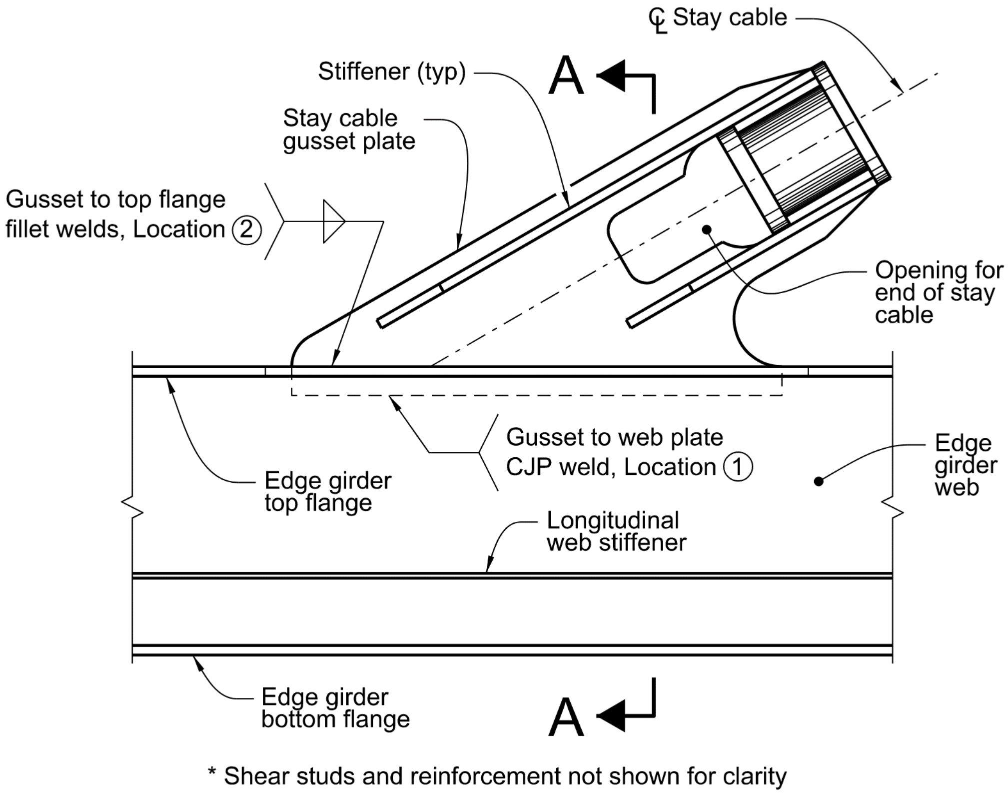
(AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)), for use in cases where it is not practical to attach the lateral connection plate to a flange, the lateral connection plate is placed on the same side of the web as the transverse web stiffener, and the detail is subjected to compression; e.g., at the intersection with a bearing stiffener. However, this type of detailing provides inefficient, indirect load paths. Modern designs typically use details where the lateral bracing frames directly into the girder flanges or into lateral connection plates that are attached to the flanges rather than into the girder webs, providing a more direct load path

If this particular case was further modified by omitting the welded connection of the lateral connection plates to the transverse web stiffeners (similar to the case illustrated in Section 5.7.4), the resulting detail would essentially be the "Hoan Bridge detail" (see Section 4.1.1). Such a detail would be subject to an elevated susceptibility to CIF unless sufficiently wide constraint-relief gaps were provided.

5.7.7 Stay Cable Anchorage Connection to an I-shaped Steel Edge Girder

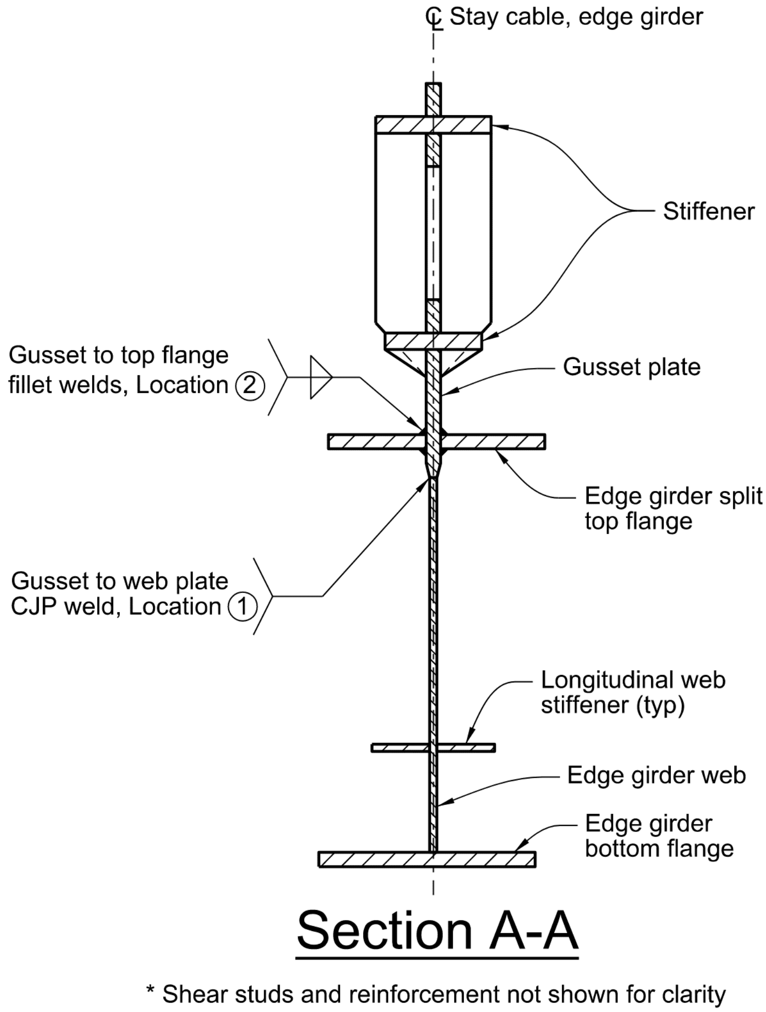
To illustrate the CIF evaluation procedure for a less common type of steel bridge detail, consider a bridge type more complicated than a typical girder-type steel bridge. Cable-stay bridges often use of unique connection details where the stay cables are anchored or otherwise attached to the rest of the bridge structure. Of particular interest in the context of evaluating susceptibility to CIF might be details connecting structural steel stay cable anchorages to structural steel deck system members.

Imagine a cable-stay bridge in which the deck system features steel edge girders and steel floor beams and a concrete deck. Assume the stay cables are anchored to steel edge girders using a projecting gusset plate detail. In this detail, a steel gusset plate might be attached to the edge girder by means of a complete joint penetration butt weld of the gusset plate to the girder web, accomplished through a slot in the top flange of the edge girder. The gusset plate might also be fillet-welded to the edge girder top flange along its sides. The slot might extend longer than the gusset plate on both ends and the ends of the slot might be left open past the leading and trailing edges of the gusset plate. See Figure 53, Figure 54, and Figure 55. The edge girder top flange and gusset plate might also have shear connectors that are eventually encased in deck concrete (not shown in the figures).



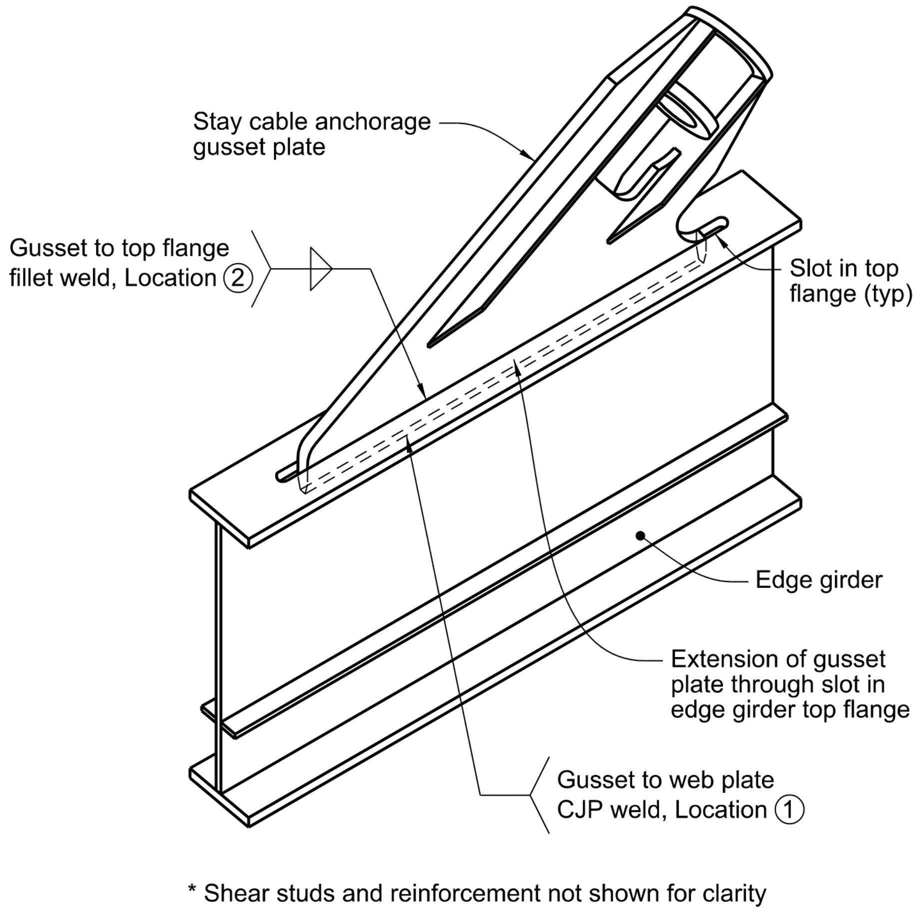
Source: FHWA

Figure 53. Illustration. Elevation view of attachment of stay cable anchorage gusset plate to I-shaped steel edge girder.



Source: FHWA

Figure 54. Illustration. Section view of attachment of stay cable anchorage gusset plate to I-shaped steel edge girder.



Source: FHWA

Figure 55. Illustration. Isometric view of attachment of stay cable anchorage gusset plate to I-shaped steel edge girder.

Many aspects of this cable anchorage detail might warrant evaluation for structural performance, in terms of strength, serviceability, fatigue, etc. Each of those evaluations is very important, but this discussion focuses on only one of those many structural evaluations – an evaluation of susceptibility to CIF.

For the purposes of this example, several simplifying assumptions are made in the interests of clearly illustrating the application of the CIF evaluation procedure to a complex steel detail. Some of these assumptions may be debatable, depending on the specific nature of the actual structure being evaluated; the reader is encouraged to lay aside those debates, accept the assumptions, and focus on the illustration of the CIF evaluation procedure.

For simplicity, it is assumed that the connection of the structural steel details to the concrete deck (via shear connectors) has no impact on the susceptibility to CIF. This is a reasonable and conservative assumption. Such connections may provide composite action for the steel edge girder, or may improve the performance or serviceability of the deck system, but realistically these connections would do little or nothing to prevent or arrest a fracture in the structural steel framing.

For this example, conservatively assume that the top flange and top of the web of the edge girder are in tension or subjected to stress reversal. Assume that the edge girder is subjected to negative moment at the stay cable anchorage and that the cable-stay system is not introducing a sufficient net compression in the deck system to fully overcome, under all loading conditions, the tension in the top flange and top part of the web of the edge girder. This assumption is dependent on the overall structural behavior of the bridge, which is beyond the scope of this example. Assuming this stress condition is conservative for the purposes of evaluating the susceptibility of this detail to CIF.

With these assumptions in place, evaluate the detail with regard to the three conditions associated with elevated susceptibility to CIF. Perform the evaluation at two distinct locations: 1) at the complete joint penetration butt weld connection of the stay cable anchorage gusset plate and the edge girder web; and 2) at the fillet-welded connection of the gusset plate to the edge girder top flange.

At location (1), the complete joint penetration butt weld connection of the stay cable anchorage gusset plate and the edge girder web:

Condition 1: A sufficiently high net tensile stress, including consideration of residual stresses. As noted in Section 3.3.1, it is reasonable to assume that this condition is present in any and all members or components subjected to a tensile stress or stress reversal, due to the presence of potentially high levels of tensile residual stresses. Thus, this detail would receive a score of 1.0 in this category.

Condition 2: A high degree of constraint, preventing local yielding. At the complete joint-penetration butt weld connection of the gusset plate to the edge girder flange, there are no external welded attachments. As a result, there is no externally introduced constraint and, at that specific location, the detail would receive a score of 0.0 for constraint.

Condition 3: A planar discontinuity approximately perpendicular to the primary flow of tensile stress. As has been suggested elsewhere in this report, a properly designed, detailed, executed, and inspected complete joint penetration weld can reasonably be assumed to be free of significant discontinuities; thus, the complete joint penetration butt weld connecting the gusset plate to the edge girder web can be assumed to be free of planar discontinuities and would receive a score of 0.0 for this condition.

A summary of the evaluation is provided in Table 19:

Table 19. CIF evaluation scorecard for the intersection of stay cable anchorage gusset plate with cable-stay bridge edge girder, at location of complete joint penetration butt weld connection of gusset plate to edge girder web.

ITEM	SCORE
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	0.0 (no constraint)
3. Planar Discontinuity	0.0 (parallel)
TOTAL	**1.0 (low susceptibility to CIF)**

From a fatigue standpoint, this would be a category D or E detail per AASHTO BDS Table 6.6.1.2.3-1, Description 6.3 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

At location (2), the fillet-welded connection of the gusset plate to the edge girder top flange:

Condition 1: A sufficiently high net tensile stress, including consideration of residual stresses. As noted in Section 3.3.1, it is reasonable to assume that this condition is present in any and all members or components subjected to a tensile stress or stress reversal due to the presence of potentially high levels of tensile residual stresses. Thus, this detail would receive a score of 1.0 in this category.

Condition 2: A high degree of constraint, preventing local yielding. As the gusset plate passes through the edge girder top flange, the fillet welds connecting the gusset plate to the top flange plate create a condition of biaxial constraint. Thus, at that specific location, the detail would receive a score of 0.5 for constraint.

Condition 3: A planar discontinuity approximately perpendicular to the primary flow of tensile stress. Four fillet welds are used to connect the gusset plate to the edge girder top flange at the point where the gusset plate passes through the top flange. At that location, there are two potential planar discontinuities, one on each side of the gusset plate, both parallel to the gusset plate. For tensile loading in the gusset plate these planar discontinuities are parallel to the primary flow of tensile stress in the gusset plate, which is through the plane of the gusset plate along the axis of the stay cable. For tensile loading in the edge girder top flange these planar discontinuities are also parallel to the primary flow of tensile stress in the edge girder top flange, which is through the plane of the top flange along the longitudinal axis of the edge girder. Since these planar discontinuities are parallel to the primary flow of tensile stress in both of these primary elements (the gusset plate and the edge girder top flange), this detail would receive a score of 0.0 for this condition at this location.

A summary of the evaluation is provided in a scorecard format in Table 20:

Table 20. CIF evaluation scorecard for the intersection of stay cable anchorage gusset plate with cable-stay bridge edge girder at location of connection of gusset plate to edge girder top flange with four fillet welds.

ITEM	**SCORE**
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	0.5 (biaxial)
3. Planar Discontinuity	0.0 (parallel)
TOTAL	**1.5 (low susceptibility to CIF)**

From a fatigue standpoint, there is no directly comparable detail in the AASHTO BDS Table 6.6.1.2.3-1 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)). From various perspectives, various components of this particular detail might be considered:

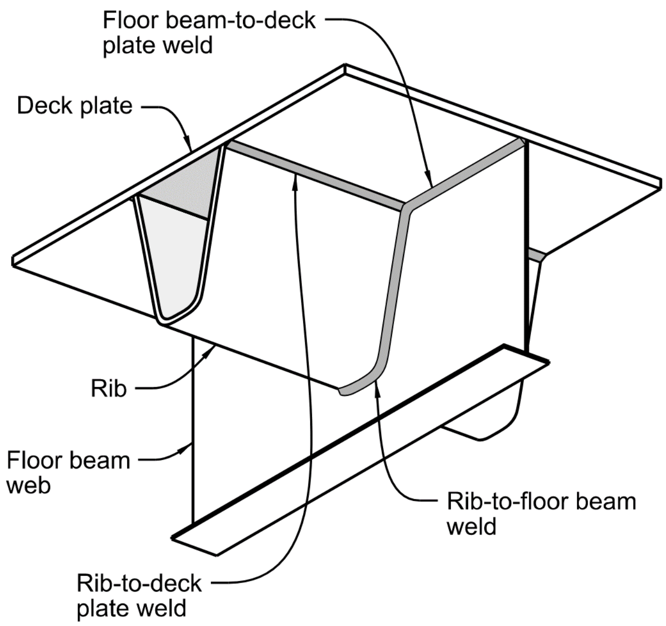
- category C per AASHTO BDS Table 6.6.1.2.3-1, Description 1.3 (AASHTO, 2017a);
- category D per AASHTO BDS Table 6.6.1.2.3-1, Description 1.5 or 3.3 (AASHTO, 2017a); or
- category B, C, D, or E, depending on the radius provided where the gusset plate is attached to the edge girder web, per AASHTO BDS Table 6.6.1.2.3-1, Description 6.1 or 6.2 (AASHTO, 2017a).

5.8 EXAMPLE EVALUATIONS OF OTHER COMMON DETAILS

Many other details are used in steel transportation structures. Two examples are discussed in this section. Each detail is subjected to the evaluation procedure described in Section 5.1.

5.8.1 Intersection of Rib-to-Deck Plate Welds with Rib-to-Floor Beam and Floor Beam-to-Deck Plate Welds in Orthotropic Steel Decks

Orthotropic steel decks often involve details featuring intersecting welds, such as the intersection of rib-to-deck plate welds with rib-to-floor beam welds and floor beam-to-deck plate welds. In bridges with orthotropic steel decks, details like this would be difficult to avoid. Consider the case of continuous ribs with fitted and fully fillet-welded floor beams. See Figure 56.



Source: FHWA

Figure 56. Illustration. Intersection of a rib-to-floor beam fillet weld with a floor beam-to-deck plate fillet weld and a rib-to-deck plate fillet weld.

Evaluate the detail for the three conditions associated with elevated susceptibility to CIF.

Condition 1: A sufficiently high net tensile stress, including consideration of residual stresses. As noted in Section 3.3.1, it is reasonable to assume that this condition is present in any and all members or components subjected to a tensile stress or stress reversal, due to the presence of potentially high levels of tensile residual stresses. Thus, this detail would receive a score of 1.0 in this category.

Condition 2: A high degree of constraint, preventing local yielding. At their juncture, various elements of an orthotropic steel deck would prevent local through-thickness yielding of other elements to some degree. In some locations, the constraint is probably triaxial; for example, the rib walls are constrained by both the fitted and fillet-welded floor beams and by the deck plate. However, the degree of constraint in that location is expected to be relatively low since all of the elements involved are quite thin. It is reasonable to assign this detail a score of 0.5 in this category.

Condition 3: A planar discontinuity approximately perpendicular to the primary flow of tensile stress. This condition is not met in this type of detail. There is no obvious crack-like or notch-like geometry (no discrete cut-outs or notches). There is also very little chance of a "hidden" plane of discontinuity in the welds. Fillet welding is often used for connection of relatively thin elements, including the deck plate, the ribs, and the floor beam webs. The likelihood of lack of joint penetration in these connections is relatively low. Even if there were a plane of discontinuity, that plane would be oriented parallel to the flow of primary tension stress in the girder in most locations. Thus, this detail would receive a score of 0.0 for the planar discontinuities parallel to the primary flow of tensile stress.

