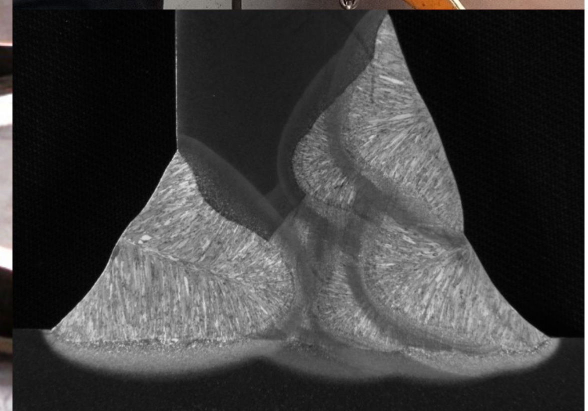

Bridge Welding Reference Manual

Publication No. FHWA-HIF-19-088
Structures

September 2019

U.S. Department of Transportation
Federal Highway Administration

FOREWORD

Welding is a versatile, complex, and essential process that facilitates the use of a broad range of steel assemblies in transportation structures. Use of welding improves design efficiency for all steel structures and modern typical sections such as trapezoidal (tub) girders and long curved I-girders cannot be achieved without welding. Further, welding facilitates rapid and flexible field connections in new construction and retrofits on existing structures. Bridge engineers need to be skilled in welding design and knowledgeable in welding science to effectively and efficiently detail welded connections during bridge design.

The reference manual explains welding specifications governing highway structures, namely the AASHTO/AWS D1.5/D1.5M “Bridge Welding Code” (AASHTO/AWS, 2015) and the AWS D1.1/D1.1M “Structural Welding Code-Steel” (AWS, 2015), and the relationships to structural performance and design intent. The manual provides a detailed overview of welding materials, equipment and processes; includes a discussion of various inspection techniques and methods; addresses the special welding requirements in the fracture control plan (FCP); and lays out the engineer’s role in welded fabrication including the creation of contract documents, approval, and dealing with unexpected circumstances. Additionally, the resource educates the user on fabrication considerations such as accessibility, closed sections, distortion and restraint, the use of partial joint penetration welds and intersecting welds. Finally, the document touches on ancillary issues such as field welding, aluminum, stainless steel, reinforcing steel, welding of coated members and upcoming welding innovations. This reference manual will benefit DOT employees, their representatives, and consultants involved with the welding and fabrication of steel highway structures. This includes designers, fabricators, field/construction/materials engineers and other paraprofessionals associated with structural design, material and procedure specifications, drawing approval, welding, construction, inspection, and repair.

Joseph L. Hartmann, PhD, P.E.
Director, Office of Bridges and Structures
Office of Infrastructure
Federal Highway Administration

Cover Photos: Source: FHWA

TECHNICAL REPORT DOCUMENTATION PAGE

<table>
<tr><td>1. Report No.
FHWA-HIF-19-088</td><td>2. Government Accession No.</td><td>3. Recipient’s Catalog No.</td></tr>
<tr><td colspan="2" rowspan="2">4. Title and Subtitle
Bridge Welding Reference Manual</td><td>5. Report Date
August, 2019</td></tr>
<tr><td>6. Performing Organization Code:</td></tr>
<tr><td colspan="2">7. Author(s)
Ronnie Medlock, P.E. (Medlock, LLC); Heather Gilmer, P.E. (HRV, Inc.); Duane Miller, ScD, P.E. (Lincoln Electric); and Anthony Ream, P.E. (HDR)</td><td>8. Performing Organization Report No.</td></tr>
<tr><td colspan="2" rowspan="2">9. Performing Organization Name and Address
HDR
Suite 800, 11 Stanwix St.
Pittsburgh, PA 15222</td><td>10. Work Unit No.</td></tr>
<tr><td>11. Contract or Grant No.
DTFH6114D00049 T0002</td></tr>
<tr><td colspan="2" rowspan="2">12. Sponsoring Agency Name and Address
Office of Research, Development, and Technology
Federal Highway Administration
6300 Georgetown Pike
McLean, VA 22101-2296</td><td>13. Type of Report and Period
Final Report
August 2017 to August 2019</td></tr>
<tr><td>14. Sponsoring Agency Code</td></tr>
<tr><td colspan="3">15. Supplementary Notes
FHWA Task Order COR (Task Order Manager): Justin Ocel, Ph.D., P.E.
FHWA Contracting Officer Representative (COR): William Bergeson, P.E.
HDR Project Manager: Anthony Ream, P.E.

The subject matter expert technical reviewers for this guide included Mary Grieco (Massachusetts Department of Transportation) and Todd Niemann (formerly of the Minnesota Department of Transportation). Their contributions during the preparation of this guide are greatly appreciated.</td></tr>
<tr><td colspan="3">16. Abstract
The objective of this manual is to produce a comprehensive reference that covers the technical aspects of welding of highway structures with an emphasis on steel highway bridges. The target audience for the reference manual is the DOT employees and their representatives and consultants involved with the welding and fabrication of steel highway structures.

The reference manual explains welding specifications governing highway structures, namely the AASHTO/AWS D1.5/D1.5M “Bridge Welding Code” (AASHTO/AWS, 2015) and the AWS D1.1/D1.1M “Structural Welding Code-Steel” (AWS, 2015), and the relationships to structural performance and design intent. The manual provides a detailed overview of welding materials, equipment and processes; includes a discussion of various inspection techniques and methods; addresses the special welding requirements in the fracture control plan (FCP); and lays out the engineer’s role in welded fabrication including the creation of contract documents, approval, and dealing with unexpected circumstances. Additionally, the resource educates the user on fabrication considerations such as accessibility, closed sections, distortion and restraint, the use of PJP and intersecting welds. Finally, the document touches on ancillary issues such as field welding, aluminum, stainless steel, reinforcing steel, welding of coated members and upcoming welding innovations.</td></tr>
<tr><td colspan="2">17. Key Words
Bridges, Steel Bridges, Welding, AWS, D1.5, NDE, WPS, PQR</td><td>18. Distribution Statement
No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161.
http://www.ntis.gov</td></tr>
</table>

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	346	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized.

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

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LIST OF ABBREVIATIONS AND SYMBOLS

AASHTO	American Association of State Highway Transportation Officials
AC	alternating current
AISC	American Institute of Steel Construction
AISI	American Iron and Steel Institute
AREMA	American Railway Engineering and Maintenance-of-Way Association
ASNT	American Society of Nondestructive Testing
ASTM	ASTM International
AWS	American Welding Society
BDS	AASHTO LRFD Bridge Design Specifications
Bridge Welding Code	AASHTO/AWS D1.5 Bridge Welding Code
CAWI	Certified Associate Welding Instructor
CC	constant current
C.E.	carbon equivalency
CJP	complete joint penetration
CSA	Canadian Standards Association
CTWD	contact tip to work distance
CV	constant voltage
CVN	charpy v-notch
CWB	Canadian Welding Bureau
CWI	Certified Welding Inspector
D1.5	AASHTO/AWS D1.5 Bridge Welding Code
dB	decibel
DC	direct current
EGW	electrogas welding
ESW	electroslag welding
ESW-NG	narrow-gap electroslag welding
FC	fracture-critical
FCAW	flux-cored arc welding
FCM	fracture-critical member
FCP	fracture control plan
FCW	fracture-critical weld
FHWA	Federal Highway Administration
FMC	full matrix capture

FWST	fillet weld soundness test
GMAW	gas metal arc welding
GTAW	gas tungsten arc welding
HAZ	heat-affected zone
HLAW	hybrid laser arc welding
HPS	high-performance steel
IIW	International Institute of Welding
IQI	image quality indicator
LOF	lack-of-fusion
MT	magnetic particle testing
NDE	nondestructive examination
NDT	nondestructive testing
NSBA	National Steel Bridge Alliance
PAUT	phased array ultrasonic testing
PJP	partial joint penetration
PQR	procedure qualification record
PT	dye penetrant testing
PWHT	post-weld heat treatment
RFI	request for information
RT	radiographic testing
SAW	submerged arc welding
SCWI	Senior Certified Welding Instructor
SMAW	shielded metal arc welding
SSPC	Society for Protective Coatings
SW	stud welding
TFM	total focus method
UT	ultrasonic testing
WPS	welding procedure specification

CHAPTER 1 - INTRODUCTION

1.1. BACKGROUND

This manual provides bridge welding guidance based on knowledge gleaned since the 1950s about practices that result in cost-effective and long-performing welded bridges. Bridge construction experience over this period has demonstrated that welding provides extraordinary versatility in bridge designs. Further, the service history of bridges over the past half century demonstrates that properly designed and produced welds consistently perform effectively for the life of the structure.

The manual is intended to support the work of the broad variety of transportation engineers who encounter welding in bridge work, from design through fabrication and inspection. Engineers who design steel bridges need welding knowledge so that their bridge designs can be readily welded, thus achieving constructability that allows bridges to be produced cost-effectively and on schedule. This manual addresses such welding fabrication and constructability knowledge as it applies to design, but it does not address design of welded connections and fatigue design. Engineers who provide fabrication and construction oversight, develop specifications, and direct the work of inspectors need to understand what aspects of welding must be controlled to ensure quality and associated long service life. Further, as welding technology advances, engineers need to understand welding so that they can help facilitate advancement of the state of the art, particularly when called upon to approve the use of innovations.

The manual is closely aligned with the AASHTO/AWS D1.5 *Bridge Welding Code* (AASHTO/AWS, 2015), which reflects the best practices in welded steel bridge construction. The Code was first published in 1988 and since then has been managed by a committee with balanced owner and industry representatives who keep the Code up to date with the state of the practice in bridge welding. Engineers who rely on this standard today can be confident that bridges welded in conformance with the Bridge Welding Code will incorporate welds with excellent workmanship and quality. In this manual, references to specific parts of the Code are shown as (clause x), where "x" represents the code clause number; statements referring to "section x" represent the numerical section of this manual where "x" is a section number. Additionally, reference to figures and tables in the Bridge Welding Code are preceded with "D1.5" to differentiate them from figures and tables occurring within this manual. Any reference to specific clause, table, or figures in the Code are based on the 2015 7th edition: D1.5:2015-AMD1.

1.2. IMPORTANCE OF WELDING IN BRIDGES

The introduction of welding revolutionized steel bridges, and its importance to steel bridge construction is paramount. Prior to the introduction of welding, steel bridges were built up from plates and shapes and joined with rivets. Web-to-flange connections were made with riveted angles, which meant that built-up steel members were much heavier; individual components were relatively short; assemblies could only be straight; and, because they were joined with angles, components needed to be square to each other.

Welding provides an effective means of joining plates and shapes, thick and thin, at any angle and along any curve and thus readily and cost-effectively allows more complex geometries (see figure 1), such as:

- Vertically and horizontally curved girders
- Haunched girders
- Bent flanges, such as for dapped-end girders and bull-nose railroad girders
- Welded box elements such as for pier caps and for arch and truss chords
- Trapezoidal box ("tub") girders

Further, welding provides options in the field, facilitating connections in new bridges and repairs and retrofits in existing structures. Hence, given the design flexibility it offers and its versatility in both the shop and the field, welding has become an innovative tool in the bridge engineer's toolbox.

Source: FHWA

Figure 1. Photos. Bridges and elements facilitated by welding.

1.3. WELDING

The American Welding Society (AWS) provides this definition of welding in AWS 3.0M/A3.0: "*A localized coalescence of metals or nonmetals produced by heating the materials to the welding temperature, with or without the application of pressure, or by the application of pressure along and with or without the use of filler material*" (AWS, 2010c). Arc welding is the most common type of welding used for structural steel, although other methods are also used.

Achieving a good bridge weld, which is fatigue-resistant and of sufficient strength for the life of the structure, depends on a number of factors:

- Use of the correct welding materials to attain proper weld metal strength, ductility, and toughness
- Sufficient cross-section and associated effective throat or weld size, as designed and deposited
- Welding technique that results in good soundness (fusion as required and minimal defects)
- Design details chosen and properly executed for good fatigue and fracture behavior

Good welds are a function of both design and production. This manual helps engineers to understand the latter, addressing fabrication practices that best achieve strong and fatigue-resistant welds and providing engineers with confidence in welded bridge construction.

1.4. APPLICABLE CODES AND SPECIFICATIONS

1.4.1. Welding Codes

Bridge welding is performed in accordance with the welding specifications that are stipulated in project contracts by owners. For bridge welding, most owners specify the AASHTO/AWS D1.5 Bridge Welding Code (hereafter referred to as "the Code"), which is part of the AWS D1 family of structural welding codes. Not all transportation structure welding is within the scope of the Code; selection of welding codes is discussed in section 10.1.

1.4.1.1. The Bridge Welding Code

Prior to the publication of the Bridge Welding Code, steel bridges were welded to a broad variety of owner specifications and requirements, resulting in inefficiency, inconsistency, and sometimes confusion on the shop floor. This eventually led state materials and bridge engineers in the United States to recognize that steel bridge fabrication would be more productive if bridge fabricators worked to one standard that provided a common set of welding and associated fabrication rules. Thus, the American Association of State Highway and Transportation Officials (AASHTO) Subcommittee on Bridges and Structures (now the Committee on Bridges and Structures) and AWS formed a partnership to develop a state-of-the-art bridge welding standard that any owner could choose to adopt as a bridge welding specification. It should be noted that the Code of Federal Regulations, 23 CFR § 652.4, "Standard, Policies, and Standard Specifications," includes the interim revisions to the 2015 7th edition as part of the documents incorporated by reference. However, these interim revisions are only published by AASHTO and not by the joint AASHTO/AWS partnership. As a result, the interims are generally not binding unless specifically required by the owner in the contract documents. Therefore, the focus of this manual is the AASHTO/AWS 7th edition.

1.4.1.2. AWS Structural Welding Standards

The AWS structural welding codes fall under the purview of AWS Committee D1, the Committee on Structural Welding. The D1 Committee is composed of volunteers who develop and modify language for the standards. Volunteers represent various stakeholders, including owners, designers, suppliers, fabricators, inspection agencies, and erectors. The Bridge Welding Code is unique among the nine D1 welding standards in that the D1 subcommittee responsible for D1.5 is a joint committee with AASHTO, and the Bridge Welding Code is a joint publication of AASHTO and AWS. Therefore, in addition to the AWS approval and ANSI oversight practice common to the other D1 codes, the Bridge Welding Code also goes through AASHTO balloting and approval by the AASHTO Committee on Bridges and Structures. In practice, changes are balloted to AASHTO after the ballots have been approved by the Bridge Welding Subcommittee and the D1 Structural Welding Committee.

There are nine AWS structural welding standards in the D1 family (see figure 2)—these address structural welding of steel, stainless steel, aluminum, and titanium, with special focus standards on sheet steel, reinforcing steel, bridges, seismic applications, and strengthening and repair. The Bridge Welding Code is part of this family. Highway projects on the National Highway System are required to adhere to minimum design standards in the Code of Federal Regulations, Title 23, Part 625.4 (U.S. Government, 2015), and some of these AWS codes are included by reference.

Figure 2. Photo. D1 family of books.

The move to the Bridge Welding Code reduced variations in requirements and improved standardization in the shop, resulting in improvements in key areas:

- Improved efficiency from use of the same practices and equipment from owner to owner
- Improved quality from increased standardization and reduced variation and also from reduced confusion caused by disparate requirements on the shop floor
- Improved schedule through avoidance of redundant practices, such as requalification of welding procedures from owner to owner

All of these factors combine to improve delivery and provide owners with better quality and reduced cost in steel bridge construction.

1.4.1.3. Support for the Engineer in the use of AWS Welding Codes

AWS may be queried with questions regarding its various D1 standards. Queries may be unofficial or an official request for interpretation. The difference between the two is that unofficial inquiries are answered by AWS staff, who may consult with committee leaders or other members, and the answer only reflects their opinion. The AWS staff members emphasize this point in their responses. By contrast, official inquiries are answered by the D1 committee and applicable subcommittee through a process that is similar to the balloting process used to change the standards, and the answers are published on the AWS website as official interpretations. Published interpretations carry more weight than unofficial inquiries. However, because they are answered by a balloting process, it generally takes much longer, potentially over a year, to get a published interpretation to an official inquiry than it takes to get a response to an unofficial inquiry. Instructions about how to request an official interpretation are found in the Code.

1.4.1.4. AWS A5 Standards

The AWS A5 standards are the documents that AWS uses to standardize requirements for various welding consumables. Under the A5 standards, consumables are categorized by process and usage. For example, there are four standards for SMAW electrodes: A5.1 for SMAW electrodes for carbon steel (AWS, 2012), A5.3 for SMAW electrodes for aluminum (AWS, 1999), A5.4 for SMAW electrodes for stainless steel (AWS, 2012a), and A5.5 for SMAW electrodes for low alloy steels (AWS, 2014a). These facilitate the specification of certain consumables for specific applications; of these, for example, the Code references A5.1 and A5.5.

1.4.2. Material Specifications

The Bridge Welding Code is intended for use with specific steels with properties that make them suitable for bridge service. As currently required by the Bridge Welding Code, these are the steels of the ASTM standard A709 (ASTM, 2018a). The A709 specification is a compilation of steels with a history of welded use in bridges plus newer, more recently developed grades of steels appropriate for use in bridges. Table 1 provides a list and description of these steels including other similar ASTM grades (where some of the A709 grades originated).

AASHTO M 270 is the AASHTO specification for steel bridge materials. Like other AASHTO material specification, M 270 is published and maintained by the AASHTO Subcommittee on Materials. The Subcommittee attempts to keep M 270 aligned with ASTM A709, following ASTM's lead when changes are made to keep them the same.

The first structural steels published by ASTM were A7 for bridges and A9 for buildings; these were later combined into A7. These grades were not developed and published as weldable steels, but they are encountered in older structures and generally are weldable, depending upon their actual alloy chemistry. In 1954, A373 was specifically published as a weldable steel; it was soon superseded by A440, A36, and A441, published in 1959, 1960 and 1960, respectively.

From the late 1960s through the mid-1990s, ASTM A36 and A572 were the materials of choice for structural shapes. As time passed, mill processing improved and yield strengths rose such that most of the structural shapes on the market actually have yield strengths over 50 ksi, even when ordered to A36. This led to the unofficial designation "A36 Modified", which was A36 with a minimum specified yield strength of 50 ksi. Later, this led to development of A992, which provides the same strength as A572 but with a limit on the ratio of yield to tensile strength and chemical limits that are useful in certain structural design applications (ASTM, 2015b). Most non-weathering structural sections are now produced to A992.

ASTM A572 is a standard for weldable plates and shapes at higher minimum specified yield strengths (ASTM, 2015); ASTM A588 does the same and adds weathering properties (ASTM, 2015a). Other higher-strength quenched and tempered steels became available and were sometimes used in the early and mid-part of the 20th century (ASTM, 1939 and ASTM, 1966). However, by the 1970s, virtually all bridges were fabricated with A572 and A588. In 1974, A709 was created to be a collection of bridge steels with Charpy V-notch (CVN) testing for tension members, and now includes the grades shown in table 1. As shown in the table, some of these grades came from other equivalent ASTM specifications that were prevalent before A709 was published.

Table 1. Bridge structural steels.

ASTM A709 AASHTO M 270 Grade	Equivalent Alternate ASTM Standard (as applicable)	Minimum Specified Yield Strength and Key Distinctions
36	A36	36 ksi
50	A572 Gr. 50	50 ksi;
50W	A588	50 ksi; weathering
50S	A992	50 ksi; structural steel for shapes
HPS 50W		50 ksi; weathering; enhanced toughness
HPS 70W		70 ksi; weathering; enhanced toughness
HPS 100W		100 ksi; weathering; enhanced toughness
50CR	A1010	50 ksi; weathering; 12 percent chromium stainless steel well suited to bridges; new to A709 but not in D1.5 or M 270 at the time of this publication.
QST 65	A913 Gr. 65	65 ksi; quenched and self-tempered structural steel for shapes; new to A709 but not in D1.5 or M 270 at the time of this publication.
QST 70	A913 Gr. 70	70 ksi; quenched and self-tempered structural steel for shapes; new to A709 but not in D1.5 or M 270 at the time of this publication.

ASTM A709 (ASTM, 2018a) lists CVN requirements for fracture critical (FC) and non-FC materials (see section 7.3 for FC materials) in ASTM A709 tables 12 and 11, respectively. It uses "T" and "F" designators to represent non-FC and FC grades—for example, 36T for non-FC grade 36 to be used in tension and 36F for FC grade 36. Fabricators use these designators when ordering material, as well as a number, 1, 2, or 3, to indicate the AASHTO temperature 3 zone, e.g., 50WF2 for grade 50W FC material in zone 2. If the fabricator orders material without a "T" or "F" and number, e.g., "grade 50W", then the fabricator is ordering material without CVN requirements. Any reference in this manual to specific paragraphs or tables in ASTM A709 are to the 2018 edition. Because the requirements are covered in ASTM A709, it is not necessary to list specific service temperature ranges, test temperature, or absorbed energy requirements (ft-lbs) on the plans as long as the proper designations described above are used.

The CVN test temperatures in ASTM A709 are higher than the lowest anticipated service temperatures for each zone. For more information about this temperature shift, see Barsom and Rolfe, 1999.

Prior to 2005, CVN requirements were mandatory supplementary tables in ASTM A709. As with the present format, fabricators then had to specify to their suppliers which materials were to be FC and the associated service zones. This "mandatory supplementary" approach was confusing, so ASTM moved the ordering requirements from supplementary requirements to the body of the specification. Therefore, no supplemental requirements need to be specified for CVN requirements.

High-performance steel (HPS) was developed in the 1990s in a cooperative research program among the Federal Highway Administration (FHWA), the US Navy, and the American Iron and Steel Institute (AISI). The goal was to produce an improved bridge steel that offered higher strength, enhanced weathering, improved toughness, and improved weldability over similar strength steels available at the time. ASTM A709 Grade HPS 70W is now a mainstream steel. In practice, owners and designers have taken most advantage of its high strength. This means that it offers the advantage of smaller cross-sections in flanges, where strength often controls design, but the higher strength does not offer advantages in webs, where stiffness controls. Hence, hybrid designs of 50-ksi webs and Grade HPS 70W flanges or mixed 50-ksi and Grade HPS 70W flanges are common. When HPS was first made available, it was not yet in the Bridge Welding Code, so to facilitate its use the FHWA, AISI, and the US Navy jointly published guides that owners used in their contracts to govern welding of HPS (AISI, 2003). When HPS was adopted into the Bridge Welding Code in 2008, the HPS guide was retired and should no longer be specified.

1.4.3. Fabrication Specifications

Bridge owners typically have a "steel structures" or "steel bridges" item in their standard construction specifications that calls out the Bridge Welding Code for welding and associated fabrication aspects and then either directly addresses other aspects of fabrication, or references other codes, as applicable. The "steel structures" items are similar from state to state because they were based on the same original AASHTO guide specifications. Since that time, most owners have adapted their specifications based on experiences in their jurisdictions and such adaptations have led to a variety of requirements for fabrication.

Just as the Bridge Welding Code standardizes requirements under its purview, a standard for other aspects of fabrication would provide a similar benefit. Fabrication requirements are included in the AASHTO Bridge Construction Specifications (AASHTO, 2017). This specification is available for adoption, but only four state DOTs do so; others use it for reference. In 1997, the AASHTO/National Steel Bridge Alliance (NSBA) Steel Bridge Collaboration was established to standardize steel bridge design, fabrication, and construction requirements. Among other efforts, the Collaboration developed and published a new steel bridge fabrication specification, S2.1, *Steel Bridge Fabrication Guide Specifications* (AASHTO/NSBA Steel Bridge Collaboration, 2018). This standard has grown to become the most up-to-date fabrication specification available, reflecting decades of experience and the state of the art in fabrication processes. However, as with the AASHTO Bridge Construction Specifications, few owners have adopted it directly and more owners use it for reference.

CHAPTER 2 - MATERIALS, EQUIPMENT AND PROCESSES

A variety of materials and welding processes are used to fabricate modern steel bridges. This section first describes the basics of welding, with an emphasis on arc welding because this type of welding is most common in steel bridge fabrication. Following the basics, the common welding processes and associated equipment used in steel bridge fabrication and listed in the Bridge Welding Code are described in detail. When properly applied, all of these processes provide good quality welds. Fabricators and erectors choose among these available processes for various applications depending upon their preferences and associated factors, such as their equipment and the skill of their workforces.

2.1. ACHIEVING FUSION

The action of welding is melting components at a common surface and allowing them to fuse at their fusion surfaces. Understanding this fusion begins with an understanding of metal structure and behavior at the atomic level where fusion occurs.

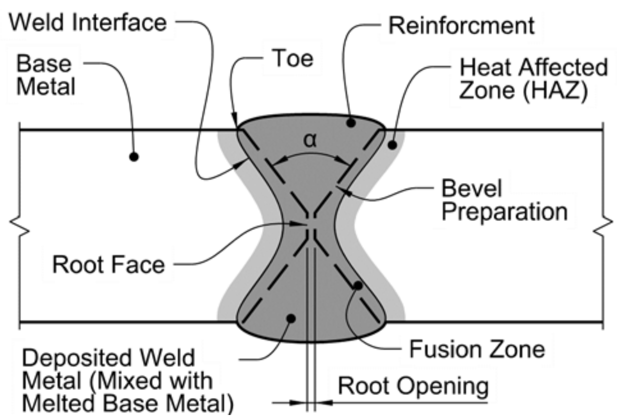
A single piece of metal is composed of millions of individual atoms. Each atom is connected to surrounding atoms, resulting in what is considered a single solid piece of metal. In the simplest terms, fusing two separate pieces of pure metal together requires forming the same bonds between the two pieces as exist within the individual pieces. In metals, these metallic bonds form when individual metallic atoms are brought close enough together that they share a common "cloud" of electrons. The required closeness of the two atoms is close indeed, measured in units of angstroms (10^{-10} m). When metallic atoms are brought into close contact, metallurgical bonds form. Thus, the first requirement for fusion is atomic closeness.

The previous paragraph emphasizes "metallic atoms" to differentiate them from oxidized metals. When pure (non-oxidized) metallic surfaces are exposed to the atmosphere, such surfaces readily oxidize, and the forces that would normally encourage the metallic atoms to bond to each other are neutralized. Instead of bonding to another metallic atom, bonding is made with an oxygen atom. To achieve fusion, the atoms must just be metallic (i.e., not combined with oxygen), leading to the second requirement for fusion: atomic cleanliness.

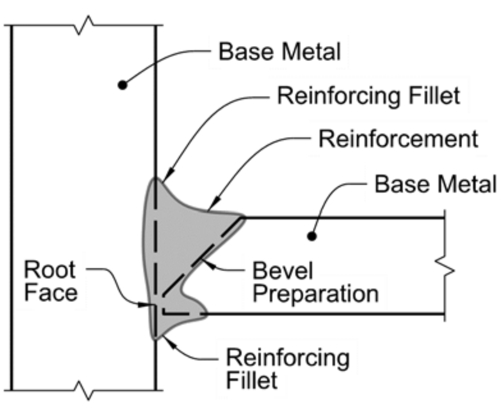
In theory, if two pieces of steel with oxide-free surfaces are brought into close contact, the attractive forces between the two pieces should cause the two to bond to each other. In practice, this does not happen because, on an atomic level, the atoms are separated by significant distances, restricting the first requirement for fusion: atomic closeness. Furthermore, surfaces will remain oxide-free for only short periods of time, and once such oxides form, the second requirement for fusion, atomic cleanliness, is violated. Thus, welding processes rely on a variety of methods of oxide removal and protection of the surfaces before welding. In theory, any metals can be fused given the right resources and processes. For the bridge engineer, the practical question of welding is, can the material in question be readily fused given the welding processes and equipment commonly used in bridge construction? While the paragraphs above describe the science of fusion for welding, for most bridge engineers, good welding practice is a matter of combining known weldable base metals with welding consumables (electrodes and fluxes) designed for use with those materials, and this in turn is readily achieved by following the Bridge Welding Code.

2.2. WELD ANATOMY

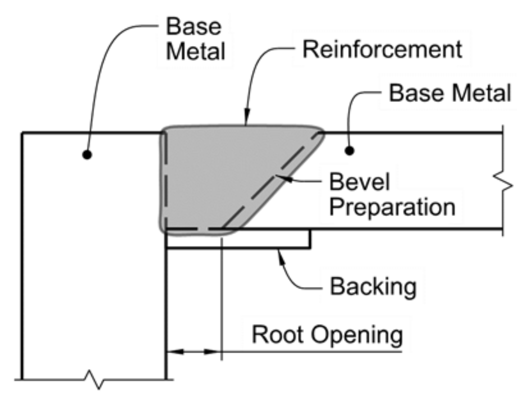
Described here are the basic parts of typical steel bridge welds. Figures 3, 4, and 5 describe key components of welds that are discussed throughout this manual. In the figures, the dotted lines represent the steel surfaces prior to welding; after welding and associated fusion, these surfaces no longer exist. The definitions given in quotation marks below are from AWS A3.0M/A3.0 (AWS, 2010c); additional explanatory notes are also provided. Many of these terms apply to other weld and joint types as well. AWS 3.0 makes the distinction between standard terms, which are preferred terminology, and nonstandard terms, which are used colloquially but are not preferred.



A. The basic parts of a typical welded butt joint.



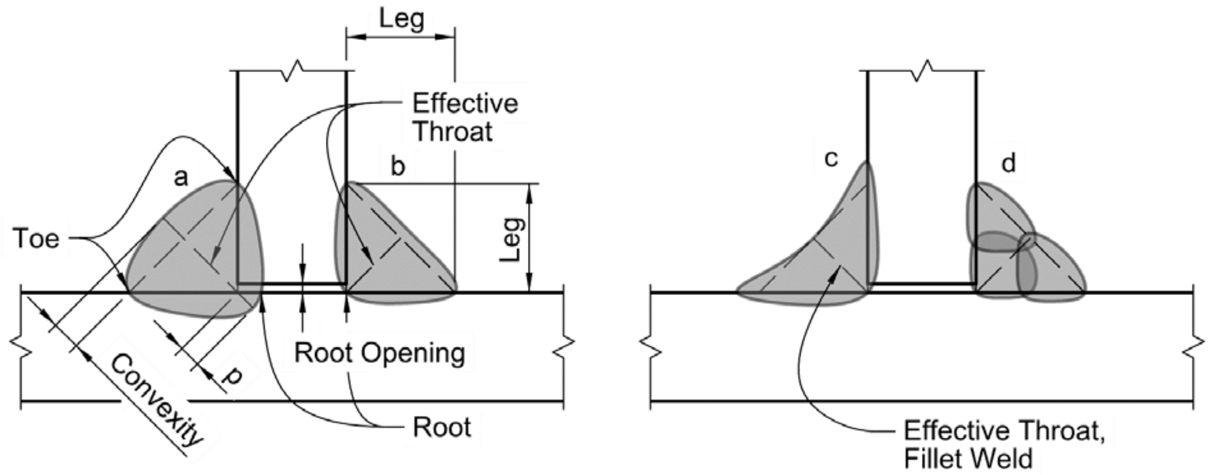
B. The basic parts of a typical welded T-joint.



C. The basic parts of a typical welded corner joint with backing.

Source: FHWA

Figure 3. Illustrations. Typical butt splice, T-joint and corner joint details.

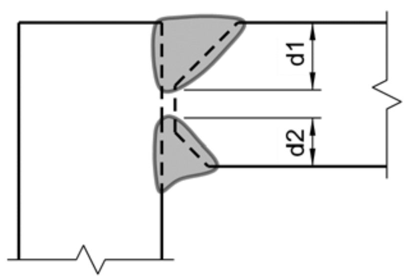


Source: FHWA

Figure 4. Illustration. The basic parts of a fillet weld

Figure 4 illustrates the effective throat of fillet welds. AWS defines effective throat as “the minimum distance from the fillet weld face, minus any convexity, and the weld root.” Hence, the convexity in “a”, “b”, and “d” do not contribute to the length of the effective throat. For the concave fillet weld in “c”, the minimum distance is measured to the root from the fillet weld surface. Figure 4 also illustrates penetration in fillet welds. Fillet weld “b” penetrates right to the root; the fillet weld “a” penetrates beyond the root by *p* amount.

Figure 5 illustrates groove weld size for partial joint penetration groove welds. The groove weld size of the partial joint penetration weld in figure 5 is the sum of the two joint penetrations, *d1* + *d2*.



Source: FHWA

Figure 5. Illustration. Example partial joint penetration weld in a corner joint.

Base Metal - “The material being welded, brazed, or cut.”

Bead - A weld bead, defined as “a weld resulting from a weld pass.” For example, fillet weld “d” shown in figure 4 above was made with three passes and has three beads.

Bevel - “An angular edge shape.” Such shapes are introduced in a joint in preparation for welding. The dotted lines in figures 3-A through 3-C and 5 indicate how plates in this cross-section are prepared for welding.

Complete joint penetration (CJP) weld - "A groove weld in which weld metal extends through the joint thickness." The welds shown in figures 3-A through 3-C are CJP groove welds.

Effective throat - "The minimum distance from the fillet weld face, minus any convexity, and the weld root." As written, this definition describes the effective throat of a fillet weld, as shown in figure 4. The effective throat of a fillet weld represents the cross-section that is used to calculate the effective cross-section of the fillet weld for carrying load.

Fusion zone - "The area of base metal melted as determined on the cross-section of a weld." In figure 3-A, the dark grey area outside the original groove face (the dotted lines), where the base metal has melted and then re-solidified in combination with the deposited weld metal, or "fused", is the fusion zone.

Groove Weld Size – "The joint penetration of a groove weld". The use of "size" in terms of groove welds is similar to the use of "effective throat" for fillet welds. Groove weld size is defined only in terms of joint penetration; it is independent of any weld reinforcing present. As shown in figure 3-A, in a CJP groove weld, the weld size is the same as the thickness of the component being joined because the entire thickness is being joined, In figure 5, the size of this partial joint penetration weld (PJP) is the combination of the two parts of the weld shown, *d1* and *d2*, and in each part, the size is the minimum distance from the groove weld face, minus the reinforcement, to the weld root. Groove weld size is also referred to as the groove weld effective throat; however, according to AWS, such use is non-standard.

Heat-Affected Zone (HAZ) - "The portion of base metal whose mechanical properties or microstructure have been altered by the heat of welding, brazing, soldering, or thermal cutting." In figure 3-A, the HAZ is shown as slightly darker than the base metal; in polished cross-sections of weld (such as macroetches, which are discussed later), the HAZ typically shows up this way. Although welding alters heat-affected zones, experience demonstrates that heat-affected zones created when welding under the Bridge Welding Code provide reliable performance in service.

Multipass weld - "A fusion weld produced by more than one progression of the arc, flame, or energy source along the joint." Most fillet welds are made in a single pass. The size of weld that can be made in a single pass depends upon the process and equipment being used and on the position of the work. For larger fillet welds (perhaps over $^{5}/_{16}$ inch), fabricators may choose to make the fillet weld with more than one pass. For example, fillet weld "d" shown in figure 4 was made with three passes. Such fillet welds are known as "multipass" fillet welds. Virtually all bridge groove welds, as in figures 3-A through 3-C and 5, are made with more than one welding pass and, if so, are also multipass welds. However, as applied to welding practices and discussed later in this manual, the distinction between single-pass and multipass fillet welds is of some importance, which is why this distinction is illustrated in a fillet weld example.

Partial joint penetration (PJP) weld - "A groove weld in which incomplete joint penetration exists." The weld in figure 5 is a PJP weld because the weld only partially penetrates the cross-section.

Penetration - "A nonstandard term when used for depth of fusion, joint penetration, or root penetration." Figures 3-A through 3-C and 5 describe joint penetration in groove welds, and figure 4 demonstrates root penetration in fillet welds. The fillet weld “b” on the right penetrates just to the root; the fillet weld “a” on the left penetrates the root beyond the fillet weld by *p*. The amount of penetration is often referred to as the “depth of penetration”.

Reinforcement (weld reinforcement) - “Weld metal in excess of the quantity required to fill a weld groove.” Examples of reinforcement are shown in figure 3-B, where the deposited weld metal extends somewhat above the top plane of the two plates on top and also extends below the bottom plane of the plates on the bottom. This term is a misnomer: this metal does not reinforce the weld; in fact it is superfluous. However, The Bridge Welding Code requires a reinforcing fillet on the inside corner of T- and corner welds; these are shown in the T- and corner examples in figures 3-B and 3-C. These fillet welds are intended to provide a smooth transition through the corner to facilitate improved fatigue performance.

Root face - “The portion of the groove face within the joint root.” In figure 3-A, the small flat plane in the center of the joint, as prepared, is the root face; in the shop this is more commonly known as the land, landing, or stand.

Root opening - “A separation in the joint root between the workpieces.” The gap between the two root faces shown in figures 3-A and 4 is the root opening. A commonly used non-standard term for root opening is “root gap”.

Weld interface - “The boundary between weld metal and base metal in a fusion weld, between base metals in a solid-state weld without filler metal, or between filler metal and base metal in a solid-state weld with filler metal.” Figure 3-A shows the interface as the point where weld metal meets base metal.

Weld pass - “A single progression of welding along a joint. The result of a weld pass is a weld bead or layer.”

Weld root - “The points, shown in cross-section, at which the weld metal intersects the base metal and extends furthest into the weld joint.” In figures 4 and 5, the root is distinct because the location where the weld metal intersects the base metal is obvious. In figure 3-A, the root is more of a “root area” where the root face is shown.

2.3. WELDING POSITIONS

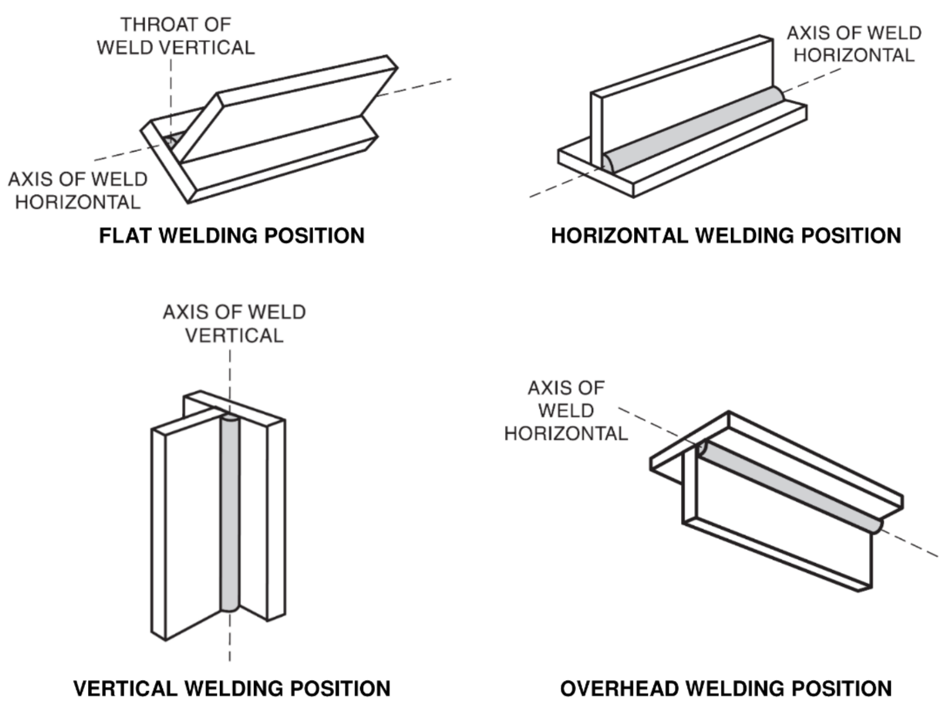
AWS nomenclature regarding welding positions used in WPSs and PQRs is explained here to facilitate the understanding of welding practice and procedures under the Bridge Welding Code.

The effort and technique needed to achieve welds effectively is highly dependent upon the position of welding. As welding progresses, gravity pulls on the molten weld puddle, and this pull has a different impact depending upon the welding position. These positions are divided into four categories: flat, horizontal, vertical, and overhead.

Gravity has a much more significant effect on vertical and overhead welding than on flat and horizontal welding, so welding in the vertical and overhead positions is sometimes called welding “out of position”, whereas welding in the flat and horizontal positions is called welding “in position”.

AWS A3.0 defines these positions (AWS, 2010c); in addition, AWS has diagrams that define the positions in terms of the angle of inclination of the weld axis and the angle of rotation of the weld itself and figures that illustrate the test plate positions. These figures are adopted by the Bridge Welding Code in D1.5 figures 5.4 through 5.7. Notably, there is often confusion between "flat" and "horizontal"; the distinction is based on the position of the weld puddle. See figure 6 for illustrations of the welding positions discussed below.

- **Flat** - The weld puddle lies flat in a roughly horizontal plane and the weld axis is roughly horizontal; when the weld is molten, gravity does not pull the molten weld to either side. In bridges, most groove welds, such as flange and web butt joints, are made in the flat position.
- **Horizontal -** Part of the weld fuses to a roughly vertical surface, and part of it fuses to a roughly horizontal surface, with welding performed from above the weld; when the weld is molten, gravity will tend to pull the molten weld away from the vertical face and towards the horizontal face. In bridges, most fillet welds are made in the horizontal position.
- **Vertical -** The weld axis is roughly vertical. Vertical welding is usually performed with an upward progression using a weave pattern to facilitate proper solidification as gravity pulls on the weld puddle.
- **Overhead** - Welding is performed from the underside of the joint. Gravity pulls on the weld puddle in the direction away from the joint, limiting pass sizes.



Original figure: © 2015 AWS

Figure 6. Illustration. Examples of flat, horizontal, vertical and overhead welding positions based on D1.5 figure 5.7 of the Bridge Welding Code and modified by the authors (AASHTO/AWS, 2015).

The standard joints are position-specific. Allowed positions vary by process and joint among flat, horizontal, vertical, and overhead positions. For example, standard joints for SAW are only listed for groove welding in the flat position. SAW is sometimes used in horizontal groove welds; in these cases, the joint is a nonstandard joint that must be qualified (section 4.4.4).

2.4. ARC WELDING

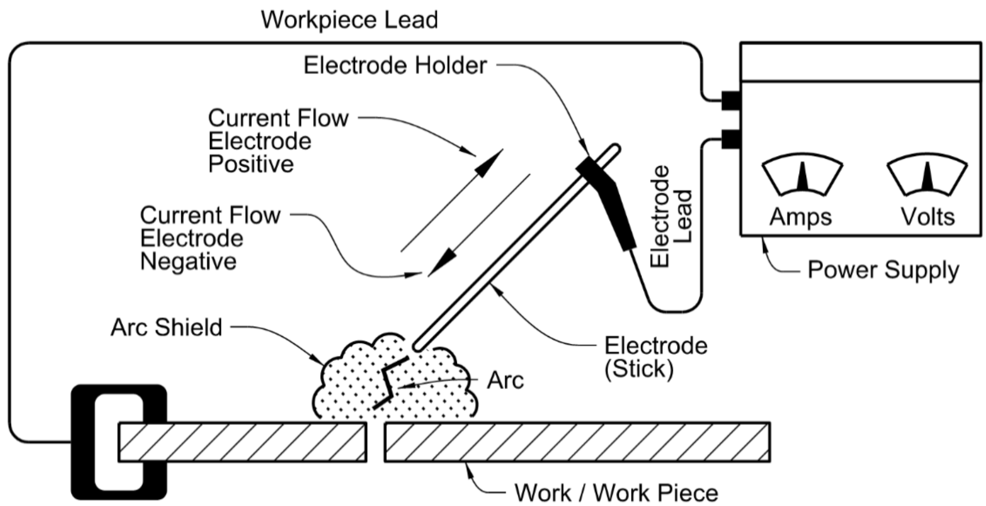
Arc welding describes "a group of welding processes that produces coalescence of workpieces by heating them with an arc" (AWS, 2010c). Arc welding provides a ready and practical way to fuse bridge structural steel in the shop or field. In arc welding, the arc provides local heat to melt the two base metals to be joined, cleanly achieving atomic closeness that results in fusion when the metal cools and solidifies. The arc welding processes used in bridges are described in detail later in this section; table 2 below provides an abbreviated description of the most common of these processes for reference in the following discussion. A detailed discussion of these processes are provided in chapter 3.

Table 2. Common arc welding processes for bridges

Process	Abbreviation	Electrode Delivery	Shielding
Shielded Metal Arc Welding	SMAW	Manual	Electrode Covering
Flux-cored Arc Welding	FCAW	Wire-fed	Flux (inside wire) Gas (can be flux only)
Gas Metal Arc Welding	GMAW	Wire-fed	Gas
Submerged Arc Welding	SAW	Wire-fed	Flux (powder)

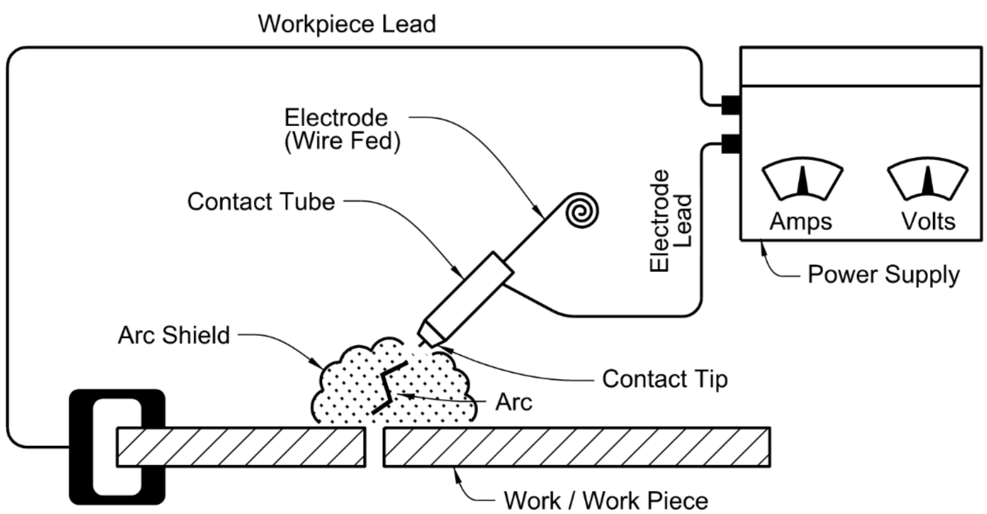
In these arc welding processes, the electrodes are consumed into the weld puddle, which then solidifies to become the deposited weld. Fundamental to arc welding is the need to shield the arc from atmosphere to help maintain the atomic cleanliness discussed above and help ensure good fusion and weld integrity. A high-level description of the shielding associated with each process is provided here; details about shielding are provided below. In some cases, flux not only provides shielding but also contributes alloys to the weld. Collectively, electrodes, fluxes, and shielding gases are known as welding consumables.

The following is a list of items related to the arc welding process. Descriptions in quotations are taken from AWS 3.0M/3.0 (AWS, 2010c). Figures 7 and 8 show the operation and parts of an arc welding circuit for manual and wire-fed welding, respectively.



Source: FHWA

Figure 7. Illustration. The arc welding circuit with manual electrode.



Source: FHWA

Figure 8. Illustration. The arc welding circuit with wire electrode.

Power source - "An apparatus for supplying current and voltage suitable for welding, thermal cutting, or thermal spraying." The power source generates the current and voltage to create the welding circuit.

> **Constant Current (CC) Power Source** - "An arc welding power source with a volt-ampere relationship yielding a small welding current change from a large arc voltage change."

> **Constant Voltage (CV) Power Source** - "An arc welding power source with a volt-ampere relationship yielding a large welding current change from a small arc voltage change."

Under CV, the power supply holds the voltage constant at the arc, reacting to conditions at the weld puddle by altering the current to keep the voltage constant. The converse is true for CC.

Lead (welding lead) - "The workpiece lead and the electrode lead of an arc welding circuit,"

Electrode lead - "A secondary circuit conductor transmitting energy from the power source to the electrode holder, gun, or torch."

Workpiece lead - "A secondary circuit conductor transmitting energy from the power source to the workpiece connector." Also called "ground lead".

Manual Processes:

Electrode holder - The electrode holder grips the electrode and conducts current to the electrode. The holder will have a switch that the welder can use to turn the current on and off.

Electrode (informally called "stick" or "rod") (a covered electrode) - "A composite filler metal electrode consisting of a bare or metal-cored electrode with a flux covering sufficient to provide a slag layer and/or alloying elements." The electrode brings the current to the work and is consumed to produce the weld. The electrode does not touch the work; this gap creates the gap for the arc.

Wire-Fed Processes:

Contact Tip - "A tubular component of an arc welding gun delivering welding current to, and guiding, a continuous electrode." The contact tip delivers the current to the electrode for wire-fed processes. Informally, the contact tip is often referred to as the contact tube, with the tip being a small replaceable component at the end of the tube. Not shown is the fixturing that holds the contact tip; such fixturing is highly dependent on the welding process, the application, and the equipment being used. Tips are separate from tubes because they wear out and are replaced over time.

Electrode (wire) - As with the manual electrode, the wire electrode brings the current to the work but does not touch the work, creating the gap for the arc.

Filler Metal - "The metal or alloy to be added in making a brazed, soldered, or welded joint." Filler metal is often used to mean the electrode consumed in the weld.

Gun (Arc Welding Gun) - "A device used to transfer current to a continuously fed consumable electrode, guide the electrode, and direct the shielding gas." The "gun" is a common name of the apparatus that the welder holds for handheld processes.

Arc (Welding Arc) - "A controlled electrical discharge between the electrode and the workpiece formed and sustained by the establishment of a gaseous conductive medium, called an arc plasma." Within the arc, the current flows from the electrode to the work through an ionized column of plasma. Ions moving within this column generate the intense heat of the arc that melts the work and, in consumable processes such as those used in bridge fabrication, transmit metal droplets from the electrode to the work.

Work/Workpiece - "An assembly, component, member, or part in the process of being manufactured." The "work" or "workpiece" is the component being welded. It is also part of the circuit.

Arc Shield - The arc shield protects the arc from air, helping ensure the integrity of the final weld. The arc shield is created in a number of ways, which is discussed in association with the arc welding processes discussed later. See further discussion of the impact of shielding on quality in chapter 3. Shielding is typically provided by slag, gas, or a combination of both (figures 7 and 8 show the use of gas).

> **Shielding by Slag** - Fluxes melt to form slag, which may coat the individual droplets of metal that leave the electrode. Additionally, once the slag contacts the weld pool, it floats to the surface and shields the pool as well. Slag acts as a mechanical barrier on the weld pool, keeping nitrogen and oxygen from contaminating the weld deposit. Additionally, in the vertical or overhead positions for some welding processes, slag constitutes a mechanical support for the liquid weld metal, helping to shape the weld bead and hold it in place.
>
> **Shielding by gas** - The second means of shielding is through gases. Such gases may be generated from fluxes contained in, on, or around the electrode, or shielding gas may be delivered directly to the weld region. Suitable shielding gases include inert gases, such as argon and helium, or carbon dioxide, which is not inert but is appropriate for shielding carbon steel weld deposits. These gases displace the atmosphere, moving nitrogen and oxygen away from the region.

Equipment Control - Much about welding processes and how they operate relates to how the equipment is controlled, and these are generally categorized into the basic types listed here. Descriptions in quotation marks below are from AWS A3.0 (AWS, 2010c). These process controls relate to the level of welder or operator attention and intervention and are associated with the process, from shielded metal arc welding (SMAW) (see section 3.2), which is fully manual, to semiautomatic handheld wire fed processes, to mechanized wire fed processes that needs continual monitoring and adjustment, to mechanized wire fed processes that need minimal involvement (automatic welding), to robotic welding.

> **Manual** - "Pertaining to the control of a process with the torch, gun, or electrode holder held and manipulated by hand. Accessory equipment, such as part motion devices and handheld material feeders may be used." Example: welding with SMAW ("stick").
>
> **Semiautomatic** - "Pertaining to the manual application of a process with equipment controlling one or more of the process conditions." Example: handheld use of a wire-fed process, such as FCAW or GMAW.
>
> **Mechanized** - "Pertaining to the control of a process with equipment that requires manual adjustment by an operator in response to visual observation, with the torch, gun, wire guide assembly, or electrode holder held by a mechanical device." Example: typical

SAW welding in butt joints and gantry web-to-flange fillet welding. The operator is not directly manipulating the welding head, but does need to watch for the need to make adjustments.

Automatic - "Pertaining to process control with equipment requiring only occasional or no observation and no manual adjustments during its operation." Example: processes like those described in "mechanized" welding but with sufficient controls to allow good welds to be produced with minimal or no observation (not common in the bridge industry). While not an arc welding process, electroslag welding (see section 3.7) is also automatic.

Robotic - "Pertaining to process control with equipment that moves along a controlled path using controlled parameters with no manual intervention once a cycle is initiated." Example: fully robotic welding, for which an operator initiates welding but then the operation is driven by a program and is capable of tracking and following the joint on its own with no other intervention from the operator.

2.5. ARC WELDING VARIABLES

Section 2.3 provides a general description of the arc welding circuit. This section discusses the settings, or variables, associated with the arc welding circuit components and how changing these settings influences the welding process and the final weld.

2.5.1. Amperage

Amperage is a measure of the amount of current flowing through the electrode and the work. Amperage is constant throughout the welding circuit. Generally, an increase in amperage means higher deposition rates, deeper penetration, and more blending of weld and base metal.

2.5.2. Wire Feed Speed

Applicable to wire-fed processes (and not manual welding), wire feed speed is a measure of the rate at which the electrode is passed through the welding contact tube and delivered to the arc. Measured in inches per minute, the wire feed speed is proportional to the deposition rate and directly related to amperage. If other variables are held constant, an increase in wire feed speed will directly lead to an increase in amperage.

Given the correlation between amperage and wire feed speed, fabricators will often choose to control wire feed speed in a combined way, adjusting welding in process and controlling amperage. The wire feed speed can be independently adjusted and measured directly, regardless of the other welding conditions; the welder may adjust the wire feed speed while monitoring the amperage readout. Consumable manufacturers publish wire feed speed-to-amperage correlation curves that provide amperages for given wire feed speeds; however, if there is a question about the actual amperage in a particular circuit during welding when wire feed speed is being used for control, the amperage can be measured with an ammeter.

2.5.3. Arc Voltage

Unlike amperage, which is constant throughout the circuit, voltage varies along the circuit, dropping across the various parts of the circuit that provide resistance, such as within the leads, within the gun and cable assembly, within the work piece to work clamp connection, as well as within the arc itself. Hence, during welding, voltage is somewhat lower at the work than at the machine. The voltage of most interest is the voltage at the work. To get a precise measurement of the actual weld voltage, it should be measured at the work as close to the arc as possible.

Arc voltage is directly related to arc length: as the arc voltage increases, the arc length increases. For welding processes that use wire electrodes, arc voltage is an important procedural variable: the higher the voltage, the longer the arc length, and vice versa. The arc length in turn affects the weld bead width and other aspects of the weld geometry. For manual welding (SMAW, section 3.2), the arc length is control by the welder and is constantly changing as the welder moves the electrode as welding progresses. Therefore, with manual processes, the voltage is not typically monitored.

2.5.4. Travel Speed

Travel speed, measured in inches per minute, is the rate at which the electrode is moved relative to the joint. If a weave is applied (i.e., the electrode is moved back and forth as it progresses), the travel speed is the rate at which welding advances, not the rate of weaving. All other variables being equal, travel speed has an inverse effect on the size of the weld beads; that is, as the travel speed increases, the weld size will decrease. Travel speed is a key variable used in computing heat input; reducing travel speed increases heat input.

2.5.5. Heat Input

Heat input (sometimes called “energy input”) is a mathematical estimate of the amount of thermal energy that is introduced into the steel when the weld is made. Heat input in turn determines the solidification and cooling rates of the weld metal, and the cooling rates that will be experienced in the heat-affected zone (HAZ). Changes in the thermal cycles experienced by the weld and the HAZ will affect the properties of these regions.

The following equation is used in the Bridge Welding Code to calculate heat input, *H* (kJ/inch):

$$H = \frac{60EI}{1000S} \tag{1}$$

where:

E = arc voltage (volts)
I = current (amperes or amps)
S = travel speed (inches/minute)

Higher levels of heat input add more thermal energy into a weld and increase the amount of material deposited in a unit of time (deposition rate). Higher heat input levels will typically result in weld beads with larger cross-sectional areas. For single-pass welds, higher heat input will mean a larger weld. For multipass welds, higher heat input will decrease the number of passes needed to complete the weld.

In many respects, lower levels of heat input have just the opposite effect of high heat input levels: lower levels add less thermal energy to a weld, and will deposit weld beads with smaller cross-sectional areas. Single-pass welds made with lower heat input will generally be smaller, and multipass welds will require a greater number of weld passes to complete the weld.

Lower heat input usually leads to slightly increased yield and tensile strength in the weld metal, but in the case of fracture toughness, both very low and very high heat input levels can result in lower fracture toughness, particularly for multipass welding. The higher cooling rates associated with low heat input cause the weld metal to increase in strength and decrease in ductility. Additionally, when subsequent weld passes are deposited on top of these welds with low heat input in a multipass weld, there is little additional energy to reheat the previously deposited beads; it is the reheating of the previously deposited weld metal that refines the weld metal and improves the fracture toughness. Qualification testing (clause 5) in the Bridge Welding Code demonstrates that the heat inputs to be used in production satisfactorily provide needed toughness.

2.5.6. Electrode Extension

For automatic and semiautomatic welding processes (e.g., SAW, FCAW, GMAW), the length of electrode that extends beyond the contact tip to the arc is known as the electrode extension. It is colloquially called “stickout” or “electrical stickout”. The proper electrode extension depends on the electrode type and diameter, and a recommended value or range is usually provided by the electrode manufacturer. Welders using semiautomatic (handheld) welding processes maintain the electrode extension by controlling the distance from the contact tip to the work. For automatic welding, the electrode extension is mechanically established by the welding equipment.

2.5.7. Contact Tip to Work Distance

The contact tip to work distance (CTWD), is the electrode extension plus the arc length. This is the physical distance from the contact tip to the work, and for mechanized welding such as gantry-mounted SAW, it is typically maintained by the mechanical apparatus that aligns the welding torch with respect to the joint. CTWD is usually the electrode extension dimension, plus $^{1}/_{8}$ to ¼ inch, which represents the arc length. Changes in CTWD have the same effect as changes to the electrode extension.

2.5.8. Electrode Diameter

The diameter of the electrode determines how much current it can carry: larger electrodes can carry higher welding currents. For SMAW electrodes, the diameter is that of the steel core, and does not include the coating.

As electrodes get larger in diameter, the amount of weld metal deposited over a unit period of time increases; hence, the larger the electrode used, the more weld metal that can be deposited over a shorter period of time, and therefore the higher the productivity. However, other factors play into this productivity. Larger electrodes run at higher current (or “hotter”), and in some cases it may not be possible to increase electrode diameter without causing burn-through. Also, higher current may require larger, more powerful power supplies, and so equipment availability may limit the size electrode a fabricator can use.

2.5.9. Polarity

Polarity is associated with direct current (DC) welding circuits and describes the direction of current flow. Positive polarity, also called reverse polarity, is achieved when the electrode lead is connected to the positive terminal of the power supply, and the work lead is connected to the negative terminal. Negative polarity (or straight polarity) occurs when the electrode is connected to the negative terminal and the work lead to the positive terminal. Alternating current (AC) is not a polarity, but a current type. With AC, the current flow is alternately positive and negative.

Submerged arc welding is the only process that commonly uses either electrode positive or electrode negative polarity with the same type of electrode. For a fixed wire feed speed, a submerged arc electrode will require more amperage on positive polarity than on negative, and the higher current will result in deeper penetration.

CHAPTER 3 - BRIDGE WELDING PROCESSES

While there are common elements among all welding processes, arc welding is differentiated from the others by the presence of the arc, which melts base metal and filler metals under some form of shielding. The following sections provide an overview of common welding processes.

The Bridge Welding Code does not dictate which welding processes are to be used for certain welds. Rather, the contractor or fabricator chooses the best process for each application, based on available equipment and skills. This information is provided to give an understanding of each type of weld and the general balance of conditions that the fabricators must address for production.

Collectively known as filler metals, the electrodes and, for SAW, electrode-flux combinations used in welding are classified by AWS. To structure these classifications, AWS publishes filler metal specifications, which are known as A5 standards.

The A5 specifications outline the required mechanical properties and chemical compositions of welds deposited by various filler metal classifications. Consumable manufacturers produce filler metals in conformance with the A5 specifications and conduct regular tests to demonstrate conformance. Certifications of conformance with the A5 specifications are available from the consumable manufacturers. The Bridge Welding Code refers to electrode classifications using the A5 specification designations.

Welding processes may be selected based upon the required welding position—flat, horizontal, vertical or overhead. See section 2.3 for a detailed discussion about these positions, including why vertical and overhead are often referred to as “out of position” and flat and horizontal referred to as “downhand” or “in position”.

3.1. MANUAL VERSUS WIRE-FED PROCESSES

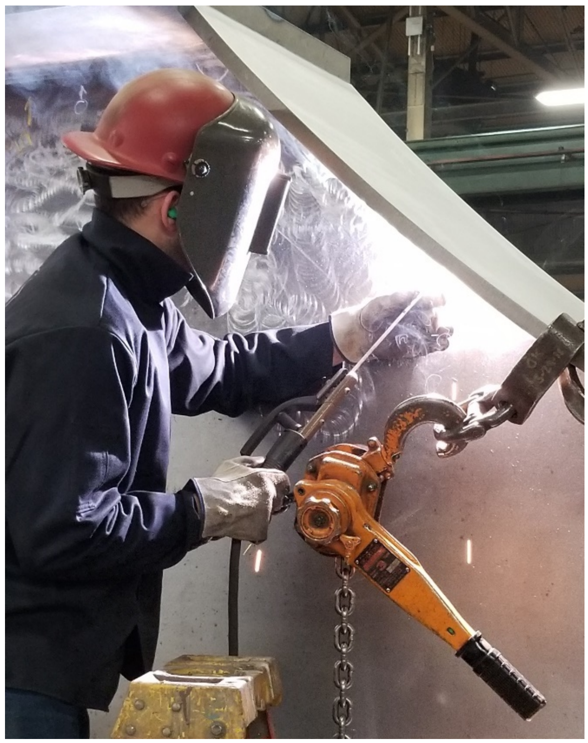
There are distinct differences between manual welding and wire-fed processes). With manual processes, the welder must maintain the arc length (the gap between the electrode and the workpiece), while manually feeding the electrode into the puddle and advancing the electrode along the joint. With wire-fed processes, the operator does not need to maintain the arc length or feed the electrode into the puddle. The power supply maintains the arc length, and the wire feeder delivers the electrode to the arc. However, the welder must establish the initial electrode extension distance. Unlike the variable resistor associated with manual welding, wire-fed processes have a fixed resistance (for a given electrode extension), and more uniform welding conditions are maintained. In some ways, less welder skill is required for handheld wire-fed processes than with manual; however, because of the higher deposition rates involved with wire-fed processes, welders must learn how to control the larger volumes of molten metal that are common to this process.

Electrodes for wire-fed processes are said to be “continuous”. In reality, they have a finite length, but since these wire electrodes are spooled onto packages that may consist of anywhere from 1 to 1,000 pounds of material, they are virtually continuous in comparison to manual electrodes.

3.2. SHIELDED METAL ARC WELDING (SMAW)

3.2.1. Applications

SMAW as shown in figure 9, colloquially called “stick” welding, is used infrequently in most bridge shops due to its lower productivity. However, because of its simplicity, it is used where access for equipment is limited, or when transporting and positioning of equipment would be a major task. A prime example is tack welding—moving equipment around a structural component may take more time than making the tack welds. Thus, SMAW is often used for tacking. Portability makes SMAW useful in field applications, both in new construction and also for field repairs.



Source: FHWA

Figure 9. Photo. SMAW.

3.2.2. Fundamentals

SMAW is “[a]n arc welding process with an arc between a covered electrode and the weld pool. The process is used with shielding from the decomposition of the electrode covering, without the application of pressure and with filler metal from the electrode” (AWS, 2010c). The process is illustrated in figure 10. The wire core of the electrode (see section 3.2.4) is melted and becomes part of the deposited weld. During welding, the welder manually maintains the arc length and “feeds” the electrode to the weld as it is consumed to create the deposited weld.

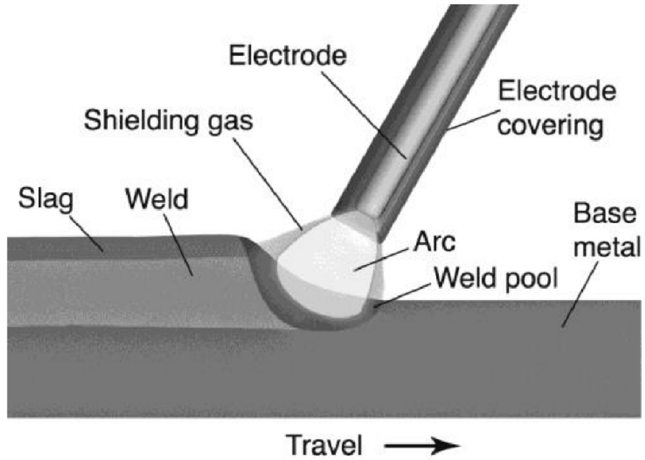


Figure 10. Illustration. SMAW process.

SMAW is characterized by versatility, simplicity, and portability. In the 1940s, 50s, and 60s, SMAW was commonly used for shop fabrication that could not be done with submerged arc welding. The advent of gas metal arc welding (see section 3.5) and flux-cored arc welding (see section 3.6), however, has displaced much of the use of SMAW in the shop. Though SMAW is still sometimes used to dependably deposit quality welds, it is slower and more costly than other methods of welding.

3.2.3. Equipment

SMAW is performed using a power supply with constant current (CC) output (figure 11). Either alternating current (AC) or direct current (DC) may be used for SMAW. If primary electrical power is available, a transformer (for AC output) or a transformer/rectifier (for DC output) is used to convert the high-voltage alternating current into high-amperage output, suitable for welding. Small, lightweight, and highly portable inverter power supplies can perform this function more efficiently, reducing operating costs. When primary power is not available, the same type of equipment may be powered by portable power generation systems. Alternatively, individual engine-driven welders, fueled by gasoline, diesel, or propane can be used to generate the necessary welding power directly.

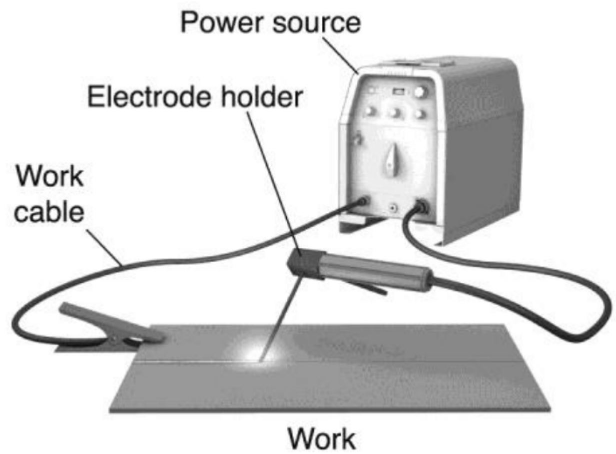


Figure 11. Illustration. Typical SMAW welding circuit.

For SMAW, the arc length determines the arc voltage; there is no control on a SMAW welding machine that permits the arc voltage to be set. SMAW welders manually control the length of the welding arc: when the arc is too short, the electrode will stick to the work; when the arc is too long, the arc is extinguished. Accordingly, there is a relatively narrow range of arc lengths that can be used, with a correspondingly narrow range of arc voltages that will result.

3.2.4. Consumables

For SMAW, this is the electrode, often called a "rod", although that is not the AWS-defined use of the term. Electrodes for SMAW consist of a solid wire surrounded by a coating of material called flux. The core wire is essentially the same for all SMAW electrode classifications, with the coating providing alloys that result in different strength properties for the weld metal as well as shielding the molten weld metal from the atmosphere. These coatings can pick up moisture, which must be avoided, so once the seal on a can of electrodes has been broken, the electrodes must be stored in an electrode oven (except for cellulosic electrodes such as E6010, which are not listed in the Bridge Welding Code); the Bridge Welding Code prescribes practices for this control. One end of the electrode has no coating; this is the grip end where the exposed core wire is inserted into the electrode holder.

SMAW electrodes range in size from $^{1}/_{8}$ to $^{3}/_{16}$ inch for most structural work, although smaller and larger sizes are available. Larger electrodes can carry more welding current, and thus melt faster, yielding higher production rates. The as-received lengths of the electrodes vary with the diameter; electrodes that are 14 to 18 inches long are typical (figure 12).

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Figure 12. Photo. SMAW electrodes.

The filler metal specification AWS A5.1 (AWS, 2012) addresses the particular requirements for carbon steel covered electrodes used with the SMAW process, while AWS A5.5 (AWS, 2014a) similarly covers the low-alloy electrodes. All AWS A5.1 electrodes produce weld deposits with either 60 or 70 ksi minimum specified tensile strength, while AWS A5.5 low-alloy electrodes will give deposits with minimum specified tensile strengths from 70 to 120 ksi. SMAW electrodes for use with A709 Grade 50 steels will likely be carbon steel electrodes governed by AWS A5.1, whereas for weathering steels such as ASTM A709 Grade 50W, when the weld deposit is required to have similar atmospheric corrosion resistance, the alloy electrode is governed by AWS A5.5.

3.2.5. Electrode Classification

AWS A5.1 and AWS A5.5 both define classification systems for identifying the various types of SMAW electrodes. The AWS A5.1 system consists of the letter "E" followed by four digits, where EXXYZ is:

E indicates an electrode

XX specifies the minimum specified tensile strength (ksi)

Y indicates the position in which the electrode may be used; for example, electrodes restricted to the flat or horizontal positions are designated with a "2"

Z designates the coating type and welding current polarity

Using the classification E7018 as an example, the "E" stands for "electrode"; the minimum specified tensile strength is 70 ksi; the "1" indicates that the electrode can be used in all positions; and the "8" indicates this electrode has a low hydrogen coating, operates on direct current (DC) with the electrode connected to the positive side of the circuit (i.e., DC positive) or alternating current (AC), and the deposited weld metal can deliver a minimum specified CVN energy of 20 ft-lb at −20 °F. Although “E70” is a common way to refer to a 70-ksi strength class electrode for any given process, that usage is not correct.

Under the low-alloy electrode specification AWS A5.5, a similar format is used to identify the various electrodes. The most significant difference is the inclusion of suffix letters and numbers indicating the alloy content. An example is an E8018-C3 electrode, where the suffix “-C3” in this case indicating the electrode nominally contains 1 percent nickel.

There are a variety of other suffixes that may be applied to either AWS A5.1 or AWS A5.5 SMAW electrodes. These suffixes address issues such as specific military classifications, extra CVN toughness capabilities, and the maximum diffusible hydrogen content of the deposited weld metal. These designations may be of great importance to the fabricator, but are typically not important to the design engineer because the Bridge Welding Code defines electrode classification requirements. Further detail on these suffixes, as well as other details regarding the electrode classification system, can be obtained from the filler metal specifications themselves, or from other sources listed in chapter 12.

3.2.6. Advantages and Limitations

SMAW continues to be a viable process, despite some inherent disadvantages. Familiarity can be an advantage in the sense that many welders know and have used this process (though this is becoming less so with the growing popularity of other processes). In addition to standard safety equipment, all which is required to weld with SMAW is a work cable and clamp, an electrode cable and holder, and the SMAW electrode. In terms of equipment, it is the simplest of all the arc welding processes.

A limitation of SMAW is that the electrode is a finite length, and when it is consumed, the remaining electrode (called a stub) must be removed from the holder, discarded, and a new electrode inserted. This has several consequences. First, the welder must stop welding, interrupting productivity. When this occurs in the middle of a weld, an otherwise unnecessary stop/start is created, which could potentially be a source of defects. Finally, because the stubs cannot be consumed, the process is less efficient in the use of purchased electrodes. Out of 100 lb of purchased electrode, approximately 20 lb of stubs will be created when following the good practice of welding down to only 2-inch stubs. From the standpoint of technique, SMAW requires more hand position adjustment than handheld wire fed processes because as the electrode shortens, the welder must maintain arc length as the electrodes is consumed and shortens.

3.3. SUBMERGED ARC WELDING (SAW)

3.3.1. Applications

Submerged arc welding (SAW) is “[a]n arc welding process using an arc or arcs between a bare metal electrode or electrodes and the weld pool. The arc and molten metal are shielded by a blanket of granular flux on the workpieces. The process is used without pressure and with filler metal from the electrode and sometimes from a supplemental source.” (AWS, 2010c). Shown in figure 13, SAW, more familiarly known as “subarc”, is the workhorse of the steel bridge fabrication industry, well suited to full-penetration welds of large cross-sections and too long, mechanized welds. Given its common use in flange splicing, web splicing, web-to-flange welding and stiffener-to-flange welding, SAW accounts for perhaps 90 percent of shop welding on steel bridges by volume. As described below, it operates with larger-diameter electrodes, higher heat input, and higher deposition than other arc welding processes, and so for many decades it has been the process of choice in bridge shops.

Source: FHWA

Figure 13. Photos. SAW in use.

3.3.2. Fundamentals

Submerged arc welding is illustrated in figure 14. Because the arc is completely covered by the flux (i.e., it is “submerged” in the flux), it is not visible and the weld is made without the flash, spatter, and sparks that characterize open-arc processes. Further, the nature of the flux is such that little smoke or visible fumes are emitted under normal conditions.

SAW is typically used automatically, although semiautomatic operation is also used. In mechanized welding, the electrode is positioned with respect to the joint, and is automatically propelled along the length of the joint (figure 15). In the semiautomatic (handheld) version of the process, the welder orients the electrode with respect to the joint, and also moves the welding arc along the weld joint.

SAW is most commonly used with one electrode, although it is possible to use more. Two electrodes may be fed through a single electrical contact. This is formally called “parallel electrode welding”, although it is often called “twin electrode” welding. When two or more separate individually controlled arcs are used, the configuration is formally known as “multiple electrode welding”, although it is often called “tandem” welding.

The granular flux must stay in place to shield the weld pool, and thus, SAW is restricted to the flat and horizontal welding positions. Because of gravity, it is not effectively possible to keep flux in place over the arc, properly shielding it, in the vertical or overhead positions.

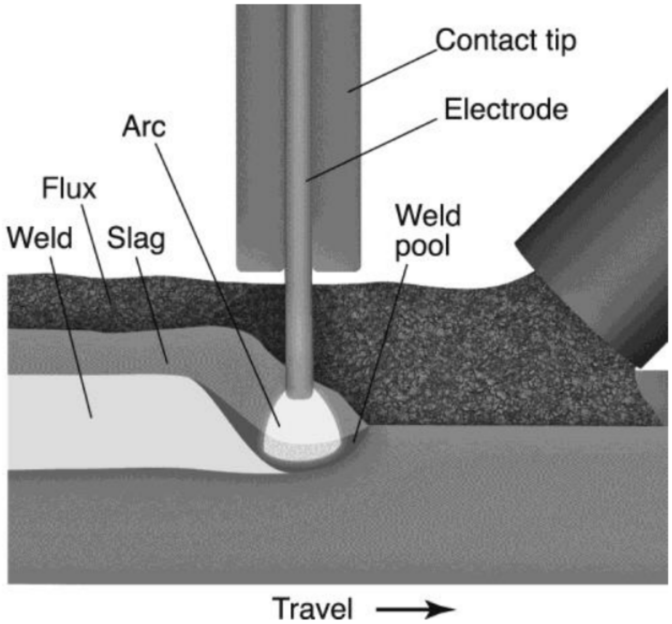

Figure 14. Illustration. SAW process.

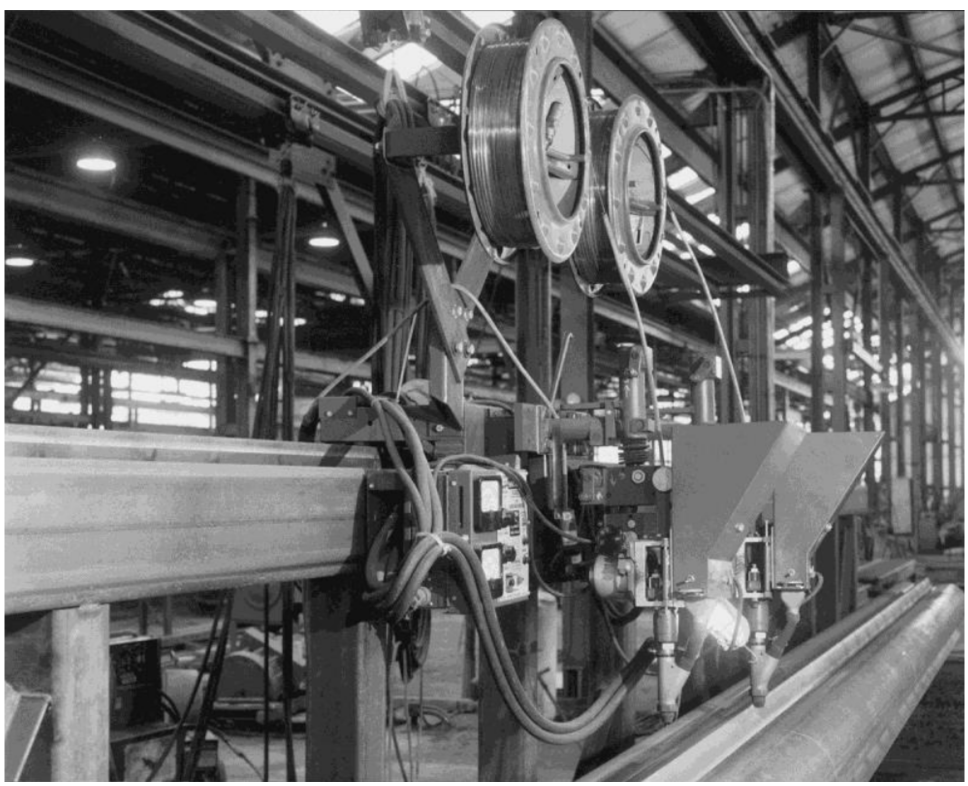

Figure 15. Photo. Automatic SAW.

3.3.3. Equipment

SAW equipment consists of a power source, a wire feeder, a device through which the electrode is fed and electrical power is introduced, and a work lead and clamp. The power supply can be either constant current or constant voltage, and SAW can be performed with either direct or alternating current. For direct current welding, either DC positive or DC negative may be used. For alternating current, the AC may be a sine wave or square wave. Advanced SAW power supplies allow the square wave output to be controlled in a variety of ways: the frequency, ratio of positive to negative cycle and other output characteristics can all be controlled. SAW can be performed with constant voltage (CV) or constant current (CC) systems. Constant current has been traditionally preferred when making large weld passes, although special CV output modes have been developed to mimic the advantages that CC has traditionally offered. The Bridge Welding Code does not prescribe current type for SAW or any other process.

For SAW, flux is separate from the electrode and requires an additional flux delivery system. For many applications, a simple hopper allows the flux to be delivered by gravity, or a pressurized container may be used, where the flux is delivered by means of compressed air (figure 16). The compressed air must be clean and dry, or the flux will become contaminated. Some SAW systems may also have an automated flux removal and recycling system (see section 3.3.4 below).

Figure 16. Photo. Pressurized SAW delivery system.

3.3.4. Consumables

Consumables for SAW consist of electrodes and fluxes. Electrodes typically are solid, but may be tubular with metal powders or flux ingredients inside. Solid electrodes usually have a thin coating of copper, which aids in electrical conductivity as the wire passes through the contact tip. For steel bridge fabrication, diameters typically range from $^{5}/_{64}$ to $^{3}/_{16}$ inch. Both larger and smaller electrodes are used but are not common; the most common are $^{3}/_{32}$ and $^{5}/_{32}$ inch in diameter.

Fluxes are granular materials that are also consumed, forming the slag coating on top of the completed weld, and are classified as active or neutral. Active fluxes contain deliberate additions of manganese and silicon and are primarily designed for single- or limited-pass welding. These ingredients enable welding on materials with heavier rust and scale. Hence active fluxes are an excellent solution for fillet welding over mill scale. Neutral fluxes, on the other hand, are appropriate for multipass welds but do not have the same ability to handle surface contaminants as do the active fluxes.

The amount of flux that is melted per pound of weld deposit is dependent on the arc voltage: the higher the voltage, the more flux is melted. When active fluxes are used, and when the voltage is increased, more manganese and silicon are added to the weld deposit. When this is done in conjunction with multipass welding, high voltages with active fluxes can lead to alloy buildup in the weld deposit. However, when voltage changes are made with neutral fluxes, the manganese and silicon content of the weld deposit does not appreciably change. Active fluxes therefore typically are used for single-pass or limited-pass welds.

Alloy flux is a distinct type of neutral flux made by adding specific alloys to the flux. When the flux is melted, the alloys are transferred to the weld. Thus, a carbon steel electrode can be used, and an alloy deposit obtained. Such alloy fluxes are often used in weathering steel (e.g., A709 Grade 50W) applications on bridges.

3.3.5. Electrode and Flux Classification

Submerged arc welding filler materials are classified under AWS A5.17 (AWS, 1997b) for carbon steel and AWS A5.23 (AWS, 2011) for low-alloy filler metals. Both fluxes and electrodes are covered under these specifications.

SAW electrodes are typically solid, but composite (metal-cored) electrodes are also used. Solid electrodes are classified based on the composition of the wire, whereas composite electrodes are classified on the deposit chemistry.

In both AWS A5.17 and A5.23, fluxes are always classified in conjunction with an electrode. Welds made with the flux–electrode combination must meet specific mechanical property requirements prescribed in the A5 specification. The A5.17 classification system follows the format of an “F” followed by a single- or two-digit number, an “A” or “P,” a single digit, and a hyphen which separates the electrode classification, where FZBC-EXXXX is:

F	indicates flux
Z	specifies the minimum specified tensile strength in units of 10 ksi
B	designates the condition of heat treatment
C	indicates the CVN test temperature (where *C* is multiplied by −10 °F) at or above which the weld metal meets the specified CVN properties

E	indicates an electrode
C	optional, indicating a composite electrode
L, M, or H	referring to a low, medium, or high level of manganese
One or two digits	the nominal carbon content in hundredths of a percent
K (optional)	when present, denotes that the electrode is made of killed steel

Thus, a typical flux-electrode combination may be classified as an F7A2-EM13K. The “F” stands for flux, and the “7” indicates all of the following: a 70- to 95-ksi tensile strength deposit, a 58 ksi minimum yield strength, and a minimum of 22 percent elongation. The “A” indicates that the deposit is tested in the as-welded (as opposed to heat-treated) condition. The “2” indicates a notch toughness of 20 ft-lb at −20 °F (where the “2” in the classification is a reference to the temperature), and the balance of the classification identifies the electrode used. EM13K is a non-composite electrode (E) with medium levels of magnesium (M), 0.13 percent nominal carbon (13), consisting of a killed steel (K).

Because of the popularity of the SAW process for pressure vessel fabrication where assemblies are routinely stress-relieved, submerged arc welding consumables may be classified in the post-weld heat-treated, or stress-relieved, condition. When this is done, a “P” replaces the “A.” For bridge work, which is seldom stress-relieved, “A” classification electrodes are used.

For low-alloy products classified under AWS A5.23, a format similar to that of AWS A5.17 is used, but at the end of the flux-electrode classification, a weld deposit composition is specified. For example, F7A2-ENi1-Ni1 indicates that the electrode, ENi1, delivers an F7A2 deposit when used with a specific flux. In addition, the deposit has a composition that meets the requirements of Ni1. In this case, a nickel-bearing electrode deposits a weld that contains nickel. It is also possible to use alloy fluxes which, with carbon steel electrodes, are capable of delivering alloy weld metal. In this case, a typical classification may be F7A2-EL12-Ni1. In this example, an EL12 electrode (a non-alloy electrode that contains a low level of manganese) is used with an alloy flux. The result is an alloyed deposit.

3.3.6. Flux Recovery

Only part of the flux deposited onto a weld is melted, or fused, in welding. The unfused, granular flux may be recovered for future reuse and is known as recovered flux. The unmelted flux does not undergo chemical changes and may therefore be used to produce quality welds when used the next time. However, if the flux comes in contact with oil, water, dirt or other contaminants during deposition and recovery, the properties of the weld deposit made with reclaimed flux may be adversely affected, so flux recovery systems are designed to provide suitably clean flux.

Loose mill scale can be picked up along with the unfused flux when it is recovered. The recovered flux can be passed through a magnetic separator to capture metallic scale. Pieces of slag can be inadvertently recovered with the unmelted flux; screens can be used to separate slag from flux.

The method of flux recovery can range from sweeping up the flux with brooms and pans, to vacuum recovery systems (figure 17); the method chosen should take into account the need to avoid contamination. In the figure, the black hose on the right is a vacuum that returns loose flux from the weld to the hopper. The vacuum can be mounted following the arc, as shown, or the welder may use it by hand. When flux is handled and rehandled, there is the potential for the mechanical breakdown of particles and the modification of the particle size distribution. This is pronounced with some vacuum recovery systems; some systems have filters to capture the fine particles called "flux flour," which is normally discarded.

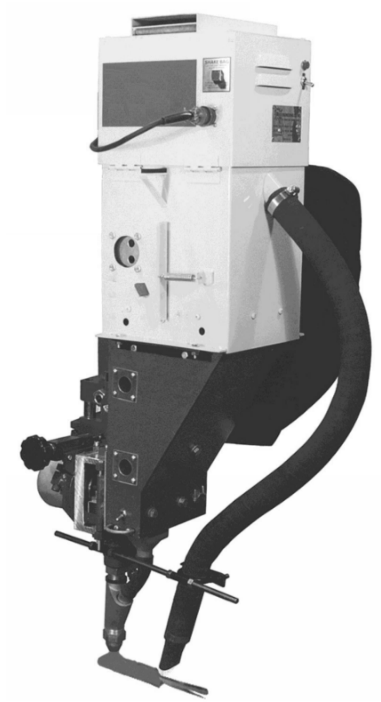

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Figure 17. Photo. Flux vacuum recovery system.

When recovered flux is used, the Bridge Welding Code requires it to be mixed with unused, new flux. Typically, the mix will be approximately 25 percent to 50 percent new flux, except that the Bridge Welding Code prescribes at least one-third new flux for welding of fracture-critical components. The condition that should be avoided is starting with a hopper of new flux which may consist of several hundred pounds of material, recovering flux and returning it unmixed to the hopper and continuing the practice until the hopper is empty. When this approach is used, the final flux used may have been recovered a half dozen times. Flux that has melted and solidified is called slag. Slag is typically different, chemically, from unfused flux. This welding byproduct can be crushed and reused in some applications; however, crushed slag, which has its own FSXXX designation, is not referenced in the Bridge Welding Code and is not used on bridges.

3.3.7. Advantages and Limitations

SAW is capable of high productivity rates because it can use higher welding currents, resulting in higher deposition rates and deeper penetration. Square wave technology may further increase deposition rates without increasing welding current. Higher deposition rates simply mean that the fabricator can deposit the required weld in less time. For even higher deposition rates, a second electrode or more can be added into the system to increase productivity. Because the process typically is mechanized, SAW welds usually are made without starts and stops for the length of the joint; considering its common use joining girder webs to flanges, continuous welds over 100 feet in length are common. Welds made under the protective layer of flux are excellent in appearance and spatter-free.

Another benefit of the SAW process is that the arc is not exposed to view. This means that the welder is not required to use the standard protective helmet, and multiple welding operations can be conducted in a tight, restricted area without the need for extensive shields to guard the operators from arc flash. The process produces little smoke, which is another production advantage, particularly in situations with restricted ventilation. Also, because the arc is covered, the process radiates less heat, which can be more comfortable in hot shops.

The arc being submerged, however, also offers a key challenge to the process: it does not allow the welder to observe the weld puddle. When SAW is applied semi-automatically, the operator must learn to propel the gun carefully in a manner that will ensure uniform bead contour. The experienced operator relies on the formation of a uniform slag blanket to indicate the nature of the deposit underneath it. Most SAW applications are mechanized. Long, uninterrupted, straight seams are ideal applications for SAW.

Finally, SAW is not suitable for vertical and overhead welding because the flux cannot be kept over the arc in those positions; it falls away due to gravity. For shop fabrication, the work can be moved such to facilitate a position suitable to SAW. However, field conditions prohibit such opportunities, and thus restrict the suitability of SAW.

3.4. GAS METAL ARC WELDING (GMAW) AND FLUX-CORED ARC WELDING (FCAW)

Compared with other welding processes, GMAW and FCAW are very similar, particularly considering practical aspects of steel bridge welding. Given these similarities (in fact, in the ASME codes, FCAW is considered to be a type of GMAW, and GMAW with metal-cored electrodes was formerly classified by AWS as FCAW), this section addresses common aspects of these processes, and then more unique aspects of GMAW and FCAW are addressed in sections 3.5 and 3.6, respectively.

3.4.1. Applications

Depending upon the application and the fabricator’s preferences and equipment, these two processes may be used semi-automatically, mechanized, automatically, or robotically. In semiautomatic welding, the welder holds the gun and controls travel speed, whereas in mechanized, automatic, and robotic welding, these functions are mechanically controlled. Generally, these two processes are popular for welds that cannot be readily made with SAW. This includes short welds, “out of position” welds (vertical or overhead), tack welds, and robotic welds. Generally, fabricators choose between GMAW and FCAW for such applications. Often the choice between the two comes down to what is customary in a given shop.

3.4.2. Equipment

GMAW and FCAW equipment is similar enough that in many cases it is used interchangeably between the two processes. Each requires a power supply, a wire feeder, a gun and cable system, a work lead and clamp, and a power lead that runs from the power source to the wire feeder (see figure 18). While the gun cable assembly may be 10 or more feet long, the electrical power is delivered to the electrode near the point where it exits the gun. As the electrode passes through a hollow copper tube, called a contact tip, the electrical energy is transferred to the electrode. Then, current is transferred through the electrode until it gets to the arc. The electrode is typically fed through the contact tip at a rate of 200 inches per minute or more, which, given the short distance, offers little time for the electrode to overheat. The electrode extension is typically ¾ to 1 $^{1}/_{8}$ inches.

GMAW power supplies may have additional controls for optimizing the output characteristics for certain modes of transfer (see section 3.5.7). For GMAW and gas-shielded FCAW, some type of shielding gas regulator and flow meter, as well as hoses, are required; for self-shielded FCAW, gas and associated equipment and controls are not needed. While the gas shield must be protected from winds and drafts, this is not particularly difficult in enclosed shop fabrication situations. The wire feeder mechanically drives the coiled electrode through the gun and cable system. Typically, when the welder depresses the switch on the gun, simultaneously, the wire feeder delivers the electrode, the gas shielding (when required) begins to flow, and the power source output is energized. Some wire feeders are small, lightweight and compact units that can be moved from one location to another, while others are larger and more likely to be part of a welding station. Gun cable assemblies are typically 10 to 15 feet long, allowing for some movement of the gun from the wire feeder.

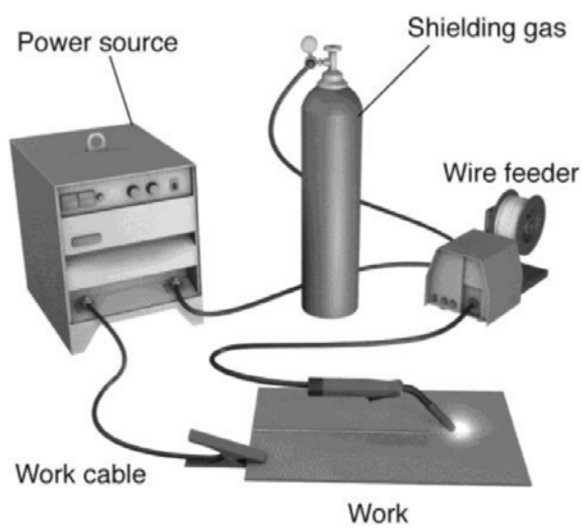

Figure 18. Illustration. GMAW and FCAW-G equipment.

3.4.3. Advantages and Limitations

As described in section 3.3, SAW is generally the most popular welding process in a bridge fabrication shop because large structural assemblies (like plate girders) with many long fillet welds or many thick full penetration groove welds lend themselves to the high-deposition welds that SAW offers. By contrast, bridge fabricators typically turn to GMAW or FCAW for tack welds; for shorter production welds, such as stiffener to flange welding; or for welds that change direction, are difficult to access, or are out of position (i.e., vertical or overhead). As discussed in section 3.2, SMAW is another reasonable but generally less productive choice for such welding.

Compared with SMAW, GMAW and FCAW welding equipment costs more and is less portable. Guns and cables are more costly to buy and maintain than are the simple electrode holders used for SMAW. To change from one size of electrode to another, the welder must change the coil or spool of electrode, perhaps change the drive rolls in the wire feeder, and make changes to the gun and cable assembly. Thus, as compared to SMAW, such changes are more complicated.

3.5. GAS METAL ARC WELDING (GMAW)

This section addresses unique aspects of GMAW. Aspects of GMAW that are the same as or similar to FCAW are addressed in section 3.4

3.5.1. Applications

GMAW is "[a]n arc welding process using an arc between a continuous filler metal electrode and the weld pool. The process is used with shielding from an externally supplied gas and without the application of pressure." (AWS, 2010c). GMAW, shown in figure 19, is also commonly known as "MIG" (metal inert gas). "MIG" is a reference to the shielding gas used in early versions of the process and in some cases today. With the use of other gases such as carbon dioxide, the process would more properly be referred to as "MAG" (metal active gas), but "MIG" remains the most popular colloquial term. In some countries, GMAW is formally referred to as "MIG/MAG".

Despite being a mature and effective welding process, GMAW has come into bridge fabrication somewhat slowly. This is probably because, for a time, many bridge owners prohibited its use and it was not allowed for use on fracture-critical members; in turn, these prohibitions were probably related to concerns about short circuiting transfer (discussed in section 3.5.8). However, short circuiting transfer is readily avoided, and GMAW with metal-cored electrodes has now been allowed for use on fracture-critical members since 2002. Notwithstanding owner prohibitions, bridge fabricators like GMAW because it is versatile and does not generate slag. Use of GMAW in pulsed spray mode (see section 3.5.7) is particularly useful for out-of-position welding.

Source: FHWA

Figure 19. Photo. GMAW in use.

3.5.2. Fundamentals

GMAW, illustrated in figure 20, uses a solid or metal-cored electrode (also called a “composite” electrode), and leaves no appreciable amount of residual slag.

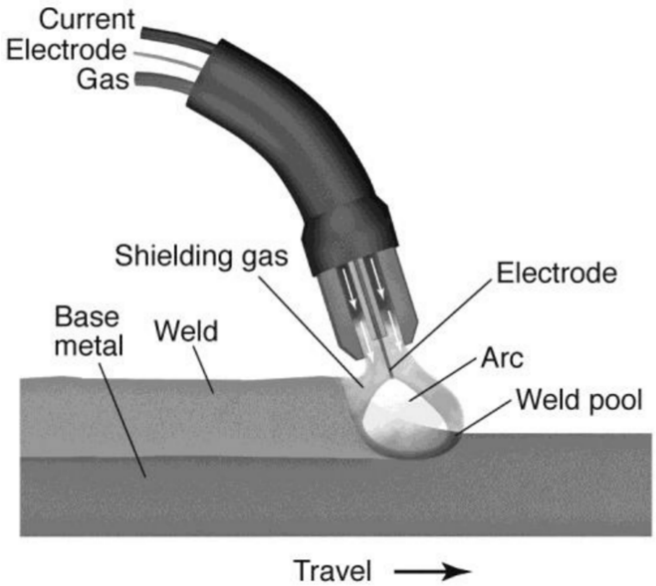

Figure 20. Illustration. GMAW process.

The transfer of metal from the electrode to the weld pool may occur in several manners, called transfer modes. While a dozen or so transfer modes have been defined, four are commonly used in structural applications. These modes of transfer are discussed in Section 3.5.7.

3.5.3. Equipment

GMAW requires a power supply, a wire feeder, a gun and cable system, a work lead and clamp, and a power lead that runs from the power source to the wire feeder (figure 18). Some GMAW power supplies and wire feeders are combined into one self-contained housing (figure 21). Additionally, a shielding gas regulator, flow meter, and hoses are required. The wire feeder mechanically drives the electrode through the gun and cable system.

For steel applications, GMAW is performed with direct current and with the electrode positively charged (DC positive). Traditionally, GMAW has used constant voltage (CV) power supplies. Some of the advanced GMAW power supplies for pulsed spray transfer are more like constant current (CC) machines.

Figure 21. Photo. Self-contained GMAW wire feeder/power supply.

3.5.4. Consumables

Usually GMAW is performed with solid electrodes or metal-cored composite electrodes. The solid electrodes, also known as “hard wire”, are normally 0.035 to 0.052 inch in diameter, and metal-cored electrodes are typically 0.045 to $^{5}/_{64}$ inch in diameter. However, in both cases, other sizes of electrode can and have been used. GMAW solid electrodes typically have a light copper plating on the surface to improve electrical contact between the wire and the contact tip.

Metal-cored electrodes are similar to flux-cored electrodes in that both are tubular, but they are different in that the core material for metal-cored electrodes is metal powder and does not contain slag-forming ingredients. The resulting weld is virtually free of slag, just as with solid wire GMAW. Metal-cored electrodes typically have increased ability to handle mill scale and other surface contaminants as compared to solid GMAW electrodes. For a given current (amperage), metal-cored electrodes offer higher deposition rates than solid electrodes. However, metal-cored electrodes are, in general, more expensive than solid electrodes. For a time, metal-cored electrodes were classified as FCAW electrodes; presumably, this was on the logic of classifying cored wires together. Later this was changed, simply because metal-cored electrodes do not have flux in their cores.

3.5.5. Electrode Classification

GMAW electrodes are covered by AWS A5.18 (AWS, 2017), AWS A5.28 (AWS, 2005a) and AWS A5.36 (AWS, 2016) filler metal specifications. AWS A5.18 addresses carbon steel electrodes (solid and metal-cored) and AWS A5.28 similarly addresses the low alloy counterparts. AWS A5.36 covers metal-cored GMAW electrodes for both carbon steel and low-alloy steel applications as well as addressing electrodes for FCAW, and will be discussed more thoroughly in section 3.6.

In AWS A5.18 and A5.28, for solid electrodes the classification is based on the electrode composition, whereas for metal-cored electrodes, it is based on the weld deposit. In both cases, the mechanical properties are based upon tests made on deposited weld metal. The GMAW electrode classification system in AWS A5.18 and A5.28 consists of ERXXY-Z, where:

ER or E	“ER” indicates a solid electrode or rod; “E” is used for a metal-cored electrode
XX	specifies the minimum specified tensile strength in ksi
Y	“S” indicates a solid electrode; “C” indicates a cored electrode
Z	designator to describe the electrode or deposit chemistry and CVN properties

As an example, for the electrode class ER70S-3, “ER” indicates that this is either an electrode or rod used for GMAW, the “70” reflects a deposited weld metal minimum specified tensile strength of 70 ksi; “S” indicates that this is a solid electrode; and “3” represents specific chemical properties and a CVN toughness of 20 ft-lb at 0 °F.

3.5.6. Shielding Gas

A variety of shielding gases or gas mixtures may be used for GMAW. The selection of gas type primarily depends on the desired mode of metal transfer and cost. Carbon dioxide is the lowest-cost gas, but it cannot be used for spray and pulsed spray transfer. Also, welding with pure carbon dioxide typically results in high spatter levels. Argon-based mixtures of gas can be used for all modes of transfer, with less spatter. Argon-based gas mixtures are considerably more expensive than pure carbon dioxide. The selection of the shielding gas can affect the weld penetration and penetration profile, as well as the overall appearance. Figure 22 compares welds made with various shielding gases at the same wire feed speed and voltage.

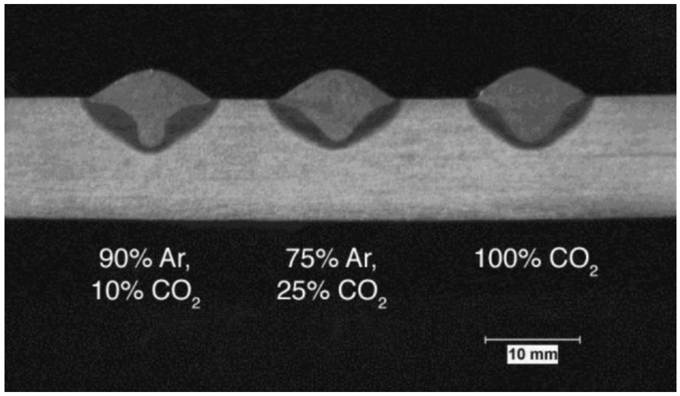

Figure 22. Photo. Effect of shielding gas on penetration profiles.

For GMAW, welding with pure argon shielding gas is not used on bridges; rather, small quantities of carbon dioxide, oxygen, or both are added. Although shielding gas is used to displace atmospheric nitrogen and oxygen, it is possible to add small quantities of oxygen into mixtures of argon, generally at levels of two to eight percent. This helps stabilize the arc and decreases puddle surface tension, resulting in improved wetting (blending of the weld metal and base metal at the weld toe). Tri- and quad-mixes of argon, oxygen, carbon dioxide, and helium are also available, offering advantages such as improved arc action, better weld appearance, and reduced fume generation rates.

Specific gas mixtures are designated for use with specific filler metals because shielding gas may influence the properties of the deposited weld metal. Documentation indicating the suitability of gas and filler metal combinations is typically available from the electrode manufacturer or gas supplier.

3.5.7. Modes of Transfer

There are four common modes of metal transfer associated with GMAW: globular transfer, spray transfer, pulsed spray transfer, and short circuiting transfer. The first three are used for steel bridge fabrication, with spray and pulsed spray being the most common. The choice of transfer mode is part of the contractor’s means and methods; however, the engineer should be aware that the fourth mode of transfer, short circuiting, poses some unique concerns. Hence, all four modes of transfer are discussed below.

Globular Transfer:

Globular transfer is “the transfer of molten metal in large drops from a consumable electrode across the arc.” (AWS, 2010c). This mode of transfer occurs when high concentrations of carbon dioxide shielding gas are used. Globular transfer is characterized by deep penetration and relatively high levels of spatter. Weld appearance can be poor and it is restricted to the flat and horizontal positions. Globular transfer may be preferred over spray transfer because of the low cost of carbon dioxide shielding gas and the lower level of heat felt by the operator.

Spray Transfer:

Spray transfer is a mode of transfer "…in which molten metal from a consumable electrode is propelled axially across the arc in small droplets." (AWS, 2010c). The fine molten droplets are smaller in diameter than the electrode diameter (figure 23). Spray transfer is characterized by high wire feed speeds at relatively high voltages, and a high level of energy transferred to the work, creating a large puddle. Because of this large puddle, spray transfer is only suited to the flat and horizontal positions. In these positions, spray transfer generates high-quality welds with particularly good appearance, and practically no spatter. Outside of these positions (in overhead or vertical positions), gravity will deform the molten weld before solidification occurs.