A summary of the evaluation is provided in Table 21:

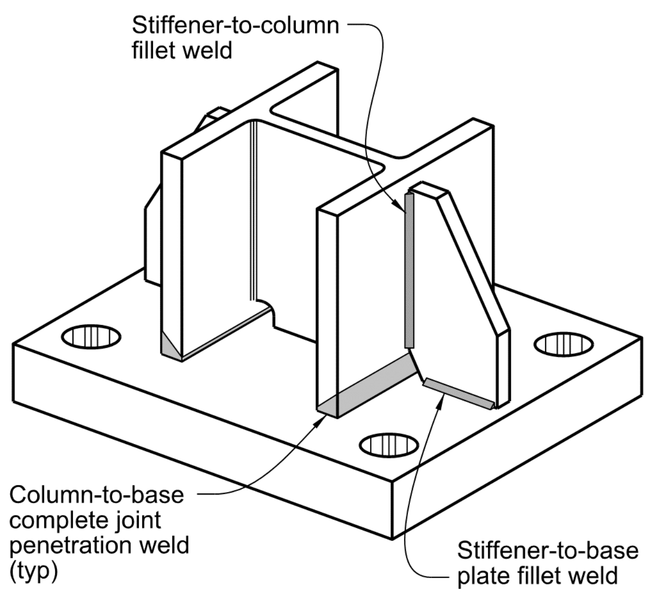
Table 21. CIF evaluation scorecard for the intersection of a rib-to-floor beam fillet weld with a floor beam-to-deck plate fillet weld and a rib-to-deck plate fillet weld.

ITEM	SCORE
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	0.5 (biaxial)
3. Planar Discontinuity	0.0 (parallel)
TOTAL	**1.5 (low susceptibility to CIF)**

From a fatigue standpoint, this would be a category C detail per AASHTO BDS Table 6.6.1.2.3-1, Descriptions 8.5 and 8.6 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

5.8.2 Intersection of Column-to-Base Plate Complete Joint Penetration Groove Welds with Stiffener-to-Base Plate Fillet Welds and Stiffener-to-Column Fillet Welds

Although not specifically found in steel bridges on a regular basis, there are many instances of stiffener-to-column fillet welds intersecting stiffener-to-base plate fillet welds and column-to-base plate complete joint penetration groove welds in other transportation structures, such as high mast light poles, steel columns, arch ribs, etc. See Figure 57.



Source: FHWA

Figure 57. Illustration. Intersection of column-to-base plate complete joint penetration groove welds with stiffener-to-base plate fillet welds and stiffener-to-column fillet welds.

Evaluate the detail for the three conditions associated with elevated susceptibility to CIF.

Condition 1: A sufficiently high net tensile stress, including consideration of residual stresses. As noted in Section 3.3.1, it is reasonable to assume that this condition is present in any and all members or components subjected to a tensile stress or stress reversal, due to the presence of potentially high levels of tensile residual stresses. Thus, this detail would receive a score of 1.0 in this category.

Condition 2: A high degree of constraint, preventing local yielding. At their juncture, the base plate would prevent local through-thickness yielding of the column flanges and the stiffeners to some degree. In some locations, the constraint could be triaxial. For example, the flanges of the column are constrained by both the base plate and the stiffener. At that same location, the column flange is also constrained by the column web immediately opposite of the stiffener. However, a constraint-relief gap is provided by means of the cutouts in the column web at the bottom of the column. This provides relief of the constraint at the location of high tensile stresses in the column flanges. The key parameter here is the size of the constraint-relief gaps. If the gaps, measured between the weld toes or ends, are sufficiently wide enough to permit through-thickness yielding of the web, at any given position, the constraint would only be biaxial, and the degree of constraint being imposed would not be severe. But if gaps are too narrow, such that they do not provide sufficient relief of the constraint, the gaps actually act more like a crack-like or notch-like discontinuity than constraint-relief gaps. Thus, this detail would receive a score of 0.5 in this category if the constraint-relief gaps are sufficiently sized, but would receive a score of 1.0 if the constraint-relief gaps were not large enough.

Condition 3: A planar discontinuity approximately perpendicular to the primary flow of tensile stress. Theoretically, there are no crack-like or notch-like planes of discontinuity approximately perpendicular to the primary flow of tensile stress in the column itself. In practicality, an imperfection in the CJP welds attaching the column flanges to the base plate might constitute such a plane of discontinuity, but CJP welds are typically subjected to a high level of fabrication inspection. In addition, there is not a high degree of constraint at the location of those CJP welds. There could be a plane of discontinuity approximately perpendicular to the primary flow of tensile stress at the bottom of the stiffeners. At that location there could potentially be incomplete fusion between the fillet welds attaching the stiffener to the base plate, especially if the stiffener is relatively thick. Therefore, this detail would receive a score of 0.0 for the lack of planar discontinuities in the CJP-welded connections of the flanges to the base plate, but a score of 1.0 for the planar discontinuities perpendicular to the primary flow of tensile stress in the fillet-welded connections of the stiffeners to the base plate.

A summary of the evaluation is provided in Table 22:

Table 22. CIF evaluation scorecard for the intersection of column-to-base plate complete joint penetration groove welds with stiffener-to-base plate fillet welds and stiffener-to-column fillet welds.

ITEM	SCORE
1. Tensile/Residual Stress	1.0 (high)
2. Degree of Constraint	0.5 (biaxial) to 1.0 (triaxial) depending on the width of the constraint-relief gaps
3. Planar Discontinuity	0.0 (not present) in the CJP-welded attachments 1.0 (perpendicular) in the fillet-welded attachments
TOTAL	**1.5 (low susceptibility to CIF) at the CJP-welded attachments, if constraint-relief gaps are wide enough** **2.5 (potentially high susceptibility to CIF) at the fillet-welded attachments, if constraint-relief gaps are wide enough** **3.0 (high susceptibility to CIF) at the fillet-welded attachments, if constraint-relief gaps are not wide enough**

The score of 2.5 would suggest that such detailing might have a high susceptibility to CIF. However, similar types of detailing are presented in the AASHTO *LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* (AASHTO, 2017c) (23 CFR 625.4(d)(1)(ix), and have exhibited reasonable fatigue performance in experimental testing (Koenigs et al., 2003, Stam, 2009) . Furthermore, the authors are not aware of reports of CIF occurring in service in high mast poles with this type of detailing. This may be due to the ability of the stiffener to yield locally just above the fillet-welded connection or to the presence of adequate constraint-relief gaps. This case illustrates the difficulty associated with trying to "quantify" the degree of constraint present in a complicated detail.

From a fatigue standpoint, this detail is not addressed in AASHTO BDS Table 6.6.1.2.3-1 (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)). See the AASHTO *LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* (AASHTO, 2017c) (23 CFR 625.4(d)(1)(ix) for related discussion.

CHAPTER 6 - MEASURES TO MITIGATE ELEVATED SUSCEPTIBILITY TO CIF

6.1 INSPECTION, REPAIR, AND RETROFIT OF DETAILS SUBJECT TO ELEVATED SUSCEPTIBILITY TO CIF IN EXISTING BRIDGES

The inspection of steel bridges with details that may be susceptible to CIF is covered in the BIRM (Ryan et al., 2012) and the FHWA/NHI *Fracture Critical Inspection Techniques for Steel Bridges – Participant Workbook* (Ryan et al., 2010). Both of these documents provide discussion of how to inspect and evaluate various details.

The evaluation, repair, and retrofit of bridges with details that are subject to an elevated susceptibility to CIF is covered by Russo et al. (2016), *Design and Evaluation of Steel Bridges for Fatigue and Fracture – Reference Manual*, and Connor and Lloyd (2017), *Maintenance Action to Address Fatigue Cracking in Steel Bridge Structures, Proposed Guidelines and Commentary*.

Russo et al. (2016) focuses more on design and detailing of new bridges, but the fundamental concepts discussed can be applied to the evaluation of existing in-service bridges as well.

Connor and Lloyd (2017) discuss suggested repair and retrofit actions for details in existing bridges that may be susceptible to CIF. For example, the report addresses a common detail susceptible to CIF in older bridges – the lateral connection plate detail, sometimes referred to as a "Hoan-like detail." The three repair/retrofit strategies discussed by Connor and Lloyd are summarized in Section 4.2. Repair and retrofit strategies for other CIF-susceptible details are also presented.

6.2 AVOIDING OR MITIGATING DETAILS SUBJECT TO ELEVATED SUSCEPTIBILITY TO CIF IN NEW DESIGNS

When preparing a design of a new steel bridge, it is important, as well as relatively easy, to avoid details that would otherwise be subject to an elevated susceptibility to CIF. Details under consideration can be assessed using the evaluation procedure presented in Chapter 5. Details found to be subject to an elevated susceptibility to CIF can be redesigned or reconfigured to mitigate one or more of the three conditions associated with elevated susceptibility to CIF. The basic concepts, listed in order of importance and ease of implementation, are as follows:

1. If it the intersection of welded elements in areas of net tension or stress reversal is unavoidable, detail longitudinal structural elements (the elements oriented parallel or approximately parallel to the primary flow of tensile stress) as continuous and interrupt transverse elements;

2. If possible, avoid details that introduce a high degree of constraint to steel elements subjected to net tension or stress reversal, particularly details that would introduce a high degree of triaxial constraint;

3. If the intersection of welded elements in areas of net tension or stress reversal is unavoidable and the longitudinal structural element cannot be detailed as continuous,

one way to mitigate the potential to develop high levels of stress triaxiality might be to provide appropriate constraint-relief gaps.

CHAPTER 7 - FINDINGS

7.1 GENERAL FINDINGS

The findings in this report are:

- Steel bridge details featuring intersecting welds are not necessarily at elevated susceptibility to CIF.

- Three conditions contribute to elevated susceptibility of steel bridge details to CIF: a high net tensile stress, a high degree of constraint, and a planar discontinuity approximately perpendicular to the primary flow of tensile stress.

- Evaluating details with respect to criteria rooted in a technical understanding of CIF can help bridge owners identify details that are candidates for redesign and retrofit.

- Retrofitting and redesigning details with intersecting welds without proper understanding of CIF can lead owners to undertake design and/or retrofit strategies that may result in poorer, not better, performance.

- The bridge community may benefit from:

    - Clarification of the term intersecting welds and the development and use of different terms to describe problematic details.

    - Clarification of the influence of intersecting welds on the behavior and performance of steel bridges.

    - Clarification of the difference between details with intersecting welds and details that are subject to an elevated susceptibility to CIF.

    - Clarification of the minimum width for constraint-relief gaps, including consideration of anticipated fracture and fatigue performance.

    - Education regarding the relative effectiveness of constraint-relief gaps along with other measures that can reduce susceptibility to CIF.

- The minimum width of the constraint-relief gaps (i.e., the gaps between weld toes and/or ends) currently prescribed in the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)) and other practice documents is based on a limited analytical study that considered only ½-inch-thick webs and only 0-inch and ¼-inch gaps between weld toes and/or ends.

- Regarding specific details:

    - Connecting lateral connection plates (lateral bracing gusset plates) directly to the girder web provides indirect, inefficient load paths, and in some situations can result in elevated susceptibility to CIF. Lateral bracing gusset plates can instead be connected directly to the girder flanges.

- There are unresolved concerns about the degree of stress triaxiality and susceptibility to CIF associated with large, thick bearing stiffeners provided at interior supports (negative moment regions) of multi-span continuous steel girder bridges – a plausible explanation of reported fractures at these locations is available, but might benefit from a more comprehensive research study.

- The implementation of seal weld detailing for transverse stiffeners, transverse connection plates, and bearing stiffeners offers a potential for improved corrosion protection.

- Implementation of such sealing weld detailing for coped stiffeners would benefit from a more thorough study of the appropriate size of constraint-relief gaps.

- Implementation of such sealing weld detailing for non-coped stiffeners (featuring continuous welding to attach transverse stiffeners, transverse connection plates, or bearing stiffeners to girder flanges and webs) would benefit from more thorough study of the susceptibility to CIF of this type of detailing and from study of the maximum permissible gaps between the corners of the stiffener and the flange-to-web welds and of the welding details that would be used for such a connection.

7.2 POTENTIALLY IMPROVED WELDING DETAILS FOR TRANSVERSE PLATES

There is potential for improved transverse stiffener, transverse connection plate, and bearing stiffener welding details. Two types of detailing are discussed in the following subsections.

7.2.1 Coped, Seal-welded Transverse Stiffeners, Transverse Connection Plates, and Bearing Stiffeners

The first potentially improved detail is a modified version of the typical transverse web stiffener, transverse connection plate, or bearing stiffener (discussed in Sections 5.6.1, 5.6.2, and 5.6.3, respectively). Currently, the detailing that is often used in new steel bridge designs in the United States involves providing a cope in the corners of the stiffener to clear the flange-to-web weld. These types of stiffeners are typically welded to the girder web and are also generally, but not always, welded to the girder flanges. The dimensions of the cope and the provisions for welding to the flanges are discussed in the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)). The detailing currently used also often involves stopping the welds short of the edges of the stiffener; a setback dimension (typically in the range of ¼ inch) is often specified. As a result, the faying surfaces between the stiffener and the girder flanges and web are not fully sealed by welding. The exposed gaps allow the penetration by pickling acids during fabrication (if hot-dip galvanizing is specified) and the development of crevice corrosion of the faying surfaces in service.

Alternate weld details could be used in which the welds are fully wrapped around each faying surface of the stiffener. This would seal the faying surfaces of the stiffener. See Figure 45, which shows a typical transverse connection plate; the welded connections would be similar for a bearing stiffener or a transverse web stiffener. See also Appendix E, which discusses welding mock-up trials of similar details and includes photos of those welding mock-ups.

The use of this detail (with coped stiffener corners) would be particularly applicable when the girder is to be hot-dip galvanized. The copes would allow the free flow of the molten zinc around all the exposed surfaces when the girder is dipped in the zinc bath, including the hard-to-reach surfaces on the insides of the copes. See also Article C6.13.3.7 of the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)) for additional information related to sealing of faying surfaces in galvanized structures.

This detail would be less appropriate when a girder is to be metalized or painted. In those cases, it could be difficult to metalize or paint the hard-to-reach surfaces on the insides of the copes. When metalizing or painting is specified, the detail discussed in Section 7.2.2 could be considered instead.

To implement this detail, the minimum size of the cope, or rather, the minimum size of the constraint-relief gaps between the toes of the flange-to-web welds and the welds connecting the stiffener to the girder flanges and web might need to be different than currently used by bridge designers. In addition, adequate setback of the edge of the stiffener from the edge of the flange to avoid undercutting of the flange when wrapping the sealing weld around the base or top of the stiffener might also need to be different. Section 8.2 suggests areas for future research. For the time being, the following can be considered:

- Minimum constraint-relief gap (measured between the toes of the flange-to-web welds and the welds connecting the stiffener to the girder flanges and web): ¾ inch

- Minimum setback of the edge of the stiffener from the edge of the flange at the base and top of the stiffener: ¼ inch plus the size of the weld used to seal the stiffener-to-flange faying surfaces

The suggested minimum constraint-relief gap dimension of ¾ inch is based on the dimension in Table 6.6.1.2.4-1 of the AAAHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

The suggested minimum setback dimension is based on engineering judgment. To be clear, the undesirable undercutting mentioned above is undercutting of the flange itself. As stated in Article C6.13.3.7 of the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)), "The undercutting of the corner of the stiffener or connection plate itself, even when severe, does not reduce the fatigue performance of the weld, which is controlled by the toe of the transverse fillet weld connecting the stiffener or connection plate to the flange (Spadea and Frank, 2004)."

7.2.2 Continuously Seal-welded Transverse Stiffeners, Transverse Connection Plates, and Bearing Stiffeners Continuously Welded to Girder Flanges and Webs

The second potentially improved detail is a variant of the detail discussed in Section 7.2.1. In this variant, instead of providing full copes in the corners of the stiffener to clear the flange-to-web welds, only a small nominal cope would be provided. The cope would be just big enough to provide an appropriate controlled fit between the stiffener and the flange-to-web weld. Then, the stiffener-to-flange welds and stiffener-to-web welds would be run continuously into the corner such that they intersect with the flange-to-web welds. This type of detailing facilitates sealing of the faying surfaces of the stiffener. See Figure 46, which shows a typical transverse connection plate; the welded connections would be similar for a bearing stiffener or a transverse web stiffener. See also Appendix C, which presents discussion and photographs of welding mock-up trials of similar details.

Key to implementation of this type of detailing would be providing an appropriate controlled fit of the stiffener not only to the girder web and flange, but also to the flange-to-web welds. An excessively large gap in this region might result in the temptation to fill the gap with excess weld metal. Such practices would likely lead to increased opportunities to introduce welding discontinuities and imperfections that could manifest themselves as crack-like or notch-like planes of discontinuity approximately perpendicular to the primary flow of tensile stress, one of the three conditions associated with an elevated susceptibility to CIF.

The use of this detail (continuously welded into the corner of the intersection of the stiffeners with the flanges and web) would be particularly applicable when the girder is to be painted or metallized. Eliminating the traditional open stiffener copes at the flange-to-web welds would remove the need to paint or metalize the hard-to-reach surfaces on the insides of the copes.

This detail would not be appropriate when a girder is to be hot-dip galvanized; the traditional open stiffener copes at the flange-to-web welds facilitate the flow of the molten zinc throughout the length of the girder without providing opportunities to trap air bubbles, leading to voids in the galvanized coating. When hot-dip galvanizing is specified, the detail discussed in Section 7.2.1 could be considered instead.

To implement this detail, various construction aspects should be considered, such as: a) the fit between the corners of the stiffener and the flange-to-web and the associated weld details, and; b) maintaining an adequate setback of the edge of the stiffener from the edge of the flange to avoid undercutting of the flange when wrapping the sealing weld around the base or top of the stiffener. Section 8.2.3 suggests areas for future research. For the time being, the following can be considered:

- Fit between the corner of the stiffener and the flange-to-web weld: Typical fit for fillet-welded connections
- Minimum setback of the edge of the stiffener from the edge of the flange at the base and top of the stiffener: ¼ inch plus the size of the weld used to seal the stiffener-to-flange faying surfaces

The suggested minimum setback dimension is based on engineering judgment. To be clear, the undesirable undercutting mentioned above is undercutting of the flange itself. As stated in Article C6.13.3.7 of the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)), "The undercutting of the corner of the stiffener or connection plate itself, even when severe, does not reduce the fatigue performance of the weld, which is controlled by the toe of the transverse fillet weld connecting the stiffener or connection plate to the flange (Spadea and Frank, 2004)."

CHAPTER 8 - GENERAL DESIGN, DETAILING, AND CONSTRUCTION CONSIDERATIONS, POSSIBLE FUTURE RESEARCH, AND CONCLUSIONS

8.1 DESIGN, DETAILING, AND CONSTRUCTION CONSIDERATIONS

8.1.1 General Design and Detailing Considerations

Article 6.6.1.2.4 (Detailing to Reduce Constraint) of the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)) and the associated commentary address common joints and avoiding details that are susceptible to CIF. In addition to Article 6.6.1.2.4, the evaluation of details to reduce susceptibility to CIF could be considered; Chapter 5 provides a discussion of the three conditions necessary for elevated susceptibility to CIF. Designers might consider evaluating various details, particularly new or unusual details, for an elevated susceptibility to CIF.

Article 6.6.1.2.4 of the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)) currently presents the minimum constraint-relief gap dimension. However, designers should be aware of Section 8.2.1 which discusses the possible future research on the thickness of the constrained member affecting the width of constraint-relief gap to avoid elevated susceptibility to CIF.