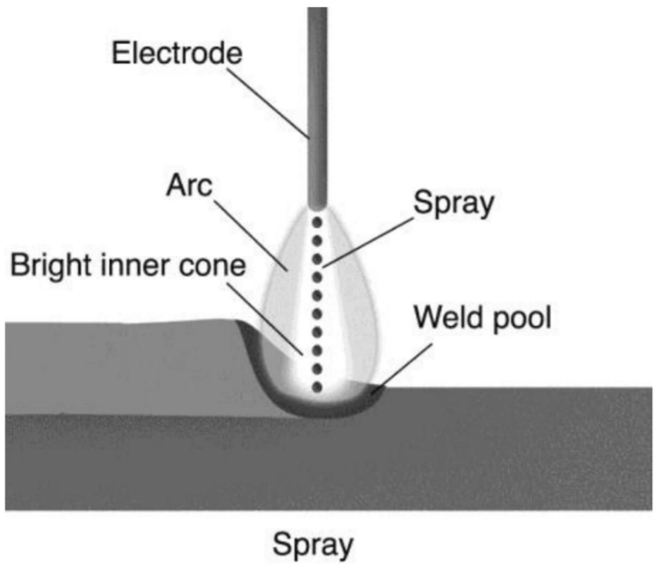

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Figure 23. Illustration. GMAW spray transfer.

The shielding used for spray transfer is composed of at least 80 percent argon, with the balance made up of either carbon dioxide or oxygen. Typical mixtures include 90/10 argon-carbon dioxide, and 95/5 argon-oxygen. Other proprietary mixtures are available from gas suppliers.

The current required to achieve spray transfer is called the transition current. Each combination of electrode diameter and shielding gas has a different transition current. Increased levels of argon in the gas mix will result in a lower transition current; a higher carbon dioxide level corresponds to a higher transition current.

Pulsed Spray Transfer:

Pulsed spray transfer is "[a] variation of spray transfer in which the welding power is cycled from a low level to a high level, at which point spray transfer is obtained, resulting in a lower average voltage and current." (AWS 2010b). The process uses a background current, which is continuously applied to the electrode, and a pulsing peak current that momentarily forces spray transfer. Metal transfer occurs during the pulse. The pulsing rate is optimally applied as a function of the wire feed speed, and ideally, a single droplet of metal is transferred by the pulse.

After the droplet is released, the power supply then delivers a lower background current, which maintains the arc. This occurs between 100 and 400-times-per-second. This mode of transfer is sometimes abbreviated as GMAW-P, and may be called “pulsed arc” or “pulse”.

One advantage of pulsed spray transfer is that it can be used to make welds “out of position”. For flat or horizontal work, GMAW-P may not be as fast as spray transfer. However, used in vertical or overhead positions, it is free of the problems associated with the short circuiting mode. Weld appearance is good, and quality can be excellent. The disadvantage of pulsed arc transfer is that the equipment is more complex and costly than that required for other modes of transfer. However, with advances in equipment technology, the machines have become easier to use and the popularity of this mode of transfer is increasing.

Many advanced pulse waveforms have been developed over the years. Pulsed waveforms vary in terms of peak current, background current, and time at those levels; frequency of pulsing; and the rate at which the current levels are reached and maintained. Pulsing waveforms may be developed to meet one specific application need such as higher travel speeds, decreased spatter levels, better penetration profiles, or all of the above.

Short Circuiting Transfer:

Short circuiting transfer is a “[m]etal transfer in which molten metal from a consumable electrode is deposited during repeated short circuits.” (AWS, 2010c). It is abbreviated as GMAW-S and is a low-energy mode of transfer, ideal for welding on thin-gauge materials. It can be used for all-position GMAW and is sometimes called “short arc” welding.

In this mode of transfer, a small-diameter electrode, typically 0.035 or 0.045 inch, is fed at a moderate wire feed speed with relatively low arc voltages. The electrode will touch the workpiece, creating a short in the electrical circuit. Once the electrode shorts, the arc is extinguished. At this point, the current increases dramatically, superheating the electrode and causing it to melt. Just as excessive current flowing through a fuse causes it to blow, so the shorted electrode will heat and melt, breaking the electric short and initiating a momentary arc. A small amount of metal will be transferred to the work at this time. This cycle will repeat itself when the electrode once again shorts to the work. This occurs somewhere between 20 and 200 times per second, creating a characteristic “buzz” to the arc.

3.5.8. Advantages and Limitations

GMAW, regardless of the mode of transfer, has some inherent advantages and limitations. Because no slag covers the weld, cleanup is simple, and the process is particularly suitable for multipass robotic welding. GMAW electrodes typically cost less than flux-cored electrodes.

An overall limitation of GMAW, regardless of mode, is that the process is more sensitive to contaminants that might be present on the steel surface, including mill scale, rust, and oil. GMAW can handle some of these contaminants, but other processes with slag systems can typically tolerate greater levels of such materials. Porosity may appear when surfaces are too contaminated, and heavy scale may inhibit fusion. Steel that is cleaned of mill scale and contaminants can be readily welded with GMAW.

Additional advantages and limitations of GMAW depend on the mode of transfer. Spray transfer permits higher deposition rates and deposits welds with good appearance, but requires the use of the higher-cost argon-based shielding gas mixtures, and can be used only in the flat and horizontal positions.

Pulsed spray arc permits all-position welding and deposits welds with good appearance. Like spray transfer, this mode requires the use of the more expensive argon-based shielding gas mixtures. The welding equipment is more expensive and complex, but technical advances in power source controls have simplified the user interfaces.

Globular transfer uses low-cost carbon dioxide shielding and offers high deposition rates, but weld appearance is inferior to spray transfer, and extensive spatter is typical. The mode is also restricted to the flat and horizontal positions.

Short circuiting transfer is a very low heat input process that is ideal for thin material but, because of this low heat input, is not reliable for achieving fusion in thicker sections. Therefore, short circuiting is not permitted for use on bridges by the Bridge Welding Code. However, concerns about short circuiting do not justify prohibition of GMAW entirely, and there are provisions for its use in the Code. Short circuit transfer is readily avoided through use of proper welding procedures, and GMAW's other modes of transfer are very beneficial in bridge fabrication.

3.6. FLUX-CORED ARC WELDING (FCAW)

This section addresses unique aspects of FCAW. Aspects of FCAW that are the same as or similar to GMAW are addressed in section 3.4.

3.6.1. Applications

FCAW is "[a]n arc welding process using an arc between a continuous filler-metal electrode and the weld pool. The process is used with shielding gas from a flux contained within the tubular electrode, with or without additional shielding from externally supplied gas, and without the application of pressure." (AWS, 2010c). FCAW has been used in bridge fabrication for many decades, and is popular for shorter welds, such as stiffener to flange fillet welds or small complete joint penetration groove welds that do not lend themselves to mechanization.

3.6.2. Fundamentals

A flux-cored electrode comprises a cored tubular wire containing a combination of materials that includes flux and sometimes metal powders. The flux ingredients inside the electrode perform the same function as the flux on the outside of an SMAW electrode and form a slag cover over the weld bead.

Within the category of flux-cored arc welding, there are two subcategories: gas-shielded FCAW (FCAW-G, figure 24) and self-shielded FCAW (FCAW-S, figure 25). Self-shielded flux-cored electrodes require no external shielding gas; the entire shielding system is generated from the flux ingredients contained within the core of the tubular electrode. The gas-shielded versions of flux-cored electrodes use an externally supplied shielding gas. Self-shielded flux-cored electrodes are better suited for field-welding situations where wind may displace the shielding gas required for FCAW-G. The FCAW-G process tends to be more operator-friendly, and is generally preferred in situations where the gas shielding can be protected from disruption.

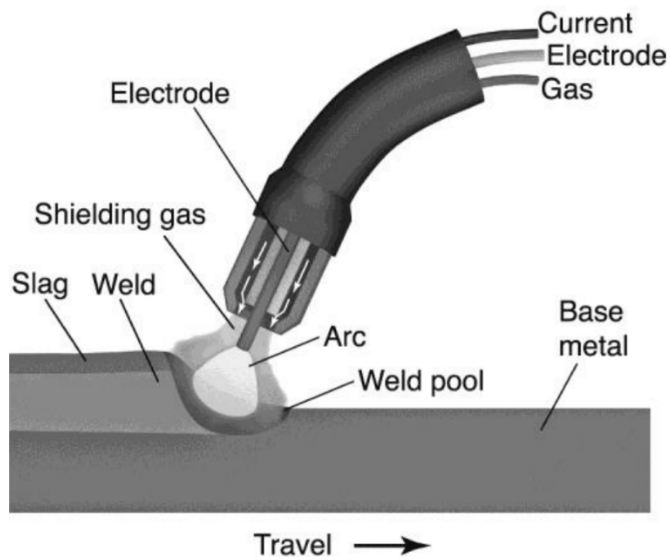

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Figure 24. Illustration. Gas-shielded FCAW process.

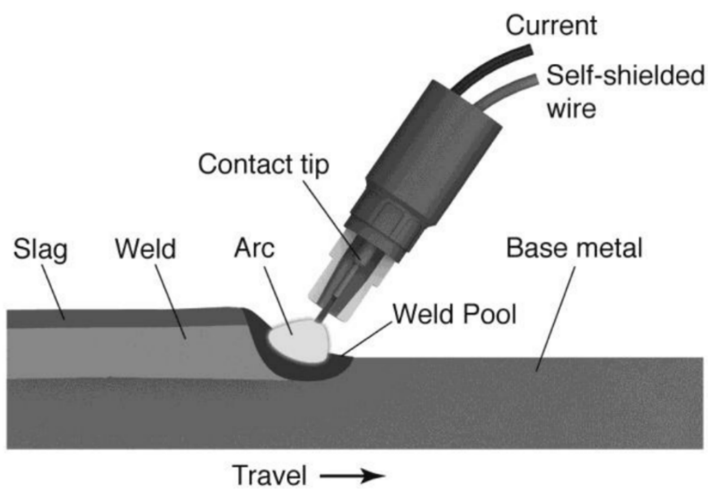

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Figure 25. Illustration. Self-shielded FCAW process.

3.6.3. Equipment

FCAW is typically and optimally performed using a constant voltage (CV) power supply. FCAW can also be performed with a constant current (CC) system. Some contractors may choose to use CC for FCAW when they convert from SMAW to FCAW, desiring to continue to use their existing SMAW power supplies. This is often the case for contractors with engine-driven welding equipment, which is relatively expensive to replace. Welding results under CC are dependent not only on the specific CC output characteristics of the power supply, but also on the specific electrode used. These factors are noted in the welding procedure to be used.

3.6.4. Consumables

FCAW electrodes are always tubular, with a metallic tube that surrounds the internal flux. The diameter of such electrodes ranges from 0.030 to $^{1}/_{8}$ inch, with 0.045 to $^{1}/_{32}$ inch being typical for steel bridge work. Electrodes that are $^{5}/_{64}$ inch and smaller are often used for out-of-position work (i.e., vertical and overhead) while those that are $^{1}/_{16}$ inch and larger are generally used for flat and horizontal welds. The various electrodes are wound on spools, coils, or reels, or inserted into drums. The package size is typically dictated by the balance between the need for portability and a desire to limit the number of electrode changes (figure 26).

For FCAW-G, an additional consumable is the shielding gas. Most of the gas-shielded flux-cored electrodes use carbon dioxide or argon-carbon dioxide mixtures for the shielding media. The shielding gas may affect mechanical properties. This is largely due to the difference in alloy recovery—that is, the amount of alloy transferred from the filler material to the weld deposit (as opposed to the slag). The shielding gas selected should be that required of the filler metal classification, or supported by the filler metal manufacturer's recommendations or suitable test data.

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Figure 26. Photo. FCAW electrode packaging.

3.6.5. Electrode Classification

AWS A5.20 (AWS, 2005), AWS A5.29 (AWS, 2010b), and AWS A5.36 (AWS, 2016) specify the requirements for FCAW filler metals. AWS A5.20 covers carbon steel electrodes, AWS A5.29 addresses low-alloy steel materials and AWS A5.36 deals with both carbon steel and low-alloy steel electrodes.

AWS A5.20 and AWS A5.29 have existed for many years; AWS A5.36 is a newer specification, introduced in 2012, that combines provisions for all steel tubular electrodes, including those currently classified under AWS A5.20 and AWS A5.29, in addition to composite GMAW electrodes currently classified under A5.18 (AWS, 2017) and A5.28 (AWS, 2005a).

AWS A5.36 uses two classification systems: “fixed” and “open”. The “fixed classification” designation and requirements are the same as those specified in A5.18, A5.20, A5.28 and A5.29: each electrode has a “fixed” set of requirements, with a “fixed” designation. AWS A5.36 lists these electrodes.

The “open classification” system uses separate designators for usability, welding position, tensile strength, impact strength, shielding gas, heat treatment condition, and deposit composition. For gas-shielded processes, more classification options are available. The foreword in AWS A5.36 explains the changes in greater detail. The additional flexibility permitted by the “open classification” system permits a better description of the characteristics of the filler metal, albeit with a more complicated method for designating the electrode.

To assist in the transition from the original filler metal specifications to the new A5.36, a transition period was enacted during which time AWS allowed manufacturers to classify electrodes under both the new A5.36 specification and their original certification. However, there is no definitive plan for the retirements of the original specifications.

A5.20 classifies electrodes as EXYT-AB, where:

E	indicates an electrode
X	specifies the minimum specified tensile strength (in 10 ksi units)
Y	indicates the position in which the electrode may be used
T	indicates a tubular electrode
A	designator to describe the shielding type (gas or self-shielded), welding polarity and single versus multipass
B	“C” indicates shielding gas for classification is CO2; “M” indicates mixed gas

For the classification E71T-1C, the “7” conveys the same information as does the “70” for the SMAW of E7018; the minimum specified tensile strength of welds made with the filler metal under prescribed conditions is 70 ksi. The “1” means the electrode is suitable for use in all positions. A “0” at this location indicates the suitability of the electrode for welding in the flat and horizontal positions only. The final “1” in the E71T-1 example conveys a variety of information: the electrode is for FCAW-G (gas-shielded), operates on direct current with positive polarity (DC positive), has a rutile slag system, is suitable for single- and multipass welds, and must be capable of depositing weld metal with a minimum Charpy V-Notch toughness of 20 ft-lb at 0 °F. The “C” in the example indicates that carbon dioxide shielding gas is used for the classification of this electrode. An “M” in this location would indicate the use of a mixed gas (e.g., argon/carbon dioxide).

Under AWS A5.29 for low-alloy electrodes, a suffix letter or letters followed by a number appears after the designation described above for A5.20. Common designations include “Ni1”, indicating a nominal nickel content in the deposited metal of 1 percent. Additional suffix designators may be used that indicate characteristics such as diffusible hydrogen limits.

3.6.6. FCAW-S Intermix Concerns

Different welding processes are often combined in a single joint for a variety of reasons. For example, tack welding may be done with SMAW, and the rest of the joint may be filled with FCAW. Under most circumstances, such intermixing of processes causes no difficulty. However, FCAW-S poses a specific exception.

For most arc welding processes, the molten weld pool is shielded by shielding gas, slag, or a combination of the two. FCAW-S is different in that it produces very little shielding gas, but rather relies on the addition of large amounts of deoxidizers and denitrifiers to the weld pool to react with oxygen and nitrogen. Aluminum is the primary element used for this purpose, but titanium and zirconium may also be used. The balance between aluminum and nitrogen, as well as carbon and other alloys, must be properly maintained to ensure that specified mechanical properties are obtained in the weld metal. Because of the relatively high amount of aluminum and magnesium present in FCAW-S, mixing other processes, including FCAW-G, with FCAW-S in a single weld joint creates the potential for negative interactions. Welding over a deposited FCAW-S weld can break apart the aluminum and magnesium compounds, and the presence of these compounds in the subsequently deposited weld can have a negative impact on the weld mechanical properties, and in particular the CVN toughness (FEMA, 2000 and Quintana, 1998). Hence, the Bridge Welding Code requires special qualification testing to determine compatibility when mixing FCAW-S with other processes.

3.6.7. FCAW-G Advantages and Limitations

Comparing FCAW-G and FCAW-S, gas-shielded flux-cored electrodes provide better arc stability and greater ease of maintaining a uniform shape than self-shielded flux-cored electrodes. Hence, they are well-suited to a broad variety of applications in the shop. Weld appearance and quality are very good. Higher-strength gas-shielded FCAW electrodes are available, while the technology current as of this publication limits self-shielded FCAW deposits to 90 ksi tensile strength or less.

For structural applications, the primary limitations of FCAW-G are related to the need for shielding gas. The Bridge Welding Code limits the maximum wind velocity around an arc to 5 mph. A shelter or screen can be erected to limit such wind, if necessary. However, when welding is performed under windy conditions with FCAW-G, porosity is likely.

3.6.8. FCAW-S Advantages and Limitations

For welding under field conditions where wind may disturb the gas shielding, FCAW-S is ideal. Welds have been made under conditions simulating wind speeds of 10 mph without harmful effects (FEMA, 1997). Some fabricators have found FCAW-S offers advantages for shop welding as well, particularly for relatively open fabrication shops (open shop doors, buildings without walls, etc.).

Because no external shielding gas is required, there is no need for gas cylinders, hoses, and regulators, and no need to move them around a shop or jobsite. Gas nozzles associated with FCAW-G can become plugged with welding spatter, while this is of no concern for FCAW-S. The gun and cable assembly is simpler and less obstructive for FCAW-S, making it more suitable for welding in some confined spaces.

3.7. ELECTROSLAG WELDING (ESW) AND ELECTROGAS WELDING (EGW)

Because of their similarities, electroslag and electrogas welding are discussed together in this section.

3.7.1. Applications

Electroslag welding (ESW) and electrogas welding (EGW) are ideal for welding on thicker materials, typically 1 inch thick or greater. Materials 12 inches thick and greater have been welded with ESW using multiple electrodes. However, ESW is not well suited for use on thinner materials because traditional processes are more efficient.

While use of EGW is not common in bridges, ESW is used for splicing bridge flanges. Further, ESW is sometimes used in CJP weld T- and corner joints; in such applications, ESW can be more efficient and also help minimize the welding distortions which are more pronounced with multipass processes.

Currently, the Bridge Welding Code does not allow for the use of ESW or EGW for fracture-critical members or high-performance steel and does not allow EGW for welding quenched and tempered steels (which includes most grades of high-performance steel) or joints in tension or stress reversal.

3.7.2. Fundamentals

ESW is "[a] welding process producing a coalescence of metals with molten slag, melting the filler metal and the surfaces of the workpieces… The process is initiated by an arc that heats the slag. The arc is then extinguished by conductive slag, which is kept molten by its resistance to electric current passing between the electrode and the workpieces." (AWS, 2010c). The process is illustrated in figure 27. EGW is "[a]n arc welding process using an arc between a continuous filler metal electrode and the weld pool, employing approximately vertical welding progression with backing to confine the molten weld metal. The process is used with or without an externally supplied shielding gas and without the application of pressure." (AWS, 2010c). The process is illustrated in figure 28. Both processes are most typically used for welding joints in a vertical position and an upward progression (although they can be operated in the flat position for cladding), with the weld pool contained by backing (usually in the form of removable "shoes") on the sides of the weld. Groove welds in butt and T joints are the most common applications for these processes. Both practices are found in the Bridge Welding Code, but use of EGW is not common in bridge fabrication.

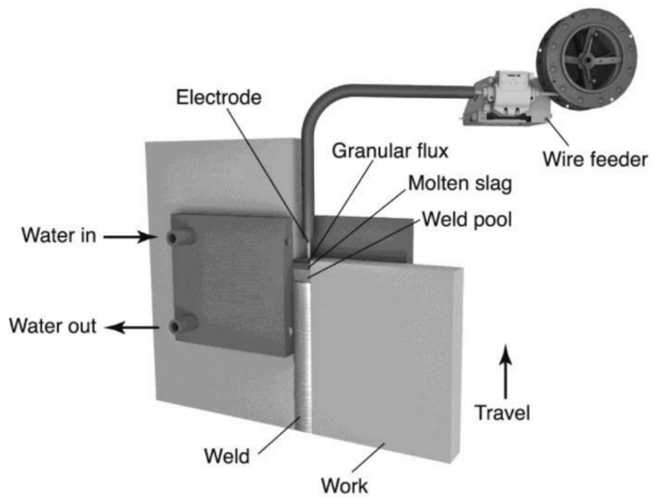

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Figure 27. Illustration. ESW process.

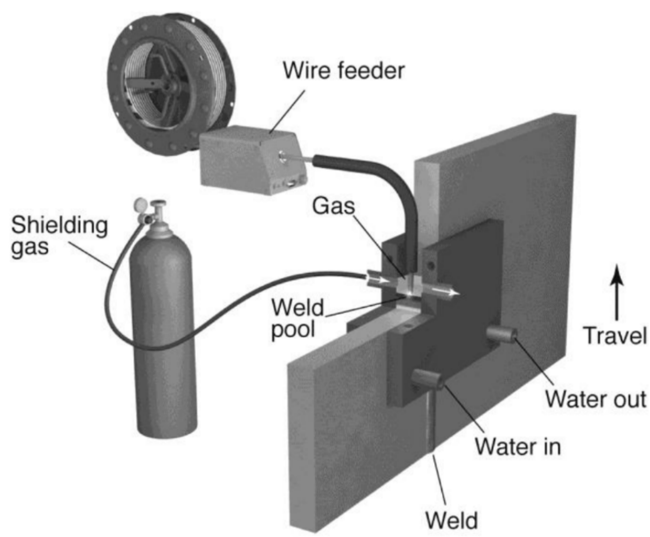

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Figure 28. Illustration. EGW process.

Although ESW and EGW are used in similar applications, the means by which the electrodes are melted is fundamentally different. The apparatus needed for electroslag and electrogas welding are mechanically similar in that both use backing to contain the weld metal in the joint. Such backing may be copper or steel. Copper is often used for reusable shoes because, despite copper having a lower melting point than steel, the weld metal freezes at the copper surface due to the high thermal conductivity of the copper. Such shoes can be water-cooled (as required in the Bridge Welding Code) or solid blocks of copper that retain all of the welding thermal energy. The copper shoe is removed from the joint after the weld has solidified and cooled. When steel backing is used, it is fused to the weld, and is typically left in place, becoming part of the weldment.

ESW is not an arc welding process, but a resistance welding process. Initially, when ESW is started, it functions like SAW; an arc buried under the flux melts the base metal, filler metal, and flux, forming a slag. Unlike for SAW, the slag for ESW is electrically conductive. After a slag blanket is established, the welding changes to a resistance process, with the electrical current conducted from the electrode, through the slag, and into the workpiece. The high currents transferred through the slag keep it hot. As the electrode is fed through this hot slag, it melts, and molten metal drips from the electrode into the weld pool. No arc is involved except when the process is started.

EGW is also performed vertically up in a single pass. A solid, flux cored or metal cored electrode is fed through a conductor and into the groove, often with an oscillation of the wire within the groove, and an arc is established between the electrode and the weld pool. A shielding gas protects the weld pool (although there is a gasless variation of this process that uses FCAW-S wire). The arc is maintained throughout the process.

Specifically for bridges, a variation of ESW known as “narrow gap” (ESW-NG), was developed in the 1980s and 1990s through research sponsored by the FHWA. The research was prompted by a brittle fracture of an improperly repaired ESW weld on a highway bridge in the 1970s, causing FHWA to impose a moratorium on the use of ESW for bridge tension applications. The new process was originally called “NGI-ESW” for “narrow gap improved”; later this was changed to “ESW-NG”. ESW-NG addresses the concerns of the traditional ESW process. The variation uses a narrower root opening of approximately ¾ inch, a tubular electrode, and a winged consumable electrode guide that helps to distribute thermal energy across the joint. The root opening was made narrower to speed deposition, thereby reducing heat input. Electrode oscillation is not permitted in this version of ESW. The new practice has improved weld and HAZ toughness, is faster and far less susceptible to piping porosity, uses less energy, and has a lower heat input than the original process. Requirements for ESW-NG were first included in the Bridge Welding Code in 2010; the version of the ESW process required by the Code is the narrow gap process, but it is not always labeled “NG”. There is an informative annex in the Bridge Welding Code, “Guide for the Use of Electroslag Welding—Narrow Gap (NG),” that provides information about this special process.

In addition to the codified ESW-NG process, there is an annex in the Bridge Welding Code that describes a practice for qualifying alternate ESW processes. For completeness, this manual also discusses conventional ESW (other than ESW-NG).

3.7.3. Equipment

Equipment for ESW and EGW consists of a power supply, wire feeder, flux delivery system (for ESW) or gas delivery system (for EGW), appropriate power and work leads and connections, an apparatus to support the electrode with respect to the joint, and fixturing to hold the backing in position. Lastly, the water-cooled copper shoes need a source of cooling water.

3.7.4. Consumables

Consumables for ESW in the Bridge Welding Code consist of cored electrodes, fluxes, and consumable guides. Consumable guides are also known as guide tubes; the Bridge Welding Code uses the term “consumable guide”, but in bridge practice the guides are still informally referred to as “guide tubes” even though the guides used for ESW-NG are not round. Consumables for conventional ESW consist of solid electrodes, fluxes, and guide tubes; EGW consumables include solid or metal-cored electrodes, shielding gas, and guide tubes.

3.7.5. Electrode Classification

Unique in the Bridge Welding Code, electrodes for the ESW-NG process do not fall under an A5 specification but rather must conform to a mandatory annex of the Bridge Welding Code that describes chemical composition requirements for ESW electrodes, consumable guides, and fluxes. AWS A5.25 (AWS, 1997a) addresses electrodes for conventional ESW flux and wire, and AWS A5.26 (AWS, 1997) addresses electrodes for EGW. Solid ESW (for conventional ESW processes) and EGW electrodes are classified based upon the composition of the electrode. Composite (cored) electrodes for these processes are classified based upon the deposited weld metal chemistry. Mechanical property requirements are based upon tests made from deposited weld metal.

Flux–electrode combinations for conventional ESW have classifications that follow the pattern FESXYZZZ-EW, where:

- **FES** indicates a flux for electroslag welding
- **X** indicates the minimum specified tensile strength in units of 10 ksi
- **Y** indicates the CVN test temperature (where *Y* is multiplied by −10 °F) at or above which the weld metal meets the specified CVN properties
- **ZZZ** electrode designation, based on chemistry
- **EW** “EW” suffix indicates electroslag welding

For EGW the classifications follow the pattern EGXYZ-A, where:

- **EG** indicates electrogas filler metal
- **X** indicates the minimum specified tensile strength in units of 10 ksi
- **Y** indicates the CVN test temperature (where *Y* is multiplied by −10 °F) at or above which the weld metal meets the specified CVN properties
- **Z** “S” designates a solid electrode; “T” designates a tubular electrode
- **A** defines the chemistry of the electrode

3.7.6. Advantages and Limitations

Very high deposition rates can be obtained with ESW and EGW, leading to productivity gains. Typically, ESW progresses at about 2 inches per minute, regardless of the thickness of the components being joined; hence, for example, the actual welding of a 30-inch-wide, 3-inch-thick joint can be completed in 15 minutes, although the time for of setting up and breaking down ESW is much longer. Using the SAW process, with many weld passes and also turning the work and preparing the back side for welding, could take an entire shift, so the gain in productivity is obvious.

Normally, ESW or EGW joint details involve square edge preparations, eliminating plate beveling costs. Material handling is reduced since plates do not need to be turned over to complete welding, as is the case for double-sided welds. However, special fixtures are needed because flange plates being joined with ESW must be supported in the vertical position. Angular distortion is reduced, as compared to traditional multipass full-penetration welds. The equipment and associated fixturing for ESW/EGW are more expensive and less flexible than those associated with other processes.

ESW and EGW have the advantage of being automatic processes, and when they are properly set up, consistently obtain good results. However, different variables are involved as compared to other processes. Fit of the copper shoes to the work, the temperature of the shoes, and the thickness of the slag layer are all factors that must be controlled in order to obtain quality welds. Further, unlike other processes where travel speed is adjusted independently of other parameters, the travel speed (or rate of rise) is a function of the groove area and the amount of wire being fed into the weld puddle, which in turn is associated with the wire feed speed and amperage being used.

3.8. OTHER WELDING PROCESSES

Shear stud welding is another welding process that is common in bridges but unique compared to other processes. Shear stud welding is addressed in section 11.2.

Resistance welding processes generate thermal energy by passing electrical current through the workpieces, where the interface between the two pieces constitutes a point of high electrical resistance. In addition, pressure is applied between the two workpieces, resulting in the need for significant fixturing that is only practical when repeatedly making the same part. Generally, then, resistance welding (other than the unique case of ESW) is not used in bridges; however, the longitudinal butt seam on structural tube and pipe is typically welded with the electric resistance welding process.

Gas-tungsten arc welding (GTAW) is "[a]n arc welding process using an arc between a tungsten electrode (non-consumable) and the weld pool. The process is used with shielding gas and without the application of pressure." (AWS, 2010c). The process is illustrated in figure 29. Unique among the other arc welding processes discussed in this manual, the electrode in this process is not consumed; rather, it simply introduces the arc that melts the base metals being joined and the filler metal, which is optional. The process is slow and not common in bridges except in unique applications, such as attaching a thin stainless steel sheet for some special purpose. It is not addressed in the Bridge Welding Code, though it is referenced in the AWS D1.1 Structural Steel Welding Code.

In the remaining broad categories of solid-state welding and other processes, there are no processes that are commonly applied to structural steel fabrication but typically not used in bridges. Solid-state welding processes include exotic and interesting processes like laser welding, diffusion welding, explosion welding, friction welding, ultrasonic welding, electron beam welding, and thermite welding. Most of these processes are highly specialized and have not found commercial application in the structural steel industry.

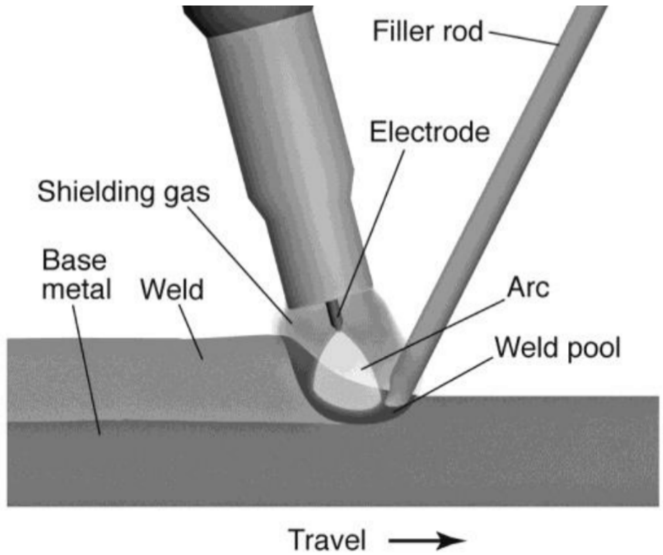


Figure 29. GTAW process.

CHAPTER 4 - QUALIFICATION OF WELDING PROCEDURE SPECIFICATIONS AND WELDERS

Welding in bridge fabrication is performed by qualified welders in accordance with welding procedure specifications that are developed and qualified by fabricators and approved by owners. A welding procedure specification is a set of instructions followed by the welder to produce the weld. The welder's use of these instructions achieves three objectives:

- **Quality -** The welding procedure prescribes requirements intended to ensure quality.
- **Conformance -** An approved welding procedure provides a set of welding instructions that are compliant with the project specifications and applicable codes.
- **Productivity -** The welding procedure codifies the operational choices that the fabricator makes to achieve the required weld in the most productive way.

There are two key terms to understand regarding the procedures used for bridge welding and their qualification under the Bridge Welding Code**:**

- **Welding Procedure Specification (WPS) -** "A document providing the required welding variables for a specific application to assure repeatability by properly trained welders and welding operators." (AWS, 2010c). This is the Bridge Welding Code name for what is also more generally known as a "welding procedure".
- **Procedure Qualification Record (PQR) -** "A record of welding variables used to produce an acceptable test weldment and the results of tests conducted on the weldment to qualify a welding procedure specification." (AWS, 2010c). The term "PQR" has become slang for the groove weld test itself in contrast to its actual meaning as a record of the test results. Also, it is most typically used to describe the record of the D1.5 figure 5.1 groove weld test (or the test itself), as opposed to other tests such as fillet weld soundness tests or nonstandard joint qualification tests.

This chapter describes the practices fabricators follow to develop and qualify welding procedures and the practices owners follow to approve welding procedures under the Bridge Welding Code. Also discussed are welder qualification tests. Specific topics include:

- Welding procedure specifications (WPSs) and the steps fabricators follow to develop them
- WPS qualification methods used in the Bridge Welding Code
- WPS qualification tests associated with the Bridge Welding Code WPS qualification methods
- Procedure qualification records (PQRs), in which fabricators document the results of welding procedure qualification tests
- Joints used for groove welds in the Bridge Welding Code, including standard joints and the testing associated with qualifying WPSs with nonstandard joints

- Recommended practices for review and approval of WPSs (and PQRs); these recommendations are tied to appendix A, which provides checklists for review of WPSs and various qualification elements, including PQRs, fillet weld soundness test reports, and nonstandard joint qualification test reports
- Qualification of welding personnel

4.1. WELDING PROCEDURE SPECIFICATIONS (WPSs)

4.1.1. Description

The Bridge Welding Code requires that all welding be performed under a WPS approved by the engineer. A WPS, with an example shown in figure 30, includes:

- Identification - name of WPS, fabricator, date, and possibly project information
- Applicable base materials
- Welding process, consumable name, and consumable classification
- Welding parameters and electrode size used for various welding passes
- Preheat and interpass temperature requirements
- Joint details and welding position
- Any special processing, such as backgouging, other root treatment or grinding flush, which are common, or special weld treatment, such as postheat, which is not common

Welding Procedure for Prequalified Joint:

WP105

Material Specification	**ASTM A709 Grade 50W**
Welding Process	**Submerged Arc Welding**
Manual, Semiautomatic or Machine	**Semiautomatic or Machine**
Filler Metal Specification	**AWS 5.23**
Weld Metal Classification	**F8A2-ENI1K-NI1-H8**
Single/Multiple Arc	**Single Arc**
Root Treatment	**Remove all scale, rust & contaminants and grind to sound metal**
Electrical Stickout	**1"**
Shielding Gas	**N/A**
Required Preheat	**See preheat chart below**
Kilojoules Per Inch	**35.1-62.3 / 50.3-93.4**

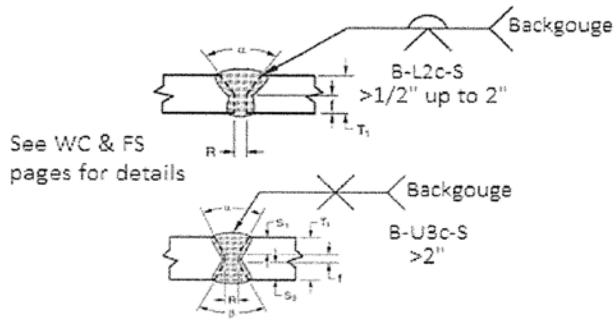
Revision:	
Original Issue:	

Pass #	Position	Amps	Wire Feed Speed (IPM)	Volts	Travel speed (IPM)	Polarity	Wire & flux combo.	Wire dia.	Gas flow (CFH)
1	1G	370-440	73-90	30-34.2	14.5-19	DC+	Lincoln LA-75 / 960	3/32"	N/A
2+	1G	450-550	93-125	33.5-38.5	13.6-18				N/A

Used for Flange and Web Splicing

(Can also be used for approved repair welds)

Back gouge and grind to sound weld metal to ensure complete removal of root pass



Minimum Preheat requirements.

Thickness at thickest part at point of welding. (Inches)	ASTM A709 Grade 50W
To 3/4 inclusive	50°F
Over 3/4 to 1-1/2 inclusive	70°F
Over 1-1/2 to 2-1/2 inclusive	150°F
Over 2-1/2	225°F
Maximum Interpass = 550°F	

Qualified in accordance with PQR:		
	expiration:	
	expiration:	

(approval stamp)

Source: FHWA

Figure 30. Illustration. Sample WPS.

4.1.2. Fabricator Development of WPSs

WPSs originate with the fabricator's intent to weld a particular application. On a bridge project, the fabricator makes choices about how to complete the welds on the job. In each case, the fabricator wants to select the most efficient welding process, consumables, parameters, and, for groove welds, joint preparation (root opening, groove angle, root face, etc.) that will result in welds that satisfy the design and conform with the Code.

For most bridges, the required welds tend to be the same or very similar. Materials usually do not vary much (the Bridge Welding Code and AASHTO LRFD Bridge Design Specifications (AASHTO, 2017a) limit allowable materials). Girders usually have web splices and flange splices, web-to-flange fillet welds, and fillet welds joining stiffeners and connection plates to the girder. However, some structures have unique welds requiring specialized procedures. In other situations, fabricators want to improve an existing process even for typical welds.

When developing a new procedure, the fabricator first makes fundamental decisions about how to approach the weld, including which:

- Process to use
- Position to weld in
- Welding consumables to use, including product, electrode diameters, and, for SAW, number of electrodes
- Equipment to use for welding and also, as needed, fixtures for supporting the work to be welded (fixtures are usually not shown on a WPS but they are an essential part of decisions related to the fabricator's WPS development)
- Joint to use (for groove welding)

Having made these decisions, the fabricator will then establish the welding parameters, including:

- Amperage (or wire feed speed)
- Voltage
- Travel speed
- Polarity and
- Electrode extension

The fabricator will likely weld mockups to refine and confirm the chosen settings for the WPS.

Once a fabricator has established the settings for a desired WPS, tests are conducted to qualify the procedure. Under the Bridge Welding Code, the following tests are required for the weld type noted:

- *CJP and PJP groove welds*: groove weld test (section 4.3.2); also, if a nonstandard joint is to be used, then a joint qualification test (section 4.4.4)
- *Multipass fillet welds*: groove weld test (section 4.3.2) and fillet weld soundness test (section 4.3.5)
- *Single-pass fillet welds:* fillet weld soundness test (section 4.3.5)

Further information on these tests is presented in the following sections.

Under the Bridge Welding Code, once a WPS has been approved, the WPS may be used only by the fabricator who created and qualified the WPS (clause 5.2). If a fabricator subcontracts work to another, the subcontracted fabricator must have its own WPSs. Unless otherwise prescribed in the contract documents, the fabricator may use the approved WPS at any of the fabricator's facilities and on any project performed for the owner under whose authority the WPS is approved. Put another way, under the code, WPS approvals are fabricator-specific but neither plant-specific nor project-specific. However, some owners do require project-specific approval of WPSs.

There are a variety of forms used for WPSs. The Bridge Welding Code provides an example form, but this form is only there as a suggestion and is not mandatory. Rather, the code specifies what must be a part of the WPS without mandating a WPS form or how it is presented. The best practice is to allow fabricators the flexibility to use their own chosen form, particularly because they perform work for multiple owners on multiple projects. If a particular form is mandated on a project that is unusual to the fabricator, the fabricator would have to use a form that is unfamiliar to the fabricator's welders, causing unnecessary confusion. Fabricators should include space on the form for the owner's approval (which may be a stamp).

4.2. WPS QUALIFICATION AND CREATION OF THE PROCEDURE QUALIFICATION RECORD (PQR)

For most welding under the Bridge Welding Code, fabricators must perform testing to qualify the WPS (see section 4.2.1 regarding prequalified WPSs for which qualification testing is not required). The Bridge Welding Code requires qualification testing to "provide assurance that the weld metal produced by welding in accordance with the provisions of this code shall produce weld metal strength, ductility, and toughness" (clause 5.1.1). Though this statement does not encompass all qualification tests (for example, the fillet weld soundness test of clause 5.10), the code does prescribe requirements for each qualification test prescribed.

The results from WPS qualification testing are recorded on a procedure qualification record, or PQR. The PQR, with an example shown in figures 31 and 32, shows all details associated with the qualification testing, including the welding consumables, welding parameters, materials used for the test, and the test results.

A PQR may support more than one WPS. The PQR may support a broad range of variables under which multiple specific WPSs can be written. For example, the heat input methods of qualification (section 4.2.2.1) qualify broad amperage, voltage, and travel speed ranges whereby any number of WPSs with specific amperage, voltage and travel speed that fall within these ranges can be developed.

Bridge Fabricator A

WELDING PROCEDURE QUALIFICATION RECORD (PQR)

Test Number: PQR-1 Codes/Specifications: AWS D1.5-15
Test Date: 4/12/2018
Welding Process: SAW Filler Metal Specification: AWS A5.17
Test Position: 1G Test Plate/Joint Detail: Figure 5.1
Qualification method: 5.12.1

AWS Classification (Flux/Electrode): F7A2-EM12K
Manufacturer (Flux/Electrode): x 960/L61
Base Metal Thickness/Specification (1): 1" ASTM A709 Grade 50W, Heat no. 12345
Backing Bar Thickness/Specification: 3/8" ASTM A709 Grade 50W, Heat no. 67890
Preheat min/max, Interpass min/max: 70°F, 550°F

Electrode	Diameter	Amps	WFS	Voltage	Current/Polarity
1	3/32"	500	N/A	35.0	DCEP

Travel Speed: 17"/min. Contact Tip to Work Distance: 1"
Weld Passes: 30 Calculated Heat Input (KJ/inch): 61.8

Visual Inspection	Radiography	Ultrasonic Test
PASS	PASS	PASS

	Reduced Section Tensiles		All Weld Metal Tensile
Tensile Strength (psi):	76,000	76,000	73,000
Yield Strength (psi):	----	----	58,000
Elongation in 1 in. (%):	66	62	(2") 30
Reduction in Area (%):	62	62	74

Side Bend Tests (**4**)
PASS

Macros (**3**)
PASS

CVN Weld Impacts (Ft.-Lbs. @-20°F): 64, 34, 39, 72, 32 (Avg. 46)

Lab Testing by: ABC Testing
Test No: X1

Based on the above testing this material **meets** the requirements of the specifications.

Weld Tech: ____________ Witnessing Agency ____________

Reviewed/Approved By: ____________

Source: FHWA

Figure 31. Illustration. Sample PQR.

PROCEDURE QUALIFICATION RECORD WORKSHEET
PQR NUMBER__________

Welder's Name __________ ID__________ Welding Test Date __________

Process __________ Position__________ Joint Detail: ☐ Fig. 5.1 ☐ Fig. 5.2 ☐ Fig. 5.3 ☐ Fig. 5.8

Electrode(s) Mfg. Designation__________

AWS Electrode Classification __________ Electrical Stick Out __________

Flux Mfg. Designation __________ AWS Flux Classification __________

Postweld Heat Treatment: Temp.__________ Hold Time__________ Heating/Cooling Rate __________

		Diam.	Current	WFS*	Voltage	Current and Polarity
Electrode	(1)					
	(2)					
	(3)					

Shielding Gas __________ Dew Point __________ Flow Rate__________ Gas Cup Size__________

Travel Speed: Min. __________ Max.__________

Base Metal Specification and Thickness__________ Heat Number __________

Backing Metal Specification and Thickness __________ Heat Number __________

Preheat Temp.__________ Interpass Temp. Min. __________ Max.__________

			FILLER METAL	CURRENT						TEMPERATURE	
Pass Number	Layer	Process	Diam.	Type & Polarity	Wire Feed Speed	Amp	Volts	Travel Speed	Stick Out	Preheat	Interpass

*Optional

Page ________ of ________

For multiple electrodes list each electrode on separate line. For parallel electrodes show "2 @ ________" under number and diameter.
Preheat and interpass temperature measured at mid length of plates approximately 25 mm [1 in] from the weld center line.

State/3rd Party Witness __________ Mfr./Contractor __________

Date __________

Figure 32. Illustration. Sample PQR Worksheet (AASHTO/AWS, 2015).

The fabricator chooses the qualification method from the Bridge Welding Code based on the fabricator's preferences. These methods are presented in table 3 and described in more detail in the sections following the table.

Table 3. Welding procedure qualification testing methods in the Bridge Welding Code.

Qualification Method	Qualification Testing	Application
Prequalification (section 4.2.1)	The WPS is prequalified, with no testing required to verify welding parameters.	Under the Bridge Welding Code, only SMAW procedures in conformance with clause 1.3.2, and for fracture-critical welding clause 12.7.1, may be used without testing.
Fillet weld soundness test (section 4.2.3)	D1.5 figure 5.8 soundness test	WPSs for single-pass fillet welds require only fillet weld soundness tests; no groove weld qualification test required. WPSs for multipass fillet welds require soundness testing and also a groove weld qualification test (see "Heat input method" and "Production method" below).
Maximum heat input method (section 4.2.2.1)	D1.5 figure 5.1 groove weld test at maximum heat input	Applies to groove weld and multipass fillet WPSs.
Maximum–minimum heat input (section 4.2.2.1)	D1.5 figure 5.1 groove weld test at maximum heat input, and D1.5 figure 5.1 groove weld test at minimum heat input	Applies to groove weld and multipass fillet WPSs.
Production method (section 4.2.2.2)	D1.5 figure 5.1 groove weld test at similar parameters to the WPS	Applies to groove welds and multipass fillet welds WPSs. WPS parameters are similar to the qualification test parameters.
ESW qualification (section 4.2.4)	D1.5 5.1 groove weld test, with modified joint and thickness adaptions (section 3.2.4); some parameters similar to production weld	Applies to ESW groove welds
Pretest / verification method (section 4.2.2.3)	D1.5 figure 5.1 groove pretest (by 3rd party) D1.5 figure 5.2 groove verification test (by the fabricator)	Applies to groove weld and multipass fillet WPSs. Using either the heat input approach or the production method approach, a third party conducts the D1.5 figure 5.1 groove weld test, and then the fabricator performs the D1.5 figure 5.2 verification test.

Each method has limits that apply once the qualification tests are completed. The parameters controlled by these limits are known as “essential variables”; they differ by method, welding process (for example, a change of flux applies to SAW procedures but not to GMAW procedures), and other WPS characteristics. If the WPS is changed outside of prescribed limits for these essential variables, a new qualification test must be performed based on the change.

The rules regarding time limits of PQRs under the Bridge Welding Code have changed over time. Under the original edition of the code in 1988, non–fracture-critical PQRs were valid for three years, and when fracture-critical PQRs were introduced, they were valid for one year. These limits have been extended over time, and under the 2015 edition of the code, non-fracture-critical PQRs no longer expire, and fracture-critical PQRs are valid for five years.

Originally, time limits were adopted to ensure consistent welding and test results. However, many years of practice demonstrated consistency in test results and associated production welding, and so the time limits were removed or reduced. The owner can “order tests of … WPSs whenever there is evidence that unacceptable welds are being or have been produced” (clause 5.2.4). However, consideration should be given to whether or not a qualification test will address the concern.

4.2.1. Prequalification

The Bridge Welding Code allows most SMAW WPSs to be developed and approved without qualification testing (clause 5.11.1). The limits of prequalification for SMAW are described in clauses 1.3.2 and 5.11. In practice, it is rare to need qualification testing for SMAW because the WPSs usually fall inside the prequalification limits.

The Bridge Welding Code also allows WPSs for tack welds that are completely remelted by SAW to be developed and approved without qualification testing (clause 5.11.1). When tack welds are remelted, the deposited tack is completely consumed by the SAW passes; as a result, there is nothing that qualification testing will demonstrate about the original tack weld in the final weld.

However, the lack of requirement for qualification testing of SMAW and of remelted tack welds does not mean that other quality requirements do not apply. Tack welds that will be remelted and SMAW welds must still be made in accordance with approved WPSs and in conformance with the quality requirements of the code to help ensure the integrity of the final bridge product.

4.2.2. Qualification of WPSs for Groove Welds and Multipass Fillet Welds

The fabricator has choices for the qualification method that is required for groove WPSs and multipass fillet WPSs. WPSs for multipass fillets welds also require a fillet weld soundness test (section 4.3.5); however, since the 2015 edition of the code, WPSs for single-pass fillets do not require groove weld qualification testing. These choices are presented in the following subsections. The requirements apply to all processes except ESW, which has its own unique qualification, found in section 4.2.4.

4.2.2.1. Heat Input Qualification

The fundamental premise of the heat input qualification method (clauses 5.12.1–3) is that current, voltage and travel speed are combined into one factor known as "heat input" (see section 2.5.5). Current, voltage and travel speed are allowed to vary, within certain broad limits, provided the heat input calculated by the current, voltage, and travel speed used falls within heat input limits determined by the tests.

Heat input is calculated for this purpose as follows:

$$\text{Heat Input (kJ/inch)} = \frac{\text{Amperage} \times \text{Voltage} \times 0.06}{\text{Travel Speed (inch/min)}} \tag{2}$$

As seen in equation 2, increases in current and voltage and decreases in travel speed increase heat input. Conversely, decreases in current and voltage and increases in travel speed decrease the heat input.

Heat input (sometimes called "energy input") is a mathematical estimate of the amount of thermal energy that is introduced into the steel when the weld is made. Heat input affects the thermal cycles (heating and cooling) of the weld and thereby influences:

- Solidification and cooling rate of the weld metal
- Cooling rate of the HAZ
- Amount of grain refinement among the passes in multipass welds (discussed below)

Thermal cycles are also influenced by preheat and interpass temperature (section 5.2).

Changes in the thermal cycles experienced by the weld and the HAZ affect the resultant properties of these regions. Higher levels of heat input and associated slower cooling usually results in slightly decreased yield and tensile strength in the weld metal and slightly increased ductility. Lower heat input and faster cooling usually results in slightly increased yield and tensile strength in the weld metal and slightly decreased ductility.

The influence of grain refinement complicates the relationship of weld metal fracture toughness to heat input. In multipass welds, the reheating of the previously deposited weld metal refines the weld metal and generally improves the fracture toughness. When heat input is too low, there is little additional energy to reheat the previously deposited beads for refinement. When heat input is too high, cooling rates slow, leading to larger grain sizes, which is the antithesis of grain refinement. Also, the number of weld passes in a given joint is reduced when high heat input levels are used, reducing the number of refinement cycles possible.

Hence, mechanical properties vary with changes in heat input. In practice, qualification testing under the Bridge Welding Code has demonstrated that WPSs within qualified heat input ranges consistently provide the strength, ductility and fracture toughness required for bridge welds. Therefore, as an indication of the collective influence of amperage, voltage, and travel speed on weld metal properties, the heat input methods are effective for bridge WPS qualification.

Qualified heat input limits can be established in two different ways:

- **Maximum Heat Input Qualification** - Under this method, one qualification test plate (section 4.3.2) is welded and the heat input of this test becomes the maximum heat input qualified. The minimum heat input qualified is 60 percent of the maximum heat input value.
- **Maximum-Minimum Heat Input Qualification** - Under this method, two qualification test plates (section 4.3.2) are welded. The heat input of the test plate with the higher heat input of the two becomes the maximum qualified heat input, and the heat input of the lower of the two becomes the minimum qualified.

Once these maximum and minimum heat input values are established, they define the qualified heat input range. Then WPSs can be written such that their current, voltage and travel speed result in a heat input that falls within this range. However, there are additional limits for current for SAW in clause 4 of the code, and allowable current ranges for all processes in D1.5 table 5.10.

4.2.2.2. Production Method

The production method is found in clause 5.12.4 of the code. Whereas the heat input method creates broad windows of qualification within which a WPS can be written, by testing at the extremes, the production method involves using test parameters that are closer to the target production welding values. As a test of parameters close to the intended WPS amperage, voltage, and travel speed settings, and also because the qualified ranges are narrower, the production method is generally considered to be a more conservative qualification method than the heat input method. For example, the code requires its use for the qualification of WPSs that use atypical practices, such as EGW, nonstandard joints (clause 5.7.5), unlisted base metals (clause 5.4.3), and welding processes not found in the code (clause 1.3). The production qualification method uses the same groove welding qualification test plate (section 4.3.2) as the heat input method.

Though the production method of qualification is intended to be more restrictive than the heat input method, some amount of flexibility is necessary. In D1.5 table 5.4, the Bridge Welding Code provides allowed variances for the various settings and characteristics of WPSs qualified by this method. D1.5 table 5.5 includes further limits associated specifically with EGW.

4.2.2.3. Pretest/Verification

Under the pretest/verification WPS qualification method, a party other than the fabricator conducts a qualification test, known as a “pretest”, and then the fabricator conducts a verification test. The pretest can be one of the two heat input qualification methods or the production qualification method. The verification test is defined by D1.5 figure 5.2. It is similar to the D1.5 figure 5.1 test (figure 33, discussed in section 4.3.2) but with less testing. However, from the fabricator’s viewpoint, executing the verification test plate takes about the same amount of effort as the full qualification test, and therefore this method is rarely, if ever, used.

4.2.3. Fillet WPS Qualification

Fillet weld soundness qualification testing (section 4.3.5) is required for all fillet weld procedures that are not prequalified. For single-pass fillet welds, no groove weld test is required (clause 5.10.1). Multipass fillet weld procedures require qualification testing by both fillet weld soundness test and groove weld test (clause 5.10.2). Groove weld qualification tests are discussed in section 4.3.2.

Prior to the 2015 edition of the Code, a groove weld test was required for all types of welds, including single-pass fillet welds. The original premise of groove weld qualification testing (for both groove and fillet weld WPSs) was to help ensure satisfactory weld metal properties using the settings of the WPS. In particular, the original authors were concerned about achieving good weld metal CVN toughness at high heat inputs. However, welding practice since the implementation of the Code demonstrated that the groove weld test was counterproductive for fillet weld WPSs. Welding consumable manufacturers have developed welding products specifically for use in fillet welding, such as active SAW fluxes that are tolerant of light rust and tight mill scale, but such products do not perform well in groove welds. Further, the lower heat inputs associated with fillet welding also do not perform as well in groove welds. As a result, qualifying fillet weld WPSs under the Code was difficult and sometimes excluded the best fillet welding products. Removing this restriction from single-pass fillet welds has facilitated use of consumables optimized for fillet welds and improved welding.

4.2.4. Electroslag WPS Qualification

For ESW, WPS qualification testing requirements are distinct from those used for other processes in the Bridge Welding Code; the code is more prescriptive about electrical parameters, with fewer choices to be made by the fabricator. The code prescribes a voltage range that is dependent upon the number of electrodes and the electrode diameter, and a travel speed (better known as “rate of rise” for ESW) range that is dependent upon the thickness of the qualification plate. Fabricators choose the thickness of the test plate with these three key considerations:

- The test plate qualifies production thicknesses equal to or less than the thickness of the qualification test plate thickness
- A change in the number of electrodes requires requalification
- The number of electrodes needed depends upon the thickness of the plates to be joined

Based on these restrictions, the most logical choice for the fabricator is to qualify the greatest thickness to be welded in production with one electrode, the greatest thickness to be welded in production with two electrodes, and (possibly) the greatest thickness to be welded in production with three electrodes.

Other essential variables requiring requalification are found in D1.5 table 5.6.

4.3. WPS QUALIFICATION TESTS

4.3.1. Qualification Testing

The Bridge Welding Code requires that fabricators conduct their own qualification tests, and that the welding of the test plate and the production of the test coupons to be witnessed by the owner or an independent third party "acceptable to the state". It is customary, but not required, for the witness to be an AWS Certified Welding Inspector (CWI). The witness will sign the PQR attesting that "the results of the tests are certified as accurate".

4.3.2. Groove Weld Qualification Test Plate

Groove weld tests are conducted in conformance with D1.5 figure 5.1, which is shown in a simplified form in figure 33. This test is used to qualify groove weld WPSs (section 4.2.2) and also multipass fillet WPSs. An example plate is shown in figure 34. The following are key details about this test:

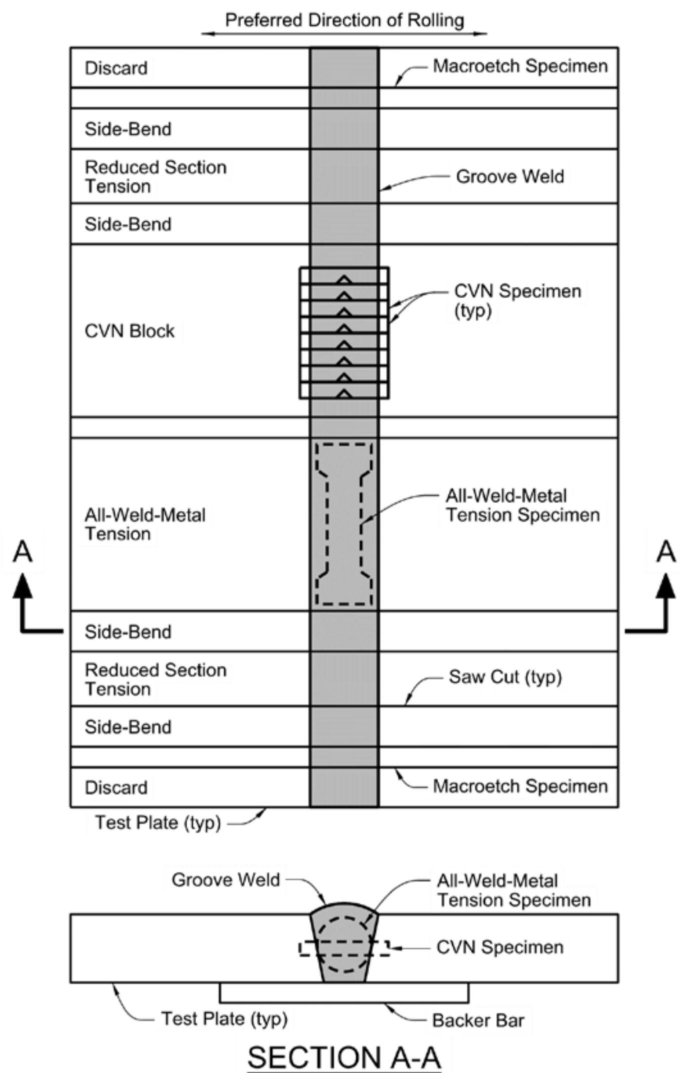
- **Materials** - The material choices for this groove weld test are found in clauses 5.4.1 and 5.4.2. There are a number of things to note regarding these choices:
    - The materials used for the test plate are similar but not necessarily the same as those used in production. ASTM A709 Grade 50W meeting requirements for particularly hardenable chemical composition may be used in the test to qualify procedures for A709 Grades 36, 50, 50S and 50W. This covers most of the steel used in bridge fabrication, and so in practice, fabricators often choose this material for qualification testing.
    - "Hardenable" relates to the hardness of the steel after welding and the associated susceptibility of its microstructure to hydrogen-assisted cracking (section 5.3.1). The hardness of a microstructure is dependent upon its chemistry; higher carbon and alloy contents in steels increase hardenability. The chemical composition of this material does not mean that it is highly susceptible to HAC; rather, it is at the outer bound of hardness for the steels it represents. Therefore, its use in qualification testing is conservative.
    - In qualification tests for WPSs that will be used for hybrid joints (joints that join two different types of base metal), the lower strength material to be joined is used for the qualification test plate. For example, if ASTM A709 Grade 50 material is to be joined to Grade HPS 70W, then Grade 50 material is used for the qualification test. When testing a hybrid plate, the transverse tension specimens fail in the lower-strength base metal; this reveals nothing about the hybrid joint. Likewise, in a bend test, the straining in the test will occur only in the lower-strength base metal and not provide information about the joint or the weld. Additionally, the design of the weld itself is controlled by the lower-strength material.
- **Welding Consumables** - Welding consumable requirements vary by qualification approach. In the heat input method, electrode diameter is not an essential variable, but in

the production method it is. There are also limits associated with consumable brand and electrode classification (clause 5.5.1).

- **Plate Thickness** - Except for ESW and EGW, the D1.5 figure 5.1 (figure 33) test plate thickness is prescribed as one inch minimum. One inch was set by AWS as a size that can readily provide a test specimen that is completely composed of weld metal. The size is prescribed as a minimum because the fabricator can choose a thicker plate if desired. Given that with most welding processes, welds are deposited in multiple passes (see chapter 2), the thickness of the test plate does not have an effect on the test properties. This is because once the groove weld test plate is thick enough to provide the required test specimens, making it thicker by adding more welding passes to complete the weld is not likely to have an effect on the test specimens.
- **Length** - The D1.5 figure 5.1 (figure 33) test plate length is shown as a minimum; this size is intended to provide sufficient material for the test specimens. A common application of the Bridge Welding Code regarding length is that because this is a minimum, the test plate may be made longer and any given continuous 23-inch length (the length of test weld required for most processes) may be used for testing. Hence, for example, if a 30-inch-long test plate is welded and there are defects in the weld at one end in the last few inches, that length and the associated defects can be ignored if there is the minimum amount of continuous length available for testing in the balance of the plate.
- **Joint** - For most processes, the joints used for the test plate are single-V or single-bevel welds with backing (see section 4.4 for examples). This is per direction from note 3 in D1.5 figure 5.1; the joint designations in this note point to the single-V and single-bevel joints in D1.5 figure 2.4. The choice of joint is adapted to the welding process and position. The Bridge Welding Code prescribes these joints because they have a wide root opening, which facilitates production of the all-weld-metal test specimen. The backing facilitates the wide test plate groove opening by providing a landing for the initial weld passes; backing thickness prescriptions are intended to prevent melting through the backing when welding the root passes. The backing is to be removed prior to testing, including radiographic testing (RT). A separation between the backing and base metal, if the backing is not removed, will appear as a discontinuity on the RT film and, presuming the discontinuity is large enough, will cause the test to fail even though this is not a defect in the weld.
- **Position -** D1.5 figure 5.6 of the Bridge Welding Code illustrates test positions (section 2.3) used for the qualification of groove WPSs.
- **Special Details for ESW/EGW -** ESW and EGW are qualified using a special version of the D1.5 figure 5.1 test plate. Items that are different:
    - The plate is modified to use a square groove joint. For ESW, the plate is modified specifically to use a square butt joint with a ¾-inch root opening, which is the same joint used in production for ESW. Given the joint restrictions for the ESW process prescribed in the Bridge Welding Code, this is the only practical joint that can be used for the qualification test.

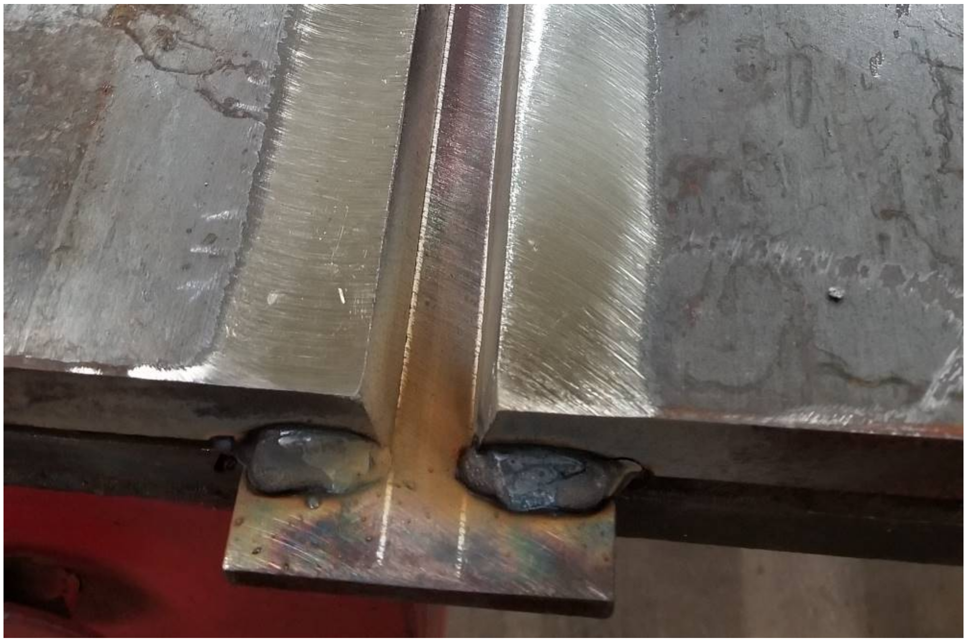
- The plate thickness varies; plate thickness is an essential variable for both processes.
- The number of CVN specimens is increased. This is because deposited ESW and EGW weld metal may have coarser grain size than that expected from traditional processes, and this grain size may result in an associated higher scatter of CVN results. A longer plate is needed to provide the additional specimens.

The Bridge Welding Code requires that the test specimens be “available to the engineer” (note 2 in D1.5 figure 5.1). This means that the actual owner (if the witness was not the owner) may inspect the specimens and expect them to be retained. However, neither practice is common. The Bridge Welding Code also requires that qualification test results be kept by the fabricator and made available to the owner upon demand (clause 5.2.5). In practice, test results are provided when WPSs are submitted for approval.



Source: FHWA

Figure 33. Illustration. WPS qualification or pretest [based on D1.5 figure 5.1(AASHTO/AWS, 2015)].



Source: FHWA

Figure 34. Photo. End of unwelded test plate.

4.3.3. Executing the Groove Weld Qualification Test

This section discusses key concepts and practices associated with executing the qualification test.

- **Average heat input and welding parameter control** - Since it will take many welding passes to complete the D1.5 figure 5.1 (figure 33) qualification plate, the following is required:
    - All passes (except the root and cap passes) must have the same amperage, voltage, and travel speed within ± 10 percent of the calculated average. Keeping the passes the same, rather than an average of a variety of passes, helps ensure test integrity. The ± 10 percent tolerance is provided to allow flexibility on each of the passes, giving the welder some room to make fine adjustments to settings as needed to complete a sound groove weld for the test. Such adjustments are typical for groove welds in production as well.
    - The official heat input represented by the WPS qualification is the average of all passes except root and cap passes.

  Root and cap passes are excluded from the heat input calculation because they do not affect test results, and are often made with different parameters from the other passes. However, in production welding, if different settings (or a different process) are to be used for the root or cap pass that are outside the allowable limits of the fill pass WPS, separate WPS qualification must be used for those passes.

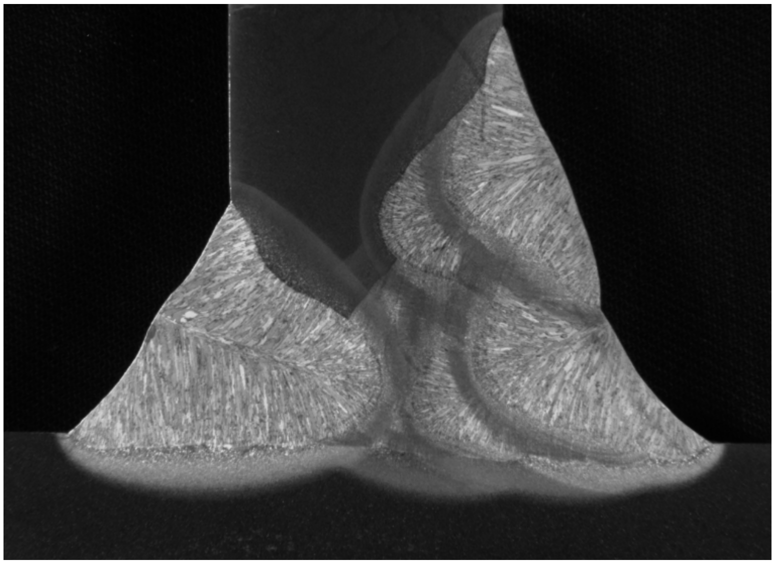
- **Cleaning** - Power tools may be used during welding of the test plate to clean slag that may occur between passes and also prepare deposited passes for subsequent passes. Such treatment is also common practice in production.

- **Aging -** The qualification test plate may not be artificially aged unless the same treatment is to be required on the WPS (clause 5.7.6). The Bridge Welding Code prohibits treatment in which the test weld is “heat treated, stress relieved, aged at temperatures above room temperatures, or modified in any way after welding” and deems this to be “artificial aging”. Natural aging by waiting before performing the test is not prohibited. Waiting is not an elevation in temperature or a modification. Hence, allowing some time to pass between welding the specimen and testing is not restricted by the code and generally results in slight improvements in ductility test results as hydrogen diffuses from the joint (see section 5.3).

4.3.4. Groove Weld Qualification Tests

The resulting tests performed on the groove weld qualification test plate are described in this section. Requirements of the tests are characterized here; specific test value requirements are found in clause 5 of the Bridge Welding Code. Much more detail is provided regarding these tests in the Bridge Welding Code, in both the code text and commentary.

- **Visual Inspection** - In addition to the tests described in this section, the test plate must also satisfy typical visual requirements for groove welds. Achieving this appearance shows good workmanship in the execution of the test plate. Visual inspection is performed before mechanical specimens are cut from the test plate.
- **Nondestructive Testing** - The test plate is radiographed and must satisfy quality requirements similar to those for production welds. Radiographic testing is primarily a soundness evaluation, with the joint examined for discontinuities such as inclusions and incomplete fusion. A passing result indicates that good workmanship was used to produce the test weld. Like visual inspection, nondestructive testing is performed before mechanical specimens are cut from the test plate.
- **Macroetch**
  - **Test** - Macroetches involve taking a cross-section of the weld, then polishing and etching the cross-section with acid to reveal the weld metal, base metal, and features of the weld such as the heat-affected zones and fusion among the passes and base metal. See the examples in figures 35 and 36. The macroetch specimens demonstrate that soundness was achieved in the test plate. Three specimens are tested, and these are located at three distinct locations to better represent the entire weld.
  - **Results** - The macroetch specimens should show complete fusion between the deposited weld metal and the base metal, and complete fusion among the weld beads.



Source: FHWA

Figure 35. Photo. Example macroetch of a T-joint mockup.

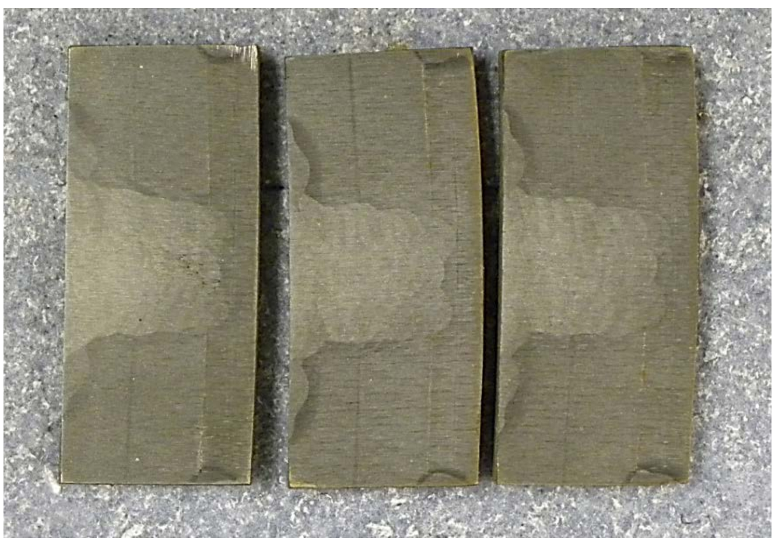


Figure 36. Photo. Typical groove weld qualification macroetch test.

- **Side Bend**
  - **Test** - In a side bend test, a relatively thin slice is taken through the cross-section of the weld and then bent to a specified radius. An example is shown in figure 37; the weld is in the middle of the specimen. The side-bend specimens demonstrate ductility and also soundness of the test plate weld, indicating properties of the weld metal and also of the combination of the weld metal and base metal. Bending the specimens puts significant strain on the weld metal, the base metal, the weld metal–base metal fusion zone, and the heat-affected zone.

- **Results** - The bend test coupons must withstand the prescribed bend without breaking, with minimal tearing, and with minimal cracking (specific limits are prescribed in clause 5).

A. Specimen before test.

B. Test setup.

C. Specimen after test.

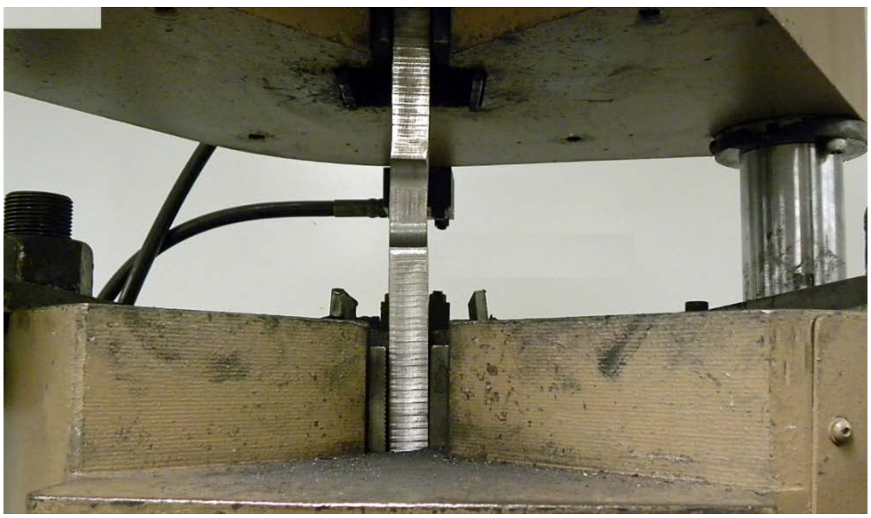
Figure 37. Photos. Side bend test.

- **Reduced-Section Tension**
  - **Test** - This specimen is a combined weld metal and base metal test for ultimate tensile strength. An example is shown in figure 38. As shown in D1.5 figure 5.10 of the Bridge Welding Code, the width of the specimen is reduced in the weld and adjacent base metal to promote failure in the area of interest rather than at the fixture. Only ultimate strength is measured and evaluated. Yield strength and elongation are not measured in the reduced-section tensile specimen because

meaningful mechanical examinations will not result from these tests due to the mismatch between the base metal and weld metal strength properties. Though the arrangement of weld metal and base metal in the test joint is typically not the same as the production weld, this test shows that that the combination of weld metal and base metal is capable of achieving the required weld strength.

- **Results** - The specimen must achieve the specified minimum tensile strength of the base metal used. The failure point for the reduced-section tensile test may be in the weld, in the HAZ, or in the base metal; all failure points are acceptable. In most cases, the failure occurs in the base metal.

A. Specimen before test.

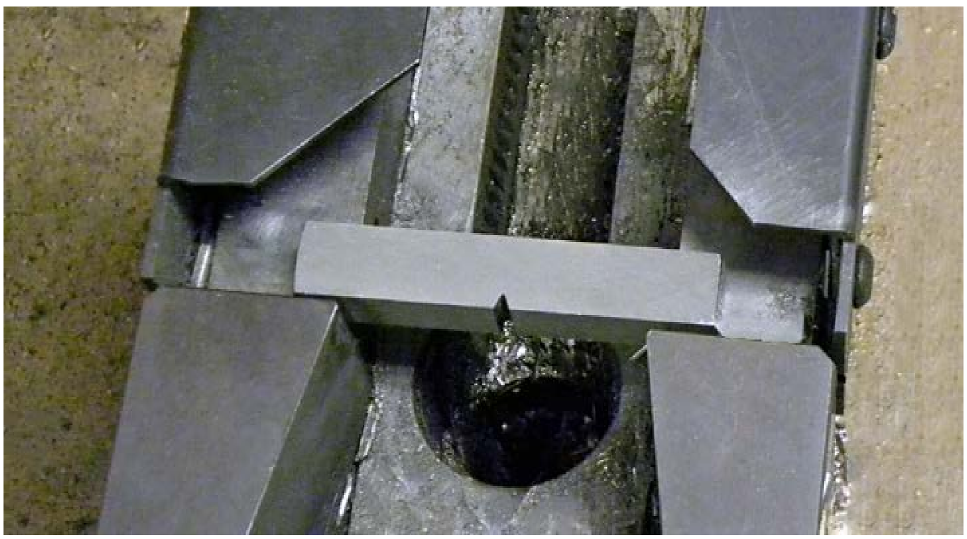


B. Test setup.

C. Specimen after test.

Figure 38. Photos. Reduced tension test.

- **Charpy V-Notch (CVN)**
    - **Test** - The Charpy V-Notch (CVN) test is a high strain rate test that measures the amount of energy absorbed by a notched specimen during fracture. Using a specimen like the one shown in figure 39, which is machined to a standard size and has a standard notch, the test fixture breaks the specimen and measures the absorbed energy; this measurement provides a surrogate measure of the fracture toughness of the material being tested. The D1.5 figure 5.1 (figure 39) specimens have the notch centered in the weld metal. Hence, the test is exclusively a weld metal toughness test.
    - **Results** - Requirements are based on the AASHTO temperature zone (I, II, or III) specified for the bridge being welded, and the grade of material being used. Requirements are found in D1.5 table 5.1.



A. Specimen before test.

B. Specimen after test.

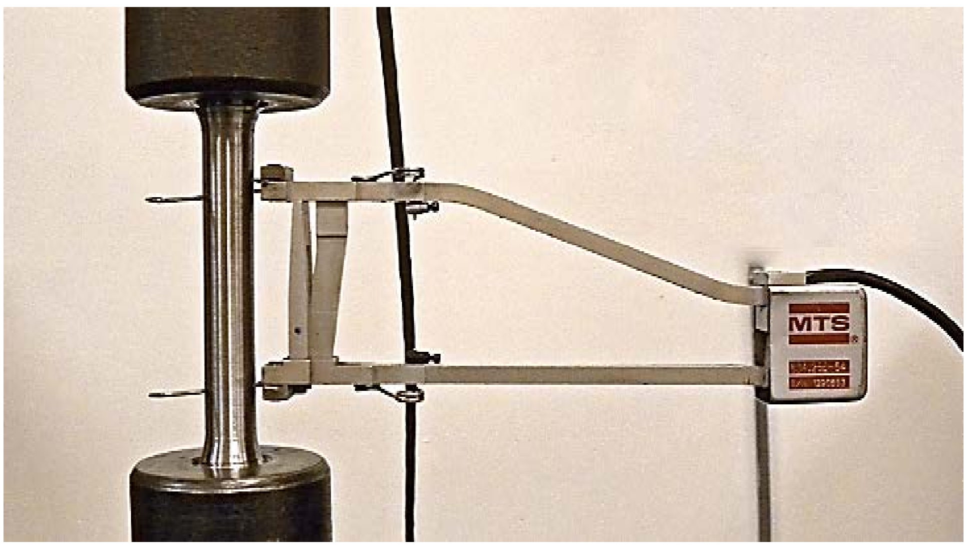
Figure 39. Photos. Charpy V-notch test.

- **All-Weld-Metal Tension**
    - **Test** - As the name indicates, this specimen is a weld metal test. The specimen, shown in figure 40, which is machined from the center thickness of the weld in

the test plate, is described in D1.5 figure 5.9. The reduced-section part of the specimen is composed entirely of weld metal, with the possible inclusion of a minimal amount of diluted base metal. The specimen is used to measure weld metal yield strength, ultimate strength, and ductility (through measurement of elongation). The specimen is known familiarly as a “505” specimen because in very early versions of the test, the diameter was once 0.505 inch because this provides an area of 0.2 square inches, which simplified load-to-stress calculations. Later, the diameter was changed to 0.500 inch, but the familiar name endures.

- **Results** - Yield strength, ultimate strength, and elongation must meet the requirements of D1.5 table 5.1. Although this test is an all-weld-metal test, the required results are related to the properties of the base metal being joined.

A. Specimen before test.



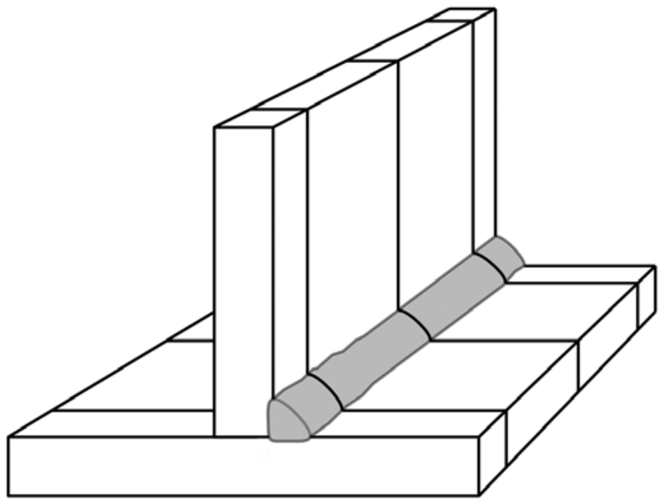
B. Test setup.

C. Specimen after test.

Figure 40. Photos. All-weld-metal tension test.

4.3.5. Fillet Weld Soundness Tests

The fillet weld soundness test is intended to demonstrate that the deposited fillet weld will satisfy quality requirements. Soundness tests are performed using the WPS that will be used in the work. Parameters are allowed to vary within the limits of D1.5 table 5.4, but generally welding of the test coupon will be conducted as it will be in production. The fillet weld soundness test is defined by D1.5 figure 5.8. The configuration of the test specimen is illustrated in figure 41.



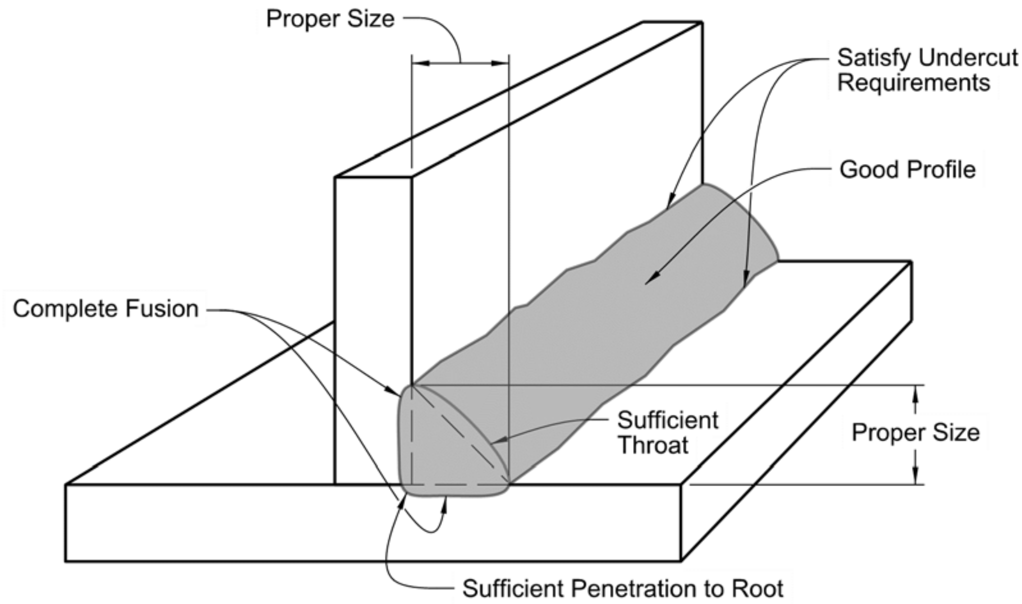
Source: FHWA

Figure 41. Illustration. Fillet weld soundness test coupon based on D1.5 figure 5.8 of the Bridge Welding Code (AASHTO/AWS, 2015).

Note the following details:

- **Material and Welding Consumables** - Requirements for these two aspects of the test are those of the groove weld test discussed above in section 4.3.2.
- **Plate thickness** - The minimum plate thickness for the fillet weld soundness test depends on the size of the weld that will be made in production.
- **Skewed fillet welds** - If the WPS is to be used to make a skewed fillet weld, the soundness test is conducted on a test plate with an orientation within 10 degrees of that skew, with the “stem” plate beveled as necessary.
- **Position** - Soundness tests are position-specific.

As shown in figure 42, the fillet weld macroetch demonstrates that the WPS will produce proper size, complete penetration to the root, complete fusion, no or minimal undercut, and acceptable profile.



Source: FHWA

Figure 42. Illustration. Required visual characteristics of a fillet weld soundness test.

After visual assessment, the specimen is cut three times at the mid-length of the weld and near the ends, as shown in figure 41, and one face of each cut is prepared as a macroetch.

4.3.6. Combinations of PQRs and WPSs

Some welds can need welding passes that are different enough from each other that one qualification test will not cover all the passes. In such cases the fabricator can use different WPSs for the different passes, or the fabricator can use one WPS with different PQRs to support different passes.

Either approach is acceptable under the Bridge Welding Code; in practice, the first is more common. The root passes of a groove weld represent an example of where a fabricator may use one of these practices—the fabricator may want to run cooler, smaller passes in the root of a groove weld where special care may be needed to avoid melt-through, and then use hotter, larger fill passes for the remainder. To do this, the fabricator might write one WPS that has root passes supported by one PQR and the balance of the passes supported by another, or the fabricator might use one WPS for the root passes and another WPS for the balance.

4.3.7. WPSs for PJP Groove Welds

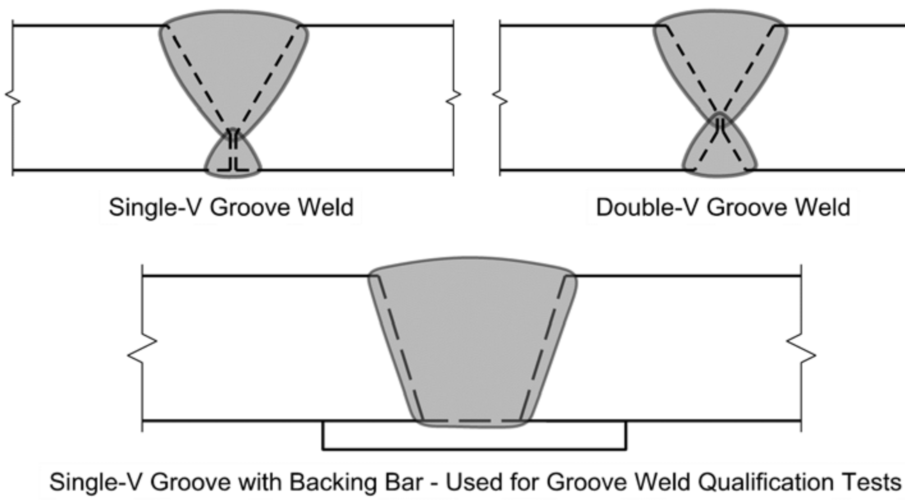
Under the Bridge Welding Code, WPSs for PJP welds are qualified in the same way as WPSs for CJP welds (clause 5.9.1). This is logical considering that even for CJP welds, the actual joint used in production is not used for qualification testing.

4.3.8. Unlisted Materials

If materials are used that are not addressed by the code (as defined in clause 1.2.2), then these specific materials are used for the groove weld qualification test plate, and the production qualification method (4.2.2.2) is used. However, use of unlisted materials under the Bridge Welding Code is much more involved than just running this test. Clause 5.4.3 provides guidance intended to ensure that the welds will have the properties needed to perform well in service. The suitability for welding should be established during the design phase of the project; the fabricator's testing is only for WPS qualification and not for welding suitability. The Bridge Welding Code qualification tests are not weldability tests. Rather, the code tests primarily evaluate weld metal and not the suitability of the base metal for welding.

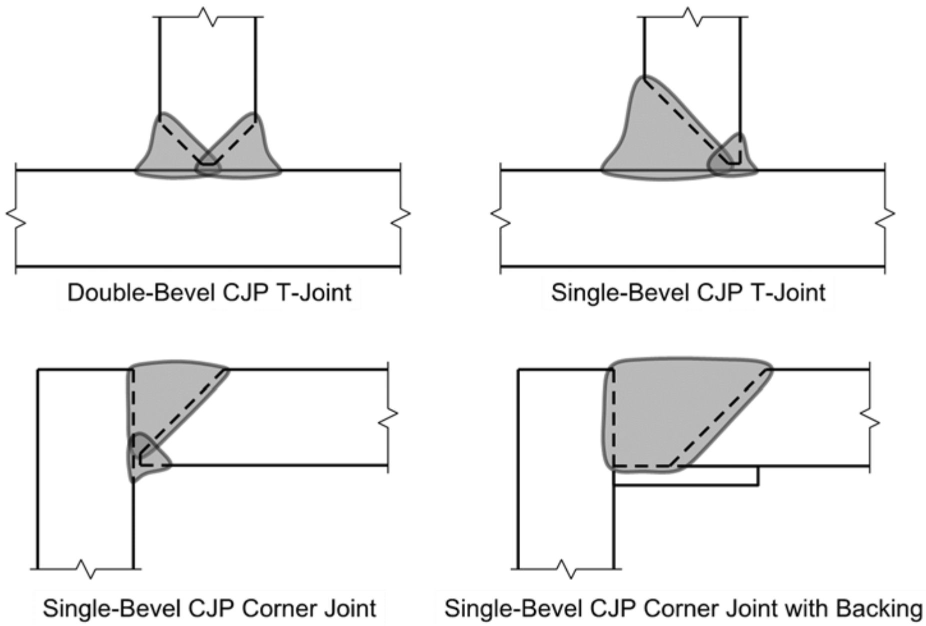
4.4. STANDARD JOINTS

The configuration of the joint being welded is an essential part of the WPS. WPSs are joint-specific, and the WPS will feature a sketch of the joint. In bridge practice, CJP weld single-V and double-V butt joints and single- and double-bevel T- and corner joints are heavily used (see figures 43 and 44). Joints with "U" or "J" shaped preparations as opposed to straight bevels are not common; generally this is because preparation of J and U bevels requires machining, which is much more time-consuming and expensive.



Source: FHWA

Figure 43. Illustration. Common CJP weld butt splice joints.



Source: FHWA

Figure 44. Illustration. Common CJP weld T- and corner joints.

The Bridge Welding Code features a series of standard joints for CJP and PJP welds in clause 2 D1.5 figures 2.4 and 2.5. The Bridge Welding Code does not call these joints “prequalified”, but in common parlance they are often referred to this way because as standard joints, they can be used on WPSs with no further testing to demonstrate the suitability of the joints themselves. However, this only means that the joint itself does not need testing; use of a standard joint does not mean that the WPS itself is prequalified. If a joint other than a standard joint is to be used in the WPS, the joint is said to be “nonstandard” and requires qualification testing. Qualification of nonstandard joints is described below in section 4.4.4.

The standard joints listed in clause 2 have been part of the Bridge Welding Code since the first edition was published in 1988, and prior to this, these or very similar joints have been a part of other welding codes, such as AWS D1.1, for decades. They have not changed significantly over this time and have a long history of successful performance.

4.4.1. Tolerances and Fit Up

The standard joints are published with “as detailed” and “as fit-up” tolerances for the groove preparation parameters such as root opening, root face, and groove angle. There are important distinctions:

- The “as detailed” tolerance is added to the nominal dimension shown on the standard joint details. The “as detailed” tolerance may be exercised on shop drawings. For example, a particular joint detail may have a required root opening of ¼ inch with an as-detailed tolerance of + $^{1}/_{16}$ inch, − 0 inch; this means that the fabricator can show this joint with either a ¼-inch root opening or a $^{5}/_{16}$-inch root opening.

- The “as fit-up” tolerance is added to “as detailed” dimensions shown on the shop drawings. The “as fit-up” tolerance is not shown on drawings but executed in production. For example, if a particular joint has an “as fit-up” tolerance for root opening of + ¼ inch, -$^{1}/_{16}$ inch, then if the joint is detailed with a root opening of $^{5}/_{16}$ inch on the shop drawings, the joint may actually have a root opening of up to $^{9}/_{16}$ inch ($^{5}/_{16}$ inch plus ¼ inch) or as small as ¼ inch ($^{5}/_{16}$ inch minus $^{1}/_{16}$ inch).

D1.5 clause 2.12.1 says, “Dimensions of [CJP] groove welds specified on design or detailed drawings may vary as shown in figure 2.4.” However, it is not the best design practice to show joints types (e.g., single bevel groove, double bevel groove) on design plans. Rather, designers should simply designate where a CJP, PJP, or fillet is needed, provide the required PJP and fillet weld size, and allow the fabricator to select and detail the joint for groove welds.

4.4.2. Joint Details

The standard joints are designed to facilitate sound welding when used with the traditional arc welding processes: SMAW, GMAW, FCAW, and SAW. Groove angles are designed to provide sufficient opening for welding passes. Except for joints that use backing, root openings are tighter for SAW and root faces are larger for SAW. This is because SAW operates hotter and penetrates deeper than other processes, and the tight root openings and larger faces provide sufficient material to effectively accept hot SAW root passes.

The joints for use with EGW and ESW are not defined in D1.5 figures 2.4 and 2.5 but are prescribed elsewhere by the code. The required joint details for EGW are found in clause 4.14, and for ESW in clause 4.18.

4.4.3. Backgouging

Backgouging is defined as “[t]he removal of weld metal and base metal from the weld root side of a welded joint to facilitate complete fusion and complete joint penetration upon subsequent welding from that side” (AWS, 2010c) Backgouging to sound metal is required for all standard CJP weld joints that are welded from both sides of the joint (i.e., the standard CJP weld joints that do not use backing). Backgouging to sound metal helps ensure that there will be no root defects after the welding that follows backgouging is complete. Omitting backgouging requires qualification of the non-backgouged joint as a nonstandard joint.

A common form of backgouging used in bridge fabrication is carbon arc gouging. AWS defines carbon arc gouging as “[a] thermal gouging process using heat from a carbon arc and the force of compressed air or other nonflammable gas” (AWS, 201b). Figure 45 shows backgouging in process; figure 46 shows a groove weld after excavation by gouging in preparation for repair.

Source: FHWA

Figure 45. Photo. Welder carbon arc backgouging test plate.

Source: FHWA

Figure 46. Photo. Example of partially gouged joint.

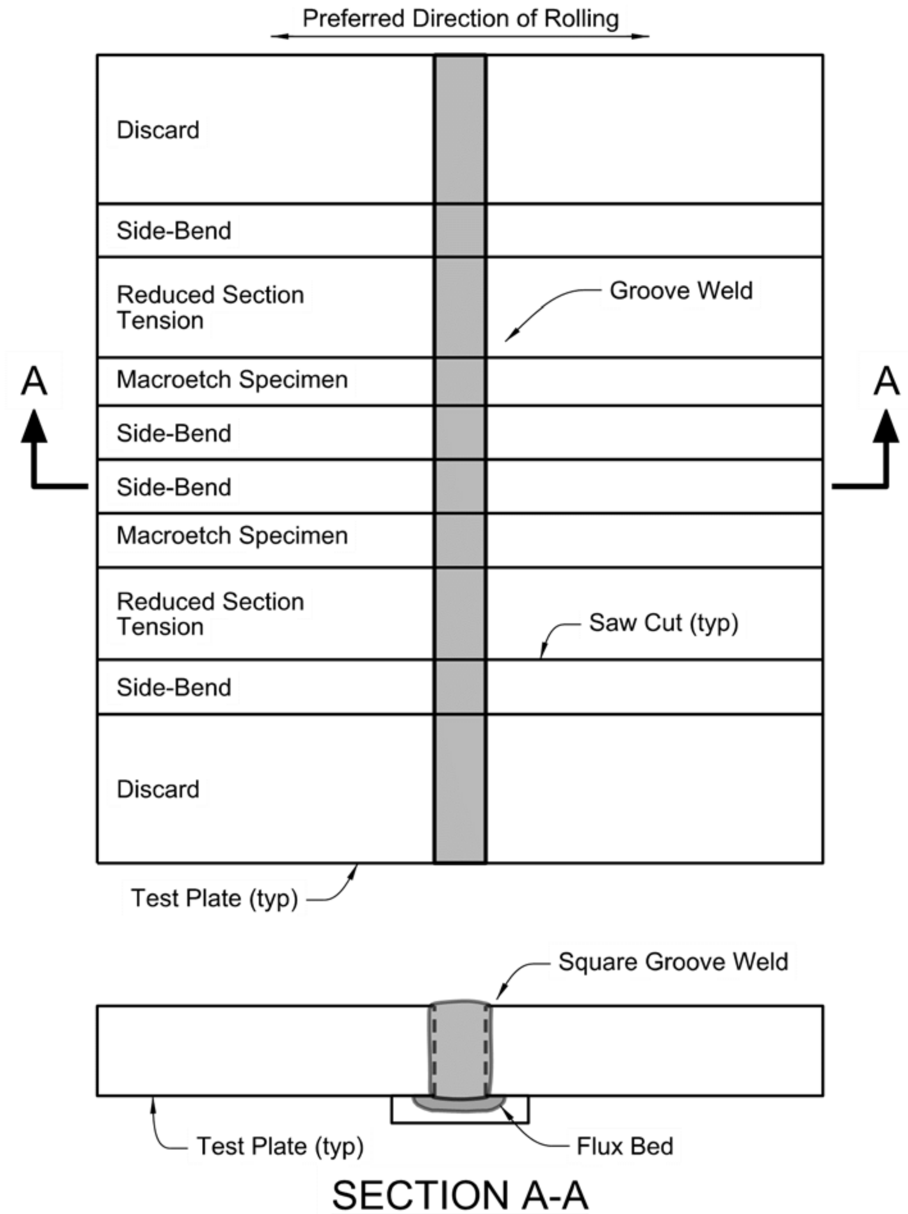
4.4.4. Nonstandard Joints

As noted in clause 2.7.1, fabricators may propose WPSs that deviate from the standard joints listed in clause 2, providing the fabricator with some flexibility. To use a nonstandard joint, the fabricator must qualify the WPS using joint details under the provisions of clause 5.7.5. Clause 5.7.5 requires that in addition to a typical WPS qualification test (section 4.2.2), an additional test must be conducted using a Bridge Welding Code nonstandard joint test plate (D1.5 figure 5.3, with a simplified version shown in figure 47 below) with the proposed joint.

The intent of the Bridge Welding Code nonstandard joint test plate is to demonstrate that weld metal can indeed be effectively deposited into the proposed joint using the proposed WPS. A successful test demonstrates that a sound weld is achieved, with proper fusion between the deposited weld metal and the base metal, with proper fusion among the passes, and without the presence of other discontinuities, including cracks.

The D1.5 figure 5.3 test plate includes these key features:

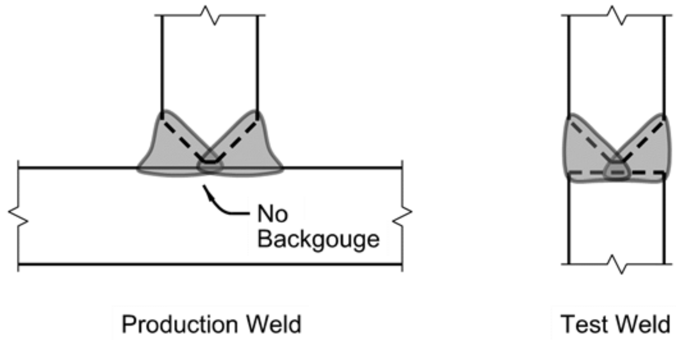
- **Joint -** The actual joint is used (the single V shown in D1.5 figure 5.3 and the joint detail in the figures below are just examples).
- **Thickness** - As with the D1.5 figure 5.1 test, the test plate thickness is to be 1 inch minimum. It may be more appropriate to use a different thickness if there are important effects associated with the joint thickness. For example, if the fabricator wants to use a joint on 2 ½-inch material that is only listed as a standard joint for material up to 2 inches thick, a test on 2 ½-inch-thick plate is appropriate. If the fabricator wants to use a joint on ¾-inch material that is only listed as a standard joint for material up to ⅝ inch thick, a test on ¾-inch material is appropriate (even though a minimum 1-inch thickness is prescribed in D1.5 figure 5.3 of the code).



Source: FHWA

Figure 47. Illustration. Nonstandard joint test plate [based on D1.5 figure 5.3 (AASHTO/AWS, 2015)].

The Bridge Welding Code nonstandard joint test is based on a CJP weld butt joint because mechanical specimens such as side-bend specimens and reduced-section tension specimens can only come from butt joints. Also, butt joints lend themselves to RT examination. The code does not explicitly address how to qualify nonstandard T- and corner joints. In practice, fabricators typically propose alternate means of replicating the essential aspects of the joint within the prescribed butt joint. For example, in figure 48 below, the fabricator may be proposing the nonstandard joint, labelled as “production weld”, which is nonstandard because it will not be backgouged. One possible test would be a plate based on the joint labelled "test weld" in the figure below.



Source: FHWA

Figure 48. Illustration. Possible alternative nonstandard joint test, with production and test welds shown.

The tests prescribed in D1.5 figure 5.3 also assume a CJP weld. A PJP weld specimen would not be expected to pass the reduced-section tensile and side bend tests. For qualification of nonstandard PJP weld details, one approach is to only require macroetches to verify adequate joint penetration and soundness.

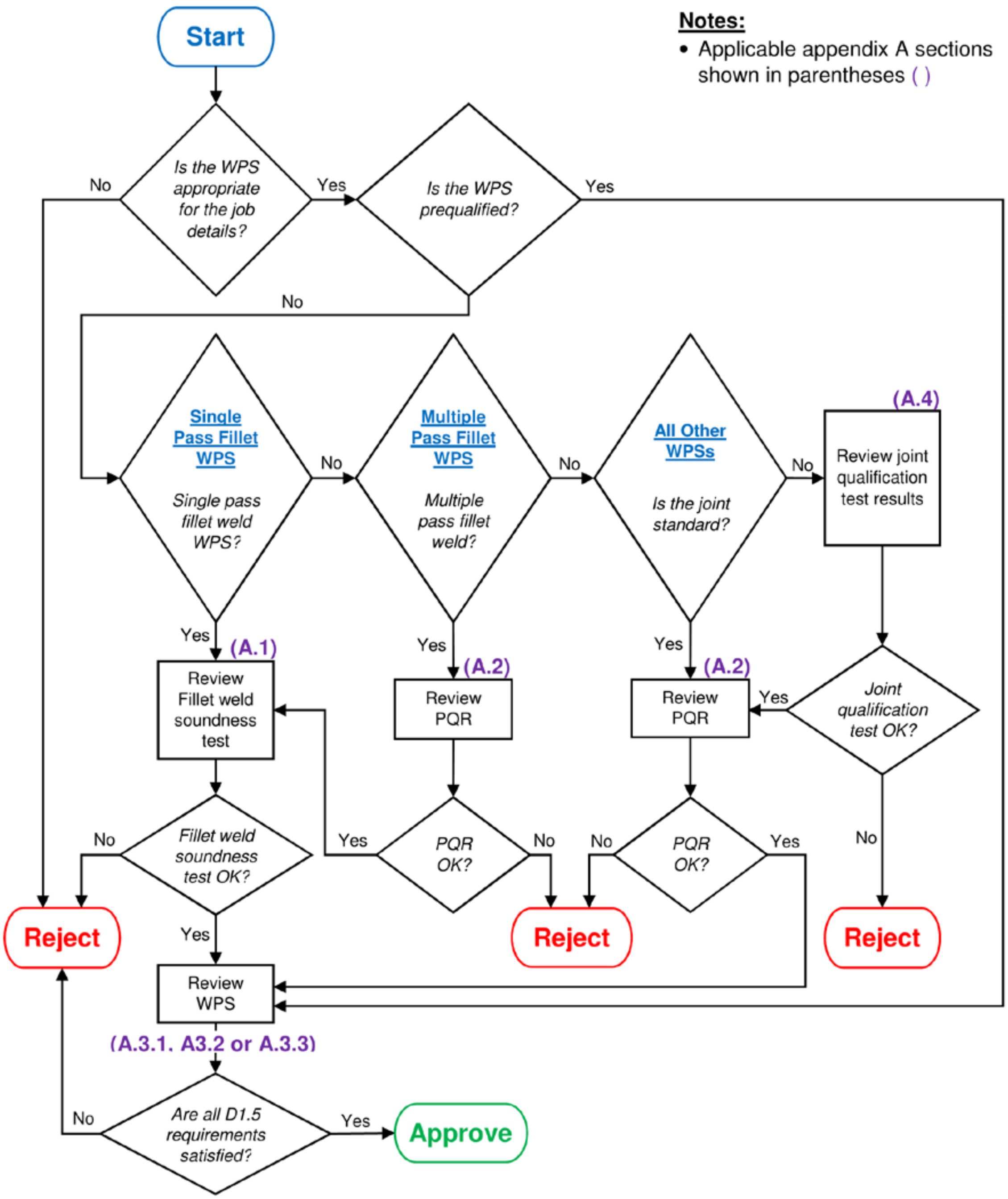
4.4.5. Plug and Slot Welds

Plug and slot welds are not represented in D1.5 figures 2.4 and 2.5 and therefore are not standard joints. Plug and slot welds are prohibited in members subject to tension and stress reversal (clause 2.14(6)) and are rare in new bridge fabrication. However, if they are used, the Code prescribes the details required for their use and exempts them from nonstandard joint qualification testing. As for all other welds under the Code, plug and slot welding must be made with an approved and qualified or prequalified WPS.