The detailing of transverse stiffeners, bearing stiffeners, and transverse connection plates is discussed in Article 6.10.11 and elsewhere in the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)). However, designers should be aware of Section 8.2.2 which discusses possible future research on potential for CIF at large bearing stiffeners at interior supports. Section 8.2.3 discusses possible future research on some elevation of susceptibility of CIF of seal-welded transverse stiffeners, transverse connection plate, and bearing stiffeners continuously welded to girder flanges and webs.

The non-binding AASHTO/NSBA Steel Bridge Collaboration *Guideline 1.4, Guidelines for Design Details* (AASHTO/NSBA, 2006) presents four pages of "Typical Girder Details." Section 2.1.2.4 ("Longitudinal Stiffeners") of the non-binding AASHTO/National Steel Bridge Alliance (NSBA) Steel Bridge Collaboration *Guideline G12.1, Guidelines to Design for Constructability* (AASHTO/NSBA, 2016) includes discussion of when to make longitudinal web stiffeners continuous at intersections with transverse web stiffeners. Designers could consider evaluating these details to assess elevated susceptibility to CIF as described in Chapter 5.

8.1.2 Construction Considerations

Article 11.4.4 (Fit of Stiffeners) of the AASHTO BCS (AASHTO, 2017b) (23 CFR 625.4(d)(1)(iv)) includes provisions and commentary associated with the detailing of transverse stiffeners, bearing stiffeners, and transverse connection plates. In addition, bridge design and construction engineers could benefit from review of Chapters 5, 6, and 7, which discuss identifying and mitigating details that might exhibit elevated susceptibility to CIF, and Section 8.2 which discusses possible future research related to CIF of transverse stiffeners, transverse connection plates, and bearing stiffeners.

The AASHTO/AWS Bridge Welding Code (AASHTO/AWS, 2015) (23 CFR 625.4(d)(1)(vii)) presents the maximum root of dimensions and other weld detail or welding procedure criteria and includes provisions and commentary associated with the detailing of transverse stiffeners, bearing stiffeners, and transverse connection plates to ensure an appropriate controlled fit between the corners of the stiffener and the flange-to-web welds. Bridge designers and fabricators should be aware of Section 8.2, which discusses possible research about potential for susceptibility to CIF of details in steel bridges.

8.2 POSSIBLE FUTURE RESEARCH

8.2.1 Influence of Web Thickness and Constraint-Relief Gap Size on Triaxiality

The minimum constraint-relief gap widths presented in various publications and reports, including the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)), the *Bridge Inspector's Reference Manual* (Ryan et al., 2012), the Participant Workbook for *NHI/FHWA Course 130078 Fracture Critical Inspection Techniques for Steel Bridges* (Ryan et al., 2010), and the *Design and Evaluation of Steel Bridges for Fatigue and Fracture – Reference Manual* (Russo et al. 2016), among others, are inconsistent. The suggestions in these documents are based on underlying research by Mahmoud et al. (2005) and Connor et al. (2005) into the minimum gap between weld toes and/or ends associated with reduced stress concentrations and triaxiality in highly constrained details; this previous research studied the problem only analytically and only considered ½-inch-thick webs and only 0-inch and ¼-inch gaps between weld toes and/or ends. During the consensus meeting discussions (see Section 4.5), it was suggested that the minimum constraint-relief gap dimension to avoid an elevated susceptibility to CIF might be related to the thickness of the member being constrained and/or other factors related to the configuration of the detail in question, and that the resulting suggestions for minimum constraint-relief gap widths may warrant further investigation.

Such investigation could focus on extending the previous work by Mahmoud et al. (2005) and Connor et al. (2005). Such investigation could also focus on extending the previous work by Pass et al. (1983) with regard to fatigue behavior and performance and the effect of the stiffener area to web thickness ratio. The investigation could include more extensive analytical studies considering other web thicknesses, other gap dimensions between weld toes and/or ends, and potentially other variations in geometric parameters (such as stiffener sizes, etc., as discussed by Pass et al., 1983). A variety of stiffener types and orientations could be considered, with the intent of addressing constraint-gap widths in a number of scenarios, locations, and orientations. Experimental study, including fatigue testing followed by low-temperature fracture testing could also be included. The goals would be to better understand the relationship between web thickness and the width of the constraint-relief gap associated with reduced susceptibility to CIF, and the effect of stiffener area to web thickness ratio on fatigue and fracture behavior and performance.

8.2.2 Potential for CIF at Large Bearing Stiffeners at Interior Supports

Cases of fracture of girders, originating at large bearing stiffeners at interior supports, have been reported (Hodgson et al., 2018, Fisher and Kaufmann 2010, among other anecdotal accounts). It appears that this behavior may be explained by the nature of the details used, which involved bearing stiffeners that were also functioning as transverse connection plates, and that were not attached to the tension flange (see Section 5.6.4 for further discussion), but the behavior of this type of detail is quite complicated and would benefit from further study, particularly with regard to considerations associated with potential retrofit actions to reduce susceptibility to CIF.

Such investigations could focus on investigating the behavior of this type of detail, with a focus on the potential for CIF. The influence of residual stresses, load-induced stresses, stress concentrations resulting from crack-like geometric conditions (whether due to weld imperfections or discontinuities, or due to connection detail geometry), and geometric constraint (imposed by the bearing stiffeners themselves, as well as by the attached cross-frames/end diaphragms) could be investigated. The investigations could also include evaluation of suggested retrofit measures to mitigate an elevated susceptibility to CIF. Such investigations could include analytical studies and, if appropriate, experimental study as well.

8.2.3 Sealed-Welded Transverse Stiffeners, Transverse Connection Plates, and Bearing Stiffeners Continuously Welded to Girder Flanges and Webs

As discussed in Sections 7.2.1 and 7.2.2, providing complete seal-welding of faying surfaces of stiffeners could potentially improve corrosion protection for steel bridges. However, there are some open questions associated with this type of detailing. One question is whether this type of detailing, particularly when used for transverse connection plates, might provide excessive constraint and lead to some elevation of susceptibility to CIF. This concern could potentially be mitigated by the lack of a planar discontinuity approximately perpendicular to the primary flow of tensile stress, but the concern remains since this type of detailing has not been previously tested for susceptibility to CIF. The other question is related to the acceptable size of the gap between the cope in the corner of the stiffener and the flange to web weld. Specifying too small of a gap might result in difficulty achieving proper fit of the stiffeners if there is variability in the size of the flange-to-web welds. Conversely, specifying too large of a gap might result in a temptation to fill the gap with weld metal, potentially increasing the chances of introducing weld imperfections that may represent a plane of discontinuity approximately perpendicular to the primary flow of tensile stress in the girder, leading to an elevated susceptibility to CIF when combined with the high level of constraint associated with this detail.

An investigation could focus on the performance of transverse stiffeners, transverse connection plates, and bearing stiffeners with no open cope provided in the corners of the stiffeners to clear the flange-to-web weld. The welds attaching the stiffener to the girder web and flanges would be made continuous around the entire perimeter of the connection to seal the faying surface. This would involve welding continuously into the corner of the intersection of the stiffener with the girder web and flanges. Only a nominal cope would be provided on the inside of the stiffener, just large enough to physically clear the flange-to-web weld by a nominal, controlled amount. The stiffener-to-flange welds and stiffener-to-web welds would run into the corner such that they intersect with the flange-to-web welds.

Such investigation could include a practical evaluation of the appropriate geometry of the gap between the stiffener cope and the flange-to-web welds. Once those parameters are established, a limited analytical study of the potential for triaxial constraint, particularly in the case of cross-frame connection plates, could be conducted. Analytical study of the fatigue performance could also be included. If appropriate, experimental study may also be considered. In particular, it may be valuable to conduct welding mock-up trials using a variety of welding processes and weld inspection methods, with subsequent dissection and microetching of the completed joints to further evaluate the joints for discontinuities.

8.3 CONCLUSIONS

This report documents the discussions and study of existing research and practices associated with the design, fabrication, and inspection of steel bridges featuring details with intersecting welds and details that may exhibit elevated susceptibility to CIF.

Lack of proper understanding of CIF could lead to unnecessary or ineffective repairs or retrofits, or the use of potentially poorer performing details.

This report presents the fundamentals of stress triaxiality, constraint, and the factors that contribute to an elevated susceptibility to CIF. The report includes a procedure that allows designers to evaluate any steel detail for the presence of the three conditions associated with elevated susceptibility to CIF under normal circumstances.

The report demonstrates the application of this procedure through the example evaluations of a number of commonly used steel bridge details. Through these illustrations, the report also shows that certain details that feature intersecting welds are not problematic in terms of susceptibility to CIF.

The report identified that guidance on minimum constraint-relief gap dimensions presented in some design specifications and industry documents is inconsistent and is based on limited prior research.

Topics of further investigation are also suggested; the Federal Highway Administration makes no commitment to completing such investigations.

The main technical finding is that steel bridge details featuring intersecting welds are not necessarily subject to elevated susceptibility to CIF. Details can be evaluated with regard to the presence of a high degree of triaxial constraint, tensile stress including the effects of residual stresses, and crack-like or notch-like planes of discontinuity approximately perpendicular to the primary flow of tensile stress, to evaluate their level of susceptibility to CIF.

APPENDIX A – LITERATURE REVIEW

The literature review encompasses 48 documents. For each, a short summary of the pertinent contents is provided. For a complete list of references cited in this report, please see the References. Like the other contents of this report, the contents of the documents included in this literature review do not have the force and effect of law and are not meant to bind the public in any way unless otherwise noted.

1. **AASHTO (1978). *Guide Specifications for Fracture Critical Non-redundant Steel Bridge Members*. American Association of State Highway Transportation Officials, Washington, DC.**

   This document represents the original American Association of State Highway and Transportation Officials' (AASHTO) fracture control plan (FCP). The document, which is not binding under FHWA regulations, describes a fracture critical member and outlines provisions to address historic fatigue and fracture problems with steel bridges, including "… unqualified personnel in inspection and nondestructive testing; design details resulting notches and difficult joints to weld and inspect; hydrogen induced cracks; improper fabrication, welding and weld repair; lack of base metal and weld metal toughness." The document includes material properties, inspection and testing criteria, welding criteria, and certification criteria. The commentary to the document discusses constraint as a parameter that influences fracture toughness, noting that thicker members exhibit greater constraint than thinner members.

2. **AASHTO. 2017a. *LRFD Bridge Design Specifications*, Eighth Edition, LRFD-8. American Association of State Highway Transportation Officials, Washington, DC (23 CFR 625.4(d)(1)(v)).**

   Article 6.6.2.1 discusses Charpy V-Notch (CVN) testing parameters and member or component designations (as either "Primary" or "Secondary"). These provisions address the toughness of the steel materials used in the structure.

   Article 6.6.2.2, *Fracture-Critical Members*, discusses the identification and designation of fracture-critical members (FCMs). A FCM is defined in Article 6.2 as: "A steel primary member of portion thereof subject to tension whose failure would probably cause a portion of or the entire bridge to collapse." This definition, and the associated provisions of this article, relate to consideration of redundancy in the design and performance of the steel superstructure of a bridge.

   Article 6.6.1.2.4 is titled *Detailing to Reduce Constraint*. This article is situated under the umbrella of Article 6.6, *Fatigue and Fracture*, and is specifically located under Article 6.6.1, *Fatigue*. Here the document states: "Welded structures shall be detailed to avoid conditions that create highly constrained joints and crack-like geometric discontinuities that are susceptible to constraint-induced fracture… If a gap is specified between the weld toes at the joint under consideration, the gap shall not be less than 0.5 in." The associated commentary discusses the concept of constraint-induced fracture (CIF) and includes reference to the Hoan Bridge failure. The commentary goes on to provide general discussion of detailing in situations where attached elements parallel to the direction of primary tensile stress (e.g., a longitudinal stiffener attached to a girder web near the tension flange) might intersect a full-depth transverse member (e.g., a vertical web stiffener). The general theme of the provisions, figures, and commentary is to maintain the continuity of the attached element parallel to the primary stress and interrupt the orthogonal element.

The commentary also states:

> If a gap is specified between the weld toes at the joint under consideration, the gap must not be less than the specified 0.5-in minimum; larger gaps are acceptable. If a gap is not specified, since the continuous longitudinal stiffener or lateral connection plate is typically welded to the web before the discontinuous vertical stiffener, the cope in the vertical stiffener should be reduced so that it just clears the longitudinal weld. The welds may either be stopped short of free edges as shown in Tables 6.6.1.2.4-1 and 6.6.1.2.4-2 or wrapped for sealing as specified in Article 6.13.3.7. A longitudinal stiffener or lateral connection plate may be discontinuous at the intersection, but only if the intersection is subjected to a net compressive stress under Strength Load Combination I and the longitudinal stiffener or lateral connection plate is attached to the continuous vertical web stiffeners as shown in Tables 6.6.1.2.4-1 and 6.6.1.2.4-2; such a detail is used at intersections with bearing stiffeners.

Table 6.6.1.2.4-1 provides sketches of "Details to Avoid Conditions Susceptible to Constraint-Induced Fracture at the Intersection of Longitudinal Stiffeners and Vertical Stiffeners Welded to the Web." Table 6.6.1.2.4-2 provides helpful sketches of "Details to Avoid Conditions Susceptible to Constraint-Induced Fracture at the Intersection of Lateral Connection Plates and Vertical Stiffeners Welded to the Web."

It can be mentioned that for both tables, a note states: "If a gap is specified between the weld toes, the recommended minimum distance between the weld toes is 0.75 in., but shall not be less than 0.5 in. Larger gaps are also acceptable."

Later, the commentary to Article 6.10.11.3.1, *General* (under Article 6.10.11.3, *Longitudinal Stiffeners*) provides similar discussion, and refers the reader to Article 6.6.1.2.4 and the associated tables. The commentary here suggests that longitudinal stiffeners be continuous in regions subjected to net tension to avoid "conditions susceptible to constraint-induced fracture due to the build-up of forces that would occur in the gap should the longitudinal stiffener be interrupted."

Throughout Section 6, "Steel Structures," there is no mention of the term "intersecting welds."

Article 6.13.3.7, "Fillet Welds for Sealing," does not list criteria related to the use of sealing welds, but instead discusses how to account for sealing welds in the determination of the strength of the connection. The associated commentary discusses undercutting that can occur when a sealing weld wraps around the end of a stiffener, mentioning that this undercutting does not reduce the fatigue performance of the weld.

Section 7, "Aluminum Structures," provides similar discussion (see Article 7.6.2.4), but presents a minimum gap of 1.0 inch between weld toes.

3. **AASHTO. 2017b. *LRFD Bridge Construction Specifications*, Fourth Edition, LRFDCONS-4. American Association of State Highway Transportation Officials, Washington, DC (23 CFR 625.4(d)(1)(iv)).**

   Article 11.1.1 discusses treatment of fracture-critical components, pointing the reader to the provisions of the AASHTO/American Welding Society (AWS) *Bridge Welding Code*, Section 12, *Fracture Control Plan (FCP) for Non-redundant Members*.

   In other locations, the code discusses other provisions related to bending and connections for FCMs.

   There is no discussion of intersecting welds, constraint, or sealing welds in Section 11 of this document.

4. **AASHTO/AWS. 2015. *Bridge Welding Code*, BWC-7 (AASHTO/AWS D1.5M/D1.5:2015). American Association of State Highway Transportation Officials and American Welding Society, Washington, DC (23 CFR 625.4(d)(1)(vii)).**

   In general, there are no defined restrictions regarding the use of intersecting welds. In Section 12, *Fracture Control Plan (FCP) for Non-redundant Members*, there is no mention of intersecting welds. Section 12 presents the parameters of the FCP for fracture critical non-redundant members.

   Section 2.14 addresses "Prohibited Types of Joints and Welds;" however, there is no mention of intersecting welds, or CIF. The section identifies as disallowed joints and welds:

   > (1) All partial joint penetration (PJP) groove welds in butt joints except those conforming to 2.17.3
   >
   > (2) Complete joint penetration (CJP) groove welds, in all members carrying calculated stress or in secondary members subjected to tension or the reversal of stress, made from one side only without any backing, or with backing other than steel, that has not been qualified in conformance with 5.7.5 and 5.12.4
   >
   > (3) Intermittent groove welds
   >
   > (4) Intermittent fillet welds, except as approved by the engineer
   >
   > (5) Flat position bevel-groove and J-groove welds in butt joints where V-groove and U-groove welds are practicable

(6) Plug and slot welds in members subjected to tension and reversal of stress

Section 6.26.2 discusses radiographic testing (RT) and magnetic particle inspection, also known as magnetic test (MT), weld inspection methods. In sub Article 6.26.2.1, intersecting flange-web-welds are noted as related to discontinuities in the weld:

> For welds subject to tensile stress under any condition of loading, the greatest dimension of any porosity or fusion-type discontinuity that is 2 mm [1/16 in] or larger in greatest dimension shall not exceed the size, B, indicated in Figure 6.8 for the effective throat or weld size involved. The distance from any porosity or fusion type discontinuity described above to another such discontinuity, to an edge, or to the toe or root of any intersecting flange-to-web weld shall be not less than the minimum clearance allowed, C, indicated in Figure 6.8 for the size of discontinuity under examination.

In Figure 6.8, C is minimum clearance measured along the longitudinal axis of the welded as is related the dimension of the discontinuity. For example, if there is a discontinuity ¼" in length, the minimum clearance to the next discontinuity, edge, or toe or root of intersecting flange-to-web weld shall be ~2 ¼".

Commentary Section C-6.26.2.1 discusses intersecting welds with respect to web-to-flange welds, stating:

1) The radiographic weld quality standards of this code are identical to the provisions of AWS D 1.1. These standards had their beginning in the ASME Boiler and Pressure Code, Paragraph UW-51, and were modified and adopted for bridges in the late 1950s. In the Boiler Code, there was no concern for ends of welds, or intersections of web-to-flange welds in bridge members, where there is a concentration of stress.

2) Restrictions on discontinuity size in areas adjacent to flange edges, or subject to high stress near intersecting welds, is considered in terms of fracture resistance since they are more susceptible to brittle fracture than areas that are completely surrounded by base metal.

5. **American Petroleum Institute (API). 2014. *API Recommended Practice 2A-WSD, Planning, Designing, and Constructing Fixed Offshore Platforms-Working Stress Design.* American Petroleum Institute. Twenty-Second Edition. November.**

This design practice document, which is not binding under FHWA regulations, addresses the design and construction of offshore petroleum drilling platforms. Deck structures of offshore platforms are framed in a similar manner as steel bridges and buildings using rolled and built up shapes. It is the norm for the weld connecting these members be sealed to prevent corrosion. Section 10.3.4 of this document discusses "Seal Welds," stating:

> Unless specified otherwise, all faying surfaces should be sealed against corrosion by continuous fillet welds. Seal welds should not be less than 1/8 inch but not exceed 3/16 inch regardless of base metal thickness. Minimum preheat temperatures of AWS Table 3.2 or Annex XI should be applied.

6. **Azizinamini, A., E. H. Power, G. F. Myers, H. C. Ozyildirin, E. Kline, D. W. Whitmore, D. R. Mertz. 2014. *Design Guide for Bridges for Service Life*. S2-R19A-RW-2, Transportation Research Board, National Academy of Sciences, Washington, DC.**

   Section 2.3.3.1.1, "Load-induced Deficiency: Fatigue and Fracture," provides some general discussion of fatigue and fracture in steel bridges, including some history of the progressive improvement in material properties with regard to resistance to the effects of fatigue and fracture.

   Section 3.2.3.2 provides some basic descriptions of ductility and toughness in steel.

   Chapter 7 of this design practice document, which is not binding under FHWA regulations, addresses fatigue and fracture of steel structures. The content of this chapter is largely adapted from Dexter and Ocel (2013). Section 7.2.3 discusses the fracture mechanics approach for investigating fatigue performance. On the whole, this chapter focuses primarily on fatigue, not on fracture. Subsection 7.7.5, Web Gusset Plates, does mention "intersecting welds" and implies they are a source of fatigue damage that should be "retrofitted by coring holes at the intersections [of the welds]; this procedure will not only remove the intersecting welds, but also reduce the web constraint." However, the discussion stops at that point without providing further explanation of the true sources of "web constraint" or how to discern good from bad intersecting weld situations.

7. **Bowman, M. D. 2002. *Fatigue Behavior of Beam Diaphragm Connections with Intermittent Fillet Welds, Part II: Brittle Fracture Examination of the I-64 Blue River Bridge*, Report No. FHWA/IN/JTRP-2001/10-II. Joint Transportation Research Program. Purdue University, West Lafayette, IN.**

   This report documents the investigation of a case of brittle fracture, stating that it addresses:

   > …the brittle fracture that occurred on the I-64 Blue River Bridge in Harrison County of southern Indiana. The fracture occurred in the middle span of a three span structure at a location where both a lateral diaphragm and a horizontal bracing members framed into a vertical and horizontal plate, respectively. The study involved experimental studies to evaluate the material performance and behavior of the bridge steel and analytical studies to assess the fracture resistance and susceptibility to brittle fracture. Recommendations for retrofit and repair of similar bridge details were formulated to decrease the fracture susceptibility from distortion related fatigue cracking.

   The report stated:

> …the brittle fracture initiated in the girder web near the intersection of a vertical connection plate and a horizontal gusset plate. Moreover, it is believed that the crack initiated as a fatigue crack in the web gap region immediately adjacent to the weld toe of the web-to-vertical stiffener weld.

The report also stated:

> … four factors are believed to have elevated the stresses in the gusset-to-stiffener connection welds: lack of positive attachment between the horizontal gusset plate and the vertical diaphragm, a small lateral gap distance between the toes of the horizontal and vertical fillet welds, loose bolts in the horizontal bracing to gusset plate connection, and impact forces introduced into the web via the horizontal bracing members.

The report is instructive in illustrating that small changes to lateral gusset plate details have significant impact on behavior and performance. The Blue River Bridge is compared to the Lafayette Street Bridge (where cracks initiated in lateral gusset plates and propagated through intersecting welds into the web) and the Canoe Creek Bridge, where lack of restraint between the lateral gusset plate and the web vertical stiffeners resulted in high out-of-plane stresses in the web. The details used in the Blue River Bridge are similar to Hoan Bridge details, with small, but significant, differences.

8. **Bowman, M. D., G. Fu, Y. E. Zhou, R. J. Connor, and A. A. Godbole. 2012. *Fatigue Evaluation of Steel Bridges*, NCHRP Report 721. Transportation Research Board, National Academy of Sciences, Washington, DC.**

This research report includes little or no discussion regarding intersecting welds and CIF. Suggestions are given based on an evaluation of existing fatigue provisions, as well as the experimental studies. Also, several revisions are suggested to the provisions in Section 7 of the AASHTO Manual of Bridge Evaluation.

The report provides suggested revisions to Section 7—Fatigue Evaluation of Steel Bridges of the AASHTO Manual for Bridge Evaluation with detailed examples of the application of the suggested revisions. The suggested revisions and examples were developed based on analytical and experimental research conducted to improve existing methods to evaluate and assess the serviceability of bridge structures for the fatigue limit state.

Items specifically identified as candidates for improvement include:

1) Improved methods utilizing a reliability-based approach to assess the fatigue behavior and aid bridge owners in making appropriate operational decisions.

2) Evaluation of retrofit and repair details used to assess fatigue cracks.

3) Evaluation of distortion-induced fatigue cracks.

To address these items, analytical and experimental studies were performed. The analytical studies were used to examine various aspects that influence the fatigue behavior. These topics ranged from truck loading effects on bridge structures to fatigue resistance related factors that affect the predicted fatigue life. Both analytical and experimental studies were used to further develop an understanding of distortion-induced deformations and the structural behavior of various retrofit details used to repair a bridge structure with distortion-induced fatigue cracking. Moreover, early in the study it was decided that it would be beneficial to perform a series of experimental tests to study the influence of tack welds on riveted joints.

9. **Bucknall, J. 2000. Review of Current Inspection Practices for Topsides Structural Components. Offshore Technology Report 2000/027. MSL Engineering for the Health and Safety Executive.**

   This design practice document, which is not binding under FHWA regulations, addresses the design and construction of offshore petroleum drilling platforms. Deck structures of offshore platforms are framed in a similar manner as steel bridges and buildings using rolled and built up shapes. It is the norm for the weld connecting these members be sealed to prevent corrosion. Section 3.3 of this document discusses welding and addresses seal welds:

   > As mentioned above, topsides tend to be of all welded construction, exceptions being secondary steelwork and lattice towers. This facilitates sealing all internal voids and spaces against possible corrosion by running fillet welds. However, the welds across load paths in joints and connections tend to be full penetration even where, structurally, there is no necessity for a full strength connection. In part, this is due to time constraints on the designer.

10. **Chajes, M. J., D. R. Mertz, S. Quiel, H. Roecker, and J. Milius. 2005. "Steel Girder Fracture on Delaware's I-95 Bridge over the Brandywine River,"** ***Proceedings of the ASCE Structures Congress 2005*****. American Society of Civil Engineers, New York, NY, pp. 1-10.**

    This is a case study of the fracture and repair of an exterior girder in a bridge carrying I-95 over the Brandywine River in DE; the bridge was constructed in 1963 and the fracture occurred in 2003. The fracture initiated from a full-penetration groove weld (butt weld) in a longitudinal stiffener in the positive moment region of the exterior girder; the weld showed evidence of lack of fusion. At the time of fracture, the girder was subjected to greater than normal live load stresses and relatively cool temperatures. The fracture was similar to the fracture of the Quinnipiac Bridge on I-95 in CT in 1973.

11. **Connor, R. J., E. J. Kaufmann, J. W. Fisher, and W. J. Wright. 2007. "Prevention and Mitigation Strategies to Address Recent Brittle Fractures in Steel Bridges,"** ***Journal of Bridge Engineering*****. American Society of Civil Engineers, Vol. 12, No. 2, March, New York, NY, pp. 164-173.**

    The abstract of this paper states:

> Brittle fracture results in unplanned loss of service, very costly repairs, concern regarding the future safety of the structure, and potential loss of life. These types of failures are most critical when there is no evidence of fatigue cracking leading up to the fracture and the fracture origin is concealed from view. Hence, the failure occurs without warning and the details are, essentially, noninspectable. In these cases, it appears desirable to take a proactive approach and introduce preventative retrofits to reduce the potential for future crack development. These efforts will help ensure that the likelihood of unexpected fractures is minimized. This paper examines the behavior of two bridge structures in which brittle fractures have developed, discusses the causes of the failures, and offers suggested design strategies for prevention and retrofit mitigation techniques. In situations where considerable uncertainty exists in the prediction of accumulated damage or in the ability to reliably inspect critical details, preemptive retrofit strategies appear to be highly desirable.

This paper discussed the behavior of two bridges (US 422 Bridge over the Schuykill River in Pottstown, PA and the Hoan Bridge in Milwaukee, WI), which experienced brittle fractures. The paper discusses the causes of the failures and offers suggested design strategies to minimize the likelihood of such failures as well as possible retrofit mitigation techniques. In both cases, the fractures were not caused by fatigue cracks that subsequently became unstable, but were attributed to CIF.

The paper notes that most highway bridges are inherently less susceptible to fracture due to the use of thinner and inherently more flexible girder web plates, lower restraining forces, good detailing, and generally higher toughness materials. However, webs can be subject to constraint provided by other plate elements such as longitudinal and vertical stiffeners and gusset plates. When such elements are located near each other such that the constraint-relief gap is very small, the web is restrained in the through-thickness direction, resulting in a triaxial state of stress. If "…the web material cannot contract freely, the stress can increase to well beyond the yield point of the material, resulting in stress intensities exceeding the fracture toughness of the web material." Meanwhile, the small gap represents a crack-like condition that produces a severe stress concentration.

The paper discusses what are called the "triaxiality factors" (descibed by Schafer, 2000), which can be used to compare the state of stress in a detail to the Von Mises effective stress, giving some measure of the degree of constraint in a given detail.

The paper discusses the fracture that occurred in the US 422 Bridge. This fracture occurred at a location where the web was restrained by a vertical stiffener and a lateral gusset plate, and was thus subject to triaxial constraint in addition to poor weld quality. Fracture occurred at two separate locations at the lateral gusset plate connection nearly simultaneously. Fracture occurred even though the material satisfied AASHTO Zone 2 fracture critical toughness criteria for Grade 36 steel. There was no evidence of an initiating welding discontinuity or fatigue crack extension prior to the fracture.

The paper also discusses the fracture that occurred in the Hoan Bridge. Fractures occurred in all three girders in the cross section, at locations where webs were restrained by transverse connection plates and lateral gusset plates, and was thus subject to triaxial constraint. Fracture occurred even though the material satisfied AASHTO Zone 2 fracture critical toughness criteria for Grade 36 steel. There was no evidence of an initiating welding discontinuity or fatigue crack extension prior to the fracture.

The paper discusses identification of details susceptible to CIF. The paper noted three conditions associated with a detail being vulnerable to CIF:

- The detail is subject to a high, localized stress concentration
- The local stress concentration occurs in a location subject to high constraint, preventing local yielding
- The detail is subjected to a sufficiently high net tensile stress

Finite element method (FEM) analytical studies conducted at Lehigh University (Mahmoud et al., 2005) suggested that "gaps greater than 6.4 mm [approximately ¼"] generally reduce the triaxial stress condition within the web gap to acceptable levels…" such that "bridge fracture should not occur." A table was provided comparing the principal stresses and triaxiality factor for a 0-inch "web gap" (constraint-relief gap) versus a 6.4 millimeter (mm) "web gap," showing 24 to 100 percent reductions in these parameters when the 6.4 mm "web gap" is provided.

The paper discusses prevention (design) and retrofit (mitigation) strategies, including avoiding any one of the three conditions associated with elevated susceptibility to CIF or increasing the constraint-relief gap dimension. The suggested details for lateral gusset plates appear to be in contrast to those suggested by Fisher et al (1998) and Mertz (2015).

12. **Connor, R. J. and J. B. Lloyd. 2017. *Maintenance Actions to Address Fatigue Cracking in Steel Bridge Structures, Proposed Guidelines and Commentary*. Purdue University, March, West Lafayette, IN.**

    The report of proposed guidelines, which is not binding under FHWA regulations, summarizes practices the authors identified from published literature, on-going research activities, and input from industry professionals, to provide suggested practices for maintenance actions to address fatigue cracking in steel bridges. In addition to fatigue, preemptive maintenance actions related to CIF are also presented. This practice document covers repair procedures, detailing techniques, maintenance suggestions, inspection suggestions and preservation actions to repair and retrofit steel bridges. The intent of the report is help mitigate initiation of fatigue cracks on details known to have low fatigue resistance, control further growth of existing fatigue cracks, and reduce or eliminate susceptibility to CIF in steel bridges.

    The report discusses in-service bridges and maintenance actions, and not necessarily new design. But, many of the noted repairs are due to poor details that could be avoided in new designs, or in fact are no longer used.

A review of current practice was conducted by the research team, with a web-based survey that was widely distributed to Class 1 North American railroads, the U.S. Army Corps of Engineers (USACE), and all State departments of transportation (DOTs). Twenty-seven surveys were completed, with 25 coming from 23 State DOTs. One item was noted with regard to CIF:

> Thirty-five percent reported having a constraint-induced fracture (CIF) occur in their inventory, while only 13 percent said that their agency currently has a policy for preemptive retrofitting of CIF-prone details. This suggests that a vast majority of owners either do not have CIF-prone details in their inventory, or are comfortable with, or unaware of, the assumed risk of not retrofitting them. Only one agency reported having a guideline for retrofitting CIF details.

Chapter 7 discusses intersecting welds at gusset plates (Hoan-like details) and intersecting welds at longitudinal stiffeners. For both, the problem is described and then retrofit practices are presented. Also in Appendix A.3, a quick reference table for repower strategies for these type of details are provided.

The authors note that there are three contributing elements to CIF, characteristic of all CIF-prone details. If one is missing, then the susceptibility to CIF reduces significantly. The following three elements are discussed:

1) There needs to be a localized area of stress concentration that intensifies the dead and live load stress level. The presence of defects within the weld, as well as certain geometry of the connection can both act as discontinuities that interrupt stress flow and cause concentrations.

2) The joint must be highly constrained, resulting in a three dimensional state of stress that prevents plastic flow, as would occur in a simple uniaxial stress state.

3) There must be an elevated level of tensile residual stresses locked into the local area. While the dominating contributor are residual stresses from welding, other factors contribute to a lesser degree, such as dead load and erection stress. As is well documented, residual stresses due to welding can easily reach the yield strength of the base metal.

A comparison of stress flow is made between two details where a longitudinal attachment to the web plate intersects with the transverse plate: one with no gap between the weld intersections, and one with a “web gap” (constraint-relief gap) between the transverse and longitudinal weld toes. It is noted that as the gap becomes larger, the effect of the residual stresses from the welds and the local stress concentrations at the termination of the longitudinal plate are reduced. The “web gap” significantly reduces the constraint, and stresses are shown to reduce. The third axis of stress, $\sigma 3$, is diffused through necking of the material while stresses along the $\sigma 1$ and $\sigma 2$ axes have been shown to reduce by 26 and 36 percent, respectively, with a "web gap" size of ¼-inch. (Mahmoud et al., 2005b; Connor et al., 2007).

The report describes two types of intersecting weld details that are typically of concern:

Intersection of Webs, Web Transverse Stiffeners, and Lateral Gusset Plates (Hoan-like details)

The report states:

> CIF details located at gusset (or shelf) plate details are often also referred to as "Hoan Details" in practice. This stems from a fracture of all three girders on one of the south approach spans of the Hoan Bridge in Milwaukee, Wisconsin, on December 13, 2000... Detailed analysis indicated that brittle fractures developed at the intersection of the shelf plate and transverse connection plate without any detectable fatigue crack extension or ductile tearing at the crack origin.

Three repair/retrofit strategies are introduced:

- Gusset Plate Cope Retrofit - The gusset plate cope retrofit removes the intersecting transverse and longitudinal welds, creating a sufficiently sized "web gap" (a constraint-relief gap, with a minimum of 1/4 inch of web exposed) to eliminate the localized constraint of the web plate.

- Web Plate Isolation Holes Retrofit - The web plate isolation hole retrofit simply installs a mechanism to arrest a fracture immediately after it initiates, isolating the web plate and flanges from further fracture propagation.

- Ball End Mill Retrofit - The ball end mill retrofit mitigates fracture at CIF details by removing the constraint and reducing the stress concentrations at the intersection of the vertical and horizontal welds. This is done by machining the intersection from the back side of the web plate using a center-cutting ball end mill bit and magnetic-based drill.

Intersection of Webs, Web Transverse Stiffeners, and Longitudinal Stiffeners

The report states:

> Longitudinal stiffener plates with welds that intersect with transverse connection and stiffener plate welds are also prone to constraint-induced fracture when located in a tensile or stress reversal zone. A few documented cases exist, such as a bridge along I-90 near Bozeman, MT, that suffered a near full-depth fracture that initiated at the intersection of the longitudinal and transverse stiffening elements… the same driving mechanisms and similar detailing as Hoan-like gusset plates are behind the cause of the fracture risk.

Retrofit strategies are similar to the Hoan-like gusset plates, including isolation holes and ball end mill retrofit methods. Also discussed is a stiffener coping retrofit that cuts back the longitudinal stiffener from the transverse stiffener/connection plate.

13. Connor, R. J., R. Dexter, and H. Mahmoud. 2005. *Inspection and Management of Bridges with Fracture-Critical Details.* NCHRP Synthesis 354. Transportation Research Board, National Academy of Sciences, Washington, DC.

This synthesis focuses on the inspection and maintenance of bridges with FCMs, as defined in the AASHTO BDS (23 CFR 625.4(d)(1)(v)). The objectives were to survey and identify gaps in the literature; determine practices and problems with how bridge owners define, identify, document, inspect, and manage bridges with fracture-critical details; and identify specific research opportunities. Among the areas examined were inspection frequencies and procedures; methods for calculating remaining fatigue life; qualification, availability, and training of inspectors; cost of inspection programs; instances where inspection programs prevented failures; retrofit techniques; fabrication methods and inspections; and experience with FCM fractures and problems details.

Intersecting welds and CIF are only mentioned a few times. As part of the author's overview of fracture, they note that: "…intersecting welds should always be avoided owing to the probability of defects and excessive constraint. Intersecting welds, or even welds of too close proximity, have caused brittle fractures [e.g., the Hoan Bridge in Wisconsin and the SR-422 Bridge in Pennsylvania.]"

Owners identified the following as the most important areas for future research as related to fracture-controlled bridges (FCBs):

- Development of load models, criteria for the extent of damage, and practice suggestions related to advanced structural analysis procedures to better predict service load behavior in FCM bridges and the behavior after fracture of an FCM, including dynamic effects from the shock of the fracture and, if warranted, large deformations.

- Development of advanced fatigue-life calculation procedures, taking into account a lack of visible cracks for fracture critical bridges.

- Investigation of field monitoring for fracture-critical bridges.

- Development of rational risk-based criteria for inspection frequency criteria and level of detail based on average daily truck traffic, date of design, and fatigue detail categories present.

- Evaluation of fracture-critical issues related to sign, signal, and light supports.

14. **Cooper, J. D. 2001. "ACTION: Hoan Bridge Failure Investigation," Memorandum. U.S. Department of Transportation, Federal Highway Administration, July 10, Washington, D.C.**

This memorandum summarizes some of the initial findings from the Hoan Bridge Failure Investigation.

Attachment A to the memorandum states that the cause of the fractures was excessive triaxial constraint with very small (1/8 inch) "web gaps" (constraint-relief gaps). Attachment A also noted that low temperatures at the time did not cause the initiation of fracture, but reduced the ability of the structure to arrest dynamic crack growth. One of the "Significant Findings" highlighted in Attachment A is of particular interest; the memorandum states:

> A narrow gap between the gusset plate and the transverse connection/stiffener plate created a local triaxial constraint condition and increased the stiffness in the "web gap" region at the fracture imitation site. This constraint prevented yielding and redistribution of the local stress concentrations occurring in this region. As a result, the local stress state in the "web gap" was forced well beyond the yield strength of the material. Under triaxial constraint, the apparent fracture toughness of the material is reduced and brittle fracture can occur under service conditions where ductile behavior is normally expected.

Attachment B illustrates details that are susceptible to CIF, using the Hoan Bridge details as examples. Attachment B discusses that CIF is a concern in elements subjected to net tension. Attachment B also suggested using the rate of crack growth as an indicator of the nature of the underlying cause of the cracking: fast crack growth suggests fracture, where slow crack growth suggests fatigue.

15. **DeLong, D. T. and M. D. Bowman. 2010. *Fatigue Strength of Steel Bridge Members with Intersecting Welds*, Report No. FHWA/IN/JTRP-2009/19. Joint Transportation Research Program. Purdue University, July, Indianapolis, IN.**

This report documents an experimental study and evaluation of steel bridge details with intersecting welds with regard to fatigue performance. The study focused on details used in bridges in Indiana. Retrofit details used to extend fatigue life were also investigated. The report includes the following summary statements:

> The purpose of this study was to examine the effect of intersecting weld details commonly used in Indiana on the fatigue strength and resistance. While the concern was initially centered on the fracture susceptibility of intersecting weld details, any detail that has welds that intersect or nearly intersect will restrain the steel in the vicinity of the weld intersection and will introduce additional welding residual stresses.
>
> It is possible that an intersecting weld detail with less constraint than the Hoan bridge detail may lead to fatigue crack initiation earlier than otherwise

anticipated, but not trigger a brittle fracture. To assess this condition a series of experimental test were conducted to examine the fatigue susceptibility of intersecting weld details commonly used in Indiana. Furthermore, the performance of drilled hole retrofits used to extend the fatigue life of these details was studied.