4.5. REVIEW AND APPROVAL OF WELDING PROCEDURES

Appendix A provides a guide to the review and approval of welding procedures, typically including WPSs, PQRs, and fillet weld soundness tests. Generally, in the review, combinations of documents will be needed. For example, a welding procedure for a multipass fillet weld will include the WPS and the associated PQR and fillet weld soundness test, all of which must be reviewed and approved. In light of this, appendix A provides a step-by-step guide to reviewing each element that might be encountered. Also, figure 49 provides a workflow outline of the documents required to review and approve WPSs. This figure can be used as a roadmap to help work alongside appendix A.

An owner may accept WPSs that have been approved by other owners. Such reciprocity of WPS approval among owners was part of the original intent of the Bridge Welding Code when it was first published in 1988. At the time, owners saw that there were cost and schedule advantages to avoiding the redundancy of requalification from owner to owner. This is also discussed in clause C-5.2.3 of the Bridge Welding Code commentary.



Source: FHWA

Figure 49. Illustration. WPS approval flow chart (refer to appendix A).

4.6. WELDER QUALIFICATION

4.6.1. The Achievement of Quality by Skilled Welders

The skill of the welder is an essential factor in the quality of a weld. As such, the bridge welding code requires all welds to be made by qualified welders. Qualification demonstrates that the welder has the skills needed to weld in conformance with the code and thereby achieve the desired quality. These skills vary among the tasks the welder performs, depending upon a number of factors:

- Welding position needed for the task
- Welding process chosen for the task
- Level of mechanization to be used (e.g., manual versus mechanized or automatic processes)

The welder must be able to understand and follow the approved WPSs on the project. Further, while good weld quality is dependent upon welder skill, not all quality problems are associated with a lack of skill. The Bridge Welding Code's welder qualification requirements are only a basic assessment of the skills needed to produce quality welds. The qualification process does not replicate every application and condition that the welder will encounter. Rather, the requirements are like an entrance exam; once a welder is qualified in a certain process and position, the welder's skills will grow as the welder gains experience. The welder must also have proper training in production applications.

4.6.2. Qualification versus Certification

Welder qualification is sometimes incorrectly referred to as "certification". "Certification" applies when an outside party conducts the qualification assessments, including tests, audits, or perhaps other types of assessments, and issues a certification in conformance with the applicable certification program. The Bridge Welding Code qualification requirements are not a certification program.

4.6.3. Fabricator Responsibility

Under the Bridge Welding Code, welder qualification is the responsibility of the fabricator. Fabricators have a high interest in the qualification of welders because welders who lack proper skills create costly defects. The fabricator administers qualification testing in accordance with clause 5 part B of the Bridge Welding Code. The fabricator conducts tests and maintains test records per the code (clause 5.21.7); the records are made available to the owner for review upon request. The code does not require approval of the qualifications by the owner, and in practice, qualification documentation is usually not submitted to the owner; rather, proper welder qualification is usually verified by the owner's inspector through review of qualification documentation and random observations of qualification practices (whether qualification by testing or by continued production welding).

4.6.4. "Welders" versus "Operators" versus "Tack Welders"

The Bridge Welding Code makes a distinction between welders and welding operators. AWS defines a welding operator as "one who operates adaptive control automatic mechanized or robotic equipment" (AWS, 2010c). Based on this definition, any welding that includes the use of equipment to hold the welding gun is considered to be done by a welding operator; any welding done where the individual is holding the gun is done by a welder.

The term operator and the distinction between welders and operators is important for qualification but otherwise, use of the term “operator” is not common. On the shop floor, the operators of mechanized and automatic welding equipment are usually referred to as “welders” and the same is done in this manual except regarding welder or operator qualification.

The code requires a slightly different qualification test for operators. However, neither the operator test nor the welder test is equipment-specific, and it is the fabricator’s choice which brand and type of equipment is used for testing.

The code provides a distinct qualification just for tack welding (clause 5.23.3). The qualification test is a simplified version of the fillet weld qualification test (section 4.6.9.3) that a welder may take just for tacking.

4.6.5. Process

Welding processes are unique from one another in the technique needed to produce welds effectively and the equipment needed to conduct the welding. Therefore, welders must be qualified separately for each process used (clause 5.24.1.2).

Not all process variations require distinct qualifications. In each of the following cases, qualification in one variation qualifies the welder in the other variations:

- SAW single wire, parallel and multi-electrode (section 3.3)
- GMAW pulse, spray, and globular transfer modes (section 3.5.7)
- GMAW solid wire and metal-cored wire (section 3.5.4)
- FCAW-G and FCAW-S (section 3.6)

Welder qualification is also not unique to the following:

- WPS
- Consumables (for given process)
- Equipment brand – for example, various power supplies
- Heat input

4.6.6. Weld Type: Groove versus Fillet Weld

Groove welding is considered more challenging than fillet welding, so the Bridge Welding Code allows welders qualified for groove welding to qualify for fillet welding, Groove welding is considered to be more challenging than fillet welding for a number of reasons:

- Groove welding typically requires numerous passes to complete the weld successfully. Hence, the welder must have the skill to achieve good welding passes repeatedly and, compared with single-pass fillet welding, not just one time.

- In groove welding, the welder must deposit weld metal in a space that is more confined that a typical fillet weld joint, offering less flexibility for positioning the electrode to successfully fuse with base metal.
- In groove welding, weld passes must be made without trapping slag among the passes, achieving complete fusion among the passes and with the base metal. This is made more difficult in the Bridge Welding Code than with other structural codes because the code does not allow grinding during the qualification test (grinding is permitted in production). This limits the welder's ability to remove trapped slag or address lack of fusion in the qualification test plate during welding.

Hence, the Bridge Welding Code allows successful groove welder qualification tests to qualify fillet welding in the same position. Additionally, given that the welding technique needed for groove welding is rather similar to horizontal fillet welding, the code allows for a flat position groove weld qualification test to qualify for both flat and horizontal fillet welding.

4.6.7. Position

There are considerable skill level differences associated with welding position (section 2.3). Welder qualification under the Bridge Welding Code codifies this skill level hierarchy in allowing qualification in some challenging positions to qualify others that are less challenging but not vice versa. This hierarchy is found in clause 5.22 and is repeated in D1.5 table 5.8.

As an example, consider the position qualification allowances for fillet welding and the skills needed to weld in various positions. In bridge fabrication as well as other structural welding, the most common production position of fillet welding is horizontal. Consequently, welders are almost always first trained to weld fillets in the horizontal position. Further, welders usually find that most of the fillet welding they encounter in the shop is in this position.

Welding in the vertical position (figure 50) is more difficult than in the horizontal position. Usually a weave technique is used to build the weld in such a way that gravity does not compromise the integrity of the molten weld puddle. Weaving creates a small shelf for the molten weld that not only helps resist gravity but also facilitates larger fillet welds. This weave pattern is evident in the photo on the right side of figure 50. Given the difficulty associated with vertical fillet welding, the code allows for a welder qualified in the vertical position with a given process to be qualified for horizontal and flat fillet welding as well (clause 5.22.2.3).

Source: FHWA

Figure 50. Photo. Example of vertical fillet welding.

Overhead welding also requires its own special technique (figure 51). Welding must proceed in such a way that gravity does not compromise the weld pool before the weld metal solidifies. This necessitates a certain minimum speed, and this means that in overhead fillet welding, pass size is limited. In the overhead position, weaving does not build a shelf as in vertical welding; however, small oscillations do allow the welder to increase the fillet weld size somewhat. Usually, the largest single-pass fillet weld that can be effectively deposited in the overhead position is about ¼ inch. The code allows for a welder qualified in the overhead position for fillet welding with a given process to be qualified for horizontal and flat fillet welding as well (clause 5.22.2.4).

Source: FHWA

Figure 51. Photo. Fillet welding qualification in the overhead position.

4.6.8. Qualification Frequency

If a welder continually uses a welding process, there is no inherent reason that the welder would need to requalify, provided there is no "specific reason to question a welder's, welding operator's, or tack welder's ability" (clause 5.21.4), and the welder can keep welding with that given process indefinitely, without requalification. There is a key exception to this allowance based on continual welding: for fracture-critical (section 7.5.5) welding, welders must be requalified on an annual basis (clause 12.8.2). If a welder stops welding with a given process for six or more months, the welder must requalify. This six-month period is not based on a study of welder skill loss over time. There is no need for particular concern about the quality of a welder's work only because of to this welding lag; the requalification simply verifies the welder's skills.

4.6.9. Personnel Qualification Tests

This section provides an overview of welder qualification requirements under the Bridge Welding Code. Not all details of code requirements are addressed. Complete details are found in clause 5 part B of the Bridge Welding Code.

Welder qualification is distinct from WPS qualification testing. However, if a welder successfully conducts a procedure qualification test, the completion of this test may also serve as the welder's qualification test (clause 5.23.2.3).

4.6.9.1. Material

Welder qualification tests are not material-dependent. Rather, any Bridge Welding Code material may be used (clauses 5.21.3 and 5.24.1.1). The only exception is for tack welding steels of yield strength greater than 90 ksi, which requires use of the same plate for the test that will be used in production, although there is not a special skill associated with tacking this material.

4.6.9.2. Groove weld tests

Test Plates

There are four groove weld welder qualification test plates in the Bridge Welding Code in D1.5 figures 5.17 through 5.20. The fabricator has the choice of which test to administer depending upon the fabricator's needs. There are two fundamental choices:

- **Thickness limitations –** the fabricator may choose between a $^{3}/_{8}$-inch-thick plate for limited thickness qualification (up to ¾ inch thickness in production welding) or a 1-inch-thick plate for unlimited thickness. The fabricator may also choose some thickness, T, between $^{3}/_{8}$ inch and 1 inch, which will then qualify the welder for up to 2T thickness or, for operators, up to thickness T. In practice, most fabricators default to using 1-inch-thick plates, particularly because groove welds over 1 inch thick are very common in bridge fabrication.
- **Position –** a test plate of one type of joint, a single-V groove, may be used for any of the positions to be tested; alternately, for the horizontal position, the fabricator may choose a

single bevel groove weld. Single-bevel grooves offer an advantage for horizontal welding in that the horizontal plane (made by the square end of the lower plate) facilitates deposition in the horizontal position. Conversely, use of a single-V joint in the horizontal position is made particularly challenging because gravity will pull the molten weld down the bevel. Note that there are two thicknesses available for this test plate as well—a $^3/_8$-inch-thick plate to qualify welding plate up to ¾ inch thick or a 1-inch-thick plate to qualify unlimited thickness.

Tests Required

Required tests are found in D1.5 table 5.9. As shown there, generally two side bends are required, except that for test plates that are only $^3/_8$ inch thick, one face bend and one root bend are required instead. This is because side bends in plates as thin as $^3/_8$ inch are not very meaningful. The bend test must not have discontinuities outside the allowances of clause 5.27.3. Failures associated with such discontinuities may be due to material issues; if so, this is still a failure, and the welder will need to retest. In lieu of testing bend specimens, the fabricator has the option to perform radiographic examination of the test plate. Specimens must satisfy visual requirements (clause 5.27).

Qualification of groove welders for fracture-critical applications requires both radiographic testing and side-bend testing (clause 12.8.2). Fabricators who perform fracture-critical work may choose to qualify all welders to the fracture-critical groove weld requirements because such qualification covers all groove and fillet welding with a given welding process.

4.6.9.3. Fillet weld tests

For fillet weld qualification only, the fabricator has the choice between the fillet weld break and macroetch test plate found in D1.5 figure 5.21 of the code or the fillet weld root-bend test plate found in D1.5 figure 5.22.

Because of its simplicity, fabricators usually opt for the fillet weld break and macroetch plate. The test must pass a visual examination, break test, and macroetch test as described in clause 5.27. If the fabricator chooses the fillet weld root bend test plate, the root bends must satisfy clause 5.27. There are also fillet weld tests for welding operators in D1.5 figures 5.26 and 5.27.

Regardless of the fillet weld test chosen, there are no thickness limits on qualification. Further, the code does not prescribe requirements related to the number of fillet weld passes.

4.6.9.4. Plug welds

Plug welding qualification is described in clause 5.23.1.5, and test requirements are found in clause 5.27.6.2. However, because plug welds are rarely, if ever, used, fabricators typically do not qualify welders for plug welding.

CHAPTER 5 - WORKMANSHIP AND WELD QUALITY

The primary goal of achieving good quality welds is performance in service. The overall successful performance of welds over the history of welded steel bridges is a testament to the efficacy of the standards and practices for welding fabrication and inspections. This chapter describes these standards and practices, with a focus on those required in the Bridge Welding Code.

There are five aspects to weld quality:

- **External condition** - external requirements related to the final shape and appearance of the weld
- **Size** - achievement of the desired weld size
- **Internal Soundness** - internal quality requirements related to internal continuity of the weld metal
- **Mechanical Properties** - production of the weld such that it has the intended mechanical properties
- **Chemical Composition** - composition of the weld metal as needed for special service requirements, such as for weathering performance

These aspects are discussed in this chapter, including how associated practices and requirements are addressed in the Bridge Welding Code.

5.1. QUALITY AND THE BRIDGE WELDING CODE

The fundamental purpose of the AASHTO/AWS D1.5 Bridge Welding Code is to provide a single document that can be specified, *typically without modification,* by bridge owners to achieve quality welded bridges.

The quality requirements of the Bridge Welding Code are generally based on workmanship rather than fitness for service. When first established, the workmanship criteria were based on what could be expected from qualified welders in the shop and not based on research that studied the effect of discontinuities on performance. The fact that workmanship criteria and not fitness-for-purpose criteria are the basis for code provisions is important: the code criteria should not be considered to be lower bounds for acceptable service. Therefore, there is room for engineering judgment when addressing quality issues that arise in fabrication. See also sections 10.5, “Tolerances”, and 10.6, “Nonconformances”.

As knowledge of weld behavior under cyclic loading grew, structural welding standards adopted non-destructive testing criteria for welds under cyclic loading. Still, the basic premise remains true: the Bridge Welding Code quality requirements are fundamentally workmanship-based criteria. The Bridge Welding Code assumes that all bridges are cyclically loaded and makes no distinction between cyclic and static loading, as opposed to the AWS D1.1–Structural Welding Code, which addresses both loading conditions.

The Bridge Welding Code quality standards have been developed very broadly, intended to cover most conditions that are encountered in the fabrication of a steel bridge. However, the code does not necessarily address every situation that may be encountered in a steel bridge. Hence, the use of some judgment on behalf of the engineer and owner is prudent when addressing conditions that are not in full conformance with the code. Further, it is sometimes more deleterious to repair minor code non-conformances than to leave them “as is”.

5.2. PREHEAT AND INTERPASS TEMPERATURE CONTROLS

Preheat and interpass temperature controls are used to help prevent cracking and help achieve mechanical properties. The preheat temperature is the temperature of the steel immediately before the arc encounters the steel. The interpass temperature is the temperature of the weld and surrounding base metal between passes.

Minimum interpass temperatures are not prescribed, per se, because the minimum preheat temperature amounts to the same thing: the minimum preheat temperature requirement applies before any pass. Put another way, the steel must be at the required minimum preheat temperature before each pass is started and not just the first pass.

Joints are heated to the required preheat temperature such that the steel is at the required temperature within 3 inches of the arc in all directions, including through the thickness of the steel (clause 4.2.2). On longer joints, the entire length of the joint may not be at the preheat temperature when welding begins at one end; rather, a preheat torch may move along the part along with the welding torch with preheat progressing as welding progresses (see figure 52). When the arc encounters the steel at a particular location along the joint, the steel must be at the required preheat temperature.

The required preheat temperature is dependent on several factors, including the steel grade and thickness. Thicker plates cool more rapidly and therefore the required preheat temperatures are higher for thicker steels. In some cases, the required preheat temperature is at or below the ambient temperature, in which case no additional preheating of the steel is required. For other situations, the steel temperature must be raised to some temperature above ambient, thus necessitating additional preheating of the steel before welding.

The maximum interpass temperature is the temperature which the steel must be at or below before any welding pass is started. For example, if the maximum interpass temperature published on the WPS is 550 °F and the temperature of the steel has risen to 600 °F because of the buildup of heat from welding (or for any other reason), then welding cannot recommence until the steel cools to 550 °F.

Maximum interpass temperatures are intended to keep the cooling rate of the welded joint high enough to avoid problems associated with extremely slow cooling rates; these problems include formation of large weld grain size, which can compromise toughness. Quenched and tempered steel are more sensitive to problems associated with excessively high interpass temperatures. The maximum interpass temperature observed during WPS qualification testing provides the upper limit for interpass temperature on the WPS.

Source: FHWA

Figure 52. Photo. Preheat applied continuously ahead of welding.

The WPS is required to list the minimum preheat and interpass temperature. The minimum required interpass temperature is the same as the minimum required preheat temperature. The WPS must also list the maximum interpass temperature. Welding may not proceed if the temperature of the base metal is above this temperature; rather, welding may only commence when the steel cools below the maximum interpass temperature.

In practice, on shorter joints like butt joints in flanges, once the steel has been heated to the preheat temperature, it will usually remain above this temperature when welding continues uninterrupted, because the welding process provides additional thermal energy. For short joints, exceeding the maximum interpass temperature is a more likely occurrence than for longer joints. If welding operations are interrupted and the joint is allowed to cool before the weld is completed, the joint will likely need to be reheated to the minimum preheat/interpass temperature before welding resumes.

On long joints, just the opposite condition is experienced from that on shorter joints: the steel cools between passes and the steel may drop below the required interpass temperature. When multipass welds are made, it is more likely that additional heating will be needed to maintain the minimum interpass temperature for each pass.

An interesting phenomenon often occurs when preheating larger sections with oxyfuel torches: moisture condenses on the steel, even forming puddles of water on plates that are lying flat. The amount of condensation is large because the combustion of the heating gas produces water as a byproduct. This moisture from combustion occurs first in the air around the torch; this air heats up and is warmer than the steel near the joint, and so the moisture condenses onto the steel into liquid water. The moisture does not come from within the steel, as is often misconceived, and the purpose of preheat does not include “driving the water out of the steel.” This phenomenon is the same as that which occurs in automotive exhaust pipes on cooler mornings where water can be seen dripping from the exhaust until the exhaust pipe heats up. After preheating is complete, the steel where the heat was directly applied will be dry but condensation may form on the steel nearby. While water must be kept away from the joint itself since it is a source of hydrogen (section 5.3), condensed water in the general vicinity of the joint is normal and not deleterious provided it is not in the joint at the time that the arc encounters the joint.

The Bridge Welding Code has minimum preheat temperature requirements in clause 4 for typical welding and clause 12 for fracture-critical welding, which is discussed in chapter 7 of this manual. Fabricators may also establish alternate preheat criteria (clause 4.2.1.1), though this is not common. The Code also has requirements regarding the extent of preheat and interpass temperatures as well as where to measure them (clause 4.2.6). Preheat is required “in all directions,” which includes through the thickness to the other side of the part to be welded. This helps ensure that the part is properly soaked in heat such that preheat will be effective in controlling hydrogen.

Fabricators typically use temperature crayons for checking preheat and interpass temperatures because their use is very convenient. The crayons are available in 25 °F increments, and they melt on surfaces at or above the rated temperature for the crayon. For example, if a preheat temperature of 150 degrees is required, the welder should regularly swipe the steel with a 150-degree crayon while welding. If the temperature is not high enough, the crayon will not melt off but will make dry marks; when the marks melt and appear wet, the welder knows that the steel is at or above 150 degrees. The reverse is the case when crayons are used to control maximum interpass temperature: the crayon makes wet marks if the steel is too hot and welding should not commence until the steel has cooled. Welding may proceed when the crayon marks are dry.

Infrared thermometers (a.k.a. non-contact pyrometers) may be used to measure the temperature of steel. Contact with the steel is not needed. The digital displays may provide a precise temperature of the steel from a short distance away. As compared to temperature crayons, they are more costly and not as user-friendly; crayons provide the welder with the most straightforward means of knowing whether or not preheat or interpass temperature is sufficient.

5.3. HYDROGEN CONTROL

Hydrogen is known to cause problems in welds when not controlled. The primary issue is hydrogen-assisted cracking (HAC), also known as “cold cracking” or “delayed cracking” because the cracks associated with hydrogen may be delayed—occurring later, after welding is complete and after the weld cools to ambient temperature. However, hydrogen-assisted cracking may occur as soon as the weld and steel cool to ambient temperature, without any delay. Hydrogen-assisted cracking is discussed in section 5.4.2, along with other types of cracks. Hydrogen can also adversely affect ductility. There are many provisions in the Bridge Welding Code in place to help control these issues. This section provides a description of the mechanism by which hydrogen causes issues, and then describes code requirements for hydrogen control of consumables. Other aspects of hydrogen control are addressed elsewhere in this manual.

5.3.1. The Mechanism of Hydrogen Problems

The mechanisms that cause hydrogen problems are complicated. They are explained here because hydrogen is an ever-present dynamic associated with welding, and engineers will frequently encounter discussions of topics such as hydrogen, diffusible hydrogen, low-hydrogen electrodes, hydrogen control and hydrogen-assisted cracking (as well as cold cracking and delayed cracking). However, engineers generally do not need to be concerned with these details. The Bridge Welding Code has many requirements in place for hydrogen control, including welding practices such as preheat; requirements for base metals, such as joint cleaning; and requirements for welding consumables, such as electrode and flux exposure limits. These have been demonstrated to be very effective for avoiding hydrogen concerns over many decades.

Generally, this is the mechanism associated with hydrogen problems in weldments:

- During welding, hydrogen dissolves into the weld pool at the high temperatures of welding.
- The dissolved hydrogen may be divided into two categories:
  - Hydrogen that is free to diffuse (travel) through the metal, and
  - Residual hydrogen which is not free to diffuse at room temperature, and does not contribute to cracking.
- The distinction between these two is emphasized here to help define “diffusible hydrogen”; generally this manual will use the term “hydrogen” to mean diffusible hydrogen.
- As the weld metal cools and solidifies, the solubility of the hydrogen reduces; diffusible hydrogen begins to diffuse out of the weld.
- Hydrogen continues to diffuse over time; the higher the temperature of the steel, the faster hydrogen diffuses out.
- Eventually the weld cools to ambient temperature and some hydrogen remains in the weld.
- The remaining hydrogen accumulates at discontinuities in the microstructure lattice; these sites of accumulation are known as hydrogen traps.
- The residual stress that results as the weld cools and shrinks pulls at the traps.
- Depending upon the amount of hydrogen and the strength of the microstructure (hence its susceptibility), the discontinuities may grow into cracks under strain from residual stress.

- If cracks do form, the cracks are now larger traps where yet more hydrogen that is still diffusing through the weld accumulates, growing the cracks.
- Small levels of hydrogen, insufficient to cause cracking, can reduce the ductility of the weld and HAZ.

The amount of hydrogen trapped in the weld therefore depends upon the amount of hydrogen available in the first place and the speed of diffusion, which in turn is dependent upon the temperature of the weld over time. The susceptibility of the weld microstructure depends upon the original susceptibility of the base metal and the weld metal and, once again, the cooling rate of the weld because the cooling rate influences the final weld microstructure and grain size.

Hence, the mechanism of HAC is dependent upon the residual stress of the weld, the amount of hydrogen available in the weld, and the susceptibility (hardness) of the weld microstructure to HAC. Residual stress is inherent in welds and cannot be prevented; however, the use of preheat helps distribute residual stresses more evenly, reducing the magnitude of local residual stresses. It is not possible (or necessary) to completely eliminate hydrogen in the weld, but it can be reduced through hydrogen control practices, and its diffusion out of the weld can be facilitated through preheat and interpass controls and, when necessary, postheat. “Postheat” in this context means maintaining the steel at an elevated temperature immediately after welding. The susceptibility of the weld microstructure can be controlled by the use of base metals and consumables prescribed in the code and also through preheat and interpass temperature controls during welding. Section 5.4 discusses discontinuities, including cracking associated with hydrogen (section 5.4.2). Practices associated with hydrogen control include preheat and interpass temperature controls (section 5.2) and welding consumable controls (discussed in the next section).

5.3.2. Welding Consumable Hydrogen Controls

The Bridge Welding Code prescribes hydrogen control practices for consumables, including requirements for packaging, storage, and handling (including exposure limits). SMAW electrodes are required to be “of the low hydrogen classification” (clause 4.5.1). Low-hydrogen electrodes are SMAW electrodes whose coating must satisfy special moisture coating limits as specified in the electrode’s governing AWS A5 specification. SMAW electrode coatings may be hygroscopic (able to absorb moisture from the air), so these electrodes are required by the code to be delivered in hermetically sealed packaging. The code also prescribes the use of rod ovens for SMAW electrodes once they are removed from packaging to prevent moisture absorption during storage. SAW flux is not as hygroscopic as SMAW electrode coatings but can also absorb moisture, so the code has requirements for flux control after it has been removed from packaging (clause 4.8.2) and drying requirements for use of reclaimed flux (clause 4.8.3).

AWS filler metal specifications include a means of classification based on the hydrogen content of welds made under test conditions. Values are published in terms of milliliter (mL) of diffusible hydrogen per 100 grams of weld metal. Electrodes capable of depositing welds with controlled hydrogen contents are classified with “H” designators. For example, an H16 electrode is capable of depositing weld metal with less than 16 mL per 100 grams of weld metal under specific (and demanding) test conditions.

5.4. WELD DISCONTINUITIES AND DEFECTS

The terms “discontinuity” and “defect” are frequently used to generally describe weld quality issues. The terms are sometimes used interchangeably, which is incorrect, because there are important differences between the two terms. A third related term, “flaw”, is often used incorrectly as well. AWS 3.0 (AWS, 2010c) defines each term as follows:

- **Discontinuity - “**An interruption of the typical structure of a material, such as a lack of homogeneity in its mechanical, metallurgical, or physical characteristics. A discontinuity is not necessarily a defect. See also defect and flaw.”
- **Flaw** - “An undesirable discontinuity. See also defect.”
- **Defect - “**A discontinuity or discontinuities that by nature or accumulated effect render a part or product unable to meet minimum applicable acceptance standards or specifications. The term designates rejectability. See also discontinuity and flaw.”

Discontinuities may or may not be rejectable; minor discontinuities are certainly acceptable. Defects are unacceptable and require repair or replacement of the defective member. Generally, the Bridge Welding Code uses the term “defect” when discussing weld quality issues.

Internal discontinuities may be broadly characterized as either planar or volumetric. Profile concerns, such as undersized welds and excessive concavity, are a third category of discontinuity. Each type is discussed below. Cracks are a type of planar discontinuity but are presented in a separate section.

The severity of a discontinuity is dependent upon its notch effect, i.e., whether or not the discontinuity in question creates a stress concentration (also known as “stress raiser” or “stress riser”) and, if so, the severity of the stress raiser. The severity of the stress raiser is dependent upon the amount of stress change in the stress field and on the size, shape, and orientation of the discontinuity.

5.4.1. Planar Discontinuities

Planar discontinuities include incomplete fusion, cracks, and tears.

5.4.1.1. Incomplete Fusion

Incomplete fusion is “a weld discontinuity in which fusion did not occur between the weld metal and the fusion faces or the adjoining weld beads.” (AWS, 2010c). Incomplete fusion (figure 53) is the result of the molten weld metal not fusing with the base metal or with previously deposited weld passes. Incomplete fusion may be called “lack of fusion”, abbreviated as “LOF”, or it may be referred to as “cold lap”.

Incomplete fusion may be caused by many factors, including:

- Improperly selected or improperly prepared weld joint detail
- Welder being unable to position the electrode properly with respect to the joint, whether due to limited access or poor skill
- Use of improper welding parameters, such as extremes in amperage and travel speed (either high and low)
- Welding on materials with excessive mill scale

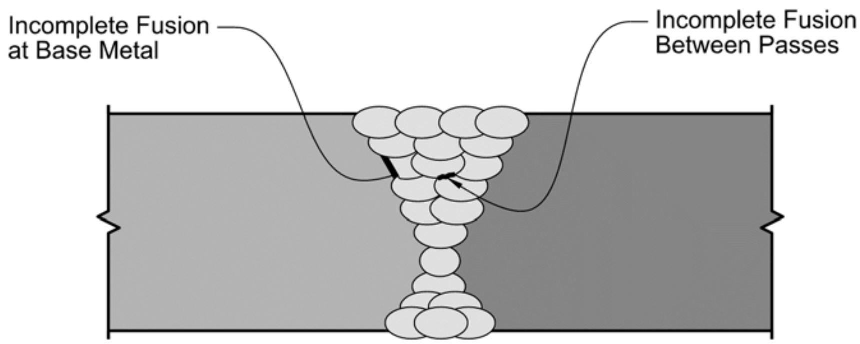


Figure 53. Illustration. Incomplete fusion (modified by authors).

5.4.1.2. Incomplete Joint Penetration

Incomplete joint penetration is "a joint root condition in a groove weld in which weld metal does not extend through the joint thickness." (AWS, 2010c). Incomplete joint penetration, also called "lack of penetration", has various causes:

- For a CJP groove weld like that shown in figure 54, incomplete joint penetration may be the result of improper backgouging (section 4.4.3) of the double-sided joint detail.
- For partial joint penetration welds where a prescribed amount of penetration is specified, incomplete joint penetration may be the result of:
    - Incorrect electrode placement
    - Improper welding procedure (typically with low current levels)
    - Improperly prepared joint

The standard double-sided CJP groove weld joint details listed in the Bridge Welding Code require backgouging. Typical examples of such welds are girder web and flange splices. Generally, experienced bridge fabricators are adept at completing these joints, and incomplete joint penetration is not common. However, if it does occur, it is readily found with nondestructive evaluation (NDE).

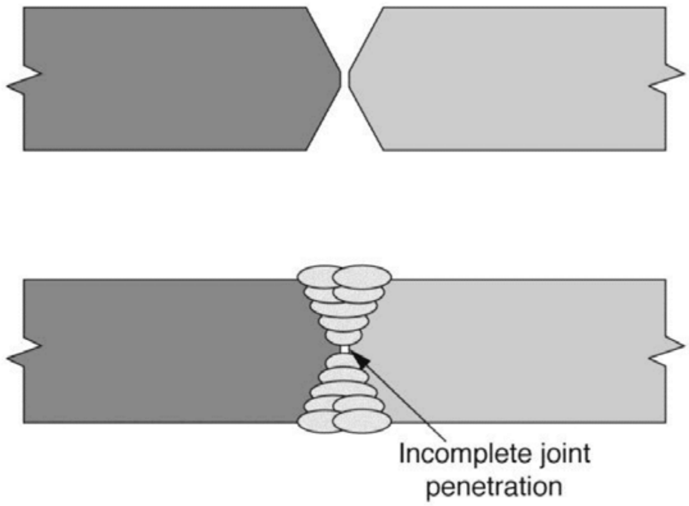


Figure 54. Illustration. Incomplete joint penetration.

5.4.1.3. Overlap

Overlap is “the protrusion of weld metal beyond the weld toe or weld root.” (AWS, 2010c). While the overlap itself is volumetric, the discontinuity it creates is planar. Overlap is an example of an incomplete fusion discontinuity that occurs on the surface of the steel as shown in figure 55.

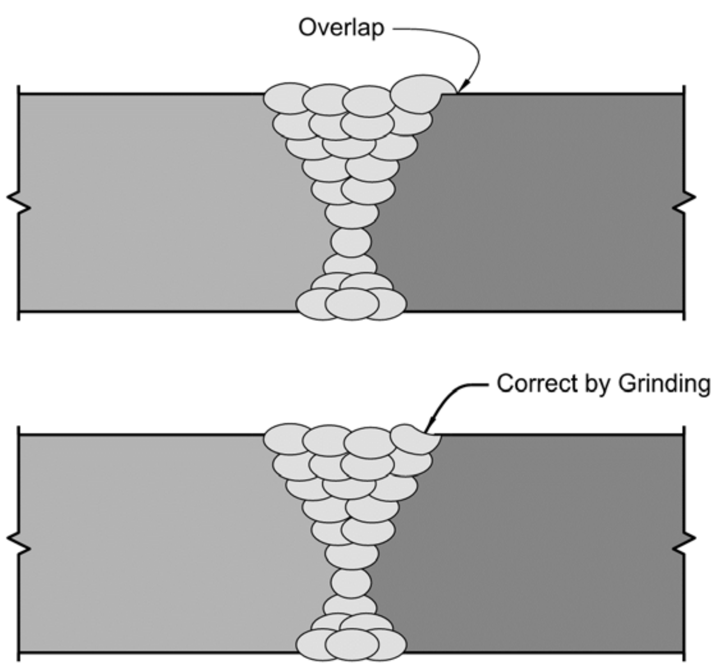


Figure 55. Illustration. Weld overlap and grinding (modified by authors).

The likelihood of overlap may be aggravated by the presence of thick mill scale, but overlap is more often associated with improper procedures or techniques. Excessively low travel speeds may cause the molten puddle to roll ahead of the arc, resulting in overlap. Often, welds with overlap can be corrected by carefully removing the overlapped weld metal by grinding. Overlap in a groove weld is shown in figure 55; the figure also illustrates repair by grinding. Overlap in a fillet weld is shown in figure 56.

Source: FHWA

Figure 56. Photo. Overlap in a fillet weld.

5.4.1.4. Laminations and Delaminations

Laminations and delaminations are planar base metal discontinuities lying parallel to the surface of the steel. The term "lamination" is used when there is essentially no gap between the two surfaces of the planar discontinuity. If the surfaces separate and a gap is formed, the term "delamination" is used. Laminations and delaminations are mill discontinuities and typically occur in the mid-thickness of the steel, whereas lamellar tearing (section 5.4.1.5) occurs during welding, and is usually located just outside the heat-affected zone, generally within ¼ inch of the steel surface. Laminations may be observed on the edge of the steel when the material is thermally cut. Non-destructive testing can be used to detect laminations that are away from the cut edge.

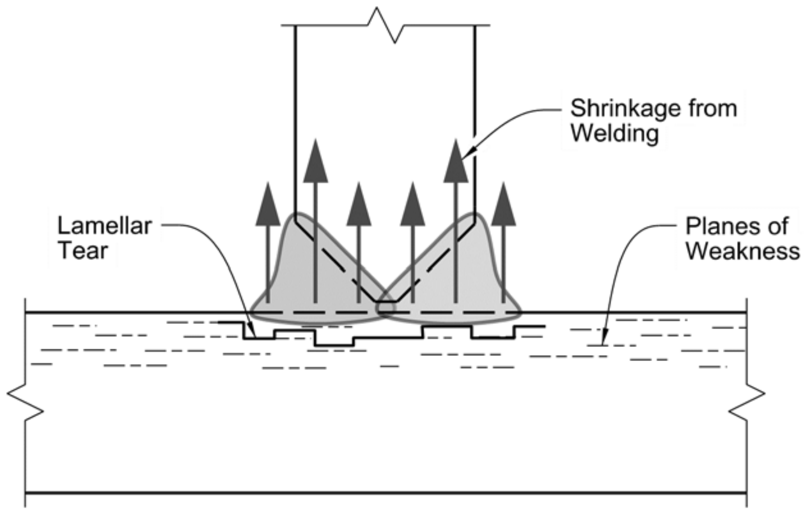
Plates or shapes that contain minor laminations are usually acceptable for service when the discontinuity is parallel to the stress field. Thus, for planar base metal discontinuities that do not open to edges of the base material, the Bridge Welding Code does not require repair (clause 3.2.3.7(1)). Delaminations may also be acceptable. Clause 3.2.3.7 provides specifics for when delaminations may remain or need to be repaired. The code allows some delaminations on edges and bevel faces (discontinuities "Y" and "W" in clause 3.2.3.7) to remain. In practice, fabricators will often repair discontinuities on bevel faces to avoid problems that can occur during welding. If delaminations on bevel faces are not repaired (by welding) before production welding begins, they can open up from preheat or the heat of the production passes. The weld metal will not fuse properly to the base metal at these delaminations if they are not repaired; such lack of fusion will be revealed by RT or UT examination. Also, if a joint is to be tested with UT, a lamination scan of the joint is required to ensure there are no laminations present to interfere with the UT of the joint. Therefore, the anticipation of UT is another reason the fabricator may choose to repair delaminations at a joint.

Laminations or delaminations that are perpendicular to a tensile stress field are of a much greater concern than planar discontinuities that are parallel to the applied stress field. Fortunately, the condition of perpendicular loading is not commonly encountered in typical plate girder bridges. Perpendicular tensile loading may occur in specialized bridge components, such as a hold-down bearing assembly.

5.4.1.5. Lamellar Tearing

A lamellar tear is a welding-related type of cracking that can occur in the base metal. The AWS definition is “[a] subsurface terrace and step-like crack in the base metal with a basic orientation parallel to the wrought surface created by tensile stresses in the through-thickness direction of the base metals weakened by the presence of small disperse, planar-shaped, nonmetallic inclusions parallel to the metal surface.” (AWS, 2010c). The tearing is caused by welding shrinkage strains acting perpendicular to planes of weakness in the steel and typically occurs slightly outside the HAZ. These areas of weakness are the result of inclusions in the base metal that have been flattened into thin, oval-shaped discontinuities during the rolling process, which are roughly parallel to the surface of the steel and separate when strained. Since the various individual inclusions are on different planes, the resulting fracture moves from one plane of inclusions to another, resulting in the “stair-stepped” pattern of fractures illustrated in figure 57.

Lamellar tearing usually occurs while the weld is shrinking and cooling; however, it can also occur later due to additional shrinkage stresses that result from welding on another part of the work. Ultrasonic inspection (section 6.5.2.3) is a good method for detecting lamellar tears. Current steelmaking practices, including the shift to continuous casting roughly from the 1970s to the mid-1990s and reduction of sulfur in the 1980s and 1990s, have helped minimize lamellar tearing tendencies.



Source: FHWA

Figure 57. Illustration. Lamellar tearing.

5.4.2. Cracks

A crack is "a fracture-type discontinuity characterized by a sharp tip and a high ratio of length and width to opening displacement." (AWS, 2010c). Cracks are a serious concern in bridge fabrication and are always required to be removed if they are detected. However, they are also unusual because welding in accordance with the code usually avoids them, particularly on routine joints. When cracking does occur, it is generally indicative of highly restrained welds or out-of-conformance conditions, such as an out-of-specification steel or filler metal, improperly preheated steel, or improper consumable storage conditions.

This section describes cracks that may occur in the shop during fabrication. Cracks that may occur in service from fatigue or fracture are not discussed in this section.

5.4.2.1. Centerline Cracking

Centerline cracking is a separation near the center of a given weld bead. If the weld bead happens to be in the center of the joint, as is always the case on a single-pass weld, centerline cracks will generally be in the center of the joint (see figure 58). In the case of multiple-pass welds where several beads per layer may be applied, a centerline crack may not be in the geometric center of the joint, although it will always be near the center of a weld bead.

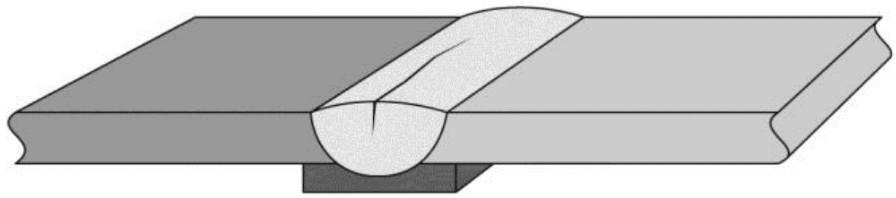


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Figure 58. Illustration. Centerline cracking.

Centerline cracking results from one of the following three phenomena: segregation-induced cracking, bead-shape–induced cracking, or surface-profile–induced cracking, as discussed in the following sections. Regardless of cause, centerline cracks are hot cracks that are solidification-related, occurring during the cooling of the weld bead. The volumetric contraction that occurs as the metal transforms from a hot liquid to a hot solid and then to a warm solid creates the driving force for this type of cracking. As hot cracks, centerline cracks will either be present immediately after solidification has taken place, or they will not occur. Centerline cracks typically extend to the surface (face) of the weld. In some situations, centerline cracks can be internal to the individual weld bead, and not be surface-breaking, although this is the exception.

5.4.2.2. Segregation-Induced Cracking

Segregation-induced cracking occurs when low–melting-point constituents from the base metal in the molten weld segregate during the weld solidification process. As the weld metal solidifies, elements and compounds with low melting temperatures are concentrated into the liquid metal and move towards the center of the weld bead or between the solidifying grains of weld metal. The enrichment of the remaining liquid material with the low–melting-point materials can lead to segregation-induced cracking. This mechanism is illustrated in figure 59, which shows the solidification sequence of a weld bead. Such cracks are rare in modern bridge fabrication because the steels used for bridge fabrication control the low–melting-point ingredients to acceptable levels. However, such controls on steels were not always present, and for welding on existing structures, particularly those that were originally riveted, this type of cracking is more prevalent.

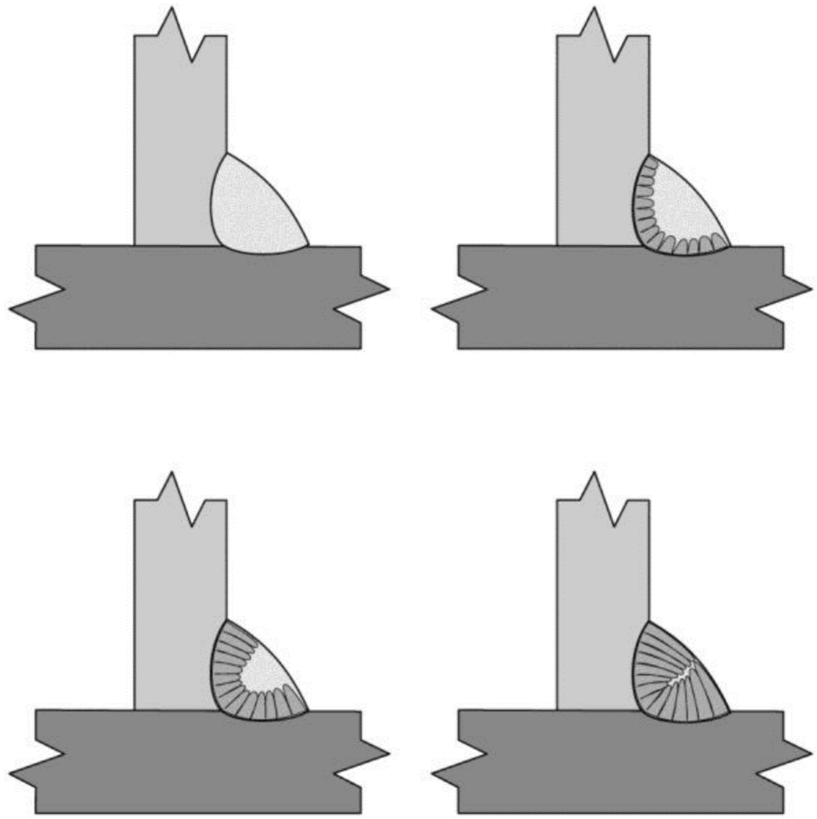

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Figure 59. Illustration. Grain growth and segregation-induced centerline cracking.

5.4.2.3. Bead-Shape–Induced Cracking

The second type of centerline cracking, illustrated in figure 60, is known as bead-shape–induced cracking. When the cross-section of a single weld bead is of a shape where there is more depth (D) than width (W), the solidifying grains grow generally perpendicular to the steel surface to which they are attached and intersect in the middle but may not achieve fusion across the joint. To correct for this condition, the individual weld beads must have at least as much width as depth; as discussed below, this is achieved through proper joint design.

Bead-shaped–induced cracking can occur on beads but is not a concern for the overall width-to-depth ratio of multipass welds. The final overall weld configuration may have a profile that constitutes more depth than width, as is very common in bridge flange splicing. Such multiple-pass welds are produced without fear of centerline cracking by welding each individual weld bead with more width than depth.

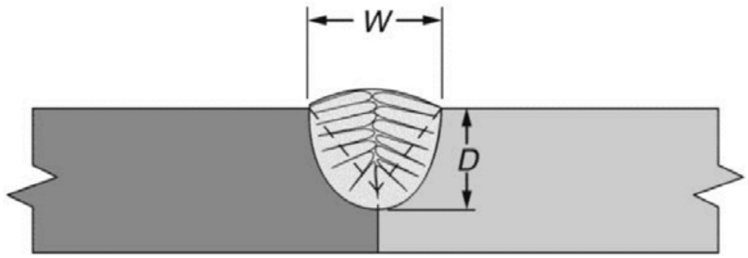

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Figure 60. Illustration. Width-to-depth ratio for bead-shape-induced centerline cracking.

Joint design affects the tendency toward centerline cracking induced by bead shape. The standard joint details in the Bridge Welding Code have taken this into account. As the included angle is decreased, the tendency toward a narrow, deep bead at the root increases. To compensate for this, a larger root opening is used. The corner joints in figure 61 illustrate this point. Standard joints have a minimum of a 60-degree included angle when the SAW process is used. With the deep penetration afforded by this process, a 45-degree included angle could lead to unacceptable centerline cracking. Other welding processes may use smaller included angles because their weld beads are shallower and do not create this width-to-depth ratio.

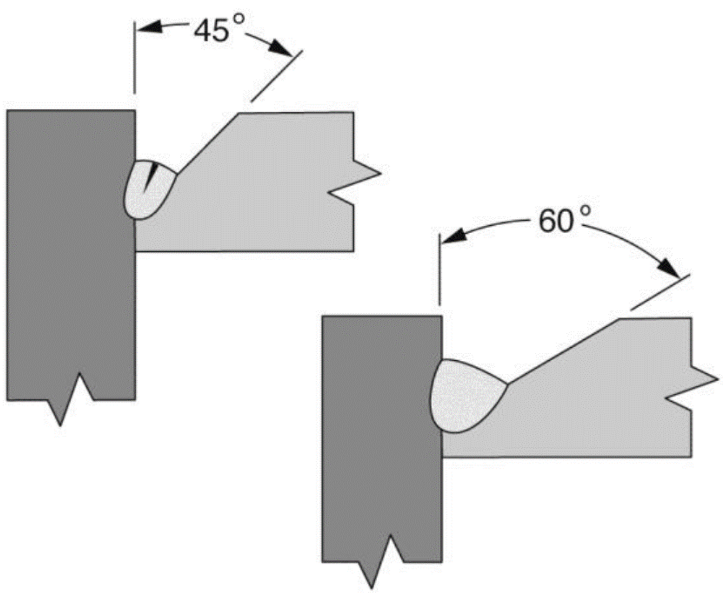

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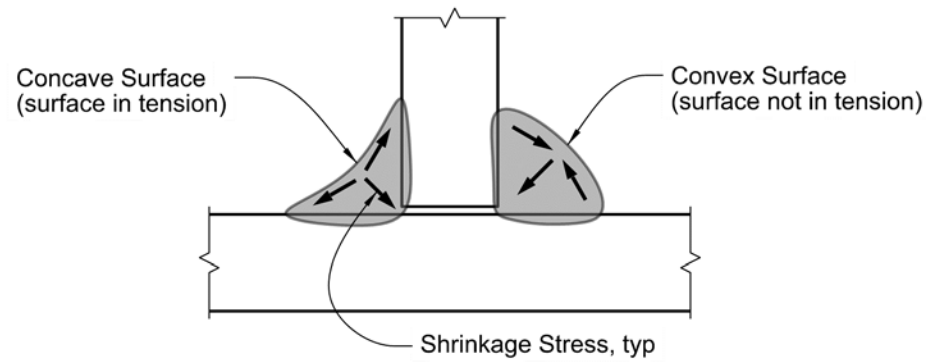
Figure 61. Illustration. Potential effect of included angle on bead-shape-induced centerline cracking.

Centerline cracking due to bead shape can occur in both groove and fillet welds. It rarely occurs in fillet welds when applied to 90-degree T-joints because the weld face width is usually about twice the weld throat dimension (for welds made with fusion to the root and not much beyond, as shown in fillet weld "c" of figure 4). For skewed T-joints, the side with the acute angle will be more sensitive to centerline cracking; such cracking has occurred on occasion when fabricating tub girders.

5.4.2.4. Surface-Profile-Induced Cracking

A third mechanism that generates centerline cracks is the surface profile condition. Depending on factors such as the amount of concavity and the size of the weld, centerline cracks can occur. When the surface of an individual weld bead is concave, internal shrinkage stresses will place the weld metal on the surface into tension. Conversely, when a convex weld surface is created, the internal shrinkage forces pull the surface into compression. These situations are illustrated in figure 62.

Concave weld surfaces usually are the result of high arc voltages. A slight decrease in arc voltage will cause the weld bead to return to a slightly convex profile and eliminate the cracking tendency. Vertical-down welding (see vertical welding, figure 6) also has the tendency to generate these crack-sensitive, concave surfaces; vertical-down welding is not permitted by the Code (clauses 4.6.8 and 4.13.1.7). Vertical-up welding provides a more convex bead (see figure 50).



Source: FHWA

Figure 62. Illustration. Surface profile and bead-shape-induced centerline cracking.

5.4.2.5. Heat-Affected Zone Cracking

Heat-affected zone (HAZ) cracking is a type of hydrogen-assisted cracking (HAC, section 5.3) that is characterized by base metal separation that occurs in the region immediately adjacent to the weld bead (figure 63). Although this cracking is related to the welding process, the crack occurs in the base metal, not the weld metal. This type of HAC is also known as “underbead cracking,” or “toe cracking”.

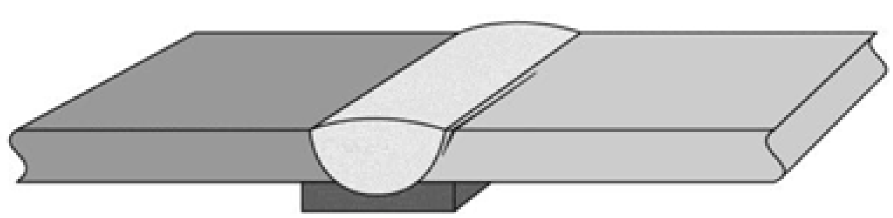


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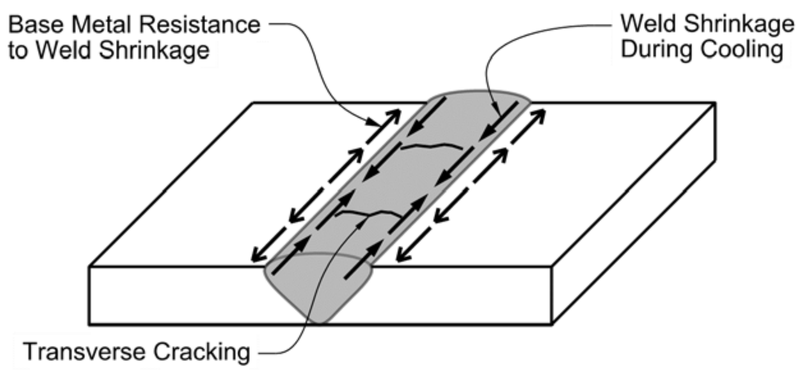
Figure 63. Illustration. Heat-affected zone cracking in based metal.