Nine steel beams containing details with varying degrees of weld intersection were tested under constant amplitude cyclic loading. Three different basic detail types were tested: a vertical stiffener welded to the web and flange, a welded horizontal stiffener terminating near a welded vertical stiffener, and a welded horizontal gusset plate coped to fit around a welded vertical connection plate. The fatigue strength of the details with intersecting welds was compared to that of details without intersecting welds. For each detail type, there were several conditions tested with varying gaps between perpendicular welds. The results of the tests were examined to determine if the web gap [constraint-relief gap] size had an effect on fatigue behavior, and more importantly, to determine if this effect could result in a fatigue strength below the appropriate design fatigue strength.

The following observations and conclusions were made based on the experimental results and evaluation:

The presence of intersecting welds connecting a vertical stiffener to the flange and web of a rolled shape was shown to have no detrimental effect on the fatigue life of the detail. All of the details tested with intersecting welds lasted well beyond the design life for a Category C' detail under the calculated nominal stress range, and all of the critical cracking in these details occurred along the stiffener-to-flange weld toe away from the weld intersection.

The size of gap between the perpendicular welds where a horizontal stiffener terminates near a vertical stiffener may have an effect on the fatigue strength of the detail. Fatigue cracking at the toe of the horizontal stiffener-to-web weld termination occurred earlier in details with a small web gap [constraint-relief gap] (0.125 in.) than it did in details with a large gap (1.0 in.). If this gap is small enough, the stress concentration factors at the longitudinal and transverse weld toes that are directly opposite each other may be additive to a certain extent. This could result in a fatigue strength that is lower than the design strength for the horizontal stiffener alone.

Within the scope of the study, the size of the web gap [constraint-relief gap] did not have an effect on the fatigue behavior of the detail with a horizontal gusset plate coped to fit around a vertical connection plate. The details in this test behaved in the same manner as many of the previous experimental tests, with the most critical location for fatigue cracking at the outside edges of the gusset plate, not in the web gap.

Small web gaps [constraint-relief gaps] may increase the risk of constraint-induced brittle fracture. Other risk factors include steels with low fracture toughness and the presence of very low temperatures. Although limited research is available, it is suggested that web gaps of less than ¼-in between adjacent weld be avoided whenever possible.

The authors made the following suggestions for action and implementation:

> No retrofit action is needed for welded stiffener or connection plate details where the vertical web welds intersect, or nearly intersect, with horizontal welds on the flange.
>
> Horizontal web attachment details with welds that are situated near a vertical connection plate or stiffener should be regularly inspected to determine if fatigue cracking has occurred at the end of the attachment plate weld toe that is situated next to the vertical plate.
>
> Horizontal gusset plate details that are coped to fit around a vertical connection plate should be inspected to determine if fatigue cracking has occurred at the weld toe located at the outside ends of the gusset plate. The weld toe in the web gap region where the plate is coped should also be inspected to determine if fatigue cracking has occurred.
>
> Fatigue cracks that are detected can typically be repaired using drilled retrofit holes. The retrofit hole diameter should be sized large enough to minimize re-initiation of the fatigue cracking. If the retrofit hole diameter is too large, then a bolted splice repair should be considered as an effective alternative.
>
> Web gaps in regions where multiple welds intersect, or nearly intersect, should be modified if necessary to increase the web gap distance to ¼-in or larger between adjacent weld toes. This distance should be sufficient to minimize the likelihood of constraint-induced brittle fracture.

16. Dexter, R. J. and J. M. Ocel. 2013. *Manual for Repair and Retrofit of Fatigue Cracks in Steel Bridges*, Report No. FHWA-IF-13-020. University of Minnesota, March, Minneapolis, MN.

This manual, which is not binding under FHWA regulations, is a comprehensive practice document, developed from a consensus workshop with experts in fatigue and fracture held in August 2002.

Early in the document there is some discussion of fracture, and of the implementation of CVN testing standards in the ASTM International (ASTM) A709 bridge steel standard in 1974. The possibility of fracture without previous fatigue cracking is mentioned, with reference to the Hoan Bridge fracture and details with high constraint. The development of the AASHTO/AWS FCP in 1978 is also briefly discussed.

The text makes the following statement: "Intersecting welds create a triaxial stress state where the material is unable to yield through its thickness resulting in localized 3-dimensionsal stresses being generated that are much higher than the material's yield."

There is also a discussion of intersecting welds in the context of web gusset plates, beginning on pg. 87. The manual states that at locations where gusset plates intersect transverse web stiffeners, the details may create “and intersecting weld at the point where the girder web, gusset plate, and stiffener all come together. There is always lack of fusion defects at the root of intersecting welds that serve as crack initiation sites.” The manual also states:

> Intersecting welds also create a triaxiality condition where the material is not allowed to yield, thus the hydrostatic state of stress increases susceptibility to fracture. In some cases, no welding defects were found at the cracked detail as in the case of the Hoan Bridge. The cracked details were brittle in nature and there were no sign of fatigue crack initiation or growth.” The manual suggests complete removal of the intersecting welds via coring as a retrofit, stating: “A clear distance between welds of 0.25 inch (6.4 mm) has been shown sufficient in reducing the triaxiality condition in these details, and in the 2010 AASHTO *LRFD BDS* it was suggested for new designs to make the minimum distance 1.0 inch (25.4 mm).

The remainder of the document focuses primarily on fatigue, including fatigue assessment and repair and retrofit practices and examples.

17. Dexter, R. J., and J. W. Fisher. 1999. "Fatigue and Fracture," Chapter 24 of *Structural Engineering Handbook*. Ed. W. Chen, CRC Press.

This chapter of this textbook focuses on both fatigue and fracture of steel girders. The discussion of fatigue is not directly relevant to the topic of CIF.

The discussion of fracture resistance gives a basic overview of fracture toughness, CVN testing, and minimum CVN energy criteria. The chapter’s overall summary briefly mentions that “high constraint of the connections” is one of several design variables that may increase the potential for brittle fracture.

18. Dexter, R. J., W. J. Wright, and J. W. Fisher. 2004. "Fatigue and Fracture of Steel Girders," *Journal of Bridge Engineering*, American Society of Civil Engineers, Vol. 9, No. 3, May, New York, NY, pp. 278-286.

The abstract for this paper reads as follows:

> This paper presents an overview of materials selection, design, and detailing of steel girders for fatigue and fracture limit states. The historical context of the FCP for bridges is presented. A discussion of fracture toughness of structural steel and weld metal is presented along with typical Charpy and fracture-toughness test data, including the new high-performance steel A709 HPS 485W. Fatigue of cover plate details and distortion-induced cracking are discussed. Methods of dealing with variable-amplitude loading are then compared to test data.

This paper discusses both fatigue and fracture of steel girders. The discussion of fatigue is not directly relevant to the topic of CIF.

The discussion of fracture provides a brief history of the development AASHTO FCP, and gives a basic overview of fracture toughness, CVN testing, and minimum CVN energy criteria. The paper comments that thicker plates have higher toughness criteria due to the increased constraint that exists in thicker versus thinner plates. The paper also states: "The existing *AASHTO Fracture Control Plan*… has done a reasonably good job of preventing fracture failure in bridge structures." However, the AASHTO FCP only addresses material criteria and does not specify design or detailing criteria for avoiding situations where CIF might be possible.

19. **Dexter, R. J., and J. W. Fisher. 2000. "Fatigue and Fracture," Chapter 53 of *Bridge Engineering Handbook*. Ed. W. Chen and L. Duan, CRC Press.**

    This chapter of this textbook focuses on both fatigue and fracture of steel girders. The discussion of fatigue is not directly relevant to the topic of CIF.

    The discussion of fracture resistance gives a basic overview of fracture toughness, CVN testing, and minimum CVN energy criteria. The chapter's overall summary briefly mentions that "high constraint of the connections" is one of several design variables that may increase the potential for brittle fracture.

20. **Ellis, R. M., R. J. Connor, M. Manoj, D. MacLaggan, M. Bialowas. 2013. "Investigation and Repair of the Diefenbaker Bridge Fracture," Paper Prepared for Presentation at the 2013 Conference of the Transportation Association of Canada, Winnipeg, Manitoba.**

    This paper describes a major fracture that occurred in the Diefenbaker Bridge, a 7-span, 1,000-foot-long bridge in Prince Albert, Canada. This paper describes the fracture investigation, diagnosis of the causes of fracture, and the innovative engineering involved in the repair of the bridge. The primary cause of the fracture was determined to be CIF. A method to mitigate possible future fractures was implemented and is described.

    A method to successfully repair the fracture is described that included supporting the structure on large towers founded on a river berm, jacking the structure to a predetermined elevation, cutting out a section of the girder and replacing it with a new section of girder. The effectiveness of the repair was verified through a load test and structural monitoring system. Additional repairs were accomplished to mitigate the 160 other locations of potential CIF in the southbound and northbound structures. The repairs consisted of coring (milling) a relieving hole at the location of the intersecting welds. The intent of the repair is to remove the intersecting welds, and points of high stress concentration that lead to CIF.

21. **Fish, P., C. Schroeder, R. J. Connor, P. Sauser. 2015. *Fatigue and Fracture Library for the Inspection, Evaluation, and Repair of Vehicular Steel Bridges*. S-BRITE Center, Purdue University, June, West Lafayette, IN.**

    This document provides both discussion and photos clearly illustrating a wide variety of fatigue- and fracture-related topics, details, and retrofit strategies.

Section 1.4 of the document provides a short but clear discussion of fracture in steel elements, with clear, succinct descriptions of brittle and ductile fractures. There is also a helpful discussion of constraint. Key text is excerpted below:

> Constraint can occur at intersecting welds, large weldments, or thick members. Constraint limits the ability of a member to deform and yield (due to Poisson's effect) under load. Since the constraint prevents the material from yielding in the classic sense, local stresses can therefore increase well beyond the nominal yield strength of the steel. This increases the vulnerability to brittle fracture.
>
> Brittle fracture due to restraint is now recognized as ''Constraint-induced Fracture'' (CIF).
>
> Highly constrained details should be avoided, as much as possible, to minimize CIF. This can be achieved by employing relatively simple techniques: provide copes to eliminate intersecting welds; avoid intersecting members by providing relief (e.g., copes); avoid large weldments by minimizing weld size, using bolted connections, or reconfiguring the joint. Avoid the combination of restrained thick members and heavy welds when possible by using built up members or higher strength steels.

Section 3.8 (B) 1, *Gusset plate connections*, discusses "web gap" (constraint-relief gap) dimensions that result in reduced constraint and thus allow for yielding of the steel in the web gap area. The text specifically mentions ¼ inch as a suggested minimum "web gap" dimension, based on previous research (Mahmoud et al., 2005).

22. Fisher, J. W. 1973. Evaluation of *Typical Welded Bridge Details for Fatigue Design*, Interim Report for NCHRP Project 12-115 Detection and Repair of Fatigue Cracking in Highway Bridges. Fritz Engineering Laboratory, Department of Civil Engineering, Lehigh University. September. Bethlehem, PA.

This report reviews existing and currently designed welded bridge details to identify these details in terms of design specification provisions, and classify them according to severity.

23. Fisher, J. W. 1984. *Fatigue and Fracture in Steel Bridges*. John Wiley & Sons, Inc., New York, NY.

This book provides a detailed a review and summary of 22 case studies of bridges that have experienced crack growth. The first portion deals with cracks that have formed as a result of low fatigue resistant details or large initial discontinuities. The second part has case studies about fatigue cracks that formed as a result of unanticipated secondary or displacement-induced stresses.

Chapter 4 addresses web connection plates, and cracks that have developed in the web at lateral connection plates because of intersecting welds. A case study of the Lafayette Street Bridge in St. Paul, MN, a two-girder bridge with floorbeams, is presented,. The primary problem was a large weld discontinuity (lack of fusion) in the welds attaching the lateral connection plate to the transverse stiffener. Since this weld was perpendicular to the cyclic stresses, and intersected with the vertical welds attaching the stiffener to the web and the longitudinal welds of the connection plate, a path was provided into the girder web. Eventually, the fatigue crack precipitated a brittle fracture in part if the web and fractured the tension flange.

24. **Fisher, J. W., J. Jim, D. C. Wagner, and B. T. Yen. 1990. *Distortion-Induced Fatigue Cracking in Steel Bridges*, NCHRP Report 336. Transportation Research Board, Washington, DC.**

    This report focuses primarily on distortion-induced fatigue cracking. The report provides only a few comments on details where lateral gusset plates intersect with web and transverse stiffeners, as related to "web gap" (constraint-relief gap) fatigue cracking.

25. **Fisher, J. W., E. J. Kaufmann. 2010. *Fracture at Web Gap of the Iowa I-80 Bridge over the Cedar River*. Letter report to Mr. Mike Koob, P.E., Wiss Janney Elstner Associates, Inc., March 31, Bethlehem, PA.**

    This letter report focuses on the investigation of a CIF failure that occurred at a bearing stiffener. Review of the report suggests that the CIF was due to a high degree of triaxial constraint at the top of the bearing stiffeners, in the gap between the end of the bearing stiffener welds and the toe of the flange-to-web welds. The bearing stiffener also functioned as a transverse connection plate, but was not attached to the top flange (tension flange) of the girder. As a result, the cross-frame forces applied to the bearing stiffener induced out-of-plane loading in the girder web. The detail exhibited the three conditions associated with elevated susceptibility to CIF: tension (resulting from out-of-plane bending of the web), a high degree of triaxial constraint, and planar discontinuities (associated with the end of the bearing stiffener). Following is an excerpt from the conclusion of the letter report:

    > The fracture of the girder web was found to be initiated from the severe geometric tri-axial restraint between the ends of the bearing stiffeners and the web-flange fillet welds. As a result the stress approached the web tensile strength. No evidence of fatigue crack growth was detected at the initiation site or at the arrested crack tip. The low levels of recorded stress range from tests (see Load Test to Assess Fatigue Detail Performance on the Westbound I-80 Bridge over the Cedar River, Bridge Engineering Center, February 8, 2010) also verified that fatigue crack development was not probable as the stress range cycles were below the fatigue limit.

26. **Fisher, J. W., E. J. Kaufmann, W. Wright, Z. Xi, H. Tjiang, B. Sivakumar, W. Edberg. 2001. *Hoan Bridge Forensic Investigation Failure Analysis Final Report*. ATLSS Engineering Research Center, Lehigh University, U.S. Department of Transportation, Federal Highway Administration, Lichtenstein Consulting Engineers, June, Bethlehem, PA.**

This report presents the findings from the analytical and forensic investigations of the Hoan Bridge failure. The descriptions of two stated tasks are excerpted below:

- Task I – identify existing defects in steel members and details of the southbound spans through visual inspection and non-destructive testing methods. Hole drilling was performed at selected locations and other short-term repairs to ensure the safety of the southbound bridge (until long-term fatigue and fracture retrofit measures can be implemented) so that southbound lanes could be opened to traffic at the earliest possible date. The work has been completed and the Southbound bridge was opened to restricted traffic (4 ton weight limit) on February 17, 2001.

- Task II – conduct a failure analysis on the failed unit to ascertain the causes and modes of failure and make suggestions.

The report presents the mechanical properties of the web and bottom flanges, the global structural analysis as well as the detailed finite element analysis of the joint E-38 location (center girder of the 3 girder cross section), observations from the fractographic and metallographic examination, and a fracture analysis at the connection.

Relevant conclusions are excerpted below:

- All three girder web cracks at Panel Point 28 initiated from the crack-like geometric condition that resulted from the intersecting shelf plate and transverse connection plate welded connections with intersecting and overlapping welds.

- The resulting geometric configuration caused extreme high levels of constraint and stress to develop in the web plate gap from the forces in the girders and the K-type lateral bracing members. This resulted in stresses in the girder web gap [constraint-relief gap] that were estimated to be at least 60% greater than the yield point of the web plate.

- Brittle fractures (cleavage) were found to develop at every web crack examined without any detectable fatigue crack extension or ductile tearing at the crack origin."

- The nature of the web crack development results in a detail that is not inspectable. The small critical crack size cannot be detected.

- Once the web fractured, it was found that the bottom flange plates of Girders E-28 and F-28 were not capable of arresting the propagating crack. Only Girder D-28 was found capable of arresting the dynamic crack at -10deg F.

- All of the flange and web steels were found to have mechanical tensile properties and Charpy V-Notch toughness that satisfied the AASHTO requirements at the time the structure was built. The Charpy V-Notch toughness was also found to satisfy current (2001) requirements for Zone II.

- The web plates were found to have sufficient toughness to tolerate through plate thickness cracks under normal conditions without the high constraint conditions that were imposed on the girder web plate by the shelf plate welds and the intersecting transverse connection plate welds.

27. Fisher, J. W., G. L. Kulak, and I. F. Smith. 1998. *A Fatigue Primer for Structural Engineers*. National Steel Bridge Alliance, May, Chicago, IL.

This design practice document, which is not binding under FHWA regulations, focuses mostly on fatigue.

Chapter 2, *Basic Fracture Mechanic Concepts*, provides a brief overview of fracture mechanics methods of analysis, but primarily in the context of fatigue evaluations. For example, the discussion of brittle fracture focuses on using steel with sufficient fracture toughness to "promote stable crack extension."

In Section 5.2, *Redundancy and Toughness* (specifically on pg. 53), there is a single mention of triaxiality. "Use of a steel that has a good level of fracture toughness and avoidance of details that create triaxial stress conditions are important to achieving the desired fatigue life and preventing premature fracture."

Later, in Section 6.3.5, *Connections for Lateral Bracing*, there is a discussion of intersecting weld details, in the context of how fatigue cracks can grow in regions where there is a "web gap" (an area in which welds are interrupted to avoid the intersection of a horizontal weld with a vertical weld) where the horizontal gusset plate is not directly connected to the vertical stiffener. Following is an excerpt from the text that is illustrative of the concepts presented in this document in this regard:

> In the illustration, the horizontal connection plate is fitted around the vertical transverse stiffener and welded to it where they are in contact. In other arrangements, the vertical stiffener passes clear through a slot in the horizontal plate without attachment at that location. In either event, it is highly likely that a gap will be left in the region shown in order to avoid intersection of the horizontal and vertical welds. Consequently, as the bracing forces push into and pull on the web at this location, the web will rotate about a vertical axis formed by the back of the vertical stiffener and the plane of the web. Because the web is very flexible in the out-of-plane direction, this causes large strains in the web in the gap region when the lateral connection plate is not attached to the transverse stiffener. The region is also a zone of high residual stresses because of the proximity of the vertical and horizontal welds. Lack of fusion in the weld or other micro-defects can also be anticipated at the weld terminations. Taken all together, these conditions mean that the type of detail shown is very susceptible to fatigue crack growth.

The document stops short of presenting an alternate detailing suggestion; however, see Mertz (2015) for such a suggestion.

28. Fisher, J. W. and U. Yuceoglu. 1981. A Survey of Localized Cracking in Steel Bridges, 1978, Fritz Lab. No. 448-2(81). Fritz Engineering Laboratory, Department of Civil Engineering, Lehigh University. Interim Report September 1978 - June 1981, Bethlehem, PA.

This report documents a survey of failures of bridge members due to fatigue and/or fracture in 142 bridges. Fifteen representative bridges were the subject of a detailed review and summary discussion in the report. Of interest is the Quinnipiac River Bridge fracture. The Quinnipiac River Bridge was completed in 1964. In November 1973, a large crack was identified in the south fascia girder. A detailed study of the fracture suggested that it initiated in an unfused butt weld in a longitudinal stiffener at this location. "Examination of the crack surface in the web adjacent to the longitudinal stiffener showed that a 'brittle fracture' had occurred following the penetration of the web thickness by fatigue crack growth." The bridge had been subjected to an estimated 14.5 million cycles of "random truck loading." At the time of construction, the AASHTO FCP was not yet in place and the web material had an average CVN impact value of 20 feet of pound force (ft-lb).

Review of drawings in the report show that the longitudinal stiffener was on the opposite side of the web from a vertical stiffener, suggesting that this area was probably not subject to excessive triaxial constraint per se. It is more likely, in retrospect, that this is the case of a fatigue crack initiating from a welding discontinuity, which grew until fracture occurred.

29. Germanischer Lloyd. Unknown Date. *Comparison of No. 47 Shipbuilding and Repair Quality Standard (1996, Rev. 1 1999, Rev. 2 Dec. 2004) with Japan Shipbuilding Quality Standard (JSQS, 1004, Appendix 0144) and Production Standard of the German Shipbuilding Industry (FS, 2003)*.

This report compares various international shipbuilding practice documents. Among other items, Table 6.4 – Pillars, Brackets, and Stiffeners, specifies a minimum gap between web "T" stiffeners and flanges or 25 mm +10 mm – 5 mm, presumably to provide a minimum "web gap" (constraint-relief gap) between weld toes and avoid intersection of the welds associated with the three elements (web, stiffener, flange).

30. Hesse, A. A., R. A. Atadero, H. N. Mahmoud. 2014. "Approach-Span Failure of the Hoan Bridge as a Case Study for Engineering Students and Practicing Engineers," Journal of Performance of Constructed Facilities. American Society of Civil Engineers, Vol. 28, Issue 2, April, New York, NY.

The paper describes the failure of the Hoan Bridge and its causes, taking much from the research conducted in other studies. The paper is geared to providing the lessons learned from the failure, and offering a case study/teaching exercise for students and practitioners. The paper provides a very good summary of the failure, the investigation, and the conclusions and suggestions based on Hoan failure studies.

31. Hodgson, I. E., R. Sause, and Y. Chen. 2018. *Pilot Retrofit Study for the Normal Wood Bridge over the Susquehanna River, York and Lancaster Counties, PA*. Final Report for the Pennsylvania Department of Transportation. ATLSS Engineering Research Center, Lehigh University, August, Bethlehem, PA.

This letter report focuses on a retrofit study and testing of a bridge that suffered a CIF failure at a bearing stiffener. Included in the report is an investigation of the failure. Review of the report suggests that the CIF was due to a high degree of triaxial constraint at the top of the bearing stiffeners, in the gap between the end of the bearing stiffener welds and the toe of the flange-to-web welds. The bearing stiffener also functioned as a transverse connection plate, but was not attached to the top flange (tension flange) of the girder. As a result, the cross-frame forces applied to the bearing stiffener induced out-of-plane loading in the girder web. The tension associated with this out-of-plane bending of the web, combined with the high degree of triaxial constraint, and the discontinuities associated with the end of the bearing stiffener, appear to have satisfied the three conditions associated with elevated susceptibility to CIF.

32. Kaufmann, E. J., R. J. Connor, and J. W. Fisher. 2004. *Failure Investigation of the SR 422 over the Schuylkill River Girder Fracture-Draft Final Report*. ATLSS Engineering Research Center, Lehigh University, October, Bethlehem, PA.

This report documents the failure analysis of a fracture in a steel plate girder in the subject bridge. The bottom flange was completely fractured, and the web was fractured for a distance of approximately 9 inches up from the bottom flange. The fracture was located at the intersection of the welds that connected a lateral bracing gusset plate and a vertical stiffener to the girder web. The lateral bracing gusset plate also developed a fracture that propagated approximately 6 inches into the gusset plate from the connection to the vertical stiffener.

The report indicated that the fracture was similar to the Hoan Bridge girder fractures, and states: "In this case fracture was attributed to constraint induced fracture from high levels of tri-axial constraint at the crack-like geometrical condition at the weld intersection of the gusset plate and vertical stiffener elevating stresses well beyond the yield point in this region and stress intensities exceeding the fracture toughness of the web material." Testing of the web, flange, and gusset plate materials demonstrated that: "Although only the flange satisfied the current AASHTO Zone 2 fracture critical toughness requirement for Grade 36 steel of 25 ft-lbs @ 40 F, the web and gusset plate were marginal." This suggests that deficiencies in the toughness of the steel materials were not a significant contributing cause of the fracture. Fracture analysis calculations suggested a stress intensity of approximately 77 kilopounds per square inch (ksi), "which likely exceeded the weld metal toughness at temperature near the minimum service temperature. This resulted in a through-thickness crack analogous to the condition that existing at the Hoan Bridge and also to the Lafayette Street Bridge after fracturing by fatigue."

33. Keating, P. B., and J. W. Fisher. 1986. *Evaluation of Fatigue Tests and Design Criteria on Welded Details*, NCHRP Report 286. Transportation Research Board, National Research Council. September. Washington, DC.

This report suggested a new set of fatigue design curves. The report states:

> [These curves are intended] to better estimate the fatigue resistance of welded bridge details. Each of the fatigue curves has been normalized to a constant slope of -3.0. A new fatigue curve, Category B', has been added to better estimate the fatigue strength of partial penetration longitudinal groove welds and longitudinal welds with backing bars. Additionally, the fatigue design criteria for non-redundant members were revised to provide a more rational and consistent set of criteria.

34. Mahmoud, H. N., R. J. Connor, and J. W. Fisher. 2005. "Finite Element Investigation of the Fracture Potential of Highly Constrained Details," *Journal of Computer-aided Civil and Infrastructure Engineering*. Blackwell Publishing, Inc., Malden, MA, Vol. 20.

The paper describes the results of a detailed finite element analysis used to investigate CIF. A detailed finite element model was developed to study the potential for fracture of the detail through linear and nonlinear analyses. The linear analysis demonstrated the effect of having welded attachments on elevating triaxial stresses in the girder's web. The nonlinear analysis was used to assess the triaxiality demand of the detail and the potential for the development of large brittle cracks in the girder. The authors state that the results of this study have resulted in a clearer understanding of the behavior of highly constrained details and the methods to minimize the triaxiality factor in welded connections.

Constraint is introduced by welding attachments to the girder web. Constrained areas have been known to be the cause of high triaxial stresses and the development of large cleavage fractures in other types of welded structures such as ships and buildings. To quantify the degree of constraint the material is subjected to, a triaxiality factor was described and introduced by Schafer et al. (2000).

The authors state:

> Nonlinear finite element analysis is needed for the calculation of the triaxiality demand and the assessment of the fracture potential of such details. The analysis should include that maximum load experienced by the bridge including service life load, dead load, and residual stresses… The results of the nonlinear analysis demonstrated that a high constraint detail with a zero inch web gap [constrain-relief gap] has high potential for fracture. A slight increase in the web gap size (to at least 1/4") will result in smaller triaxial stresses and less potential for fracture. In general, the following was concluded from the analyses:
>
> 1. High triaxial stresses through the web thickness exist in details with zero web gap.

2. For proper assessment of triaxiality demand, nonlinear finite element analysis should be conducted. The triaxiality demand obtained using the nonlinear analysis was higher than that obtained using a linear analysis.

3. Welded connections should be detailed to minimize the degree of constraint and stress concentrations to lower the possibility of brittle fracture.

The paper does not mention how the ¼-inch minimum gap dimension was determined to be sufficient. The paper does not mention any analytical study as justification of this dimension.

35. Mertz, D. 2015. "Design for Fatigue," ***Steel Bridge Design Handbook*****, Volume 12: Design for Fatigue, Publication No. FHWA-HIF-16-002, Vol. 12. U.S. Department of Transportation, Federal Highway Administration, December, Washington, DC.**

This design practice document, which is not binding under FHWA regulations, is largely a condensed version of Fisher, et al (1998). The document focuses primarily on fatigue. Section 2.3, *Fracture*, provides a very brief discussion of fracture, but avoids the topic of CIF, and focuses instead on the topics of toughness, the AASHTO FCP, and Charpy V-Notch testing. There is no mention of CIF or triaxial stress conditions.

Section 2.4.2.6, *Connections for Lateral Bracing*, provides a discussion identical to that provided in Section 6.3.5, *Connections for Lateral Bracing*, of Fisher, et al (1998), but in this case an alternate detailing suggestion is provided: "Due to the potential for fatigue cracking, when lateral bracing is required, and when the flange width and girder design allows, the designer should consider bolting the lateral-bracing members directly to the flanges."

36. Miller, D. K. 2017. ***Welded Connections – A Primer for Engineers, Steel Design Guide 21, 2nd Edition*****. American Institute of Steel Construction, Chicago, IL.**

This design practice document, which is not binding under FHWA regulations, focuses on the welded connections in general, starting with applicable design codes, and including discussion on the welding and thermal cutting process, discussion on the types of welded connections (fillet, CJP, PJP, etc.), and discussion on weld cracking, distortion caused by welding, welding procedures, weld quality, fatigue considerations, and special welding applications. The document includes discussion of highly constrained connections where residual stresses can exceed the uniaxial yield strength of the material, and mentions that "Under severe restraint, normally ductile weld metal or base metal may crack instead of yielding." The document also notes that "High toughness values alone will not ensure adequate structural performance when stresses are too high, when members are highly constrained or when severe geometric stress raisers exist." There is an entire chapter on fracture-resistant welded connections, which mentions (among other items), "Highly restrained connections should be avoided. Triaxial constraint results in a state [of] stress where there is little or no shear stress, yet shear stresses are essential for ductile behavior."

37. Pass, J. D., K. H. Frank, and J. A. Yura. 1983. *Fatigue Behavior of Longitudinal Transverse Stiffener Intersection*, Report FHWA/TX-83/9+247-5F. Center for Transportation Research, The University of Texas at Austin, May, Austin, TX.

This is an older report, focused on a single case study of a bridge designed before the implementation of modern fatigue design provisions in later AASHTO bridge design specifications. However, the report yielded interesting and valuable results.

Specifically, the structure under investigation was a twin-girder, two-span, continuous, steel plate, girder bridge with floor beams, where the floor beam connection plates intersected with longitudinal web stiffeners. In some instances, these intersections occurred near inflection points, such that the longitudinal stiffener was subjected to tension under live load. The clear distance between the transverse stiffener plate (vertical stiffener) and the longitudinal stiffener plate was ½ inch. The distance between the weld toes (the transverse stiffener was attached to the web with 5/16-inch fillet welds, while the longitudinal stiffener was attached to the web with ¼-inch fillet welds) was less.

Fourteen fatigue specimens were tested, 10 to evaluate the fatigue performance of the detail, and four to evaluate retrofit details. Section 2.5 of the report mentions the importance of gap size in reducing the stress concentration in the web in this region, and describes how some test specimens were designed with both a ½-inch gap and some with a 2-inch gap (both with a +/- 1/16-inch tolerance). Two retrofit options were considered: first was increasing the gap width to reduce stress concentrations, and second was to make the longitudinal stiffener continuous by adding supplemental plates to the longitudinal stiffener and welding them to the transverse stiffener.

The results of the fatigue testing suggested that increasing the gap size from ½ inch to 2 inches resulted in only a moderate improvement in fatigue life. Reducing the ratio of the area of the stiffener to the thickness of the web (the A_{st} / t_w ratio) produced a greater improvement in fatigue life than increasing the "web gap" (constraint-relief gap). The results of the fatigue testing also indicated that the retrofit plate detail exhibited much better fatigue performance; the intent of the detail was to mimic a Category B detail, but the nature of the retrofit also introduced some Category C and E details and the resulting net effect was to produce performance similar to a Category E detail, which was a significant improvement over the performance of the original detail, which exhibited performance worse than a Category E' detail.

The report did not discuss CIF or triaxial stress conditions. The retrofit plate details did not include attachment of the retrofit plates to the girder web, so the original ½-inch "web gap" dimension (between plates, not between web toes) was maintained and the effective local constraint of the original detail was not directly affected.

38. Platten, D. A. 1980. "Analytical Study of the Fatigue Behavior of a Longitudinal-Transverse Stiffener Intersection," Thesis. The University of Texas at Austin, May, Austin, TX.

The objective of this study was to establish an analytical estimate of the fatigue life of a longitudinal-transverse stiffener intersection detail, as well as develop an experimental test specimen to determine the adequacy of the fatigue designs practices current at the time.

It is noted that when a longitudinal-transverse stiffener intersection exists, that the current (1980) fatigue design practices suggest that fillet welds for longitudinal stiffener be terminated short of web-to-transverse stiffener welds by a distance of at least 4 to 6 times the web thickness. Previous tests (by others – Fisher NCHRP Report 147 – Fatigue Strength of Steel Beams with Transvers Stiffeners and Attachments) have indicated that failure to terminate longitudinal stiffener welds a suitable distance short of the transvers stiffener welds can results in adverse behavior due to restraint stresses induced by weld shrinkage.

The influence of detail geometry on the stress concentration at the longitudinal stiffener to web weld toe was investigated. Increasing the longitudinal stiffener width and/or thickness resulted in a more severe stress concentration at the weld toe. Increasing the girder web thickness decreased the stress concentration. The size of the gap between the longitudinal stiffener and the transvers stiffener was found to affect the stress concentration. When the gap size was increased from ½ inch to 2 inches, a drop in stress concentration at the weld toe of approximately 65 percent was observed.

Additionally, fracture mechanics principals were used to estimate the fatigue life of the longitudinal-transverse stiffener intersection detail (all welded, not gaps). It was found that the detail fell below Category E' at the time.

It was also noted that more experimental research was warranted in this area.

39. Ricles, J. 2000. *SAC Task 7.05 – Inelastic Cyclic Analysis and Testing of Full-scale Welded Unreinforced Flange Connections*. ATLSS Engineering Research Center, Lehigh University, August, Bethlehem, PA.

A summary of the report is excerpted below:

> Analytical and experimental studies were conducted to evaluate the effect that various parameters have on connection ductility in order to improve the cyclic inelastic performance of welded unreinforced flange moment connections. The analytical studies included nonlinear finite element analysis of connections subjected to monotonic and cyclic loading. A low-cycle fatigue failure formulation was developed and applied to analytically evaluate the cyclic ductility of various connection details. The experimental studies consisted of inelastic cyclic tests of full-scale connection specimens.

These types of connections are more common in building structures, and in fact the research in this paper was conducted, in part, in response to the performance of building structures during the Northridge earthquake. Suggestions included improving the toughness of the weld metal used to make the flange groove welds.

The effects of triaxial constraint, quantified in the report through the use of the triaxiality ratio (same as the T_1 triaxiality ratio described by Schafer, et al., 2000) and the "Rupture Index," which considers the ratio of the effective plastic strain (PEEQ) to the yield strain, denoted as the PEEQ Index, as well as the triaxiality ratio. The PEEQ Index is "a measure of local ductility," and is essentially a material property.

Ultimately, the details investigated in this research are not common in steel girder bridge construction and the research focused primarily on low-cycle fatigue performance, which is of greater importance in evaluating the response of buildings to seismic events, where ductile moment frames are an important structure type.

40. **Russo, F. M., D. R. Mertz, K. H. Frank, and K. E. Wilson. 2016. *Design and Evaluation of Steel Bridges for Fatigue and Fracture – Reference Manual*, Report No. FHWA-NHI-16-016. U.S. Department of Transportation, Federal Highway Administration, National Highway Institute, December, Washington, DC.**

This reference manual, which is not binding under FHWA regulations, explains the relevant issues related to fatigue and fracture in steel bridges, including analysis, design, evaluation, repair, and retrofit. The manual is to accompany a National Highway Institute (NHI) training course. Its objective is to provide engineers with technical information to effectively design and evaluate steel highway bridge structures for the limit states of fatigue and fracture to improve safety, economy, and longevity of infrastructures. An additional objective is to educate engineers about the history, scope, methodologies, assumptions, limitations, and application of AASHTO design specifications as related to design for fatigue and fracture.

Section 7.2.4 discusses detailing to avoid constraint. The manual suggests that it is the responsibility of the design engineer is to choose details that are not susceptible to fracture due to excessive constraint. Two sources of constraint in welded steel structures are described: elements joined with intersecting welds and the constraint from thick plates and highly constrained joints.

The manual states:

> CIF can occur when the web gap [constraint-relief gap] area is not allowed to contract due to the proximity of the longitudinal and vertical stiffeners and their attaching welds. As flexural tension is applied to the girder web, its tendency to contract in the through thickness direction is highly restrained by the intersection of the longitudinal attachment, the transverse stiffener, and the multiple intersecting welds that all join at a common location.

Several discussion points and figures are taken from Connor and Lloyd, 2017.

The manual includes a list of findings about the Hoan Bridge fracture, excerpted below:

- The failure was confirmed through laboratory investigation as brittle fracture.
- There was no evidence of fatigue cracking prior to fracture initiation.
- The narrow gap between the gusset plate and transverse stiffener increased the local stiffness and prevented yielding and stress redistribution in the constrained area.
- Prior similar cracks were found several years earlier and were at the time assumed to be fatigue cracks. These prior cracks did not result in the chain reaction fracture of multiple elements of the bridge.
- The States were advised through a series of technical advisories and memos from the FHWA to identify bridges that might have similar details, conduct inspections, and implement selected retrofits as necessary.

The manual notes that to guard against CIF, the AASHTO *LRFD BDS* provides an approach for engineers to use in the design of new steel structures. The manual quotes the following text from the non-binding AASHTO *LRFD BDS* (7th Edition, up to and including the 2016 Interim Revisions):

> To the extent practical, welded structures shall be detailed to avoid conditions that create highly constrained joints and crack-like geometric discontinuities that are susceptible to constraint-induced fracture. Welds that are parallel to the primary stress but interrupted by intersecting members shall be detailed to allow a minimum gap of 1 inch between weld toes.

In the AASHTO BDS, 8th Edition, (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)) the specified dimension for the minimum gap was reduced from 1 inch to 0.5 in, and the language was also changed slightly. The text from the AASHTO BDS, 8th Edition, (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)) is excerpted below:

> Welded structures shall be detailed to avoid conditions that create highly constrained joints and crack-like geometric discontinuities that are susceptible to constraint-induced fracture, as summarized in Tables 6.6.1.2.4-1 and 6.6.1.2.4-2. If a gap is specified between the weld toes at the joint under consideration, the gap shall not be less than 0.5 in.

This provision focuses on maintaining a minimum separation between weld toes as the key practice to minimize the potential for CIF.

Section 7.2.4 of the manual also includes six figures that provide illustrations and commentary of practices that are encouraged and discouraged with respect to CIF. Many of these, or similar, are not presented in the AASHTO BDS, 8th Edition, (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v)).

41. **Ryan, T. W., J. E. Mann, B. T Ott, Z. M. Chill, M. P. Rosick, R. Hartle, L. J. Danovich, P. Fish. 2010. *Fracture Critical Inspection Techniques for Steel Bridge*, Participant Workbook for Course 130078, Report No. FHWA-NHI-11-015. Federal Highway Administration, National Highway Institute, Washington, DC.**

This is the Participant Workbook for NHI Course 130078, the training course for bridge inspectors seeking to be certified to inspect fracture-critical bridges, which is not binding under FHWA regulations. The course includes discussion of triaxial constraint and intersecting welds, both of which are included in the grouping of "problematic details." The workbook discusses both topics, including a simple, but clear, explanation of the implications of triaxial constraint as a potential source of fracture at low stress levels. The workbook further continues in the same section to describe "intersecting welds," characterizing them as "welds that run through each other, overlap, touch, or have a gap between their toes of less than ¼ [inch]." The figures included in the workbook show cases that are more like a triaxial constraint situation.