As a type of hydrogen-associated cracking, heat-affected-zone cracking occurs due to the three factors described in section 5.3: applied or residual tensile stresses, sufficient level of hydrogen, and a susceptible HAZ microstructure. Weld HAZ microstructures are generally harder than other base metal due to effects from the heating and cooling of welding and are therefore more susceptible to HAC. Reiterating from section 5.3, however, HAC is usually avoided through use of proper preheat and interpass temperatures in conjunction with Bridge Welding Code materials and consumables.

5.4.2.6. Transverse Cracking

Transverse cracking, also called "cross cracking", is characterized as a crack within the weld metal, perpendicular to the longitudinal axis of the weld (see figure 64). Transverse cracking is another form of hydrogen-associated cracking and, like HAZ cracking, is driven by the same three factors: excessive hydrogen, a susceptible microstructure, and applied or residual tensile stress. As the weld shrinks longitudinally, the surrounding base material resists this shrinking force. The strength of the surrounding steel in compression restricts the ability of the weld material to shrink. When the weld material is high in strength, it has a reduced capacity to plastically deform, and if hydrogen is excessive, the weld metal may crack in the transverse direction.



Source: FHWA

Figure 64. Illustration. Transverse cracking.

As with HAZ cracking (section 5.4.2.5), transverse cracking is rare in bridge fabrication because of bridge fabrication practices and Bridge Welding Code requirements regarding combinations of base metal and weld metal. Transverse cracking was common when ASTM A852 steel was used in welded fabrication, but with the development of HPS 70W, these problems were greatly reduced. Transverse cracking is generally associated with high-strength weld metal whose strength greatly exceeds that of the base metal (overmatching). The code effectively prohibits overmatching. Of the base metals listed in the Bridge Welding Code, welds on grade HPS 70W and HPS 100W steel are the most sensitive to transverse cracking. Undermatching weld metal for the web-to-flange connections of these higher strength steels significantly mitigates transverse cracking tendencies.

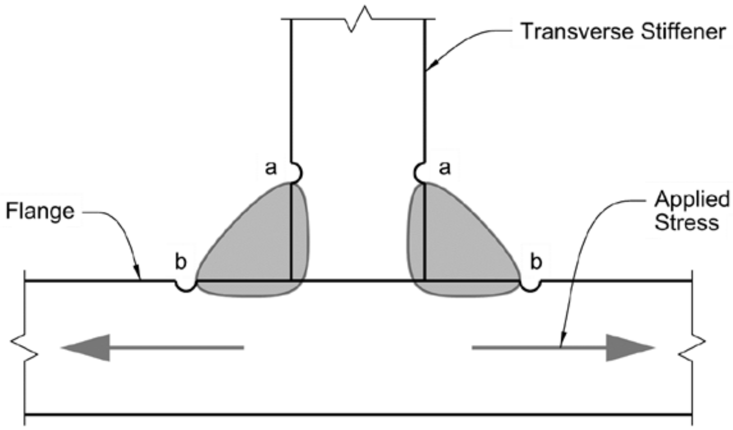
5.4.3. Volumetric Discontinuities

Volumetric discontinuities are three-dimensional imperfections located in and around the weld. Some volumetric discontinuities have rounded or blunted edges that create a less severe stress concentration than the crack-like edges of planar discontinuities.

5.4.3.1. Undercut

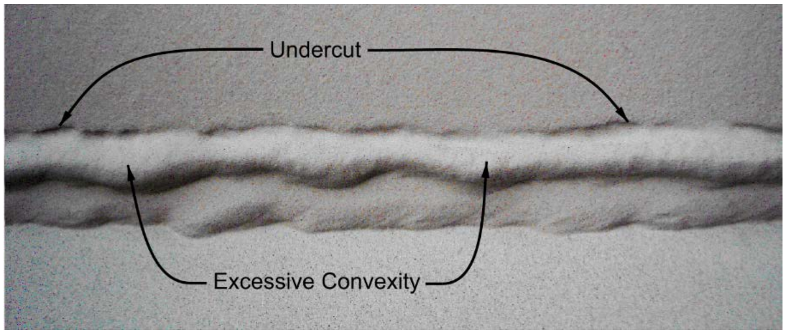
Undercut is "[a] groove melted into the base metal adjacent to the weld toe or weld root and left unfilled by weld metal." (AWS, 2010c). See figure 65 for an example. Excessive undercut is usually associated with poor welding procedures or techniques, such as improper electrode placement, high arc voltage, or the use of improper welding consumables. There are two structural concerns associated with undercut. First, undercut may create a small, notch-like cavity that will act as a stress raiser. Secondly, undercut may reduce the section of the base metal in reference to design stresses to an unacceptable level, particularly on thin members.

Undercut that is perpendicular to the applied stress on a bridge tension component is a stress raiser and detrimental to fatigue performance, so the Bridge Welding Code only accepts a depth of 0.01 inch for this condition (clause 6.26.1.5(1)). Undercut parallel to the tensile stress on a main member component may be $^{1}/_{32}$ inch deep (clause 6.26.1.5(2)). For example, if the welds for the joint shown in figure 65 are stiffener-to-flange fillet welds, then an undercut of $^{1}/_{32}$ inch is allowed at "a", and the allowable undercut at "b" is 0.01 inch. For practical purposes, an undercut of 0.01 inch is difficult to measure and quantify in the shop, and therefore, a common practice on the shop floor is that any detectable undercut perpendicular to the applied tensile stress is addressed. An undercut that is $^{1}/_{32}$ inch deep is more readily measured quantitatively in the shop. Figure 66 shows undercut as well as excessive convexity.



Source: FHWA

Figure 65. Illustration. Undercut at stiffener-to-flange fillet welds.



Source: FHWA

Figure 66. Photo. Examples of weld undercut and excessive convexity.

5.4.3.2. Porosity

Porosity refers to “cavity-type discontinuities formed by gas entrapment during solidification or in a thermal spray deposit” (AWS, 2010c). Porosity assumes the form of spherical or cylindrical cavities in the weld. See figure 67 for details. Porosity may be surface-breaking or may be internal to the weld. See figure 68 for a photo of surface-breaking porosity in a fillet weld.

Porosity occurs as the result of inadequate shielding of the weld metal, contamination of the weld joint, or both. The products used for shielding weld deposits must satisfy code requirements for quality, storage, and, for gases, gas flow rate to provide adequate shielding. Excessive surface contamination from oil, moisture, rust, or mill scale increases the demand for shielding. Porosity can be minimized by providing proper shielding and ensuring joint cleanliness in accordance with Bridge Welding Code requirements (clause 3.2.1). For SAW, flux that is reclaimed can become contaminated and can cause porosity unless code provisions are followed to keep reclaimed flux clean (clause 4.8.3), including magnetic separators to remove metal contaminants.

The Bridge Welding Code allows some amount of porosity to remain depending upon its type, size, distribution, loading condition, and location. This is described in clause 6.26 of the Bridge Welding Code. Welds with excessive porosity can be repaired by removing the unacceptable portions and restoring the removed material with sound weld metal. Defect removal is typically done with grinding or air carbon arc gouging (section 4.4.3). Only the weld metal that contains the excessive porosity must be removed; it is not necessary nor is it desirable to remove more weld metal than necessary.

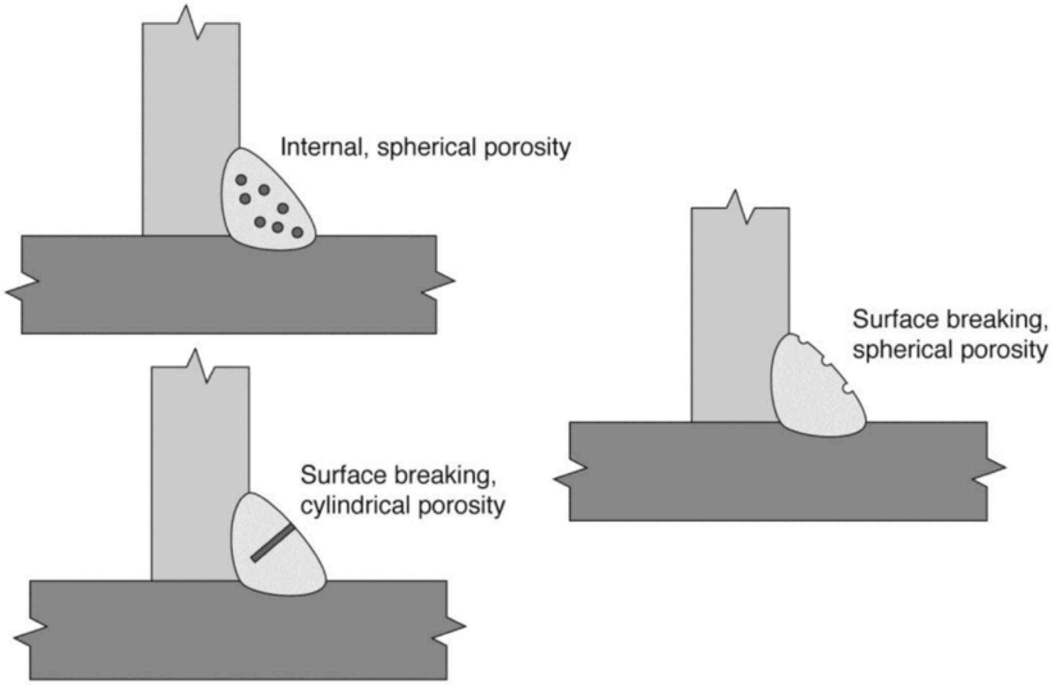


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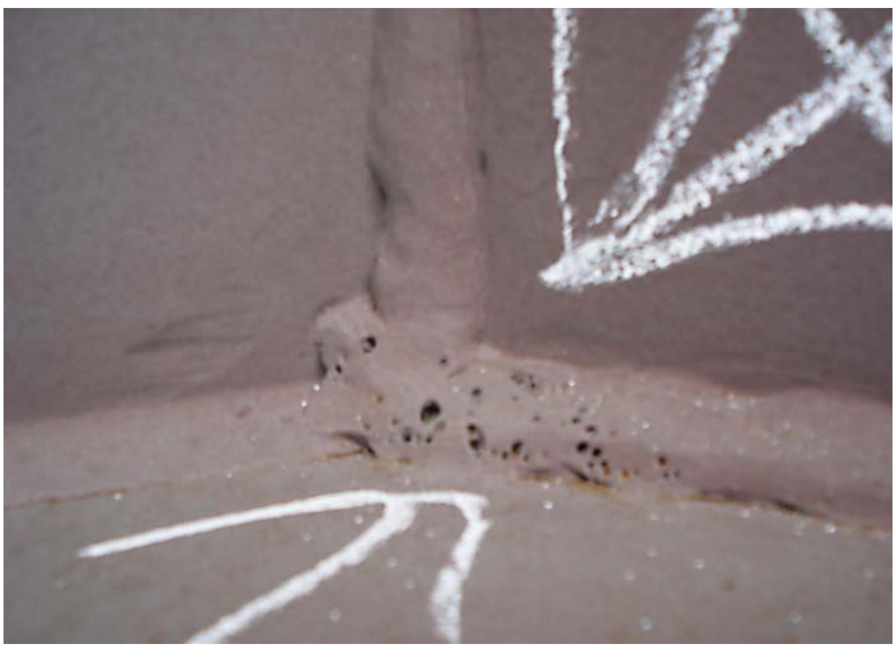
Figure 67. Illustration. Types of porosity.

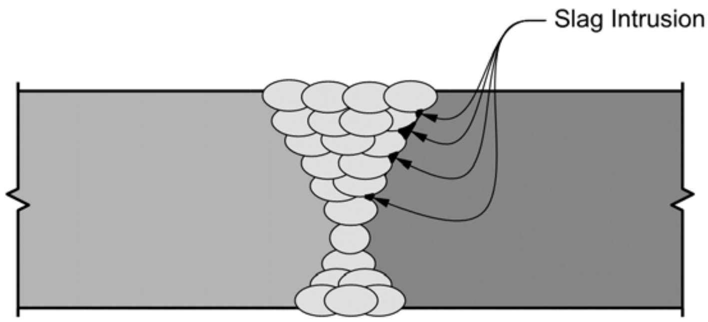


Source: FHWA

Figure 68. Photo. Example of porosity.

5.4.3.3. Slag Intrusions

A slag intrusion is “a discontinuity consisting of slag in weld metal or along the weld interface.” (AWS, 2010c). See figure 69 for an example. Slag intrusions are often referred to as “slag inclusions”. Slag intrusions are generally attributed to slag from previous weld passes that was not completely removed before subsequent passes were applied. Slag intrusions in completed welds are typically detected by ultrasonic or radiographic nondestructive testing.



Original figure: © 2018 Lincoln Electric

Figure 69. Illustration. Slag intrusions (modified by authors).

With proper welded joint designs, welding procedures, and techniques, slag can be easily removed from the joint as welding progresses, preventing the formation of slag intrusions. However, when welding conditions are sub-optimal, slag removal may be difficult. The typical location for trapped slag is at the toes of weld passes. Careful grinding of weld toes before the application of a subsequent weld pass is effective in minimizing the possibility of slag intrusions. During welding, the welder will determine whether or not grinding is necessary on each pass. Figure 70 shows slag that has been trapped in a CJP weld; this weld has been ground, but slag is still present.

Source: FHWA

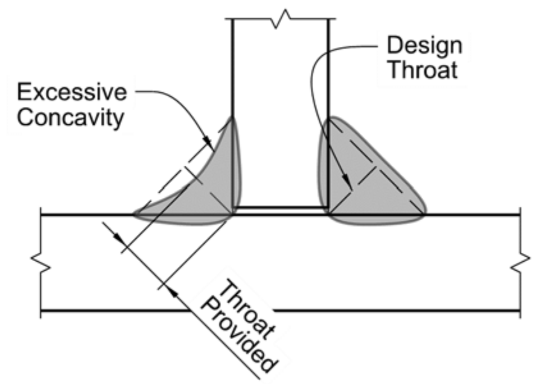
Figure 70. Photo. Example of slag trapped in a CJP weld

5.4.4. Weld Profile Requirements

The Bridge Welding Code has requirements for the profile of the surface of groove and fillet welds The Code provisions address both expectations for good workmanship and, in some cases, conditions that can affect in-service performance.

5.4.4.1. Excessive Concavity

Concavity refers to the profile of the surface of the weld as shown in figure 71. Excessive concavity can lead to centerline cracking (see section 5.4.2.1) and undersized welds for design as shown in the figure. Excessive concavity is typically caused by an improper welding procedure or welding technique. Reducing the welding current and voltage, where applicable, will usually remedy this problem. Welds with concave surfaces and adequate throats are not a problem. Inadequate weld throats created by excessive concavity are easily corrected by depositing another weld pass on the concave surface.



Source: FHWA

Figure 71. Illustration. Excessive concavity.

5.4.4.2. Excessive Convexity

Convexity is considered excessive when it exceeds the limits in clause 3.6 of the Bridge Welding Code. As shown in figure 72, excessive convexity wastes weld metal and may increase the stress raiser at the weld toe if the stress field is perpendicular to the weld axis. Improper procedures and technique are generally responsible for this condition.

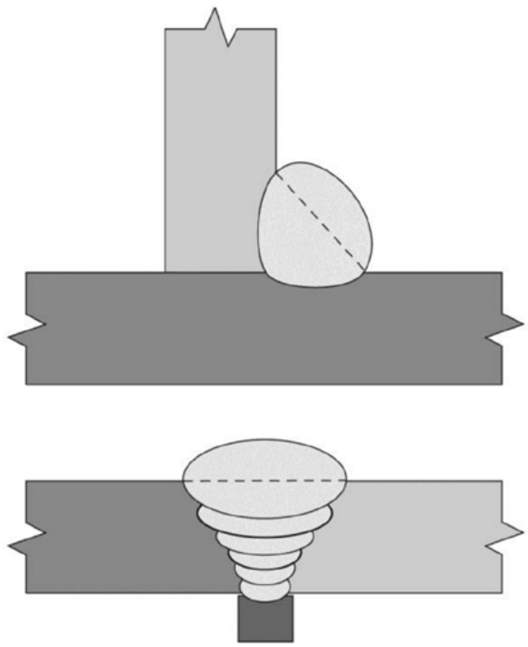


Figure 72. Illustration. Excessive convexity.

For welds made with excessive convexity, corrective measures typically involve removal of the excessive metal by grinding, with the toe region ground as necessary to smoothly transition from the base metal to the weld as shown in the bottom image of figure 73. A common mistake is to grind away the weld metal that created the excessive convexity and yet leave behind problematic stress raisers at the weld toe.

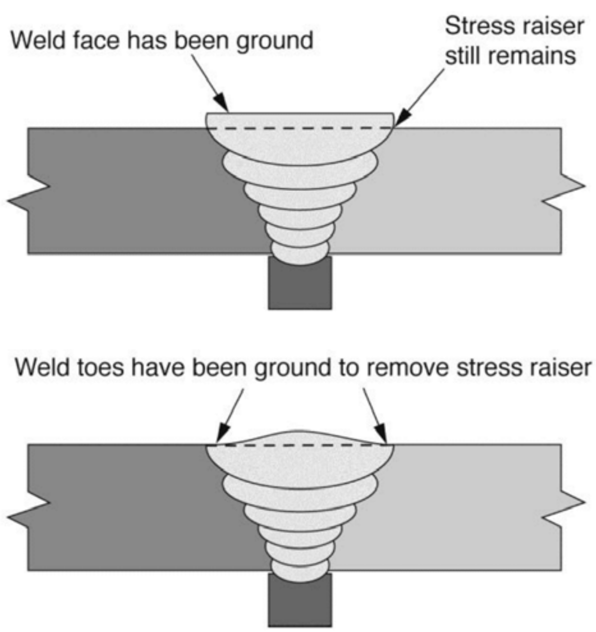


Figure 73. Illustration. Repair of excessive convexity.

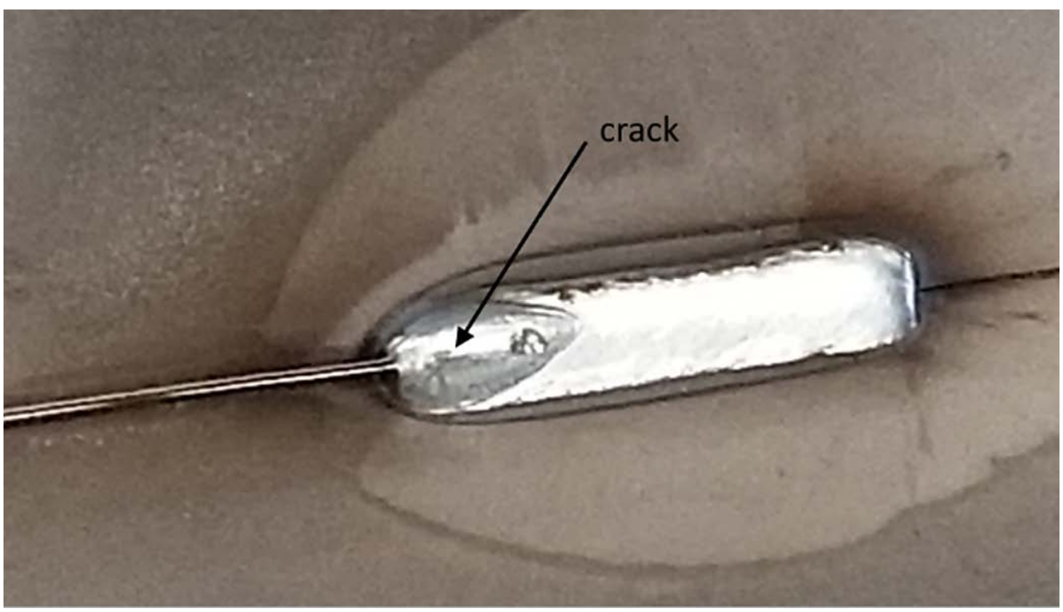
5.4.4.3. Inadequate Weld Size

Welds may be too short or too small for a given application. Fabricators routinely make fillet welds slightly larger than specified to avoid the problem of slightly undersized welds. Undersized welds are typically indicative of workmanship or procedural problems, often resulting from travel speeds that are too high. The Bridge Welding Code permits welds to be undersized within certain limits and at certain locations (clause 6.26.1.7), particularly because a certain amount of size variation is to be expected in long fillet welds.

Undersized welds may be remediated by depositing additional metal to the undersized weld. When the welds are only slightly sub-sized, the repaired weld will likely be significantly larger than required as is it difficult (and often undesirable) to deposit the very small repair welds needed to bring the weld into exact size compliance.

5.4.4.4. Underfilled Weld Craters

An underfilled weld crater is a concave depression at the end of the fillet weld; as a depression, in this localized area the weld throat is reduced. Underfilled weld craters are typically due to workmanship problems on the part of the welder. Normally, a slight pause at the end of a weld will fill a weld crater. All weld craters must be filled (clause 3.7.2.2). Underfilled weld craters can be repaired by depositing additional metal in the crater. However, underfilled weld craters can develop cracks with a star-like pattern (see figure 74); if so, the crater cracks should be repaired by removal of the cracked portion by grinding and replacing the removed material with sound metal.

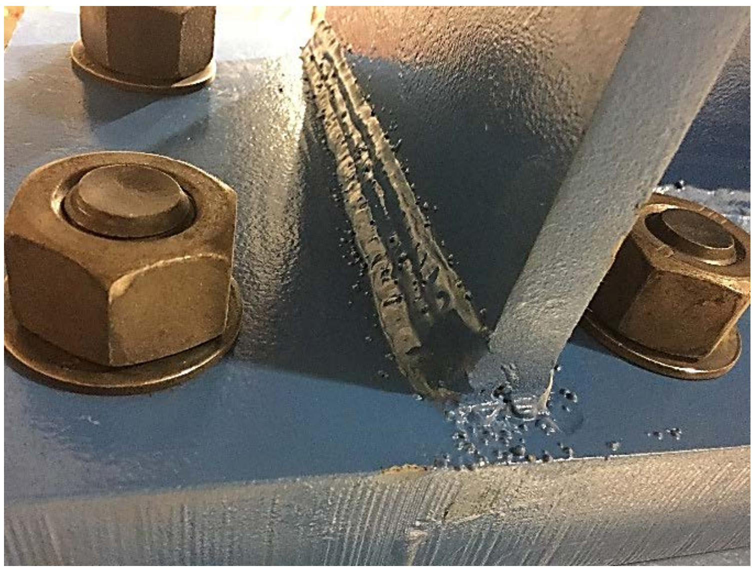


Source: FHWA

Figure 74. Photo. Example of crack in underfilled weld crater.

5.5. SPATTER

Spatter is defined as "the metal particles expelled during fusion welding that do not form part of the weld." (AWS, 2010c). Spatter consists of the roughly spherical particles of molten weld metal that fuse to the base metal outside the weld joint, or to the weld metal surface (see figure 75). Spatter is generally not considered to be harmful to the performance of welded connections, but spatter must be removed on surfaces that are inspected with magnetic particle testing (MT) (clause 6.7.6.3) or UT (clause 6.19.3). Though not specifically mentioned by the Code, it should also be removed before conducting RT or PT. Further, excessive spatter may affect the integrity of coating systems. Loose spatter is easily removed by scraping, while more tightly adhering spatter can be chiseled or ground off. In all cases, excessive spatter is indicative of less than optimum welding conditions and suggests that the welding procedures may need to be adjusted.

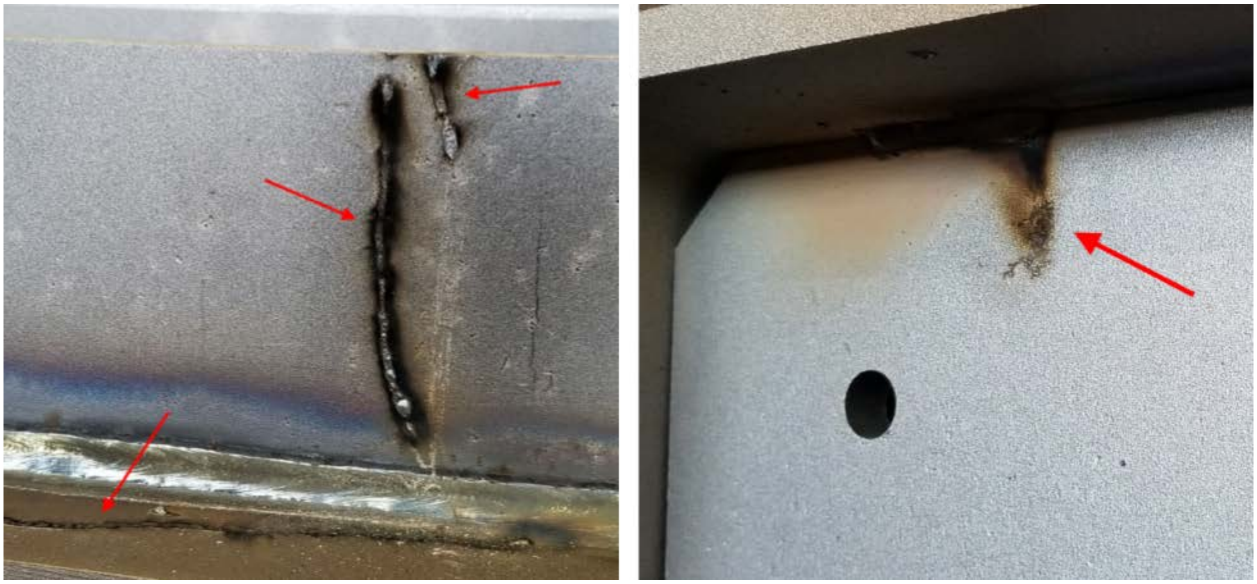


Source: FHWA

Figure 75. Photo. Example of weld spatter.

5.6. ARC STRIKES

An arc strike is "a discontinuity resulting from an arc, consisting of any localized melted metal, heat-affected metal, or change in the surface profile of any metal object." (AWS, 2010c). Arc strikes are caused by inadvertent arcing between electrically charged elements of the welding circuit and the base metal. Welding arcs that are initiated outside the joint leave behind these arc strikes (see figure 76). SMAW is particularly susceptible to creating arc strikes since the electrode holder is electrically "hot" (i.e., energized) when not welding, so inadvertent contact of the holder with the work causes strikes. For any of the welding processes, arcing from work clamps improperly attached to the base metal can cause arc strikes, and welding cables with damaged insulation can result in arc strikes. Welding cables should be insulated and in good condition. Proper welding practices minimize arc strikes.



Source: FHWA

Figure 76. Photos. Examples of Arc Strikes.

Arc strikes should be avoided; when arc strikes cause cracks or blemishes, they are to be ground smooth and checked to ensure soundness (clause 3.10). Removal of the affected metal by grinding will eliminate any potential harm from arc strikes. This includes the formerly melted metal, as well as any excessively hardened base metal.

5.7. DISTORTION AND SHRINKAGE

Distortion is the geometric deviation from the intended shape of a part or an assembly that is observable after welding is complete. Distortion is often a cosmetic issue, such as can be observed on fascia girders with stiffeners applied to the opposite side. Generally, the distortion limits in the code are based on workmanship and aesthetics and not the threat of the distorted condition as a buckling failure.

Distortion is caused by a four-step mechanism in which:

- Hot, expanded weld metal that has solidified and then shrinks as it cools from elevated temperatures
- A portion of the base metal is heated by welding, then expands, upsets (bulges because it is restrained by surrounding base metal), and cools from elevated temperatures
- The shrinkage (straining) as the weld cools creates stresses that act on the surrounding steel
- Flexible surrounding steel moves to accommodate the stresses created by the shrinking metal

As described in the types of distortion listed in section 5.7.1, the amount of distortion varies by the size of weld, the associated amount of heat, and local and system restraint.

5.7.1. Types of Distortion

5.7.1.1. Angular Distortion

Angular distortion is caused by the transverse shrinkage of the weld. When the hot metal contracts laterally, it pulls flexible members toward the weld as shown several ways in figure 77. Angular distortion can be caused by groove or fillet welds and can occur in any joint type. An example of angular distortion in a groove weld qualification test plate is shown in figure 78 and figure 79-A.

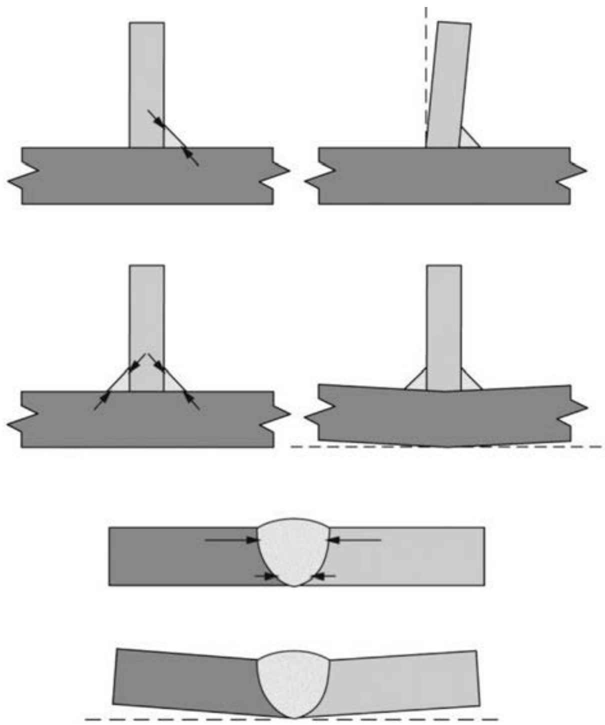


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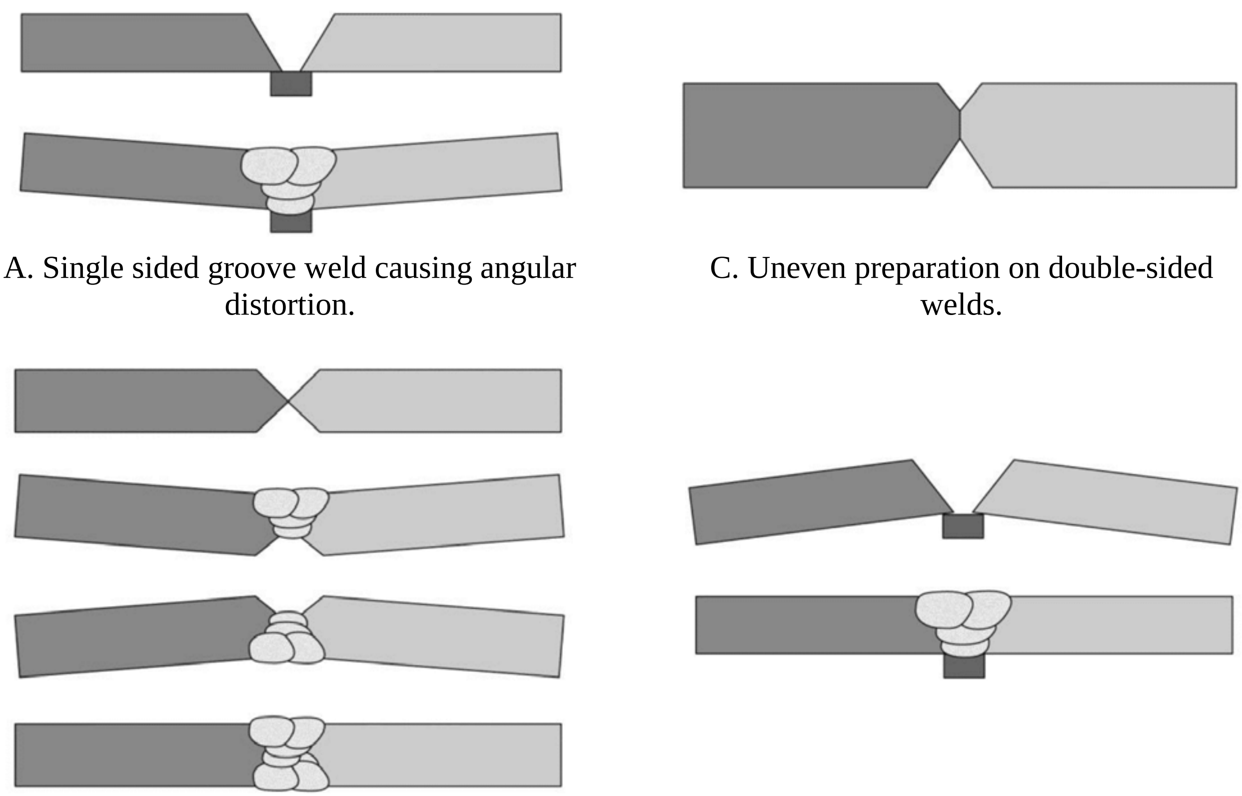
Figure 77. Illustration. Angular distortion.

Source: FHWA

Figure 78. Photo. Angular distortion on groove qualification test plates.

In some cases, angular distortion can be offset by using double-sided welds. For example, in web and flange butt joints, angular distortion can be managed to a great extent through use of double-V groove joints (figure 79-B), where the distortion caused by welding on the first side is offset by welding on the second side. The first welds are made when the assembly is more flexible, so the distortion associated with the first side weld is typically greater than from the second side. To compensate for this, the two sides of the double-sided welds can be prepared with different depths, as shown in figure 79-C. In this example, the smaller groove weld is made first, allowing the larger groove weld on the other side to offset the greater rigidity that would be experienced after the first weld is in place. Members can also be preset to minimize distortion, as shown in figure 79-D. Double-sided welds do not always eliminate all forms of angular distortion. For example, the double-sided fillet welds still cause the horizontal member to angularly distort in figure 77.



A. Single sided groove weld causing angular distortion.

B. Double sided groove weld compensates for angular distortion.

C. Uneven preparation on double-sided welds.

D. Presetting parts.

Figure 79. Illustrations. Techniques to limit angular distortion.

5.7.1.2. Transverse Shrinkage

As welds contract in width, they will cause transverse shrinkage when the edges of the parts being welded are free to move as shown in figure 80. As an example, flange and web butt joints slightly shrink the welded flanges and webs, making these components slightly shorter in length than they were before welding. Fabricator may compensate for this shrinkage by predicting it or by making initial components slightly longer and then trimming them to the proper length.

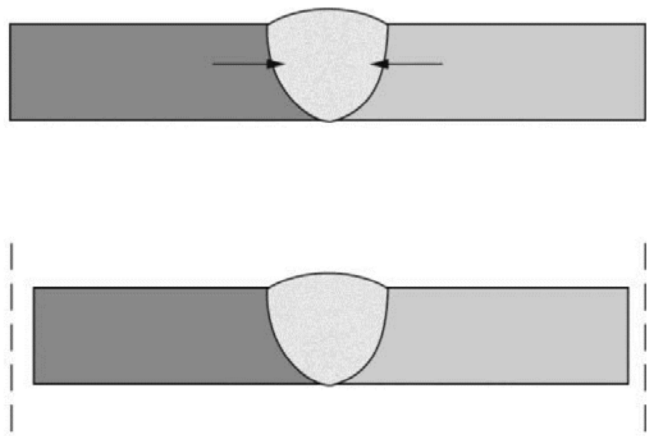


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Figure 80. Illustration. Transverse shrinkage.

5.7.1.3. Longitudinal Shortening

Longitudinal shrinkage of welds will cause an assembly to shorten, as shown in figure 81. The same shrinkage may result in twisting, longitudinal sweep or camber, or buckling and warping, which are discussed in the next three subsections. Longitudinal shortening is typically negligible except for long joints, such as bridge girder web-to-flange welds. In such cases fabricators usually make girder webs and flanges long and then trim them to length.

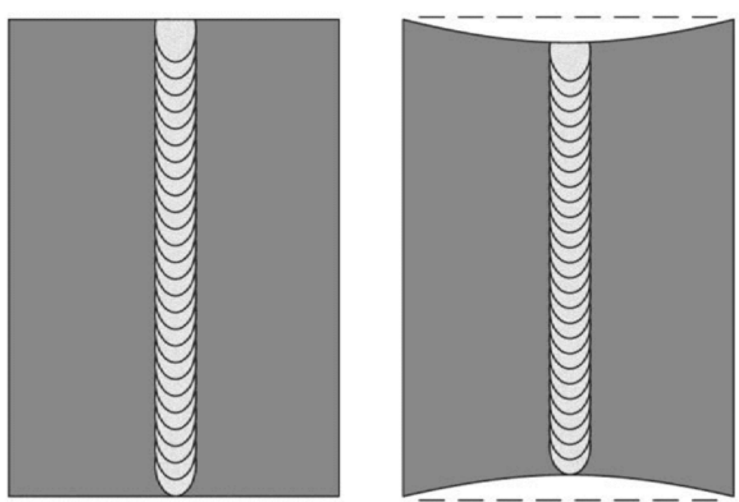

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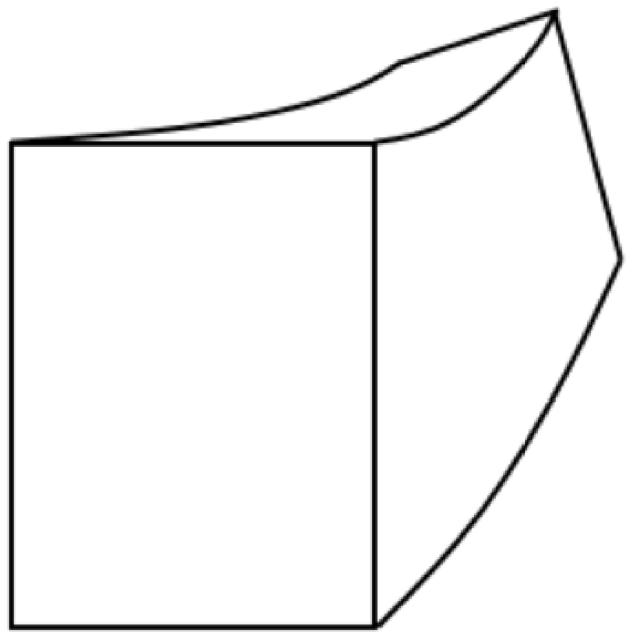
Figure 81. Illustration. Longitudinal shortening.

5.7.1.4. Twisting

Longitudinal shrinkage causes the weld region to shrink, while the outside steel remains unchanged in length. If the shrinkage stresses are large, and if the assembly has little resistance to twisting, the shrinkage can cause a longitudinal twisting of the assembly, as shown in figure 82. Such twisting occurs in box sections, as shown in figure 82 (note that the twisting shown in this figure is exaggerated), where twisting can occur in the earlier stages of fabrication, while the box is open and relatively flexible. Bridge girders may also twist, resulting in members that look deformed when the girder is hung from a crane. However, I-girders are torsionally flexible, and such twist is typically not a concern for field assembly or serviceability.



Source: FHWA

Figure 82. Illustration. Twisting due to longitudinal shortening.

5.7.1.5. Longitudinal Sweep or Camber

Longitudinal shrinkage of the weld or group of welds used to make long, built-up members may cause a curvature in the longitudinal direction. Depending on the relationship between the center of gravity (C.G.) of the welds and the neutral axis (N.A.) of the section, the deviation may form a negative or positive camber (e.g., deviation in the vertical plane), or a horizontal sweep to the left or right (e.g., deviation in the horizontal plane). Figure 83 illustrates welds that have resulted in a positive (upward) camber.

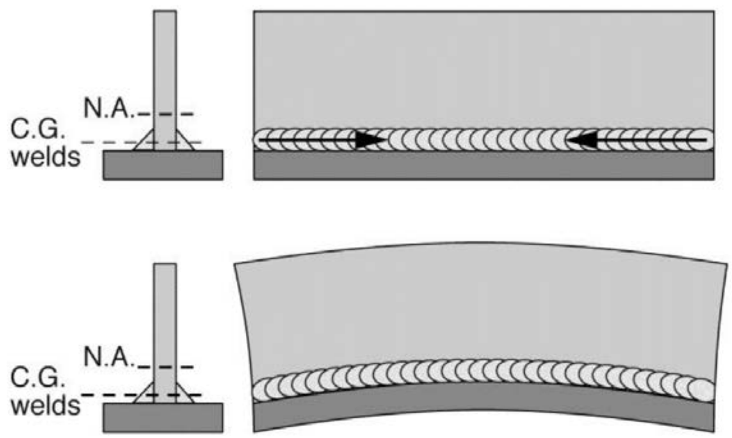


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Figure 83. Illustration. Longitudinal camber due to longitudinal shortening.

5.7.1.6. Girder Web and Flange Distortions

Longitudinal shrinkage may cause thin plates to distort, as illustrated in figure 84. This occurs because thin base metal has little resistance to compression. Distorted welded assemblies usually have elements of angular distortion as well. As would be expected, the tendency toward these forms of distortion depends on the critical buckling stress associated with the localized section. Such buckling resistance significantly increases with increases in thickness. For applications where control of distortion is critical, using slightly thicker members is helpful.

Distortions from longitudinal shrinkage are common in bridge plate girders in the form of web distortion between stiffeners. Web distortion is also referred to as “out-of-flatness” and also “web buckling”. Although the term “buckling” could be taken to imply that a critical stress has been exceeded, leading to failure, this is not the case with common warped conditions such as girders with web distortions. As reflected in the out-of-flatness web tolerances in the Bridge Welding Code (clause 3.5.1.6(1)), plate girders can safely withstand some amount of web distortion. Design limit states for buckling have been based on full-scale distorted specimens, and therefore the AASHTO *LRFD Bridge Design Specifications* (BDS) accounts for web distortion (although the AASHTO BDS does not allow shear buckling during construction or under fatigue loading) (AASHTO, 2017a).

The Code web distortion tolerances are based on appearance and not strength or stability. Even small levels of web distortion can be visually disconcerting, especially on fascia girders in sunlight when light-colored glossy paint is applied because slight waviness can cast shadows that accent minor variations in the flatness of panels (see figure 85 for an example of an unpainted girder).

Distortions can also occur where one edge is free (figure 86). Commonly known as warping, the condition is less common than web distortions and may be encountered on thinner, wider flanges.

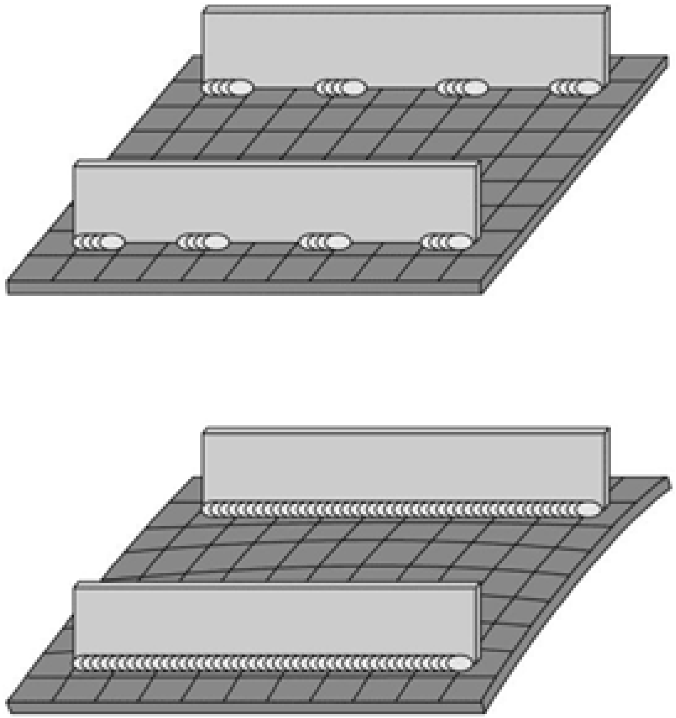

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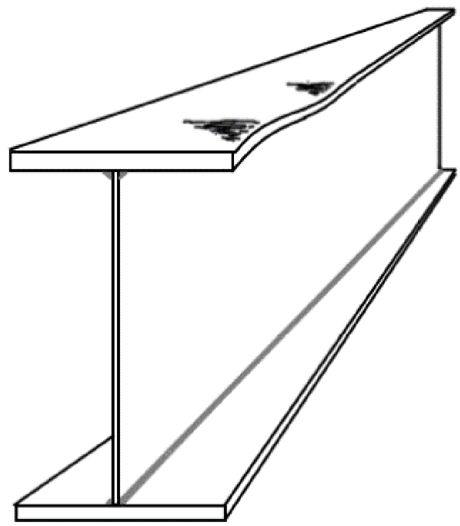
Figure 84. Illustration. Warping and buckling due to longitudinal shortening.

Source: FHWA

Figure 85. Photo. Warped (out-of-flat) girder webs.



Source: FHWA

Figure 86. Illustration. Warping of thin, wide flange.

5.7.1.7. Rotational Distortion

Rotational distortion is caused by transverse shrinkage and is typically associated with thinner members that are relatively narrow compared to their length. However, rotational distortion also occurs with electroslag welding (ESW, section 3.7) regardless of the thickness of sections being welded by ESW. This type of distortion is illustrated in figure 87. With rotational distortion, the joint can either close tight (or, in the case of a thin member, have the joined pieces overlap on top of each other) during welding, as shown in "B", or open as shown in "C". The speed of welding and the heat input determines whether the joint opens or closes; when high travel speeds are used, the joint tends to close, whereas for slower travel speeds, it tends to open. At issue is the rate of thermal conductivity as compared to the rate at which the shrinking weld metal advances with respect to the joint. Rotational distortion can be mitigated by clamping the members rigidly to increase the restraint against rotation. For heavier sections, such as those often welded with ESW, the weight of the plates being joined resists rotational distortion in addition to tie-downs.

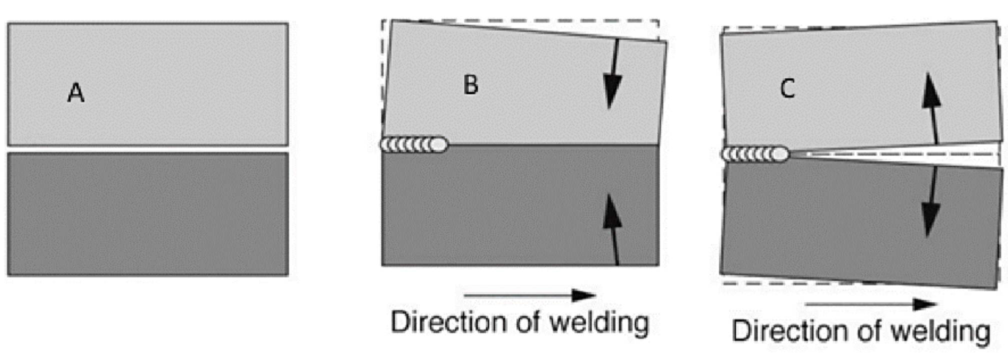


Figure 87. Illustration. Rotational distortion.

5.7.2. Distortion Control Measures

5.7.2.1. Adding Restraint

Restraint can be added to resist the shrinkage stresses. Means of restraint include tack welds, strongbacks, welding fixtures, clamps, or other methods. The Bridge Welding Code advises that members should be "welded with as little restraint as possible" (clause 3.4.5) because restraint can be associated with cracking. However, there are situations where use of restraint is necessary; controlling distortion is a primary reason for providing additional restraint. In these situations, anything that can keep the parts from moving when the hot metal begins to shrink is helpful. See figure 88 for an example. Here, clamping is used to keep the plates from rotating about the weld axis as the weld shrinks. There will be some final distortion in this plate due to welding; the purpose in this case is to keep the joint from becoming narrower, and therefore more difficult to weld, as welding proceeds pass by pass.

Source: FHWA

Figure 88. Photo. Clamping on a groove weld qualification plate.

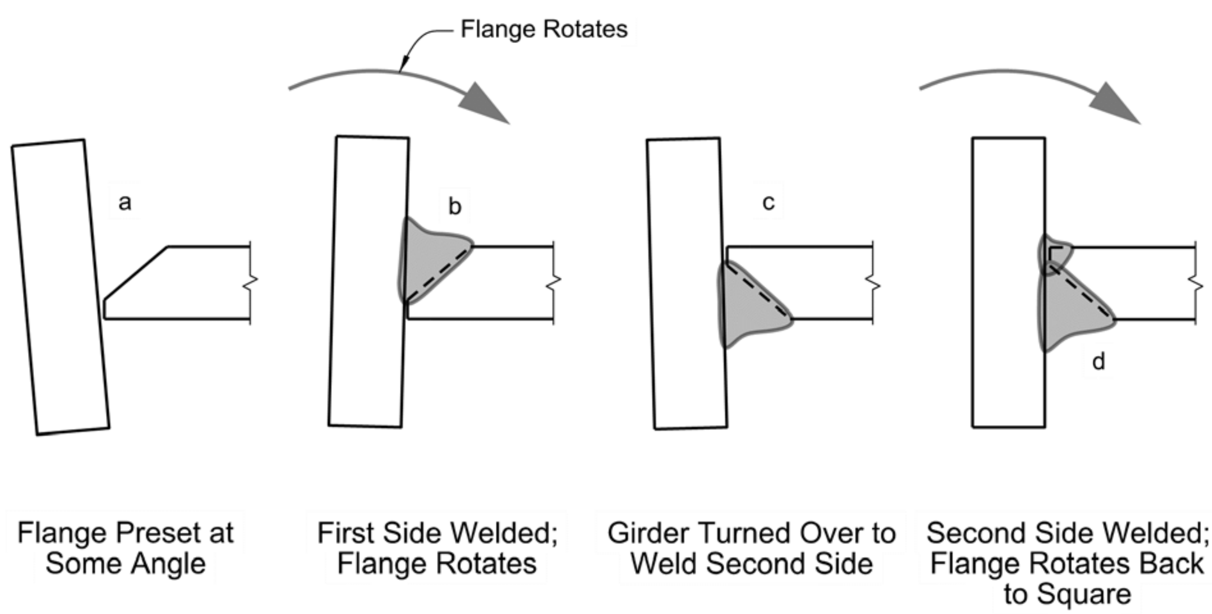
5.7.2.2. Weld Placement

For symmetrical shapes, welds may be balanced around the neutral axis of a section to help control distortion. However, in order to be effective, the whole cross-section must work as a unit when the welding is performed, and this may not be practical. For example, if all four web-to-flange welds of an I-girder could be made at the same time, then the welds would not introduce camber or sweep, but it is not possible to make all four welds at the same time.

5.7.2.3. Welding Sequence

Distortion can sometimes be minimized or eliminated through a properly planned and executed welding sequence. Consider the full penetration T-joint weld sequence in figure 89. At “a”, the flange is preset to some angle such that at “b”, the flange pulls in, but after turning the assembly over and welding “c”, the flange rotates to square, as shown in “d”, when the weld on the back side cools. Such sequencing minimizes the distortion observed in the final condition but may not eliminate it entirely; if not, the fabricator can use heat straightening to achieve the desired final condition.

As another example, when splicing I-sections (such as rolled beams) end to end, some initial passes should be welded in each of the three joints (both flange joints and the web joint) before any of the joints are welded to completion so that the two I-sections will shrink transversely together in an even manner.



Source: FHWA

Figure 89. Illustration. Welding sequence to balance distortion in a CJP weld.

5.7.2.4. Stress Relief

For parts that will be machined after welding, thermal stress relief may be used in order to achieve dimensional stability. Components of moving bridges, for example, may necessitate close-tolerance machining after welding. During machining, the residual stresses in an as-welded assembly may cause the part to “move” (i.e., distort) as steel is removed and residual stresses are redistributed. To overcome this problem, the welded assembly may be stress-relieved prior to machining. With proper stress relief, the residual stresses are reduced by approximately 90 percent and the amount of movement in the part during machining will be correspondingly reduced (Miller, 2018).