42. **Ryan, T. W., J. E. Mann, Z. M. Chill, B. T. Ott. 2012. *Bridge Inspector's Reference Manual (*BIRM*)*. Federal Highway Administration, National Highway Institute, Washington, DC.**

This manual, which is not binding under FHWA regulations, discusses the inspection of bridges in accordance with the National Bridge Inspection Standards (NBIS) found in 23 CFR Part 650, Subpart C. The BIRM includes discussion of intersecting welds and triaxial constraint in steel bridges. The BIRM provides good discussions of both topics, including a simple, but clear, explanation of the implications of triaxial constraint as a potential source of fracture at low stress levels. The BIRM further continues in the same section to describe "Intersecting Welds," characterizing them as "welds that run through each other, overlap, touch, or have a gap between their toes of less than ¼ inch (see Figure 6.4.48)." The BIRM description of intersecting welds includes the intersection of a flange or web butt splice with the flange-to-web fillet welds, but does not differentiate these from other details that include intersecting welds. The BIRM's Figure 6.4.48 shows a case that is more like a triaxial constraint situation. The BIRM states:

> This problematic detail allows for alternate, unanticipated stress paths that may act as stress risers, leading to crack initiation. Intersecting welds are not fatigue related or material dependent and may consequently occur under low stress levels in a ductile material with good toughness properties. Additionally, intersecting welds may leave large residual stresses after welding, leading to possible cracking and reduced fatigue strength. Welds are terminated short of the intersection by at least ¼ inch to avoid intersecting welds. In most cases, it is desirable to allow the longitudinal weld (parallel with the applied stress) to be continuous. This avoids Category E type detail at the weld termination if it is interrupted. The end termination of a transverse weld does not directly affect its fatigue strength and is classified as Category C' for plates.

43. Schafer, B.W., R. P. Ojdrovic, and M. S. Zarghamee. 2000. "Triaxiality and Fracture of Steel Moment Connections," *Journal of Structural Engineering*. American Society of Civil Engineers, Vol. 126, No. 10, October, New York, NY, pp. 1131-1139.

This paper describes the concept, and parameters, of triaxiality. "In this paper, triaxiality is defined as the ratio of the maximum principal stress to the von Mises stress." The paper provides two descriptions of triaxiality:

$$T_1 = \frac{\sigma_{hydrostatic}}{\sigma_{eff}}$$

$$T_2 = \frac{\sigma_1}{\sigma_{eff}} = \frac{\sigma_{max}}{\sigma_{eff}}$$

Where:

$$\sigma_{hydostatic} = \frac{\sigma_1 + \sigma_2 + \sigma_3}{3}$$

$$\sigma_{eff} = \sqrt{\frac{1}{2}}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_1 - \sigma_3)^2} = von\ Mises\ stress$$

$$\sigma_1, \sigma_2, \sigma_3 = principal\ stresses\ in\ the\ x, y, and\ z\ direction, respectively$$

These ratios are helpful in evaluating constraint, as they provide a measure of tensile stresses in directions orthogonal to the direction of primary stress. These orthogonal tensile stresses represent restraint of the ability of the material to "flow," i.e., restraint of the ability of the material to experience Poisson's effect.

The paper offers another description of triaxiality: "In its simplest terms, triaxiality is the ratio of the state of stress a material undergoes to the stress that contributes to yielding." The paper later offers an example of a case where the T_2 triaxiality ratio is 1.5, but the ratio of σ_{ult} / σ_y is only 1.4; in this case the maximum stress reaches 1.5 times the nominal yield stress before the material can actually yield. Since the ultimate strength of the material was only 1.4 times the nominal yield stress, the stress exceeds the ultimate strength before the material yields.

44. Spadea, J. R. and K. H. Frank. 2004. *Fatigue Strength of Filled-Welded Transverse Stiffeners with Undercuts*, Report FHWA/TX-05/0-4178-1. Center for Transportation Research, The University of Texas at Austin, July 2002, Revised October, Austin, TX.

This report documents fatigue performance of stiffeners with undercuts evaluated in a factorial design fatigue experiment. Undercuts at the flange edge and filling of the stiffener cope with the fillet weld were also evaluated. The undercuts did not influence the fatigue performance of the stiffener weld even when very large undercuts were purposely introduced. Filling of the cope with a continuous weld and flange undercuts also did not change the fatigue performance of the connection. Wrapping of welds in locations were corrosion at the stiffener to plate interface may occur should be allowed. The undercuts in the stiffener that may result are not of any consequence and may be left in place. Covering the stiffener cope by continuing the fillet weld is an alternate method of sealing the weld.

The authors suggest that based upon the results of these tests, wrapping the stiffener-to-flange welds to prevent corrosion of the stiffener should be employed on exposed exterior stiffeners to seal the interface between the stiffener and the flange. The attendant undercutting that may occur was not found to be detrimental to the performance of the structure and can be left in place.

The research report does not provide any information with regard to intersecting welds related to CIF.

45. Unknown. 2004. *Guide for Evaluating and Retrofitting Bridges for Constraint-Induced Fracture*, ATLSS Report No. Unknown. ATLSS Engineering Research Center, Lehigh University, October, Bethlehem, PA.

This draft report documents at least some of the background research that is discussed in Mahmoud, et al. (2005), specifically the analytical studies that contributed to the conclusion: “The results of the nonlinear analysis demonstrated that a high constraint detail with a zero inch web gap [constraint-relief gap] has high potential for fracture. A slight increase in the web gap size (to at least 1/4”) will result in smaller triaxial stresses and less potential for fracture.”

46. Verma, K., K. Frank, W. Wright, R. Medlock, B. McEleney, P. Loftus, B. Kase, H. Campbell, A. Wilson, J. Fisher, K. Lineham, M. Cress, R. Anderson, A. Shriole, D. Mertz, J. Uttrachi, R. Sathre, J. Hamilton. 2001. *Steel Bridge Fabrication Technologies in Europe and Japan*, Report No. FHWA-PL-01-018. Office of International Programs, Office of Policy, Federal Highway Administration, U.S. Department of Transportation. March. Washington, DC.

This report documents a scanning tour that observed steel bridge fabrication methods, technologies, and techniques in Europe and Japan. Among other observations, the report documents the wrapping of welds at the ends of stiffeners and the intersection of welds where various plates intersect without cope holes, both observed in Japan.

47. Wahr, A. S. 2010. “The Fatigue Performance of Cross Frame Connections,” Thesis. The University of Texas at Austin, August, Austin, TX.

The research described in this thesis is related to investigations of the fatigue performance of the “half-pipe stiffener” detail used for connection of severely skewed cross-frames to girders in steel plate girder bridges. Both experimental (laboratory testing) and analytical (FEM studies) studies were conducted.

The experimental testing featured half-pipe stiffeners welded to a W21x101 rolled beam. A continuous fillet weld was used to connect the half-pipe stiffener to the beam flanges and web, without any special preparation to achieve a tight fit.

The investigations focused primarily on fatigue performance. Hot spot stress analysis was used in the analytical investigations to evaluate areas of high stress concentration. There was no direct discussion of CIF or triaxial stress conditions. The topic of intersecting welds was not discussed in this research; the beam used in the testing was a rolled section, not a welded plate girder. However, the effects of triaxial constraint should be similar for welded plate girder structures.

48. Wright, W. J., J. W. Fisher, and E. J. Kaufmann. 2003. "Failure Analysis of the Hoan Bridge Fractures," ***Recent Developments in Bridge Engineering*****, Mahmoud ed. Swets & Zeitlinger, Lisse.**

This paper is generally as summary of the research presented in "*Fisher, J. W., E. J. Kaufmann, W. Wright, Z. Xi, H. Tjiang, B. Sivakumar, W. Edberg. 2001. Hoan Bridge Forensic Investigation Failure Analysis Final Report*." The conclusions in the paper are the same as noted in the final report.

The paper states:

> The original structure was designed as a noncomposite system. The calculated dead load stress in the girder web at the lateral gusset plate was about 105 Mpa [15 ksi]. Although the bridge was designed without composite action, strain measurements verified that full composite action for friction was present between the girders and deck for all live load conditions.
>
> Charpy V-notch tests were carried out on the girder and flange plates at the fracture locations… It is apparent that all three web locations satisfy AASHTO requirements for Zone 2 fracture critical applications were the lowest anticipated service temperature is -34 deg C… The dynamic toughness was in the lower transition region, providing limited resistance to dynamic crack propagation.
>
> In general, all fracture surfaces examined were covered with a layer of corrosion of varying thickness resulting from exposure to weather and salt prior to the removal from the structure.
>
> The fracture origin of Girder E had been traced to the girder web at the lateral bracing connection where the shelf plate partial penetration weld joint to the web terminated adjacent to the vertical connection plate... Cleavage chevron marks on the web fracture surface were observed to point from both directions to the shelf plate weld termination thus confirming this area to be the fracture origin.

APPENDIX B – SUMMARY OF CURRENT OWNER-AGENCY POLICIES AND PRACTICES

A review was performed of several published owner-agency documents related to design, fabrication, fabrication inspection, in-service inspection, and repair and retrofit of steel bridges, with particular focus on documents related to intersecting welds or constraint-induced fracture. Short excerpts, with added discussion, are provided in this appendix. This does not constitute a comprehensive review of all fifty States and other owner-agencies; the reviewed documents were selected by the authors based on their past knowledge of the owner-agencies' practices.

Federal Highway Administration (FHWA)

The BIRM (Ryan et al., 2010) includes discussion of intersecting welds in steel bridges. In Section 6.4.7, *Fracture Criticality*, the BIRM states:

> Initial deficiencies, in many cases, are cracks resulting from poor quality welds between attachments and base metal (see Figure 6.4.45). Many of these cracks occurred because the groove-welded element was considered a 'secondary' attachment with no established weld quality criteria (e.g., splices in longitudinal web stiffeners or back-up bars). Intersecting welds can provide a path for the crack to travel between steel members (see Figure 6.4.46).

In Section 6.4.6, *Inspection Methods and Locations*, the BIRM identifies intersecting welds and triaxial constraint as part of a grouping of what it calls problematic details, stating:

> Problematic details may exist on a variety of steel bridges such as girder, frame, truss superstructures, and substructure components. The following are problematic details, which can lead to fatigue cracking:
>
> - Triaxial constraint
> - Intersecting welds
> - Cover plates
> - Cantilevered suspended span
> - Insert plates
> - Field welds: patch and splice plates
> - Intermittent welds
> - Out-of-plan bending
> - Pin and hanger assemblies
> - Back-up bars
> - Mechanical fasteners and tack welds.

The BIRM continues in the same section to describe triaxial constraint, using the Hoan Bridge fracture as an example, stating:

> Triaxial constraint leads to plastic constraint and brittle fracture. This fracture condition can be produced by a narrow gap between the gusset plat and transverse connection/stiffener plate (see Figure 6.4.47). Elastic stress results indicate that triaxial constraint will prevent yielding of the steel until the stress exceeds approximately 1.3 times the yield strength of the material. Under high plastic, constraint, local stresses can reach 2 to 3 times the average stress.

The BIRM further continues in the same section to describe intersecting welds, characterizing them as "welds that run through each other, overlap, touch, or have a gap between their toes of less than ¼ inch (see Figure 6.4.48)." The FHWA/NHI *Bridge Inspector's Reference Manual*'s (BIRM) description of intersecting welds includes the intersection of a flange or web butt splice with the flange-to-web fillet welds, but does not differentiate these from other details that include intersecting welds. The BIRM's Figure 6.4.48 shows a case that is more like a triaxial constraint situation. The BIRM states: "This problematic detail allows for alternate, unanticipated stress paths that may act as stress risers, leading to crack initiation. Intersecting welds are not fatigue related or material dependent and may consequently occur under low stress levels in a ductile material with good toughness properties. Additionally, intersecting welds may leave large residual stresses after welding, leading to possible cracking and reduced fatigue strength. Welds are terminated short of the intersection by at least ¼ inch to avoid intersecting welds. In most cases, it is desirable to allow the longitudinal weld (parallel with the applied stress) to be continuous. This avoids Category E type detail at the weld termination if it is interrupted. The end termination of a transverse weld does not directly affect its fatigue strength and is classified as Category C' for plates."

Section 10.1.4, Inspection Methods and Locations, also mentions triaxial constraint and intersecting welds as "Problematic Details." Section 10.2.4, Section 10.3.4, Section 10.4.4, 10.5.4, and 10.6.4, have the same language.

Alabama

The Alabama DOT *Structural Design Manual*, (Alabama Department of Transportation, 2017), Section 6, "Steel Structures" (subsection Steel Fabrication – Shop Connections), states:

> 1. Intersecting welds shall not be permitted.
>
>   a. Corners of transverse stiffeners shall be clipped and welded as follows:
>
>   1) Stiffener-to-web welds shall be terminated 1 inch plus or minus ¼ inch from the clip.
>
>   2) Stiffener-to-flange welds shall be terminated ½ inch plus or minus ¼ inch from the clip and the edges of the stiffener plate.
>
>   b. Longitudinal stiffeners shall be cut back a minimum of 2 inches to avoid intersecting welds.

The Alabama DOT *Bridge Inspection Manual* (Alabama Department of Transportation, 2014), Chapter 12, "Fracture Critical Members and Fracture Critical Bridges," instructs inspectors to focus on key areas, particularly areas where stress concentrations may exist; among other details and conditions, the manual states:

> Examples of details that are normally checked closely include the following:
>
> 1. Intermittent welds between the web and tension flange
> 2. Areas of sudden change of cross-section near the ends of cover plates
> 3. Locations of stress risers such as nicks, scars, flaws, and holes that have plug welds, irregular weld profiles and areas where the base metal has been under cut
> 4. Locations where stiff bracing members of horizontal connection plates are attached to their webs and girder flanges
> 5. The floor beam and girder web adjacent to a floor beam connection plate
> 6. Gusset plates, improperly coped members re-entering corners and the gap between web stiffeners and flanges
> 7. Longitudinal and vertical stiffener intersections
> 8. Longitudinal stiffeners that have been connected together with butt welds
> 9. Location of welds at gusset-transverse-web intersections
> 10. Flanges that pass through a web such as girder flange passing through a box girder pier cap
> 11. Box beam to column intersection
> 12. Eyebars / Truss Members.

Colorado

The Colorado DOT *Standard Specifications* (Colorado Department of Transportation, 2017), Section 509.20 (i) states that "Intersecting fillet welds will not be allowed." It also states:

> Weld Termini Treatment. All gussets, stiffeners, diaphragms, or other attachments at a corner of intersecting plates joined by a fillet or groove weld, shall be clipped 1 ½ inch minimum. Intersecting fillet welds will not be allowed. Treatment of all end weld termini on transverse secondary attachments to main members shall be such that the welds terminate ¼ inch short of the end of the attachment.

Florida

The Florida DOT *Structures Detailing Manual* (Florida Department of Transportation, 2019), Section 16.10.D.1, states: “Intersecting welds: Do not use this type of detail.”

Georgia

The Georgia DOT *Bridge and Structures Policy Manual* (Georgia Department of Transportation, 2018), Section 3.6.6, states:

> No intersecting welds will be allowed on structural steel bridge plans or shop drawings to prevent crack propagation from welds in that area. Base metal in the intersection area of welds shall be coped 4 times the thickness of the web or 2 inches, whichever is greater.

Iowa

The Iowa DOT, Office of Local Systems, issued an electronic memorandum (Iowa Department of Transportation, 2011) addressing the “potential for brittle fracture to occur at certain details on Iowa welded steel girder bridges, specifically at details which involve triaxial constraint. Triaxial constraint occurs when multi-directional welds intersect or come in close contact.” The memorandum includes a Iowa DOT white paper (Iowa Department of Transportation, 2010) titled: “Potential for Fracture to Occur in Iowa DOT Steel Bridge Bridges due to Triaxial Constraint.”

The white paper states:

> There have been 34 fractures in Iowa steel bridges since the mid-1960s. At the time these occurred, none were identified as due to triaxial constraint. As explained herein, it is likely that 9 of the fractures occurred due to triaxial constraint.” The white paper goes on to discuss triaxial constraint, with reference to the Hoan Bridge fracture in December 2000. The white paper also describes a forensic investigation of a bridge that experienced fracture at the top of the pier bearing stiffener in an exterior girder; the investigation by Dr. John Fisher and Dr. Eric Kaufmann determined that: “The fracture of the girder web was found to be initiated from the severe geometric triaxial restraint between the ends of the bearing stiffeners and the web-flange fillet welds.

The white paper also discusses the inspection and evaluation of bridges for the potential for constraint-induced fracture (CIF), stating:

> In view of what has recently been learned, the potential for brittle fracture on Iowa DOT two girder bridges should be addressed as follows:
>
>> The girder web plate at the top of the pier bearing stiffeners should be inspected to determine if the stiffener to web welds intersect the web to flange welds or if the gap between the toes of the welds is less than ¼ in. If this condition is found, 2 in. or 3 in. diameter holes should be cored

> through the web on both sides of the stiffeners. The core hole would penetrate the horizontal and vertical fillet welds by approximately 1/8 inch. This retrofit is shown on Attachment 8.
>
> Gusset plates for lateral bracing in tension areas should be inspected to determine if there are intersecting welds in the corner formed by the web, gusset plate and connection plate. If this condition exists (or if the gap between the toes of the welds is less than ¼ in.), a hole should be cored thru the gusset plate corner to increase the gap length. This retrofit is shown on Attachment 9.

> The ¼ inch minimum gap between welds is based on research conducted subsequent to the Hoan Bridge fractures. The results of research is covered in an article in the Journal of Computer-Aided Civil and Infrastructure Engineering entitled, 'Finite Element Investigation of the Fracture Potential of Highly Constrained Details' and in the previously mentioned ASCE article, 'Preventive and Mitigation Strategies to Address Recent Brittle Fractures in Steel Bridges.'
>
> It is likely that triaxial constraint at the top of bearing stiffeners exists in many multi-girder bridges. However, it is not suggested that these structures be retrofitted due to their redundancy and the rarity of brittle fractures."

In addition, the Iowa DOT Office of Bridges and Structures *Bridge Inspection Manual* (HDR, 2015) discusses triaxial constraint and how it can lead to brittle fracture, specifically in Section 2.4.2.4.1, "Fatigue-Prone Details," and 4.6.1, "Fatigue-Prone Details." Intersecting welds are also cited in these sections as one of the "fatigue-prone details."

Kansas

The Kansas DOT *Bridge Design Manual*, (Kansas Department of Transportation, 2016) Section 6.4.3, "Longitudinal Web Stiffeners," states:

> The longitudinal stiffeners will be continuous where practical and should be welded to intersecting connection stiffener, a space of four to six times the web thickness should be left between the vertical connector weld and the longitudinal stiffener weld. To avoid weld intersection, the transverse stiffeners are coped to clear the longitudinal stiffener to web fillet weld. See Figure 6.4.3-1 Longitudinal Stiffener Details for additional information. The longitudinal stiffeners should be placed on the opposite side of the web from the transverse stiffeners. This will minimize the number of places where longitudinal and transverse stiffeners intersect. The exceptions to this are diaphragm or cross-frame connection stiffeners.

Maine

The Maine DOT *Bridge Design Guide* (Maine Department of Transportation, 2018), Appendix D.7, "Standard Notes Structural Steel," includes the typical limitations regarding minimum offsets between flange butt weld shop splices and web butt weld shop splices, but also prohibits such splices within 50 feet or 10 percent of span length from maximum negative or positive moment points, and also prohibits such butt welds from being closer than 3 feet from other transverse welds such as connection plate to web welds. The guide states:

> No transverse butt-weld splices will be allowed in the flange plates or web plates within 10 feet or 10% of the span length (whichever is greater) from the points of maximum negative moment or maximum positive moment. Butt-weld splices in flanges shall be not less than 3 feet from the transverse butt-welds in the web plates and no transverse web or flange butt-welds shall be located within 3 feet of other transverse welds (e.g. connection plates to web welds) on either flange or web. No transverse butt-weld splices will be allowed in areas of stress reversal.