Given the success of stress relief when used to control dimensional stability during machining, stress relief is sometimes inappropriately considered as a remedy for the distortion caused by welding. Under most conditions, stress relief is not effective for correcting distortion, and it is possible for distortion to actually increase as a result of stress-relieving operations. This may be the result of the part changing shape as the residual stresses are relaxed. The more common situation is that the mass of the steel component causes deflection at elevated temperatures where the steel experiences a reduced yield strength and reduced modulus of elasticity. Thermal stress relief can eliminate elastic distortion, but this is generally a very small part of the overall distortion; stress relief will not eliminate plastic distortion. Hence, it is not prudent to require stress relieving to address the types of common distortion described in section 5.7.1.

5.8. WELD REPAIRS

5.8.1. General

Weld that contain defects can usually be repaired. Such repairs range from minor in-process remediation to major repairs that need approval from the engineer. Minor repairs include adding additional weld passes to correct undersized welds, replacing small portions of welds that have excessive porosity, and repairing unacceptable undercut. Examples of major repairs include replacing components (such as stiffeners and connection plates), extensive groove or fillet weld repair, and addressing delayed cracking and lamellar tearing. In various locations, the Bridge Welding Code defines repair situations that require approval.

When problems are encountered, it is important to determine the cause of the initial defect and to institute corrective actions. Failure to do so will often result in duplicating the conditions that caused the initial problems.

Fabricators choose the welding process used to make repairs to welds. Only welding processes listed in the Bridge Welding Code may be used, and an approved WPS must be followed. The process selection depends on the circumstances surrounding the work, just as is the case for other welding. Given the broad variety of weld repairs that occur, the approved WPS used for repair may not include an actual sketch of the joint associated with the repair. Rather, it may simply show a fillet weld or groove weld. For unusual repair geometries, more details of the repair technique may be required. In all cases, the WPS applicable for the repair must be followed.

Because repair welding may be done under conditions of greater restraint, preheat temperatures beyond the level associated with normal production welding may be needed for the repair, as well as postheat.

The cost to repair a defective weld is several times greater than the cost to deposit a quality weld in the first place. As a rule of thumb, a six-fold increase for repair welds as compared to initial welds is a good estimate (Miller, 2018). This is based upon the following assumptions: the evaluation of the extent of defective weld to be removed will take twice as long as the initial welding; the metal removal will take twice as long as was associated with the initial welding; and re-welding also takes twice as long due to the use of slower welding processes and procedures.

The Bridge Welding Code does not have limits on how many weld repairs are permitted. The data suggest that a properly made weld repair has the same quality whether it is the first repair attempt, the fifth, or more. Repair welds made on fatigue-sensitive ship structures exhibited the same fatigue life as the original welds if the quality was the same (Kelly, 1997).

5.8.2. Internal Defects Discovered by NDE

When internal defects are discovered by NDE, the defect must be removed and the removed metal restored. Such defects include incomplete fusion, slag inclusions, internal porosity, and cracks. Crack repair is discussed separately in section 5.8.4 of this manual. Not all discontinuities discovered by NDE are defects; only defects need to be repaired (section 5.4).

Locating and removing the defect in a complete joint penetration weld can be a challenge. The goal of the repair is to remove the entire defect while removing no more sound material than necessary. NDE does not precisely pinpoint the location of defects; accordingly, a series of shallow excavations of weld metal with careful inspection of the cavity is appropriate. When the defect is identified, it is essential that it be completely removed before repair welding begins.

The resulting cavity created by the removal of the defective weld must provide a geometry conducive to good fusion. There must be ample access to the root of the joint, with enough width to the joint to allow manipulation of the electrode. The ends of the cavity should gradually taper into the base or weld metal. The cavities that result from defect removal are typically U-shaped in cross-section; it is advisable to maintain a general profile similar to one of the prequalified U-groove details in D1.5 figures 2.4 and 2.5 of the Bridge Welding Code. The length of the tapered ends should be around 2.5 times the depth of the cavity, but this is not prescribed in the code. The cavity surfaces should be clean and free of notches and gouges.

5.8.3. Undercut Repair

Minor undercuts may be repaired by careful grinding to reduce any notch-like feature of the undercut as shown in “b” of figure 90. While not mentioned in the Bridge Welding Code, the concept is similar to the repair of small edge discontinuities as addressed in clause 5.14.8.4. The suitability of grinding to address undercut generally depends upon whether or not sufficient weld and a good profile will remain after grinding. In the case of figure 90, the depression created by grinding can be evaluated under the allowances of clause 3.6.2 in the Bridge Welding Code.

Undercut may also be repaired by welding as shown in “d” of figure 90. Such welding is performed in accordance with an approved WPS. When undercut is properly repaired by welding, the repaired region inevitably has a weld that is substantially larger than the required size; this may need to be addressed to satisfy appearance or function requirements.

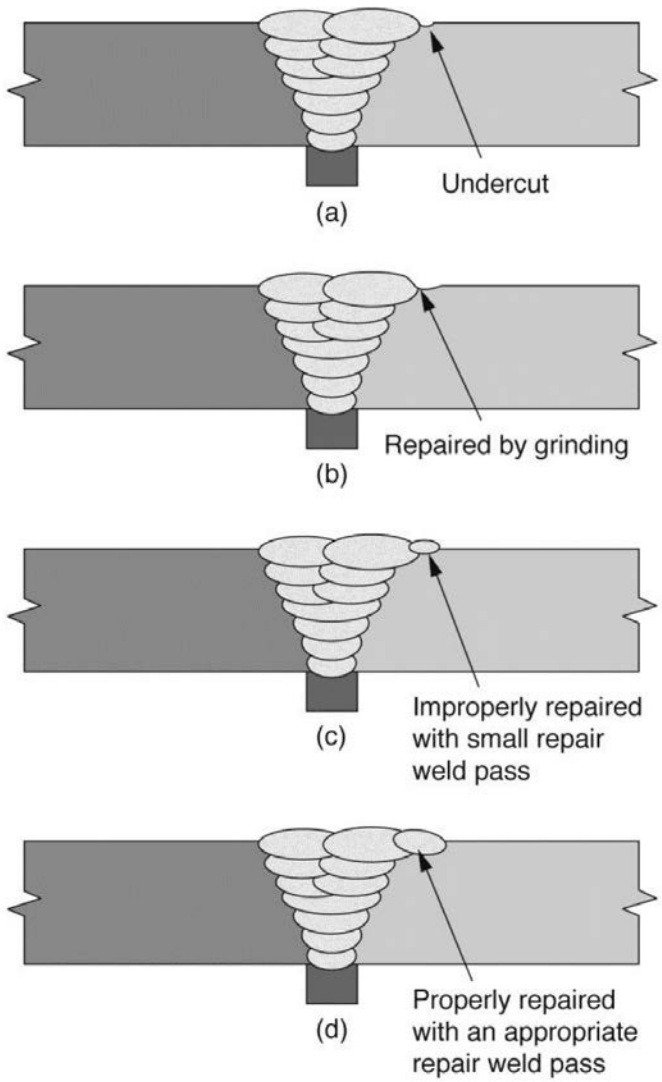


Figure 90. Illustration. Undercut and proper repair methods.

5.8.4. Cracks

Cracks in welds, regardless of size and orientation, are considered defects and are unacceptable according to the Bridge Welding Code (clause 6.26.1. 1). Cracks may be contained within the weld metal, the heat affected zone, the base metal, or in multiple regions. The practice of rendering all cracks unacceptable is a conservative position and generally reasonable, although it should be recognized that some cracks, such as those that are parallel to the direction of stress, may have no consequence on the performance of the structure. For example, the gap under web-to-flange fillet welds, between the edge of a web and the flange, is crack-like and yet is not considered a defect. Welds with cracks in their throat may be acceptable in service, providing the required weld throat is still achieved. Still, this condition is not permitted by the Bridge Welding Code and would require the engineer's careful evaluation of the in-situ condition.

When repairs of major or delayed cracks are required, the engineer is to be notified before repairs are made (clause 3.7.4). The interpretation of "major" is not specifically defined in the code, and this status is usually established on a case-by-case basis. If, for some reason, there is no ready agreement about what is considered "major", the default is to consider all cracks major.

The whole crack must be removed. To ensure complete removal, the extent of cracking is to be determined with magnetic particle testing (MT), dye penetrant inspection, acid etching or other methods; MT is probably the best option for most situations. Not only is the full length of the crack to be removed, but 2 inches of sound material beyond the end of the crack must also be removed (clause 3.7.2.4). Despite the confidence many welders will express, it is unlikely that subsequent welding will "burn out the crack" (i.e., welding over a crack will probably not remelt the material surrounding the crack such that it no longer exists); removal of the whole crack is a more certain approach.

A crack that has totally severed a member will relieve all the residual or applied stress, but a member with a partial crack may be under considerable stress. While the Bridge Welding Code appropriately permits a variety of mechanical and thermal methods for crack removal, caution is recommended regarding thermal methods, particularly when the region to be repaired is under high residual stress.

A helpful technique to mitigate the enlarging of a crack during the repair process, regardless of the crack type and orientation, is to initiate arc gouging from sound steel or weld metal beyond the end of the crack, and gouge toward the crack. When the opposite approach is used, the metal that is heated and expanded may cause the crack to propagate. When repair welding is begun, it is preferred to first repair portions away from the point where the crack terminated.

CHAPTER 6 - INSPECTION AND QUALITY CONTROL

Proper weld quality is essential to the successful performance and long life of steel bridges. Achieving quality is the responsibility of the welder, and quality assurance and quality control help make sure that responsibility is fulfilled. The engineer and detailer also share in this responsibility by designing and detailing welds that are achievable and facilitate the best constructability (see chapter 8). Quality assurance activities include American Institute of Steel Construction (AISC) plant certification, use of appropriately qualified and approved welding procedures (chapter 4), and welder qualification (section 4.6). Quality control includes visual inspection and nondestructive evaluation (NDE).

Within the bridge industry, "quality assurance" and "quality control" are sometimes used to mean that the owner verification activities are quality assurance, with the fabricator's activities being quality control. This terminology is changing, but "quality assurance" is currently used with two different meanings among different users and different documents—the older meaning of the owner's activities, and the more modern usage to refer to the higher-level functions of the fabricator's quality management system.

6.1. FABRICATOR CERTIFICATION

AISC operates a quality certification program for steel structure fabricators and erectors, including specific categories for bridge fabricator certification. The program is defined in AISC 207, *Certification Standard for Steel Fabricators and Erection, and Manufacturing of Metal Components* (AISC, 2017).

The certification program includes categories and associated requirements for fabrication of simple, intermediate, and advanced bridges. There is also a special additional endorsement for fracture critical fabrication, known as the Fracture Critical Endorsement (FC) and a certification for coatings application under the *Certification Standard for Shop Application of Complex Protective Coating Systems* (AISC, 2010), which is a joint standard between AISC (AISC 420) and The Society for Protective Coatings (SSPC-QP 3). The coatings certification under AISC 420 is still referred to within the AISC program as the "Sophisticated Paint Endorsement" (SPE), after the original name for the AISC standard.

Achievement of certification by a fabricator is based on a successful audit by AISC with annual audits for certification renewal. Welding falls within the scope of the three bridge fabricator certification categories and, for FC welding, the FC endorsement.

The Bridge Welding Code requires that fabricators be certified (clause 1.4); bridge owners in the United States also require the AISC bridge fabricator certification in their standard specifications. The Code allows a fabricator to be certified to an acceptable equivalent alternate program if acceptable to the engineer. However, the authors are not aware of any alternate programs in use by fabricators or permitted by owners in lieu of AISC certification for bridge fabricators as of this writing.

Certification ensures that the fabricator meets certain minimum requirements associated with achieving quality, including the following:

- A quality policy
- A quality manager
- A quality management system
- A quality manual
- Documented procedures for maintaining quality records
- Documented procedures for welding, including use of WPSs and PQRs, preheat, welding consumable storage, welder and welding operator qualifications, and welder traceability
- Inspector qualification and inspection practices
- Equipment calibration
- Nonconformance control and corrective actions
- Fabrication personnel training

Details about each of these requirements are found in AISC 207. The certification does not ensure weld quality itself. Rather, certification ensures the fabricator has the practices in place that will help ensure quality is achieved.

6.2. VISUAL WELD INSPECTION

Visual inspection, also called visual testing or visual examination "…is a nondestructive method whereby a weldment, the related base metal, and particular phases of welding may be evaluated in accordance with applicable requirements. Per AWS B1.11, *Guide for the Visual Examination of Weld,* all visual examination methods require the use of eyesight to evaluate the conditions which are present; hence, the term visual examination." (AWS, 2014). As implied by the term, visual inspection can only detect what can be visually observed. For completed welds, this limits the method to detection of discontinuities on the surface. This has caused some to discount the value of visual inspection. However, the power of visual inspection lies in the ability to examine "particular phases of welding." Alternatively stated, visual inspection allows for examination of the whole process of welding, from start to finish.

All welds are required to be visually inspected (clause 6.26.1). This includes welds that are subject to other NDE as well, and visual inspection should be performed before those examinations. Visual inspection is a very powerful inspection method. It can be used to improve the quality of a given weld. For example, visual inspection of the weld joint preparation and the adequacy of the root opening dimension can ensure that conditions conducive to obtaining good fusion are present before welding begins, thereby minimizing the probability of incomplete fusion in the completed weld.

To be effective, visual inspection must take place before, during and after welding. The "before" and "during" aspects are often overlooked. While specific visual inspection responsibilities are assigned to inspectors, everyone associated with welding on a project can, and should, participate in visual inspection, including welders and foremen as well as inspectors. With visual inspection, minor irregularities can be detected and corrected during the fabrication process, helping avoid the need for more expensive and complicated repairs after the weld is complete.

“Before welding” tasks include quality management responsibilities, such as verifying that welders are qualified, that WPSs are available and suitable for the application, that the joint is properly cleaned and fit, and that the welding equipment is properly maintained and calibrated as needed. “During welding” tasks include the activities needed to ensure quality, such as adhering to the WPS (including preheat and interpass requirements, electrical and other parameters, and cleaning between weld passes as discussed in chapter 4), ensuring that the profile of previously deposited passes is appropriate for deposition of additional passes, and confirming the quality of each pass (including proper fusion and the absence of cracking, porosity, cratering and undercutting). “After welding” tasks include verification of the final weld for conformance, such as checking the weld size and examining the weld for cracks, porosity and undercut. AWS B1.11 provides a useful expansion on the topic of visual inspection (AWS, 2014).

6.3. NONDESTRUCTIVE EVALUATION METHODS

Nondestructive evaluation (NDE) methods are effective for discovering welding discontinuities (section 5.4). A high-level summary of NDE methods used in bridge fabrication is as follows:

- Radiographic Testing (RT)
    - Required for flange and transverse web CJP weld butt splices, including FC
- Ultrasonic Testing (UT)
    - Required for CJP weld T- and corner joints
    - Required for FC flange and web butt splices
    - Used by some owners as an alternative to RT for flange and web butt splices
    - Used by fabricators to discover the depth of defects detected with RT
- Phased Array Ultrasonic Testing (PAUT)
    - Allowed by the Code as an alternative to UT
    - Used by some owners as an alternative to RT for flange and web butt splices
    - Used on some projects for special joints, as the owner may require
- Magnetic Particle Testing (MT)
    - Required for fillet and PJP welds
    - Sometimes used by fabricators to verify the soundness of surface conditions in process, such as for verification of backgouging to sound metal, complete removal of defects, and soundness after arc strike removal
- Dye Penetrant Testing (PT)
    - Used by fabricators to check surface conditions, particularly surface prepared for subsequent welding

These methods, requirements for them, and their application are explained in more detail elsewhere in this section. Visual inspection is also considered to be a form of NDE. However, in the Code, certification requirements are different for visual inspection (section 6.2) and other NDE methods (section 6.3). The term “NDT” (see below) is often used in the Code in a sense that excludes visual inspection. The Code dictates which methods are used for which joint types and the extent of inspection (samplingversus 100 percent).

The use of the term “nondestructive evaluation” is replacing the term “nondestructive testing” (NDT) in the fabrication community. However, this change has not yet been made in the Code. Often the two terms are used interchangeably, but there is a distinction: like NDT, NDE includes testing but it also includes the evaluation of the tested materials for conformance. Considering that nondestructive testing is always accompanied by evaluation, even if under certain circumstances the testing and evaluation may not be performed by the same individual, the term “NDE” is used in this manual.

Broadly, NDE methods may be categorized as either volumetric methods, which are used for examination inside welds, or surface methods, which are used to evaluate weld and base metal surfaces for discontinuities.

6.4. RADIOGRAPHIC TESTING (RT)

Radiographic testing (RT) has been used for evaluating bridge CJP welds since the 1930s and has been required since 1974. In spite of its disadvantages compared to UT, RT is an excellent method of verifying quality in butt joints. However, as explained in section 6.9, UT offers distinct advantages that should be considered when making choices between the two methods.

6.4.1. Mechanism

RT is familiar to most people because they have encountered RT in the medical field (e.g., x-rays). The mechanism of RT is as follows:

- A radiation-sensitive medium (film, panel, or screen) is placed on one side of the welded joint (see figure 91).
- A source of radiation is positioned on the opposite side of the joint at a set distance. Historically the medium has been film, but more recently electronic imaging has also come into use for steel fabrication (see section 6.4.4).
- The medium is exposed to the radiation. Because the joint is between the source and the medium, radiation can only reach the medium by passing through the joint; thus the joint blocks a certain amount of the radiation. The medium is exposed for a closely calculated amount of time such that enough radiation reaches the medium to produce an image but not too much radiation reaches the medium and overexposes it.
- If the medium is film, it is developed to create the image that can be read. With electronic media, the image is immediately available for reading.

The amount of radiation that reaches the medium depends upon the density of the joint and any other material between the source and the medium. The weld metal and base metal have slightly different densities, so on the image, the weld is distinct from the base metal. If there are discontinuities in the weld, they are a local lack of density (a lack of material), and so they appear as darker areas on the image. Further, other items can be purposefully included in the image as well, such as lead letters and numbers for exposure documentation and artifacts for image quality verification (see section 6.4.3).

Source: FHWA

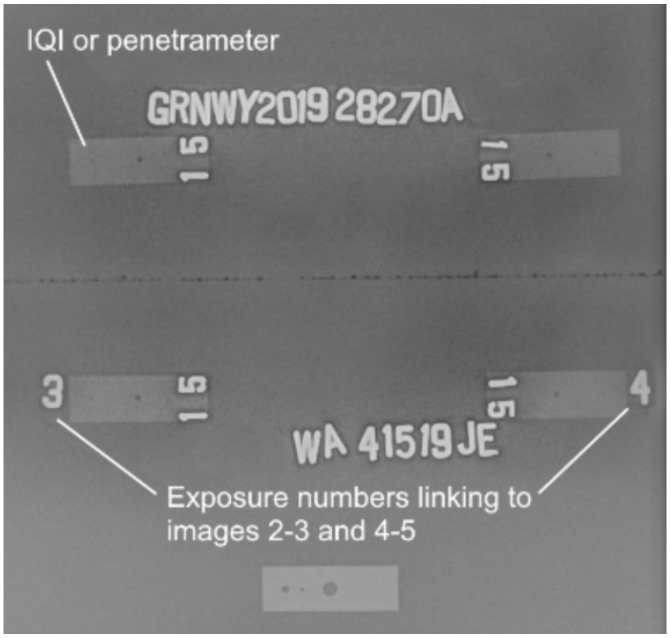
Figure 91. Photo. Radiography in a bridge shop.

6.4.2. Method

The basic method of producing RT images in the shop is as follows (see figure 92 for an example image):

- The joint is prepared for the exposure:
    - When required, the weld reinforcing and backing (if present) are removed from the weld, typically with grinding.
    - Image quality indicators (IQIs; see section 6.4.3) are placed on top of the joint.
    - If more than one exposure will be needed, the numbers "1", "2, "3", etc., or letters "A", "B", "C", etc., are placed on the joint to provide indexing and demonstrate that the entire length of interest was examined. See numbers "3" and "4" in figure 92. This is the third shot in this series. Typically film is 17 inches long and electronic media are similar in length; inspection of longer welds will require multiple exposures and proper indexing (clause 6.10.8).
- The film is prepared and then placed beneath the joints:

    - Lead letters used for identification are placed on the base metal adjacent to the weld being examined. Code requirements for identification marks are found in clause 6.10.13.
    - A special letter "B" used for checking for backscatter (see section 6.4.3) is attached to the back of the film.
- The source of radiation is positioned above the weld. The Code requires either x-ray or gamma ray radiation (clause 6.10.1) at the fabricator's option. X-rays are produced by x-ray tubes, and gamma rays are produced by a radioactive source, usually cobalt-60 or iridium-192. Tubes and sources are equally capable of producing radiographs that conform with the Code. Sources cannot be turned off and therefore require more stringent safety management. For this reason, tubes are often preferred. However, sources are more powerful, and therefore even in situations where tubes are being used for most exposures, a source will be used for thick sections, typically splices over 3 inches thick.
- Radiation safety protocols (see section 6.4.3.3) are implemented in the vicinity of the exposure.
- The exposure is made by turning on the tube or exposing the isotope. Emitted radiation passes through the weld to the film or to the digital medium. The duration of the exposure is calculated so that the medium is exposed to the right amount of radiation to produce a satisfactory image (see section 6.4.3).
- If film is used, once the exposure is complete, the film is developed. This process is similar to processing camera film into negatives to make prints, but instead of making prints, the negative is viewed. In figure 92, fusion defects are seen as a dark line across the image.
- The radiograph is evaluated by the technician for conformance with quality requirements of clauses 6.10.10 (blemishes), 6.10.11 (density), and practice requirements (edge blocks, IQIs, backscatter "B", etc.).
- The image is read and interpreted for conformance by the technician:
    - As processed, film is very dark, and a film viewer that uses a bright light behind the film is needed to read it as seen in figure 93.
    - Electronic files are read on computer monitor.
- Depending on the owner's requirements, the owner may also review and approve the film.



Source: FHWA

Figure 92. Image. A weld radiograph.

Source: FHWA

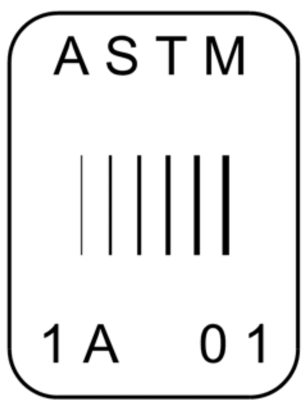
Figure 93. Photo. Viewing radiographs on a light table.

6.4.3. Application

6.4.3.1. RT Image Quality Control

- **IQIs -** With RT, exposing the medium to the correct amount of radiation and thereby achieving the proper sensitivity is essential to correct examination of the weld. Both underexposure and overexposure can preclude discontinuities from appearing on the image. To ensure the sensitivity is correct, the Code requires the use of image quality indicators (IQIs), which use a feature of known size to determine that the appropriate resolution can be displayed in the radiographic image. The IQI is placed on top of the joint before the exposure is made, as shown in D1.5 figure 6.1D. When the exposure is made, the IQI blocks a set amount of radiation so that the IQI appears on the final image. If the IQI does not appear correctly, then the image is not acceptable.

- The Code permits two types of IQIs: hole type and wire type (for wire type, see figure 94; for hole type, see figure 92). IQIs are also known as penetrameters, and the hole type IQI is often referred to as a "penny". The hole-type IQI is a small metal plate with three holes in it: the 1T hole, the 2T hole, and the 4T hole (see figure 92). "T" is the thickness of the penetrameter; the 1T hole diameter is the same diameter as the penetrameter thickness; the 2T hole diameter is twice the thickness, and the 4T hole diameter is four times the penetrameter thickness. The ability to see a given hole relates to the ability to see discontinuities that would require the same energy to reveal. Therefore, the holes are said to define the sensitivity of the radiograph.
- The penetrameters are numbered based on thickness, with the number representing the thickness of the penetrameter in thousandths of an inch (e.g., a "25" penetrameter is 0.025 inch thick); the Code (D1.5 table 6.1) specifies the number of the penetrameter to use and which hole is "essential" based on the thickness of the joint being examined with higher number (thicker) penetrameters and smaller holes required as joints get thicker. The essential hole must be visible on the image for the image to be acceptable. Similarly, if wire type IQIs are used, the correct diameter wire as specified in the Code must appear on the image.



Source: FHWA

Figure 94. Illustration. Wire type IQI.

- **Backscatter -** Clause 6.10.8.2 requires that a lead letter "B" be attached to the back side of film to protect the integrity of the exposure from backscatter. Backscatter refers to radiation that reflects back onto film from behind the film during the exposure. In bridge fabrication, flange and web butt splices are often tested near the floor, so there is the potential for radiation to reflect off the floor and further expose the film. If it occurs, this additional exposure can cause a lack of contrast and possibly result in discontinuities being overlooked. If the "B" shows up in the image, this is a sign that the radiograph has been exposed to backscatter and, per the Code, the radiograph is unacceptable.

- **Blemishes** - Radiographs must be free from any blemish that might mask defects (clause 6.10.10). Such blemishes include scratches and processing blemishes, such as streaks, watermarks, and chemical stains. Electronic images are not handled and processed like film is, so electronic images are not likely to have blemishes. However, over time, digital media can degrade such that some pixels quit operating (this does not affect the images previously generated, which are stored elsewhere). If there are too many dead pixels, the lack of image information associated with the dead pixels produces an effect that is like film blemishing. Only a minimal amount of pixels may be non-operational.
- **Density** - Film must be of a certain darkness range to provide contrast for viewing, but the film must not be so dark or light that it cannot be read. Radiograph darkness is known as "film density". Density requirements are found in the Code (clause 6.10).
- **Edge Blocks** - For joints greater than ½ inch thick, the Code requires the use of edge blocks to facilitate proper evaluation of plate edges (clause 6.10.14). Edge blocks are needed because without the edge blocks, the lack of material along the edges creates a shadow at the plate edge that can mask discontinuities. Such edge blocks are visible in figure 91. To ensure edges are completely covered, edge blocks must be at least as thick as the material being joined. At flange thickness transitions, edge blocks need to be as thick as the thickest part of the weld.

6.4.3.2. Acceptance Criteria

The RT acceptance criteria in the Code are based on the size of discontinuities as directly measured on the radiograph. With film, the radiograph is exactly to scale, so discontinuities can be measured on the film with a ruler. With electronic imaging, measurements are made on the monitor using measurement tools in the reader software.

The acceptance criteria are found in clause 6.26.2.1 and D1.5 figure 6.8 (tension welds) and clause 6.26.2.2 and D1.5 figure 6.9 (compression welds) of the Code. These figures graphically define the largest discontinuities that are allowed based on the thickness of the joint, with larger discontinuities allowed as the thickness of the joint increases. This is because as the joint thickness increases, discontinuities of discrete size are a smaller proportion of the weld cross section. The allowable discontinuities also vary with proximity to the edge of the joint: the closer to an edge, the smaller the allowable discontinuity. If multiple discontinuities are present and close to each other the discontinuities and the space between them are counted together. There is further discussion of acceptance criteria in section 6.9.

6.4.3.3. RT and Safety

Avoidance of exposure of NDE technicians and shop personnel to radiation in the radiograph process is of paramount importance. This topic is discussed in this manual as a point of awareness because ensuring safety in the RT process has a significant impact on workflow and is fundamental to how shops conduct RT. Whatever practices and control are put in place, fabricators have the responsibility to comply with national, state, and local regulations.

6.4.4. Digital Radiography and Computed Radiography

Compared to other industries, transition to electronic media for RT for steel fabrication has been very slow and long-delayed. However, fabricators have begun this transition, and many owners now allow the use of electronic images. Provisions that address electronic imaging are planned for the next edition of the Code.

Electronic imaging is quite different from that of film but yields the same result: an accurate image that reveals weld discontinuities which can be read and interpreted. As with film, controls are used to help ensure the integrity of the exposure; the same IQI protocols (section 6.4.3) that work for film work with electronic imaging.

There are two types of electronic imaging in use: digital radiography and computed radiography. Digital radiography is based on the use of a digital detector array, which ASTM E2736 (ASTM, 2017) defines as:

> *"[A]n electronic device that converts ionizing or penetrating radiation into a discrete array of analog signals which are subsequently digitized and transferred to a computer for display as a digital image corresponding to the radiation energy pattern imparted upon the input region of the device. The conversion of the ionizing or penetrating radiation into an electronic signal may transpire by first converting the ionizing or penetrating radiation into visible light through the use of a scintillating material."*

Put another way, the device captures the radiation that passes through the steel, converts the signal to visible light, then converts the light into electronic signals, and finally converts these into digital data that can be displayed on a computer screen and stored as a data file. These multiple steps are all handled within the device.

Source: FHWA

Figure 95. Photos. Viewing radiographs digitally.

To control noise, the device captures multiple exposures of the same location and averages them. Making multiple exposures is not a throughput issue; digital panels are highly sensitive and capture exposures so quickly that even using this protocol, exposure times with digital RT are significantly shorter than with film. The radiograph shown in figure 92 is a digital radiograph.

Computed radiography also produces an electronic image but in a different way. With computed radiography, luminescent screens are used instead of a digital panel or film. A cassette containing the screen is positioned beneath the weld and exposed similarly to traditional film. After exposure, the cassette is put into a reader that converts the data to a digital image. By contrast, processing steps of digital radiography take place within the panel when the exposure is made, and the image is immediately available for viewing on a computer.

AWS has adopted verbiage different from ASTM and ASNT to refer to the electronic imaging processes. In this manual the term "digital radiography" is used in the same way it is used in ASTM E2736. However, the 2015 edition of D1.1 uses "digital radiography" as a general term for both digital radiography (as described above), which D1.1 terms "direct radiography", and computed radiography. The same definitions will also be used in the 2020 edition of the Bridge Welding Code, for consistency between the two codes. This terminology may be revisited in future editions of both codes.

In addition to the advantages already described, the use of electronic images offers additional advantages over film:

- The digital files of electronic film are much easier to store than film.
- Electronic images are easy to share; if an engineer at a remote location needs to view the RT results, the digital file can be emailed, and then engineer can view it instantly.
- Electronic media (computed radiography screens and digital panels) are reusable; film can only be used once, making it more costly and less environmentally friendly, although the up-front cost of equipment for electronic imaging is also costly.
- The hazardous chemicals required for film development are not needed.

6.5. ULTRASONIC TESTING (UT)

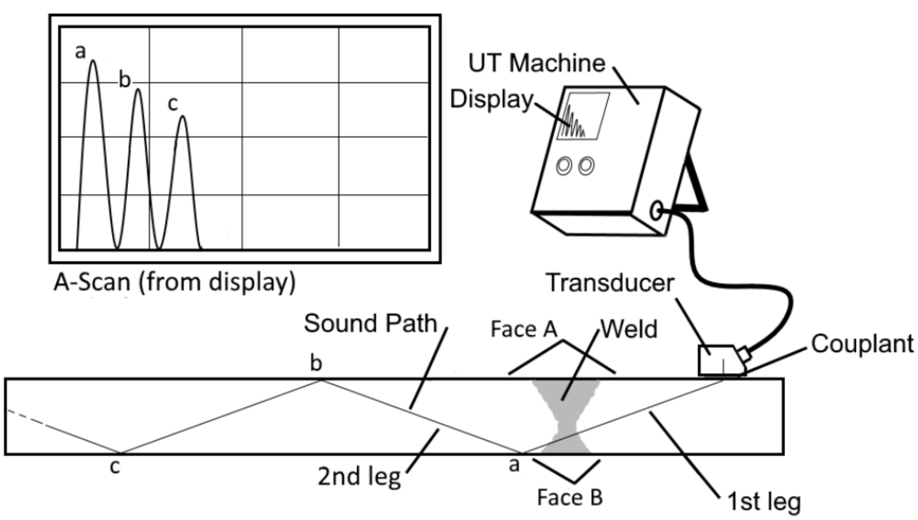
Ultrasonic testing (UT) was introduced in the 1969 editions of AWS D1.0 and AWS D2.0. The requirements have evolved somewhat over time, but generally the current UT practice in D1.1 and D1.5 is the same as the 1969 practice. A good general article that provides excellent insight into the development of the method adopted by AWS is "Ultrasonic Testing Requirements of the AWS 1969 Building Code and Bridge Specifications" (Schenefelt, 1971).

6.5.1. Mechanism

The basic mechanism of UT is as follows:

- Couplant is spread near the joint to be scanned. The couplant improves sound transmission between the transducer and the material. Typically glycerin is used because it is well suited to rough surfaces and does not evaporate quickly, providing sufficient time to complete UT scanning.

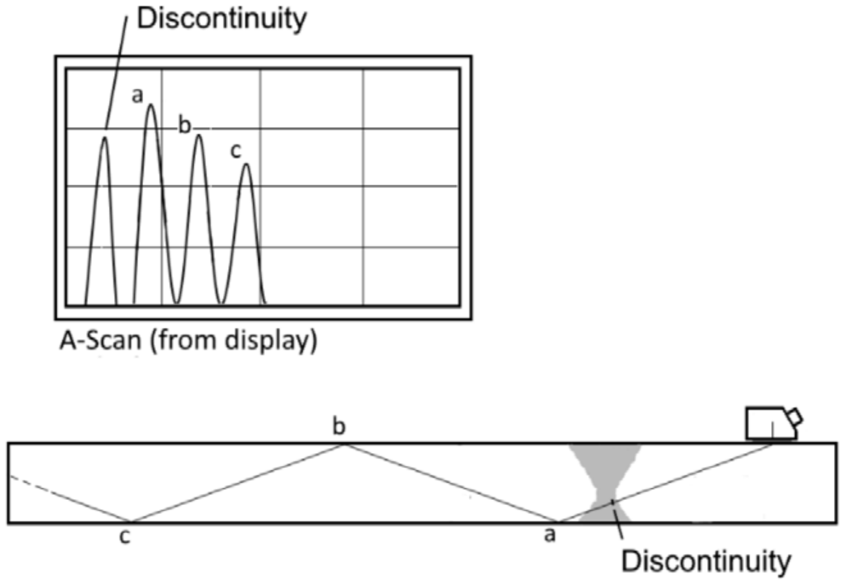
- Ultrasonic vibrations, or sound, are generated by a UT machine and transducer, which are transmitted through the weld and base metal.
- The sound reflects off of surfaces and then returns to the transducer for interpretation; this is called the “pulse-echo” technique.
- The sound travels at a constant rate in steel, so the distance from the transducer to the reflected surface is calculated using time. As shown in figure 96, each plate surface reflects sound back to the transducer. Each reflection, *a*, *b*, *c*, etc., returns sound to the transducer at a later time. Also, due to losses (attenuation), each reflector returns a weaker signal, which is why the signals are shown progressively shorter.
- The returning sound is read on the UT machine display; it is presented as a plot of amplitude versus time known as the A-scan format.
- If a discontinuity is encountered in the weld, an anomalous signal is returned to the transducer (figure 97). The amount of sound reflected by the discontinuity is measured and documented.
- The location and length of the discontinuity are determined and documented (section 6.5.2.3).
- The discontinuity is rated for acceptance or rejection in accordance with Code requirements based on sound amplitude and length, as well as inspection angle and whether or not the weld will be subject to tensile or compressive stresses.



Source: FHWA

Figure 96. Illustration. UT general schematic for butt joints.

- Discontinuities include slag, incomplete fusion, porosity, or cracks (see section 5.4). However, the returning sound does not definitively characterize the type of discontinuity and its geometry. Code allowances are based on the amount of sound reflected from the discontinuity.



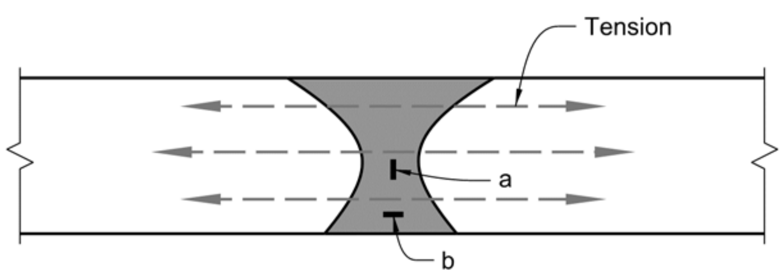
Source: FHWA

Figure 97. Illustration. Discontinuities return anomalous sound reflections in UT.

6.5.2. Method

6.5.2.1. Transducer Angles and Discontinuity Orientation Correction

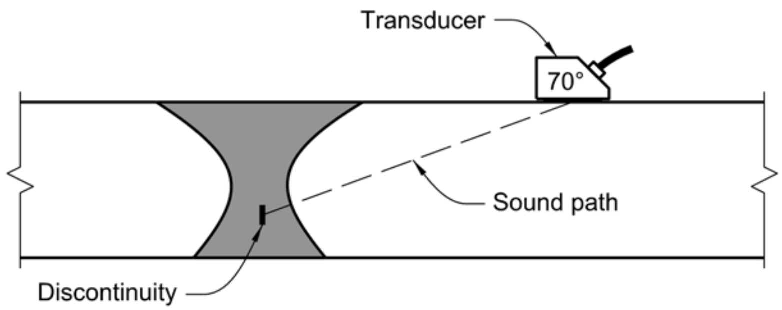
The Code requirements for UT include transducers of 45-, 60-, and 70-degree angles. In D1.5 figure 6.2, the Code specifies the transducer angle to be used based on the type of joint and material thickness. The required transducer reflects a preference for striking the worst-case discontinuity at an angle that is as close to perpendicular as practically possible. For example, as shown in the butt joint in figure 98, the vertical discontinuity, “a”, is the worst-case discontinuity because it would result in the greatest loss in cross-section. Conversely, a horizontal discontinuity, “b”, would be of least concern.



Source: FHWA

Figure 98. Illustration. Idealized vertical, "a", and horizontal, "b", discontinuities.

Given that a vertical discontinuity is the worst case, the ideal examination would be with a straight-beam transducer at the end of the plate, but this is a practical impossibility given the length of plates being spliced. The next best thing is a 70-degree transducer, as shown in figure 99.



Source: FHWA

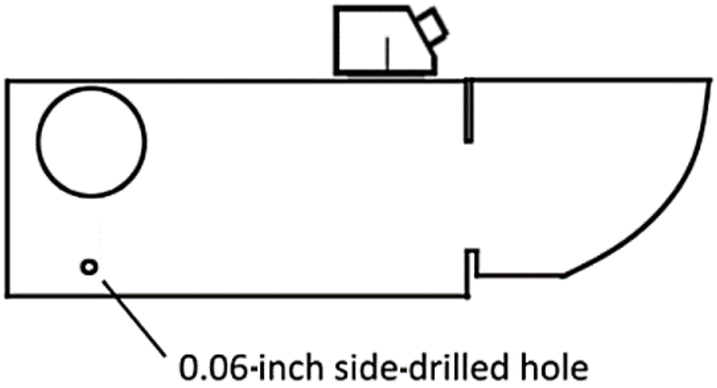
Figure 99. Illustration. Inspection of vertical discontinuity with a 70-degee transducer.

The 70-degree transducer is the closest of the three transducer choices to being normal to the discontinuity, but its use becomes ineffective on thicker joints because as joints get thicker, the sound path gets longer and eventually losses become too great. So, for thicker joints, the Code requires use of the 70-degree transducer for the top part of the joint and the 60-degree transducer for the bottom part. As joints become thicker still, use of the 60-degree transducer also becomes ineffective due to losses, and so the Code requires the use of the 45-degree transducer for the bottom part of very thick joints.

Each step from normal to the discontinuity, to 70 degrees, and then 60 degrees, and finally 45 degrees represents an examination that is progressively more oblique to the worst-case vertical discontinuity, and as such each step is more of a compromise between sound path length and optimal angle to the worst-case discontinuity. Therefore, the Code incorporates amplitude correction factors into the acceptance criteria, with more correction added as the angle becomes more oblique to a theoretical vertical discontinuity. These correction factors are 6 dB for the 70-degree transducer, 9 dB for the 60-degree transducer, and 11 dB for the 45-degree transducer.

6.5.2.2. Calibration

The amount of sound lost as it travels depends upon the material being inspected. Therefore, before conducting inspection, it is necessary to calibrate the testing equipment to the base material. The Code requires this calibration just prior to testing at each weld (clause 6.18.2). Calibration is conducted on an IIW block (clause 6.18.5.1). An outline of the block is shown in figure 100); complete details are found in D1.5 figure 6.5A of the Code. “IIW” stands for “International Institute of Welding”. The calibration block in the Code is a standard block adopted from IIW. IIW blocks are made of steel with properties that are similar to those of common bridge steels. Acceptance and rejection of discontinuities are based on the reflected signal from the discontinuity compared to this reading.



Source: FHWA

Figure 100. Illustration. Outline of IIW calibration block

6.5.2.3. Examination

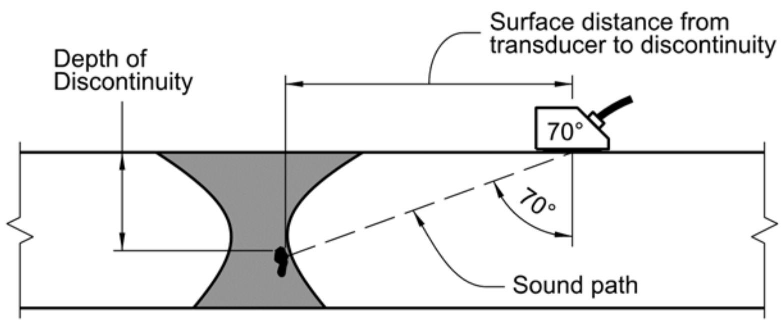
The basic steps of evaluation are as follows:

- The weld examination area is cleaned (clause 6.19.3).
- An "x" line is drawn at a set distance parallel to the weld axis (clause 6.19.1).
- Couplant is applied to the scanning surface of the plate (clause 6.19.4).
- The entire surface above any part of the base metal that will be scanned is given a straight beam evaluation to check for laminations within the base metal (see section 5.4.1.4). If laminations are discovered, adjustments to the scanning procedure may be needed (clause 6.19.5).
- The weld is scanned per the following requirements:
  - Scanning angles and procedures per D1.5 table 6.2
  - Scanning patterns of per D1.5 figure 6.7
  - Additional sound (dB) based on the sound path length
- If discontinuities are discovered, their location and length are established.
  - Given that the sound path, speed of sound in steel, and transducer angle are known, the depth of the discontinuity and the surface distance from the transducer to the discontinuity are determined geometrically (see figure 101):

    Surface distance = sin (transducer angle) x sound path

    Depth of discontinuity = cos (transducer angle) x sound path
  - The length of the discontinuity it determined by moving the transducer parallel to the weld until a significant drop in reflection amplitude occurs. (The Code defines this drop as 6 dB.)

- Discontinuity length, location and rating are documented in the UT report. Further, if discontinuities are defects, they are also typically marked on the steel to facilitate the repair.



Source: FHWA

Figure 101. Illustration. The depth and location of discontinuities determined geometrically.

Scanning is such that sound is passed "through the entire volume of the weld and the HAZ in two crossing directions, wherever practical" (clause 6.19.6.2). Longitudinally, as shown in figure 102, the transducer is passed along the length of the welding using scanning movements A, B, and C. The scanning distance transverse to the weld, *b*, used is such that the transducer will provide the intended coverage of the weld. The progression distance along the weld, *c*, is such that the entire length of the weld is examined. To scan for discontinuities transverse to the weld, scanning pattern D is used when the weld is ground flush; when the weld is not ground flush, pattern E is used.

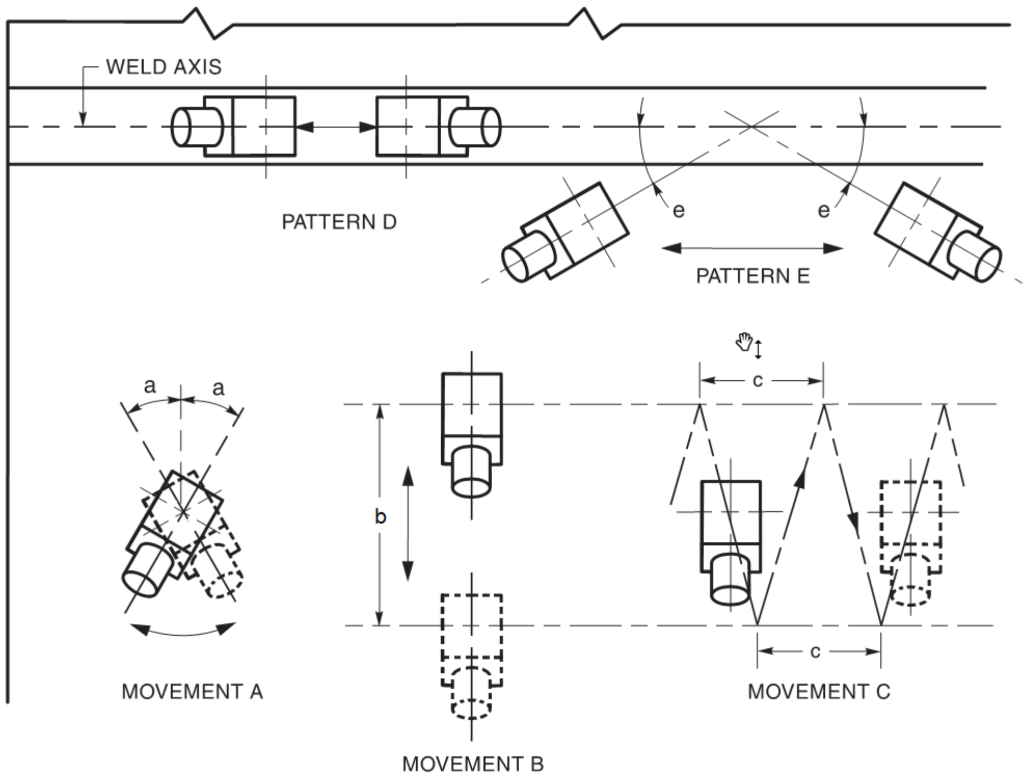


Figure 102. Illustration. UT scanning patterns from D1.5 figure 6.7 (AASHTO/AWS, 2015).

6.5.2.4. Discontinuity Classification

If a discontinuity is discovered during testing, it is evaluated for acceptance in accordance with the criteria of either D1.5 table 6.3 or 6.4. Acceptance is based on the severity of the discontinuity and possibly (depending upon the severity) its length.

When the discontinuity is discovered, scanning is progressed until the maximum attainable indication, or “peak” indication is found. This includes movement A of figure 102; as the transducer is translated with movement B, it is also rotated with movement A. This helps find the orientation at which the sound hits the discontinuity most squarely; this motion is informally known as “rastering”. Once the peak is established, this value is recorded along with the associated sound path length. Then, the attenuation factor is calculated for the sound path length (clause 6.19.6.4); the attenuation factor accounts for the fact that the transmitted sound does not all travel exactly on the intended angle but rather fans out somewhat, much in the same way that light leaves a flashlight. The attenuation factor is subtracted from the peak reference; the difference between this number and the reference level establish the rating for the discontinuity.

Using the rating, the discontinuity is classified as A, B, C, or D based on D1.5 table 6.3 or 6.4, whichever is applicable. The classification depends upon the transducer angle; as described in section 6.5.2.1, the rating differences among the angles incorporate an orientation correction. Class “A” discontinuities are rejectable, and class “D” discontinuities are acceptable. The acceptability of discontinuities that fall within Class B and C depend on length, separation, and distance from the edge (reference notes 1 and 2 in D1.5 tables 6.3 and 6.4).

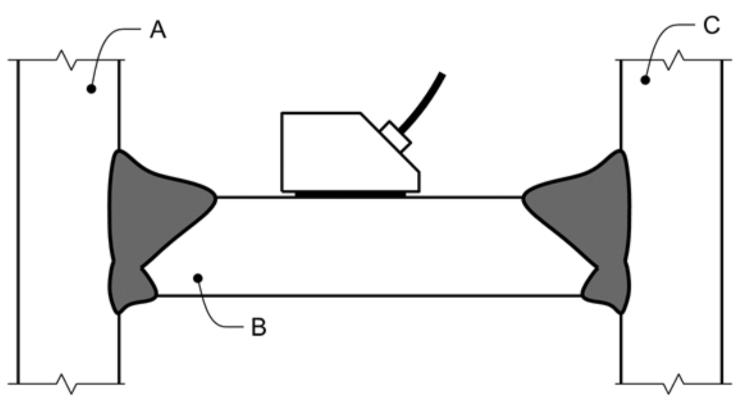
6.5.2.5. Application

UT is a highly versatile NDE method well suited to the evaluation of CJP welds. Specifics about application of UT are discussed in section 6.5.1. Summarizing, UT is effective for volumetric inspection of CJP welds, including butt, T-, and corner joints. The Code requires UT for CJP weld T- and corner joints, and requires both UT and RT for FC butt joints. Some owners prescribe UT instead of RT for non-FC butt joints, generally because they prefer UT's versatility over RT (section 6.9) and UT's superiority for finding cracks.

Caution is warranted in specifying UT for non-traditional CJP weld joints. For example, consider these two concerns:

- Skewed CJP weld butt joints - while practices are well established for both butt and perpendicular or near-perpendicular corner joints, CJP weld joints at some angle in between are unusual and may require special considerations.
- Joints with limited access - for example, consider figure 103. UT is required for the CJP welds between A and B and B and C. The traditional approach for UT would be to place the transducer on plate B, but there is insufficient space to move the transducer and properly scan the weld. Further, assuming that joint A to B is made first, that joint could be examined before plate C is added, but plate B may not be long enough for the transducer to be in the correct position to allow sound to pass through the joint as required. It may be possible to examine the joints from the opposite sides of plates A and C, but this would require a special procedure.

In special situations such as these, it may be prudent to require submittal of a special UT procedure in the contract documents.



Source: FHWA

Figure 103. Illustration. Example of difficult configuration for UT.

6.6. PHASED ARRAY ULTRASONIC TESTING (PAUT)

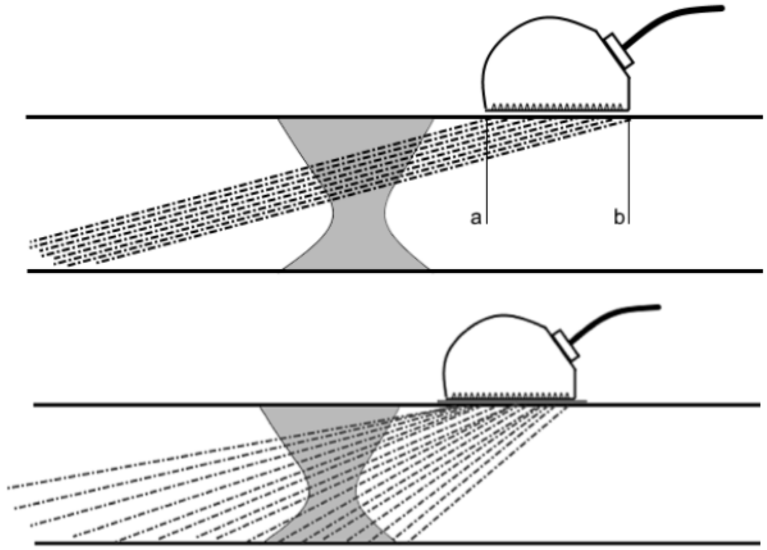
Phased array ultrasonic testing (PAUT), often referred to as "phased array", is an enhanced method of ultrasonic testing (section 6.5). PAUT is relatively new to bridge fabrication, having first been adopted into the Code in the 2015 edition. In clause 6.7.8, the Code allows PAUT to be substituted for traditional UT. Practice requirements for PAUT are defined in Annex K. As emphasized by the Code commentary (clause C-6.7.8), PAUT may be substituted for RT when approved by the engineer.

PAUT operates on the same principle as traditional UT: sound is transmitted through the base metal to the weld and reflected back to the transducer, and the characteristics of the reflected sound provide information about the soundness of the weld. As with traditional UT, if discontinuities are present in the weld, PAUT discovers them based on the reflected sound.

There are two fundamental differences between traditional UT and PAUT: use of multiple element transducers and encoding. Instead of just one element transmitting sound, the PAUT transducer has multiple elements, or an array of elements. During examination, all the elements transmit sound in sequence, one element after the other; hence, the sound transmission from the elements is phased.