Missouri

The Missouri DOT online *Engineering Policy Guide*, (Missouri Department of Trnasportation, 2019), Sections 751.14.5.6, "Longitudinal Stiffeners," and 751.14.5.7, "Lateral Bracing," show details that include copes and constraint-relief gaps, effectively avoiding intersecting weld situations for these types of details.

Montana

The Montana DOT *Structures Manual* (Montana Department of Transportation, 2002), Section 18.7.2.5, "Design of Welds," states in Note 2 on pg. 18.7(5): "Intersecting Welds. These should be avoided, if practical."

The Montana DOT *Bridge Inspection Manual*, (Montana Department of Transportation, 2015), Section 4, "Steel Bridge Inspection," recognizes intersecting weld details as susceptible to fatigue and fracture.

New York

The New York State DOT *Bridge Design Manual* (New York State Department of Transportation, 2017), Section 8.6.3, "Longitudinal Stiffeners," states:

> Longitudinal stiffeners shall be continuous for their entire length, with intermediate transverse stiffeners and connection plates cut short to avoid intersecting welds. Exceptions are when the longitudinal stiffener is interrupted by a field splice in the girder, or when the stiffener is no longer required by design. In these circumstances, the designer shall be responsible for providing the appropriate termination details that comply with the NYSDOT Steel Construction Manual on the contract plans.
>
> When longitudinal stiffeners are required, show them placed on one side of the web only. On fascia girders they shall be placed on the web surface exposed to view. The intermediate transverse stiffeners, if necessary, shall be placed on the opposite side of the web. The longitudinal stiffeners shall be attached to the web

plate with full-length, continuous, 5⁄16” fillet welds. Fabrication details including transverse connection plate and longitudinal stiffener intersection details shall be in accordance with the NYSDOT Steel Construction Manual.

Ohio

The Ohio DOT *Bridge Design Manual* (Ohio Department of Transportation, 2019), Section 302.4.3.1, prohibits the use of longitudinal stiffeners in steel plate girder bridges, stating: “Longitudinal stiffeners shall not be used.” There is no other criteria in the design manual with regard to the use of intersecting welds.

Pennsylvania

The Pennsylvania DOT *Design Manual 4* (Pennsylvania Department of Transportation, 2015), Part B, “Design Specifications,” Section 6.13.3.8P, “Intersecting Welds,” states:

> Intersecting welds which provide a potential crack path into the web or flange of a girder from an attachment will not be permitted. The termination of the fillet weld to prevent the intersection shall provide a minimum clearance of 1 1/2 in., unless another clearance is required by other design documents. Transverse groove welds shall not be terminated to prevent the intersection.

The Pennsylvania DOT *Bridge Safety Inspection Manual* (Pennsylvania Department of Transportation, 2010), Section 2.4.9.2, “Intersecting Welds,” discusses the inspection of intersecting welds, which are described as “… welds that run through each other, overlap, touch, or have a gap between their toes of less than ¼”. The intersecting welds of the web-splice-to-flange or flange-splice-to-web are not of concern here. Three-dimensional details with intersecting welds are the critical intersecting welds.” The document lists examples of critical details, actions to take if such welds are discovered, and commentary about the potential failures associated with these types of details, stating:

> Intersecting welds on some bridge details in tension or reversal zones have led to brittle fracture and failure of main longitudinal bridge members without warming. Intersecting welds are defines as welds that run through each other, overlap, touch, or have a gap between their toes of less than 1/4". The intersecting welds of the web-splice-to-flange or flange-splice-to-web are not of concern here. Three-dimensional detail with intersecting welds are the critical intersecting welds. These types of details introduce an out of plane force at the intersecting weld. Examples of some details that may have critical intersecting welds include:
>
> 1. Wind bracing connections to girder webs.
> 2. Floor beam connections to girder webs.
>
> More often than not, the bridge design and shop drawings will not indicate that intersecting welds are present on a bridge. However, excessive welding during fabrication or repairs have resulted in intersecting welds. Because these welds have caused girder failures on redundant, as well as, fracture critical girders, inspectors should examine steel bridges carefully for their presence.

If intersecting welds are found on a structure, immediately notify the District Bridge Engineer. Bridge closure may need to be considered. F&F inspections should be used to inventory and then monitor them.

Every F&F inspection must include a 'hands-on' inspection of all intersecting weld details in tension or stress reversal zones, including the use of NDT when cracks are noted or suspected. Give priority and emphasis to bridges with Fracture Critical Members (FCMs) with intersecting welds. For instances where holes have been drilled to arrest cracks, inspectors must ensure that the crack is not propagating on the opposite side.

Important points to note about the failure caused by these intersecting welds:

1. The failure mode is brittle fracture which may be sudden and without warning.

2. The failure is not fatigue related and may occur under low stress levels or with very low cumulative truck traffic.

3. The failure is not material dependent and may occur in ductile material with good CVN properties.

4. While the welds may crack, failure of the member in compression zones is unlikely.

5. On a non-redundant FCM, the fracture of the weld may lead to complete structure failure.

Because the failure mode of intersecting welds is sudden and unpredictable nature, repairs and retrofits to cracked and uncracked locations must be given high priority.

The intensity and frequency of these inspections are discussed in Section IP 2.4.6 and IP 2.4.7.

South Carolina

The South Carolina DOT *Bridge Design Manual* (South Carolina Department of Transportation, 2006), Section 16.7.2.4, "Design of Welds," states:

> The following types of welds are prohibited: intersecting welds; intermittent field welds (except for the connection of stop bars at expansion joints), and; partial penetration groove welds (except for the connection of tubular members in hand rails).

Utah

The Utah DOT *Structures Design and Detailing Manual* (Utah Department of Transportation, 2017), Section 15.7.2.4, “Design of Welds,” states: “The following types of welds are prohibited: Field welded girder splices; Intersecting welds; Intermittent fillet welds; partial penetration groove welds (except for the connection of tubular members in hand rails).”

Vermont

The Vermont DOT *Structures Design Manual* (Vermont Department of Transportation, 2010), Section 6.4.3, “Welded Connections,” includes one brief statement suggesting that the use of intersecting welds be avoided.

Wisconsin

The Wisconsin DOT *WisDOT Bridge Manual, Chapter 24 – Steel Girder Superstructures* (Wisconsin Department of Transportation, 2018a), Section 24.11, “Longitudinal Stiffeners,” states:

> It is preferred that longitudinal stiffeners be placed on the opposite side of the web from transverse stiffeners. At bearing stiffeners and connection plates where the longitudinal stiffener and transverse web element must intersect, a decision must be made as to which element to interrupt. According to LRFD [6.10.11.3.1], wherever practical, longitudinal stiffeners are to extend uninterrupted over their specified length, unless otherwise permitted in the contract documents, since longitudinal stiffeners are designed as continuous members to improve the web bend buckling resistance. In such cases, the interrupted transverse elements must be fitted and attached to both sides of the longitudinal stiffener with connections sufficient to develop the flexural and axial resistance of the transverse element. If the longitudinal stiffener is interrupted instead, it should be similarly attached to all transverse elements. All interruptions must be carefully designed with respect to fatigue, especially if the longitudinal stiffener is not attached to the transverse web elements, as a Category E or E′ detail may exist at the termination points of each longitudinal stiffener-to-web weld. Copes should always be provided to avoid intersecting welds.

The Wisconsin DOT *WisDOT Bridge Manual, Chapter 40 – Bridge Rehabilitation* (Wisconsin Department of Transportation, 2021), Section 40.18, “Retrofit of Steel Bridges,” subsection 40.18.2, “Rigid Connections,” states:

> A connection is too rigid when it is fitted into place and allowed to move but the movement can only occur in a refined area which introduces high stresses in the affected area. Examples are welded gusset connection plates for lower lateral bracing that are fitted around transverse or bearing stiffeners.
>
> Other partial constraint details are:

1. Intersecting welds
2. Gap size-allowing local yielding
3. Weld size
4. Partial penetration welds versus fillet welds
5. Touching and intersecting welds

The solution is to create spaces large enough (approximately 1/4” or more) for more material to flex thus reducing the concentration of stress. For gusset connection plates provide a larger gap than 1/4” and no intersecting welds. For existing conditions holes can be drilled at locations of high stress concentrations. For new conditions it would be better to design a rigid connection and attach to the flange rather than the web. For certain situations a fillet weld should be used over a partial penetration weld to allow slight movement.

American Railway Engineering and Maintenance-of-Way Association (AREMA)

The AREMA *Manual for Railway Engineering* (American Railway Engineering and Maintenance of Way Association, 2014), while not addressing highway bridges, does address similar steel bridges for railroad applications. Chapter 15, “Steel Structures,” Subsection 1.10.2, “Prohibited Types of Joints and Welds,” of the *Manual* includes a list of prohibited types of joints and welds which includes “highly constrained joints.” The list is excerpted below:

> Highly constrained joints. Welded connections shall be detailed to avoid welds that intersect or overlap. Welded attachments should be detailed so that the welds parallel to the primary stresses are continuous and the transverse welded connection is discontinuous. If unavoidable, welds in low stress range areas that are interrupted by intersecting members shall be detailed to allow a minimum gap of at least one inch between weld toes and weld terminations and shall be properly designed for the applicable fatigue limit state.

Furthermore, the commentary to Chapter 15, “Steel Structures,” Subsection 9.1.10.2, “Prohibited Types of Joints and Welds” goes into more detail on this topic, stating:

> Because of fatigue considerations, several types of joins and welds are added to types prohibited by AWS D1.5.
>
> g. Highly Constrained Joints:
>
> Welded structures are to be detailed to avoid conditions that create highly constrained joins and crack-like geometric discontinuities that are susceptible to Constraint-Induced-Fracture (CIF). Avoid intersecting welds by using a preferred detail (see Figure 15-9-5) or by using high-strength bolted connections. This article is not intended to apply to the intersection of:
>
> - Welded flange splices with flange-to-web welds.
> - Welded web splices with flange-to-web welds.

- Welded web splices at longitudinal stiffener-to-web welds.

Constraint-Induced-Fracture is a form of brittle fracture that can occur without any perceptible fatigue crack growth and more importantly, without any apparent warning. This type of failure was documented during the Hoan Bridge failure investigation (Reference 179) as well as in other bridges that have exhibited very similar fractures (References 45, 46 & 53). Criteria have been developed to identify and retrofit bridges susceptible to this failure mode (References 45 & 102).

Although it is common to start and stop an attached element parallel to primary stresses (e.g., gusset plate or longitudinal stiffener) when intersecting a full-depth transverse member, the detail is more resistant to fracture and fatigue if the attachment parallel to the primary stresses is continuous and the transverse connection is discontinuous. (See Figure 15-9-5 and Figure 15-9-6).

High-strength bolted connections are not susceptible to Constraint-Induced-Fracture and should be considered where practical and economical.

The commentary also includes figures illustrating examples of details at the intersection of longitudinal attachment and vertical attachments welded to girder webs.

APPENDIX C – TRANSVERSE STIFFENER AND CONNECTION PLATE SEAL WELDING MOCK-UP TRIALS

One of the topics discussed during the consensus meeting (see Section 4.5) was the idea of sealing faying surfaces between the transverse stiffeners/connection plates and the girder webs and flanges by welding all the way around each faying surface. The value of such seal welding primarily focused on corrosion protection; sealing these faying surfaces would reduce opportunities for the initiation of crevice corrosion and accomplishing such sealing by welding was perceived to provide a more durable and reliable seal than other methods such as caulking.

Shortly after the consensus meeting a structural steel bridge fabricator, conducted a number of welding mock-up trials in which they performed welding of transverse stiffener and connection plates to plate girders using a variety of details, focused on sealing the faying surfaces between the transverse stiffeners/connection plates and the girder webs and flanges.

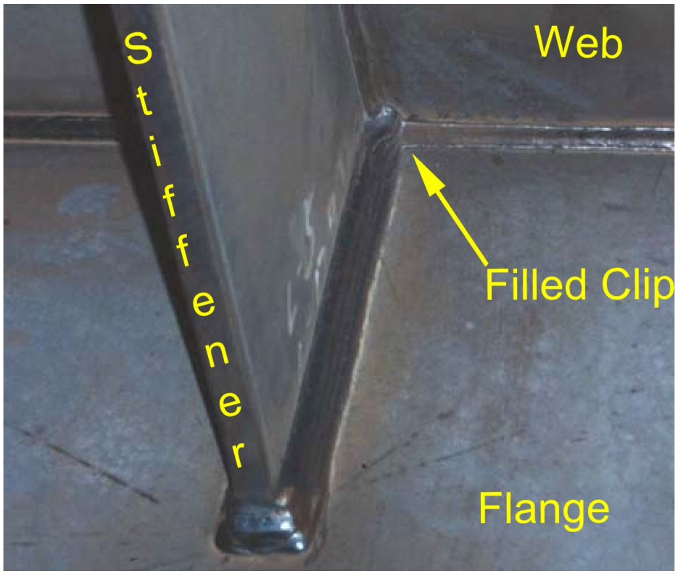
The conclusion of these, albeit limited, trials was that the sealing of stiffeners/transverse connection plates appears to be relatively straightforward from a fabrication standpoint. Section 8.2 discusses investigations that may provide helpful information related to the possible use of these details.

Figure 41 shows the typical detailing currently used when welding a transverse stiffener to a steel plate girder. Similar details are used to attach transverse connection plates and bearing stiffeners to a steel plate girder. Figure 58 shows a photo of a transverse connection plate with similar detailing. A cope is provided in the corner of the connection plate to clear the girder flange-to-web fillet welds. The fillet welds attaching the transverse connection plate to the girder web and flanges are stopped short of the cope; the dimension from the end of the weld to the edge of the stiffener is called the “hold-back” dimension. This type of detailing leaves the faying surfaces between the transverse connection plate and the girder web and flanges unsealed in the unwelded areas. In addition, some fabricators have noted that the practice of various owner-agencies specifying different hold-back dimensions sometimes leads to confusion and fabrication errors.

Source: FHWA

Figure 58. Photo. Transverse stiffener to a steel plate girder.

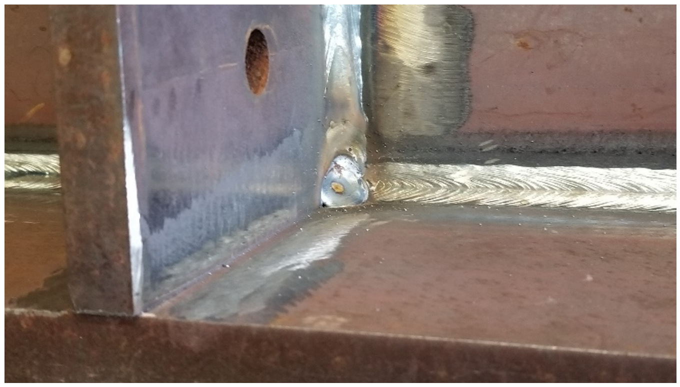
One type of alternate detailing that may merit consideration would be to eliminate the large cope (clip) in the corner of transverse stiffeners and transverse connection plates, weld around the free end of the stiffener and weld "into the corner" (into the point of intersection of the transverse stiffener / connection plate, the girder web, and the girder flange), thus fully sealing the faying surfaces. See Figure 59. This type of detailing might be preferable for painted or metallized steel structures; by eliminating the cope there is no need to paint or metallize the inside of the cope, which is difficult to access.



© 2004, Texas DOT

Figure 59. Photo. Fully wrapped stiffener with no cope or clip (Spadea and Frank, 2004).

The mock-up trial demonstrated that this type of welding is not difficult to accomplish, at least for a stiffener or connection plate oriented approximately perpendicular to the web and flanges. There are some remaining questions about the specific welding procedure and sequence. These questions are mostly related to trying to avoid discontinuities in the welds, which might represent crack-like or notch-like planes of discontinuity and might contribute to an elevated susceptibility to CIF. Figure 60 shows such a detail partially completed, with the stiffener-to-web weld completed but the stiffener-to-flange weld not yet completed. Figure 61 shows the fully completed detail. Both the stiffener-to-web and the stiffener-to-flange weld stop in the corner; at that location there are greater chances for introducing weld discontinuities.



Source: FHWA

Figure 60. Photo. Transverse stiffener-to-web weld continuous into the corner at the intersection of the stiffener, web, and flange.

Source: FHWA

Figure 61. Photo. Transverse stiffener-to-flange weld continuous into the corner at the intersection of the stiffener, web, and flange.

Another type of detailing would be to retain the cope and wrap the welds completely around the transverse stiffener or connection plate. See Figure 62, Figure 63, and Figure 64. This type of detailing might be preferable for hot dip galvanized steel structures; when dipped in the galvanizing bath the molten zinc can flow around the stiffener base and coat the inside surface of the cope. See also Article C6.13.3.7 of the AASHTO BDS (AASHTO, 2017a) (23 CFR 625.4(d)(1)(v) for additional information related to sealing of faying surfaces in galvanized structures.

Source: FHWA

Figure 62. Photo. Transverse stiffener-to-flange weld completely wrapped around the base of a coped stiffener for sealing.

Source: FHWA

Figure 63. Photo. Transverse stiffener-to-flange weld completely wrapped around the base of a coped stiffener for sealing.

Source: FHWA

Figure 64. Photo. Transverse stiffener-to-flange weld completely wrapped around the base of a coped stiffener for sealing.

For either type of detailing, it would be important to limit the width of the transverse stiffener/connection plate, or to clip the outside corners, to avoid undercutting the flange itself when wrapping the weld around the outside edge of the transverse stiffener/connection plate. See Figure 65 for photos of examples of the type of undercutting that can occur when the width of the transverse stiffener/connection plate corresponds to the flange width.

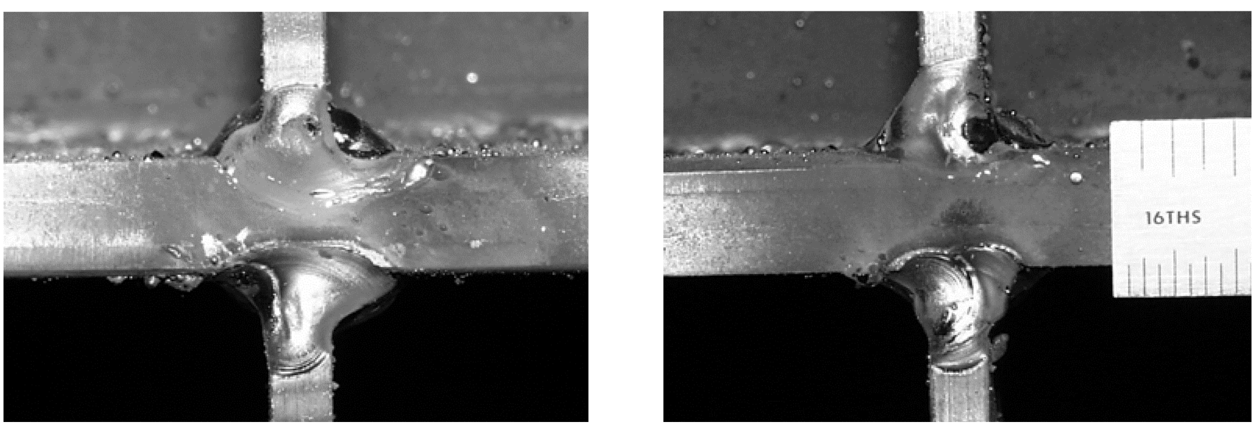


Figure 65. Photo. Example of flange undercuts when the transverse stiffener/connection plate width corresponds to the flange width.

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