Sounds from the elements can be transmitted in two types of scans:

- **Linear scan –** In a linear scan, all elements transmit sound at the same angle (figure 104, top). The advantage of linear scans is that instead of moving a traditional UT probe back and forth to get weld coverage, the PAUT probe covers a segment of the weld with one pass, getting broader coverage. For example, in the left figure, a traditional UT probe with one element would have to be moved from point "a" to "b" to get the same coverage the PAUT probe is getting without such movement.
- **Sectorial scan –** In a sectorial scan, all elements transmit sound at different angles (figure 104, bottom). The advantage of sectorial scans is that, like linear scans, they provide broad coverage and also provide coverage from a multitude of angles, thereby providing an increased opportunity to impact discontinuities squarely, with maximum sound reflection.



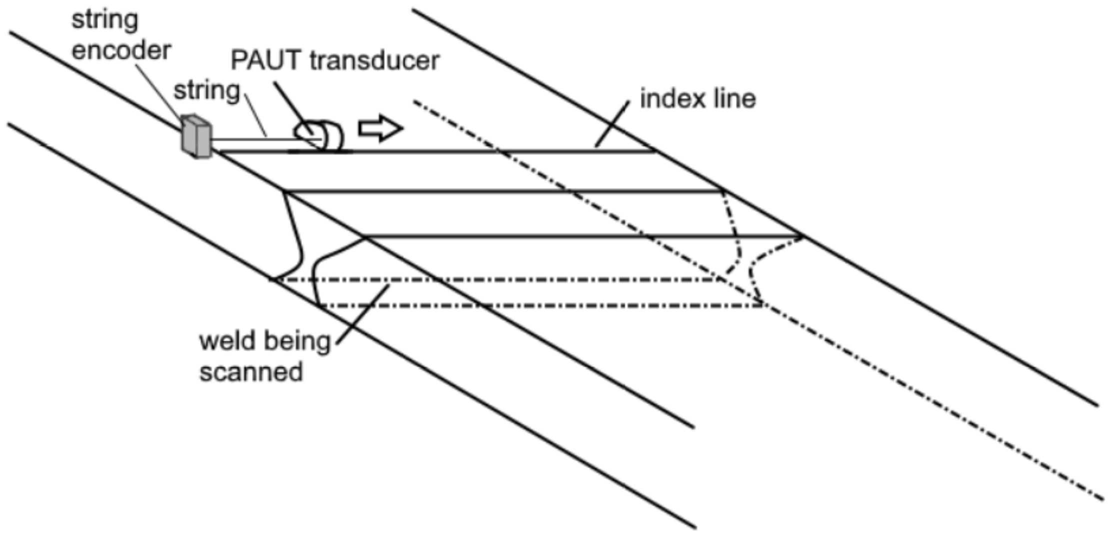
Source: FHWA

Figure 104. Illustrations. A PAUT linear scan, top, and a PAUT sectorial scan, bottom.

PAUT offers the opportunity for encoding. To encode, PAUT records the inspection results electronically such that the actual evaluation of the weld can be reviewed at any time. Thus, like RT, PAUT with encoding provides a permanent record of the actual examination results that are reviewed for acceptance of the weld. Encoding uses a device called an encoder that tracks the position of the transducer along the length of the weld during testing and records the position of the encoder in association with collected data. Under the Code, encoding is required (clause K9.3.2) when PAUT is used.

6.6.1. Operation

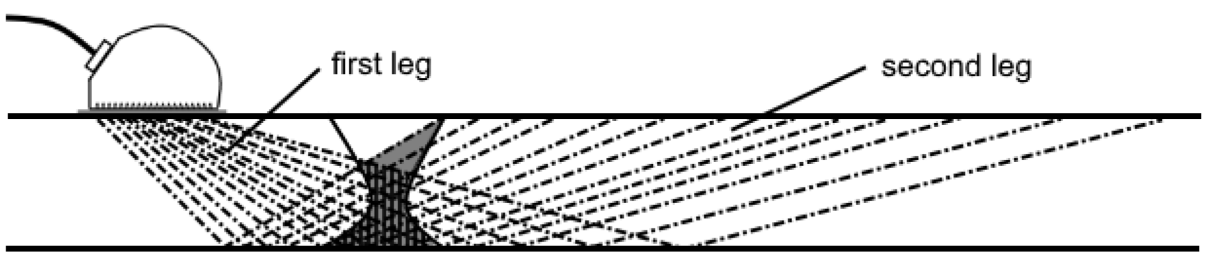
Figures 105 and 106 provide a representation of how a weld is scanned using PAUT. In figure 105, the transducer is shown at one point on the chosen index line; during examination it will be moved along the index line as indicated by the arrow. The transducer moves parallel to the weld along the index line such that the same cross-section of weld is scanned along the entire length of the weld. Also shown is an encoder; in this case, it is a string encoder. As the transducer is moved along the index line, scanning the weld, the string extends, simultaneously recording the position of the transducer and associating the scan information with the location of the transducer. Another common type of encoder is a wheel recorder. The wheel is attached to the transducer and positioned against the plate surface. As the transducer is translated, the wheel rotates, recording the location of the data being collected in the examination.



Source: FHWA

Figure 105. Illustration Details of a PAUT examination.

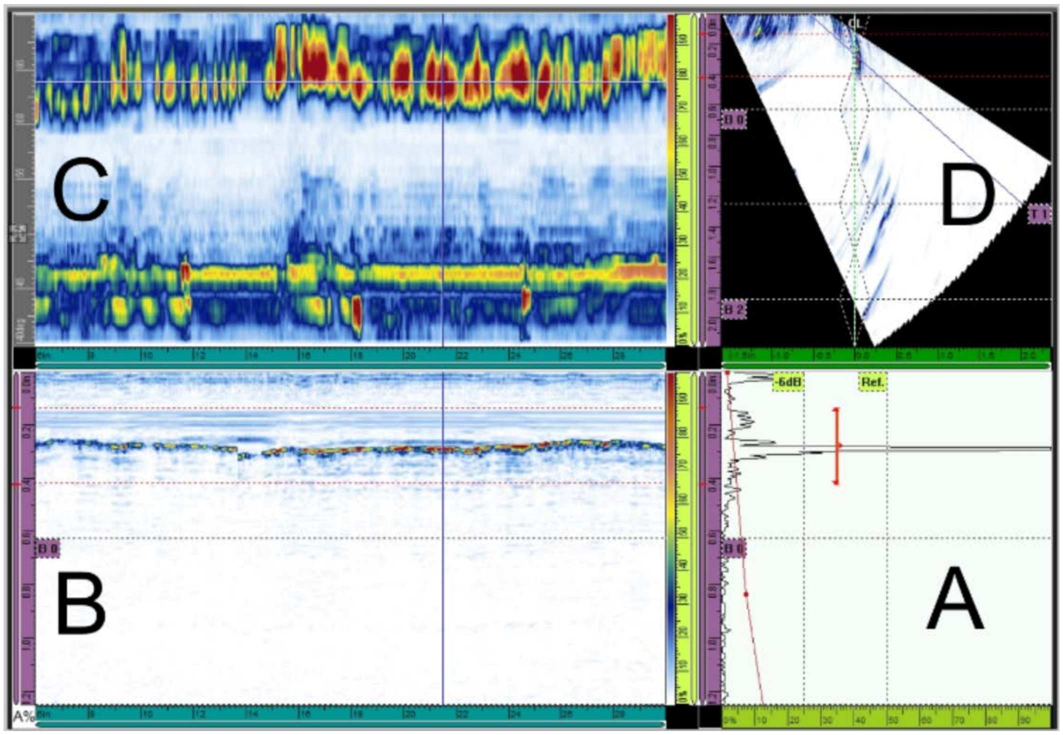
In figure 106, a sectorial scan is shown, with the transducer transmitting sound at a range of angles. For ease of understanding, only the first and second legs of the scan are shown in this figure; as with traditional UT (see figure 96), there may be more. The part of the weld being scanned by the first leg is shaded in vertical lines. The part of the weld being scanned by the second leg is shaded in gray. Note that in this scan, the lower part of the weld is being examined by both the first leg and second leg, and the second leg is also examining part of the top of the weld.



Source: FHWA

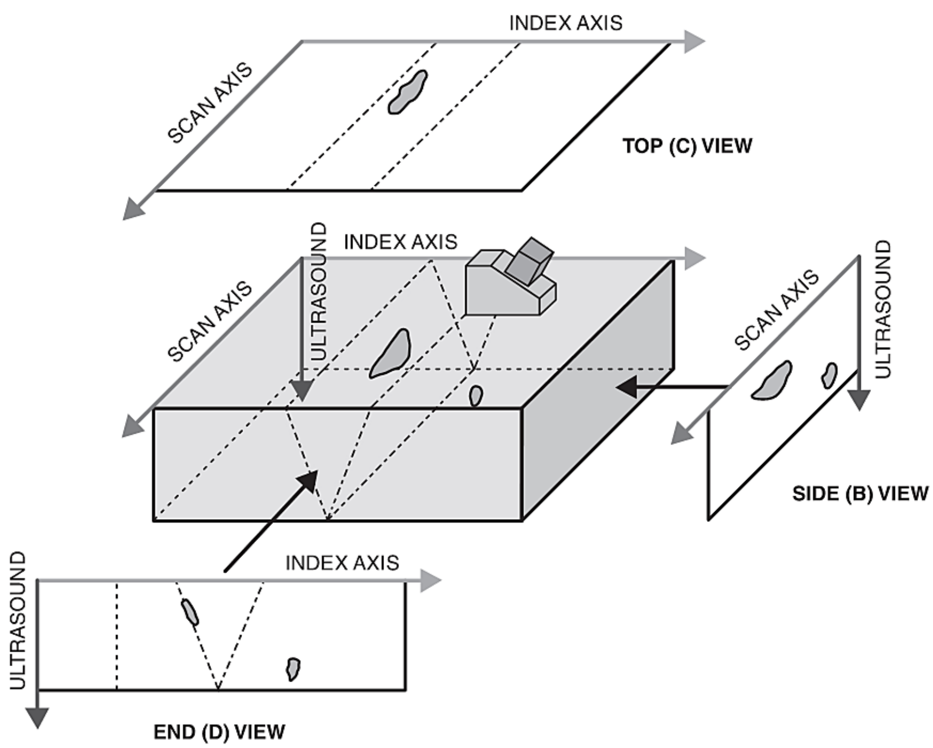
Figure 106. Illustration. Elevation of scanned weld.

PAUT examination results are shown on the PAUT machine display in four views. A screenshot of an actual PAUT test result in shown in figure 107. This screenshot happens to be from the same weld shown in the figure 92 digital RT image, which depicts incomplete fusion. The "A" view in figure 107 shows the same output as traditional UT. The other three views in figure 107, "B", "C", and "D", show weld cross-sections. These cross-sections are explained by figure 108, which is D1.5 figure K1 from the Code. The letters "B", "C", and "D" in figure 107 correspond to the same letters in figure 108.



Source: FHWA

Figure 107. Image. PAUT examination output.



Source: © 2015 AWS

Figure 108. Illustration. D1.5 Figure K.1 from the Code explains PAUT imaging views (AASHTO/AWS, 2015).

6.6.2. Practice Requirements

The Code requirements for the practice of PAUT are found in Annex K. This section of the manual highlights a few of the key requirements.

- **Scan Plans -** There are many choices to be made when using PAUT, and these depend upon several factors, including the joint configuration. The Code requires that a set of examination details be defined for the weld to be evaluated. The collection of these details is known as the “scan plan”. For example, as is apparent from figures 104 and 106, a sectorial scan using a PAUT transducer will typically examine part of a weld but not necessarily the entire weld. The part of the weld that will be examined depends upon the array of sound produced by the transducer, the weld joint geometry (width and depth or thickness), and the position of the transducer relative to the joint. Given a certain transducer and joint geometry, the transducer will need to be positioned multiple times at various index lines to thoroughly examine the joint. The scan plan will define the number and location of scanning positions.
- Use of a scan plan is required under the Code (clause K7.1). The essential variables (or required attributes) of the scan plan are defined in D1.5 table K.2; in addition to index position (required by clause K7.2.1), the essential variables include details about the equipment, technique, and software used. The scan plan must demonstrate coverage of the weld (clause K7.1.1), and calibration must be performed in conjunction with the plan (K7.1.2).
- **Coverage -** The Code requires full coverage of the weld, including the HAZ, in two crossing directions (clause K7.4). Further, sectorial scans must cover fusion faces within ±10 percent normal to the face. Therefore, when establishing the scan plan, angle must be chosen that satisfy this requirement (and defined in the scan plan). For complicated welds, a mock-up may be used to help demonstrate that the weld and HAZ are fully covered; see “mock-up” in this section.
- **Certification –** Like other NDE methods, the Code requires that PAUT be conducted by a Level II inspector (clause K4.1), and that the Level II be certified by a Level III (clause K4.2). However, as of the writing of this manual, ASNT has not established requirements for PAUT Level III certification. Therefore, the Code prescribes some detailed requirements for these certifications in clauses K4.1 and K4.2.
- **Scan type -** The Code requires use of sectorial scans, with linear scans to be used as needed to supplement the sectorial scans (clause K7.2).
- **Calibration –** The Code requires calibration to a supplemental calibration block (clause 5.7.1).
- **Mockup –** The Code addresses use of a mockup in clause K5.7.2. Mock-ups are not necessary for straightforward joints where the effective scanning plan is readily apparent, such as for flange or web butt splices. A mockup is not required but is prudent on joints with unusual configurations to demonstrate that the desired full coverage of the weld is obtained. The mockup should be a full-scale cross-section with side-drilled holes located to demonstrate coverage.

- **Longitudinal scanning –** In addition to scanning normal to welds, as figures 104 and 106 show, welds must also be scanned longitudinally to inspect for transverse discontinuities (clause K7.4.4).
- **Straight beam scanning –** As with conventional UT (section 6.5.2.3), a straight beam scan of the entire area to be examined with PAUT must be performed (clause K9.2). This is to ensure there are no discontinuities, particularly lamellar type discontinuities, present in the base metal that will interfere with the PAUT examination.

6.6.3. Practice

The use of PAUT in bridge fabrication has been growing since the 2000s, particularly with the addition of PAUT into the Code in 2015. Owners and operators have discovered that it can be useful for addressing unusual situations. For example, the joint shown in figure 109 is a CJP weld joining a curved plate to a tube. The owner desired NDE for this joint, but traditional UT was not suitable for the curved surfaces. PAUT was used instead, which required the development of a scan plan that specifically addressed the joint configuration and the scanning patterns used in the test.

Source: FHWA

Figure 109. Photo. Use of PAUT for an unusual CJP bridge weld.

Other owners are taking larger steps. Florida DOT allows use of PAUT in lieu of the RT required by the Code, including for FC members. To facilitate this step, the DOT conducted a study to demonstrate that PAUT would be effective (Wilkinson, 2014). Other DOTs are conducting parallel studies of butt joints in conjunction with fabricators. PAUT is appealing for several reasons:

- As a type of UT, PAUT is superior to RT in discovery of planar discontinuities (section 6.9).
- PAUT provides more detail about discontinuities than traditional UT. There have been examples of confusing results from traditional UT examination that have been clarified with PAUT.

- PAUT is highly efficient. Scanning is faster than traditional UT, and use of the method is faster and less intrusive than RT.
- Because it is encoded, it provides a permanent digital record that can be reviewed at any time. Further, this digital record is a potential substitute for the permanent record of RT.

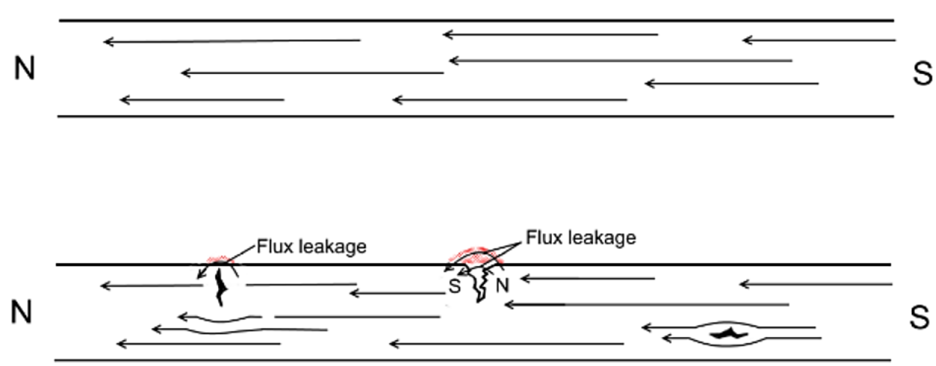
Work continues to develop the method. Following the adoption of PAUT to AWS D1.1 for the 2020 edition, efforts began to align the two codes, and this will lead to some changes to the language in D1.5. Given its advantages, use of PAUT will grow, particularly as a substitute for traditional methods.

6.7. MAGNETIC PARTICLE (MT)

Along with PT (section 6.8), MT is one of two visual NDE methods commonly used in bridge fabrication shops. It is quick and easy to use, and it readily reveals surface and near-surface discontinuities on welds and base metal. In practice, most of the discontinuities that MT reveals, such as undercut and cold lap, are also discoverable with a close visual inspection, but tight cracks are an exception. MT can also reveal discontinuities that are slightly below the surface, but in practice in bridge shops, observation of such discontinuities with MT is not common.

6.7.1. Mechanism

MT works through the interruption of a magnetic field by surface or near-surface discontinuities. Magnetic fields can be introduced by placing a permanent magnet or electromagnet on the material or by passing a current through the material. Discontinuities in the magnetic field interrupt the field and force some of the field to leak at the discontinuity (see figure 110). Open discontinuities, such as shown in the middle of the lower section, have significant leaks. As shown on the lower left and lower right, subsurface discontinuities may or may not cause surface leakages and, if so, the surface leakage will be less than that of open discontinuities. To make leaks observable, fine iron particles are dusted onto the surface being inspected. Drawn by the leaked field, these particles accumulate at surface discontinuities that are present (see figure 110).



Source: FHWA

Figure 110. Illustration. Iron dust accumulates at surface discontinuities in the presence of a magnetic field.

For MT to work, the material being tested must be ferromagnetic. MT is suitable for use on all A709 steels except, partially, for 50CR. Grade 50CR, like some other stainless steels, is ferromagnetic; however, the consumables used for welding 50CR are not ferromagnetic. Therefore, MT can be used to examine 50CR base metals but not the welds that join 50CR materials.

MT can be conducted with permanent magnets, but permanent magnets that would be large enough for inspection are heavy and require 50 pounds of force to remove, and so in the shop, electromagnets are far more practical. The MT methods prescribed by the Code (section 6.7.2) use electromagnetics (yoke) or current (prod).

Currents used for MT include the following:

- **AC -** With AC, the magnetic field is concentrated near the surface of the steel, typically reaching depths of about $^{1}/_{16}$ inch. Therefore, it is well suited to discovery of discontinuities that are open to the surface but not useful for discovery of subsurface defects.

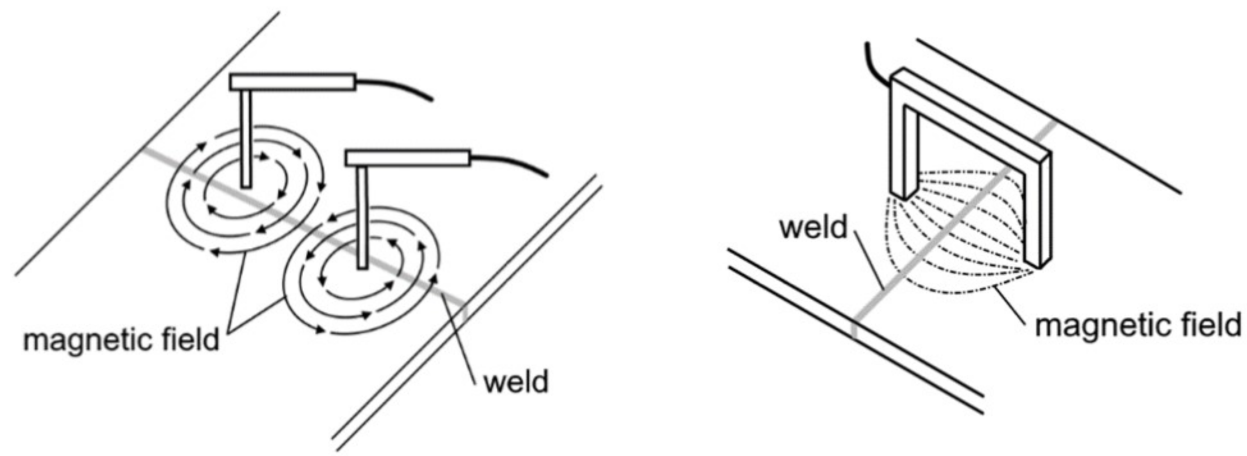
- **Half wave rectified DC –** This mode is actually an AC current that is converted to a one-directional current by removing the bottom half of the AC wave. With no current flowing for half of the wave cycle, the current repeatedly switches on and off, creating a pulse. The pulse action improves detection somewhat because the pulse helps move the particles.

- **DC –** Unlike AC, which concentrates near the surface, with DC, the magnetic field fills the cross section. Therefore, DC is better suited than AC for discovery of subsurface discontinuities; however, as shown in figure 110, this leakage is generally not significant and is difficult to detect at the surface. Generally, MT is not well suited to discovery of subsurface discontinuities. Also, because the magnetism fills the cross section, magnetism at the surfaces is not as strong, and therefore is it less effective at detection of surface discontinuities than AC.

- **Pulsed DC –** Pulsed DC is the equipment switching the DC current on and off to create a pulse. Like the pulse in half-wave rectified DC, this pulse action improves particle movement.

6.7.2. Method

The Code allows the use of two different methods of MT: the prod method and the yoke method (clause 6.7.6; see figure 111 below). In bridge shops, the yoke method is generally preferred and for FC is required, particularly because prods are prone to producing arc strikes (section 5.6). The yoke probes are adjustable to facilitate contact between the probes and the base metal on either side of the weld. The lifting capability of the yoke can be checked to verify that the yoke creates sufficient current for proper testing. The Code specifies minimum lifting forces for AC and DC in clause 6.7.6.2(2), and this is verified prior to inspection by lifting a calibrated weight.

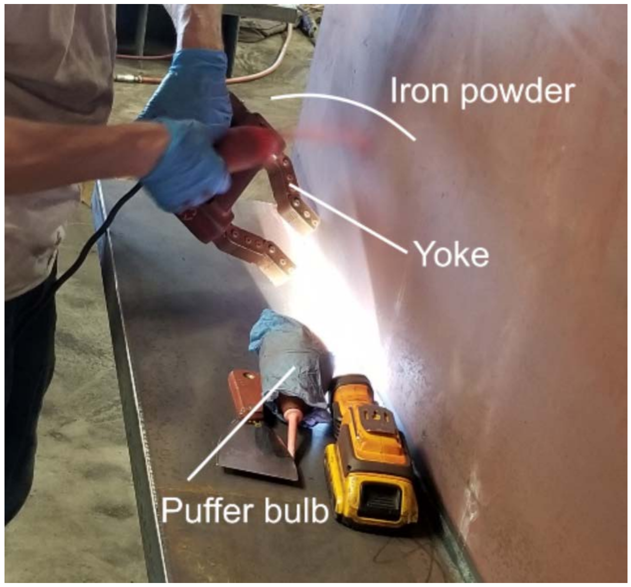


Source: FHWA

Figure 111. Illustration. Prod (left) and yoke (right) magnetic fields.

The basic steps of performing MT are as follows:

- The weld to be examined and the base metal in the near proximity of the weld are cleaned (clause 6.7.6.3).
- The yoke (or, less commonly, prods) is placed over the weld with one probe each touching the base metal on either side of the weld.
- Small iron particles are dusted on the surface in the inspection area. The particles are dyed for improved contrast and visibility on the joint (see figure 112).
- Excess iron particles are gently blown away using a puffer bulb; a puffer bulb is also shown in figure 112. Sufficient particles are blown away such that built-up dust does not mask discontinuities, but blowing is also light enough that particles are not blown out of discontinuities that are present.
- The test is interpreted immediately by the technician performing the testing. Deleterious conditions such as cracks are rare, but if cracks are indeed discovered, these are documented on the test report. Occasionally MT indicates conditions such as overlap or undercut that need to be addressed. Also, the somewhat abrupt but acceptable contour change that occurs at weld toes will attract powder, but these indications are not a concern.
- The yoke is then repositioned at an orientation 90 degrees to the first position (clause 6.7.6.4) and the dust protocol and interpretation are repeated. To facilitate the two perpendicular placements, the yoke may be oriented at 45 degrees to the weld.
- When the second orientation is completed, the technician moves to the next adjacent position and repeats this until inspection of the required length is achieved, moving about six inches per position.



Source: FHWA

Figure 112. Photo. Magnetic particle inspection.

6.7.3. Application

The Code requires MT for fillet welds and PJP groove welds on main members (clause 6.7.2). The Code requires 12 inches in every 10 feet to be tested.

In practice, it is rare to discover deleterious conditions such as cracks when using MT. This is not because MT is incapable of finding cracks. Rather, this is probably due to the fact that cracks in fillet welds are indeed rare.

If a bridge is to be coated in the shop, MT is performed before the weld is coated. MT using the yoke method can be performed through coatings if they are thin and uniform, but a special procedure is needed. Such a procedure can be developed by creating and testing a mockup of the condition to be tested.

6.8. DYE PENTRANT TESTING (PT)

Dye penetrant testing (PT), is the other surface NDE method commonly used in bridge shops. It is rather time consuming to use, particularly compared to MT, but it is very good for detecting fine discontinuities.

6.8.1. Mechanism

With PT, penetrant is sprayed onto the surface of concern and then through capillary action enters any surface discontinuities that are present. A variety of penetrants exists for various applications. In bridge fabrication, dye penetrant is most common, and in fact PT is often referred to as dye penetrant testing. Excess dye is cleaned from the surface, and the dye in discontinuities remains behind. The remaining dye may be readily visible on the now bare surface, or developer can be used to provide better contrast and help draw the dye from the discontinuities.

6.8.2. Method

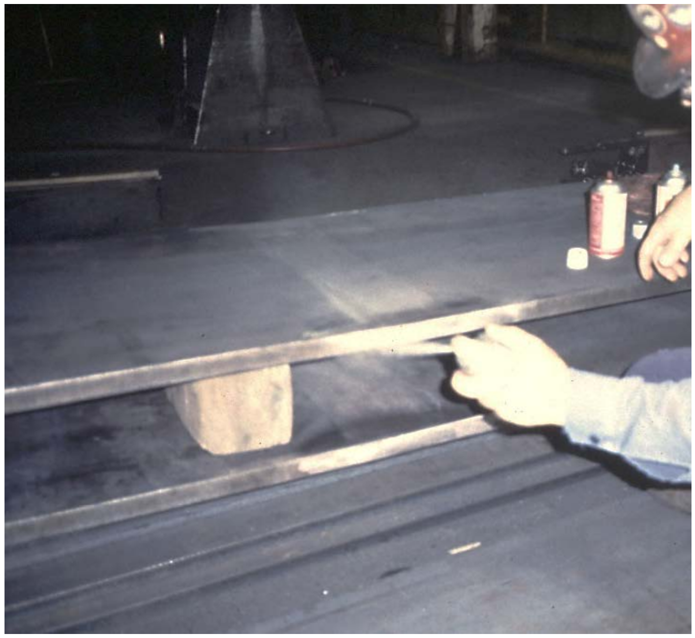
The Code does not require PT, and therefore the Code does not have detailed requirements for performing the method. However, clause 6.7.7, does require PT to be performed in accordance with ASTM E165, *Standard Practice for Liquid Penetrant Testing for General Industry* (ASTM, 2018). Therefore, when PT is required in the project specifications or for conformance verification, such as in a repair, ASTM E165 applies, and also as stipulated in clause 6.7.7, clause 6.26 applies for acceptance criteria.

The basic steps of PT per ASTM E165 are as follows:

- The surface to be examined must be cool.
- The surface is thoroughly cleaned, including solvent cleaning if any oil, grease, or cutting fluids are present.
- The penetrant dye is sprayed or brushed onto the surface of concern; there is no particular limit to the amount applied because the excess will be removed.
- The penetrant is allowed to dwell, allowing penetrant to draw into any openings through capillary action. The dwell time will be recommended by the product manufacturer and may be up to half an hour.
- After the dwell time, excess penetrant is removed from the surface. The product manufacturer's recommendations should be closely followed in the operation in order to avoid removing the penetrant from discontinuities.
- Developer is applied. The developer draws out any penetrant that seeped into openings, thus making the openings obvious. Many developers also provide a white background which improves contrast with penetrant.
- Inspection takes place after the developer has had time to draw out the penetrant, typically after a few minutes.

6.8.3. Application

PT is a useful tool for exploring surface discontinuities, or where there is a particular concern for avoiding them. For example, if cracks are discovered on the surface of a plate, PT can be used to make examination of the surface easier and also to facilitate photo documentation. Further, if cracks are remediated with shallow excavations, PT can be used to check the surface of the excavation to ensure cracks are entirely removed. Fabricators may also choose to use PT to help ensure soundness when material is removed. For example, when run-off tabs are removed from butt splices, it can be difficult to ensure by visual inspection that there are no small nicks on the flange edge; if small nicks are left there, they will be detected by subsequent RT and potentially cause a rejection of the weld. PT can be used to make sure the flange edge is free of discontinuities before RT or UT is performed (see figure 113). See section 6.10 for further discussion, including differences in application of MT and PT.



Source: FHWA

Figure 113. Photo. PT used to verify flange edge condition.

6.9. COMPARING VOLUMETRIC METHODS - RT, UT, AND PAUT

The generally homogeneous nature of steel and welds facilitate the effectiveness of the volumetric NDE methods used in bridge fabrication. Radiographic testing (RT - section 6.4) was the first volumetric examination method used regularly in weld evaluation. Later, in the late 1960s, ultrasonic testing (UT - section 6.5) was introduced for volumetric weld inspection. Both RT and UT are specified by the Code for weld examination. In 2015, phased-array ultrasonic testing (PAUT) was adopted into the Code as an enhanced form of enhanced UT. As of the 2015 edition, PAUT is not required by the Code but is allowed as a substitute for UT and, with the engineer's permission, for RT. This section discusses RT and UT at length because these are historic methods; PAUT is discussed in section 6.6.

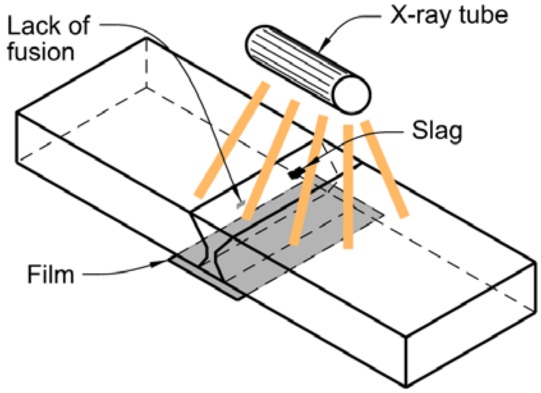
RT, UT, and PAUT are not suitable for examination of PJP or fillet welds in most bridge applications. By their nature of not having complete penetration, PJP welds have significant unfused zones at their root. In the case of RT, this unfused zone would show up as a dark area on the radiograph that would mask any discontinuities above or below this zone. In the case of UT and PAUT, the unfused zone would disrupt UT or PAUT signals and interfere with accurate evaluation. Similarly, these three techniques are not suitable for fillet weld evaluation. As with PJP welds, the unfused gap just behind fillet welds disrupts UT and PAUT signals. Fillet welds cannot be evaluated by RT both because the unfused zone would interfere with evaluation of the radiograph and more so because RT is not effective for evaluating welds in corners.

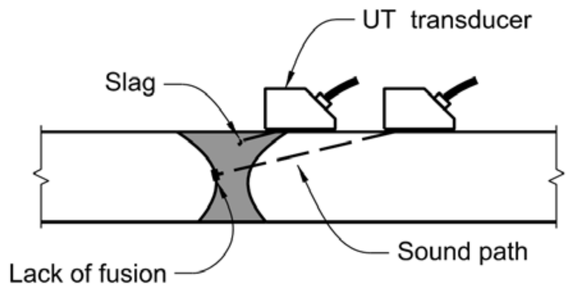
The acceptance criteria for volumetric methods originated with the introduction of RT. Later, when UT was developed, acceptance criteria were adopted that were intended to result in a similar quality levels. For both methods, the criteria are more stringent for welds that are in tension than those that are in compression or shear, based on the premise that if flaws are too large, dynamic vehicular loading could cause cracks to grow from flaws. However, the criteria are based on workmanship and not fitness for purpose. Put another way, the criteria reflect the workmanship that can be expected from a proficient welder making a CJP weld; the criteria were not adopted based on research or analysis showing that larger flaws would be deleterious for the bridge. However, although the criteria are not based on fitness for purpose, it is important to recognize the effectiveness of the criteria in practice: in the many decades of making CJP bridge welds under the criteria, no known weld that satisfied these criteria has cracked in service.

In terms of inspection effectiveness, there are advantages and disadvantages associated with both RT and UT:

- **Joints -** A key difference between the two methods is that RT is only effective for examining butt joints whereas UT can be used for T- and corner joints as well as butt joints.
- **Documentation** - RT offers an operational distinction that is popular with owners because the results are interpreted from a film (or, more recently, a digital image), and so the results can be re-interpreted by a second party for verification or to offer a second opinion at any time. With UT, the interpretation occurs as the screen is read and decibel (dB) levels are documented onto the UT report, so this interpretation cannot be re-evaluated or verified without reinspecting the weld (this is changing with PAUT—see section 6.6). It is for this reason that many bridge owners prefer RT to UT, and RT remains the volumetric method of choice in the Code: if there are questions about inspection, owners can re-interpret radiograph at any time, but such re-evaluation is not possible with UT. Rather, the report and the original test must be trusted.
- **General discontinuity evaluation** - Fundamentally, both RT and UT are excellent methods for evaluating the soundness of CJP welds. However, RT and UT do not detect discontinuities in the same way. For example, in the case of porosity, RT is somewhat better at discovery than UT because internal porosity, being spherical, tends to scatter sound. This does not mean that UT does not discover porosity; rather, compared to a discontinuity such as slag which is not spherical, UT will not get as strong a reflection. With RT, slag and porosity are clearly distinguishable. More importantly, UT is superior at discovery of planar discontinuities (see discussion in this section). Some discontinuities accepted based on RT are not acceptable based on UT, and the reverse is true as well, but generally such disagreement in acceptance is the exception; only a very small percentage of discontinuities are encountered that would be of just the right size or orientation and character that they would pass one method and not pass the other.
- **Planar discontinuity evaluation** - UT is better for discovery of thin, planar discontinuities than RT. Very thin discontinuities such as lack of fusion may not create enough of a void to show up on a radiograph. This is particularly true if the discontinuities are normal to the direction of the radiation. For example, consider figure 114. The weld being investigated has two discontinuities—a thin discontinuity on the left and slag on the right. Shown left, the radiation from the x-ray tube penetrates the weld and projects images of both discontinuities on the film (see section 6.4.1 and figure 115). The slag, which reduces the total thickness of the material between the source and the film, shows clearly on the film, but the thin discontinuity, which has little effect on the total thickness of material in the path of radiation, is barely visible. In practice, such thin

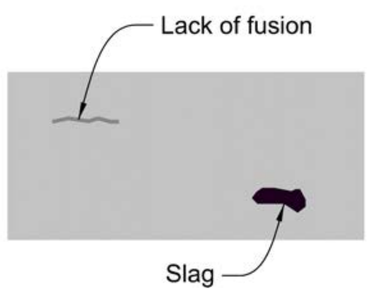
discontinuities might not show at all, or, if they do as faint images, may easily be overlooked. By contrast, shown right, UT is well suited for finding both the slag and the thin discontinuity (section 6.5.2). Cracks are generally thin and planar. In practice, cracks are rare in typical flange and web butt splices, but if they were to occur, UT would be much more likely to find them than RT.





Source: FHWA

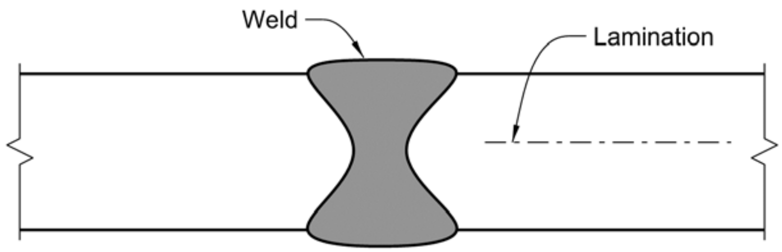
Figure 114. Illustrations. Discovery of planar and volumetric discontinuities using RT (left) and UT (right).



Source: FHWA

Figure 115. Illustration. RT film.

Laminations (section 5.4.1.4) are another type of planar discontinuity in which there are important distinctions between RT and UT. Laminations tend to run parallel to base metal in relatively large planes (see figure 116). As such, they generally are not detected by RT, but are readily detectable by UT.



Source: FHWA

Figure 116. Illustration. Plate lamination.

From an operations standpoint, UT offers significant advantages on the shop floor compared to RT:

- **Result availability**- UT results are available immediately upon testing versus waiting for film processing (this benefit does not exist when compared to digital RT (section 6.4.4), which also provides instant results). Film can be processed in about an hour, but this time is much greater if a subcontractor is performing the RT and must return to a lab to process the film and then return it to the shop. Further, if there is a quality problem with the film (such as a streak or an issue with density), then the RT process starts over again.
- **Portability** - UT equipment is highly portable. A UT technician can readily carry to the weld everything needed to conduct testing in one trip. RT equipment is heavier and bulkier, taking much longer to get set up for testing than UT.
- **Safety** - UT does not have the risk of radiation exposure that RT has.
- **Workflow impact** - UT does not have the standoff distance for safety that RT requires. When UT is being performed, other fabrication operations can continue in the immediate vicinity of the inspection. With RT, no operation can take place within a certain distance, which can easily be the width of a shop or bay, and so during radiography, workers must leave the surrounding area, or RT must be scheduled off-shift.
- **Discontinuity depth** - UT results include the depth of the discontinuity. RT shows discontinuities on a flat plane, through the weld in plan view, with no indication of the depth of the discontinuity. In fact, when defects are discovered with RT and need repair, UT is usually used first to determine the depth of the discontinuity to facilitate an effective repair.
- **Cost -** Both initial equipment costs and equipment maintenance costs are lower with UT. This is true in comparison with both film and digital RT; with digital RT there is no cost associated with films, processor, and chemicals for processing; however, there are costs associated with replacing digital panels and screens (section 6.4.4), which do wear out eventually.
- **Inspector availability -** Generally, there are more UT technicians than RT technicians. Many fabricators have in-house UT personnel but not in-house RT personnel. Thus, RT is more likely to be outsourced, often delaying inspection and making it more costly.

- **Operator Dependence** - One potential disadvantage of UT compared to RT is its greater reliance on skilled operators.

From an operations standpoint, PAUT (section 6.6) is similar to UT with its advantages over RT, and has the following advantages over traditional UT:

- **Speed** - Because sound is sent at so many angles at one time, inspection at discrete indices are done with one pass of the transducer.
- **Encoding -** PAUT can be encoded. This means that, like RT, a permanent record of the test as-conducted can be stored as well as shared electronically and, if desired, evaluated again.

6.10. COMPARING SURFACE METHODS

There are two NDE surface methods commonly used in steel bridge fabrication: magnetic particle inspection (MT - section 6.7) and dye penetrant inspection (PT - section 6.8). The Code mandates certain inspection by MT but does not require any PT. They are similar in that they both reveal essentially the same surface discontinuities, but there are also fundamental differences between the two methods, making each useful in different ways. MT is a quick test, making it ideal for quick verification of weld soundness and reasonable for inspection of long welds. Further, unlike PT, MT can be performed on hot steel, making it more versatile for in-process testing. However, MT is not suited to revealing the surface condition for more than a few moments, and PT is much more sensitive than MT. Therefore, if there is a need to study the evaluated surface or document the evaluated surface with photography, PT may be a better choice. In practice, application of PT tends to be at the fabricator's option (such as for a material mill claim by the fabricator) rather than for final acceptance. However, if MT is not available or appropriate, particularly if the material is not ferromagnetic, then PT is a good substitute.

6.11. INSPECTOR CERTIFICATION

6.11.1. Visual Inspection Certification

The Code requires that inspectors be qualified to perform visual inspection (clause 6.1.3; section 6.2). Qualifications help ensure that inspectors have the knowledge and experience to understand Code requirements, their application, and make inspection as objective as possible.

There are three options for inspector qualification in the Code:

1. **AWS certification -** The inspector may be certified by AWS as a Certified Welding Inspector (CWI). This is by far the most common certification in the United States; to the authors' knowledge, CWIs are found in all bridge shops.
2. **Canadian Welding Bureau (CWB) certification -** The CWB program is certification to the Canadian Standards Association (CSA) standard W178.2, *Certification of Welding Inspectors*, (CSA, 2018). In the United States, inspectors who carry the CSA certification typically also carry a CWI. AWS will issue a CWI to inspectors who carry a CSA certification; however, the reverse is not true.

3. **Equivalency -** The engineer may accept inspectors who are not certified but that the engineer accepts as being equivalent to one of the other certifications based on their training and experience.

Generally, certifications for CWI and the CWB program are expected to be current, but clause 6.1.3.2 does allow inspectors to be only previously certified provided, “there is acceptable documentation that the Inspector has remained active as an Inspector...and there is no reason to question the Inspector’s ability”. However, qualification of inspectors with this approach is not common nor known to be acceptable to owners.

The CWI program is defined in AWS standard QC1 (AWS, 2007), which was first published in 1975. There are three levels of certification: Certified Associate Welding Inspector (CAWI), Certified Welding Inspector (CWI), and Senior Certified Welding Inspector (SCWI). It is the CWI who has the authority to accept welds for conformance with the visual requirements of the Code. Informally, the CAWI is intended to be a CWI in training. However, certification as a CWI does not first require certification as a CAWI, and the CAWI certification is not common in bridge fabrication. There is no defined role for an SCWI in the Code, so this certification is also uncommon.

To be certified as a CWI, an inspector must have five years of experience as a welder or inspector, must pass a vision examination, and must pass an examination administered by AWS. The examination is challenging; many inspectors do not pass the examination on their first try. The examination assesses skills in performing visual inspection, knowledge of the Code or other welding codes (at the inspector’s choice, from a limited selection), and more general welding topics. To keep their certification current with AWS, inspectors must remain professionally active, pay an annual fee, and take a recertification examination or training every nine years.

6.11.2. NDE Technician Certifications

To help ensure that NDE is performed properly, the Code requires that it be performed by technicians certified in conformance with the requirements of standard SNT-TC-1A of the American Society of Nondestructive Testing (ASNT). SNT-TC-1A’s provisions are recommendations (ASNT, 2016); the Code makes the recommended requirements mandatory. The following are noted aspects of this program:

- **Certification Levels** - The SNT-TC-1A program has three levels of certification that apply to each method:
  - Level I - These are technicians who are in training. They can help perform testing in cooperation with a Level II technician, but they cannot provide an official assessment of a test.
  - Level II - These technicians are authorized to perform NDE without supervision and to provide an official assessment of the test results.
  - Level III - This technician has two distinct roles:
    - Develop and authorize a written practice for specific NDE methods (next bullet).

        - Oversee the training, experience, and testing of Level II technicians, and approve them as Level II technicians when they have satisfied the requirements for certification.

    Technicians achieve Level III status by passing a Level III examination administered by ASNT. The Level III technician may work for the same company as the Level II technicians, or the Level III technician (as with other certification levels) may be a subcontractor.

- **Written Practice** - SNT-TC-1A is not a standard practice that is followed directly. Rather, SNT-TC-1A provides recommendations for an NDE written practice. In turn, the written practice is followed for NDE technician certification. SNT-TC-1A recommends that the written practice be developed and authorized by a Level III technician.
- **Certification requirements** - SNT-TC-1A recommends minimum training, experience, and testing requirements for technicians in the NDE method for which the technician is certified. A Level III technician in that NDE method verifies that the minimum training and experience requirements have been satisfied and administers the test.

In summary, under the SNT-TC-1A program, NDE is performed by Level II technicians, with assistance from Level I technicians, in conformance with a written NDE practice that has been authorized by a Level III technician. The Level II technicians achieve certification through experience, training, and passing an examination overseen and administered by a Level III technician.

In addition to SNT-TC-1A, ASNT defines another NDE qualification program under standard CP-189, *Standard for Qualification and Certification of NDE Personnel* (ASNT, 2016a). CP-189 is similar to SNT-TC-1A but while SNT-TC-1A is a recommended practice, CP-189 mandates minimum requirements and employers cannot deviate from these requirements when CP-189 is specified. There is some movement afoot in the structures community towards CP-189 to narrow the variations encountered with the written practices developed and adopted by employers under SNT-TC-1A, although practices in the bridge fabrication community are generally uniform. CP-189 is not often required but is sometimes specified as an alternative to SNT-TC-1A.

6.12. DOCUMENTATION OF INSPECTION

Thorough and accurate documentation is an important component of steel bridge fabrication. Requirements vary by owner, but certain documentation requirements and expectations for inspection of bridge welds are standard, as follows:

- Visual inspection - project documentation affirms that visual inspection was performed and that visual requirements of the Code were satisfied. This is a statement about the acceptability of the component and not details about the results of inspection. It is common to find some defects during visual inspection, but if so, these are typically marked on the steel by the inspector and not documented on paper or otherwise. The acceptance statement in project documentation means that if any defects were found during visual inspection, they were successfully addressed.
- Required NDE - documentation includes reporting of any required NDE:

- RT documentation includes RT reports and the actual radiographs
- UT documentation includes UT reports
- MT documentation includes MT reports

- In-process NDE performed by the shop for shop purposes, such as use of PT to verify soundness in preparation for welding, is not included in project documentation.
- Documentation of significant welded repairs, i.e., repairs serious enough to require engineer approval (section 10.6).

CHAPTER 7 - WELDING TO THE FRACTURE CONTROL PLAN

"AASHTO/AWS Fracture Control Plan (FCP) for Nonredundant Members" is the name of clause 12 in the Bridge Welding Code. Clause 12 is a collection of special fabrication requirements that relate to FC bridge members. A fracture-critical member (FCM) is "[a] steel primary member or portion thereof subject to tension whose failure would probably cause a portion of the entire bridge to collapse" (AASHTO, 2017a). The Code has a slightly different definition in clause 12.2.2, but it is the same in concept. Less formally, FCMs can be thought of as nonredundant members subject to tension. The FC condition only applies to tension because members will not fracture in a brittle manner under compression loading.

In design, engineers designate bridge members and components that are FC in accordance with AASHTO BDS requirements. Fabricators then apply the clause 12 rules when fabricating these members and components. In addition to special treatment of materials, welding, and inspection for FCMs during fabrication and an extra FC certification for fabricators, there are special design and hands-on in-service inspection requirements for FCMs; these requirements are not discussed here.

7.1. HISTORY

There are many published resources that document the historic events that led to the creation of the FC designation and associated fabrication and in-service inspection requirements. Two documents that are particularly useful are:

- "SS-H-2 Highway Accident Report, Collapse of U. S. 35 Highway Bridge, Point Pleasant, West Virginia, December 15, 1967," (NTSB, 1970).
- "Evolution of Fatigue-Resistant Steel Bridges," (Fisher, 1997).

The following provides a synopsis of the background of the AASHTO FCP.

The Point Pleasant Bridge

The collapse of the Point Pleasant Bridge (also known as the Silver Bridge) over the Ohio River at Point Pleasant, West Virginia, and Gallipolis, Ohio, precipitated the establishment of the FCP. The bridge was built in 1928 and collapsed on December 15th, 1967 after almost 40 years in service, resulting in 46 fatalities.

The bridge used an eyebar suspension superstructure design. At the time of the Point Pleasant Bridge's construction, eyebar suspension bridges were common, but the Point Pleasant Bridge was unusual in that each link in its suspenders was composed of only two parallel eyebars. The entire bridge collapsed when one eyebar in a link pair fractured at a pin connection. The pin connection then twisted out of plane and the second eyebar slipped off the pin, severing the entire suspender and resulting in complete collapse of the entire bridge. Although much of FC practice is focused on welding and weld inspection, note that the Point Pleasant Bridge failure was not associated with a weld.

The collapse of the Point Pleasant Bridge demonstrated that failure could occur due to a lack of redundancy in a bridge's structure. Failure of the eyebar initiated from a small flaw that may have been caused by stress corrosion cracking, but did grow in fatigue under cyclic loading. The fracture toughness of the eyebar was low and could only tolerate a crack about ¼ inch long, which could not be seen because the pin was covered with a retaining plate. With the confluence of the cold December temperature, heavy traffic on the Ohio-bound side of the bridge, and the presence of a small crack, the eyebar failed by brittle fracture. One outgrowth of the subsequent investigation was the modern-day AASHTO FCP.

The AASHTO Fracture Control Plan

The Fracture Control Plan originated as the AASHTO Guide Specification for Fracture-Critical Non-Redundant Bridges Members, which was adopted by AASHTO and endorsed by the FHWA as a reaction to the Point Pleasant Bridge failure to help avoid a similar catastrophe. The guide specification was first published in 1978 and addressed design, materials, and fabrication of FC members. Fracture control plans were issued as special specifications based on the guide specification; owners incorporated these fracture control plans into their contracts on specific projects that had FC elements. In 1995, the fabrication and shop inspection aspects of the FCP were adopted as clause 12 of the Bridge Welding Code. These changes resulted in the current practice: owners require use of the Code for steel bridge fabrication on their projects; engineers designate FCMs on plans; and then, because the Code is already specified on the project, the clause 12 FCP applies for fabrication. The engineer need only designate the FCMs on their project plans and the clause 12 FCP will apply for fabrication provided the Code is already specified.

7.2. IDENTIFICATION OF FRACTURE-CRITICAL MEMBERS

Bridge design plans must clearly identify which, if any, members or components are considered to be FC. This practice is prescribed in the AASHTO BDS (AASHTO, 2017a), which states in section 6.6.2.2:

> *"The Engineer shall have the responsibility for identifying and designating on the contract plans which primary members or portions thereof are fracture critical members (FCMs). The contract documents shall require that all members meeting the definition of an FCM be fabricated according to the provisions of Clause 12 specified in the AASHTO/AWS D1.5M/D1.5* Bridge Welding Code."

Clause 12.1 of the Bridge Welding Code states the following:

> *"This clause shall apply to fracture critical nonredundant members. All steel bridge members and member components designated on the plans or elsewhere in the contract documents as fracture- critical shall be subject to the additional provisions of this clause. All other provisions of the code shall apply to the construction of fracture critical members (FCMs), except as modified or supplemented herein. Should any provision of this clause be in conflict with other provisions of the code, the requirements of this clause shall apply."*

In practice, engineers are required to clearly identify FCMs on design plans as directed by AASHTO BDS, and fabricators produce those members in accordance with clause 12 of the Code.

7.2.1. Proper Use of the FC Designation

Welding practices associated with FC requirements (section 7.5) have been adopted as an extra level of conservatism towards avoiding weld discontinuities. Engineers should recognize that the requirements of the code for non-FC members represent already-conservative welding practices that have demonstrated appropriate weld performance over decades of steel bridge construction. It is neither necessary nor prudent to indiscriminately designate redundant members, or compression members or elements, as FC on the premise that doing so will make the bridge better. Rather, designating redundant members as FC makes them unnecessarily more expensive to fabricate as well as extraordinarily more expensive to maintain, considering mandatory in-service inspection requirements for FC members. The Bridge Welding Code commentary advises

> *"The Fracture Control Plan should not be used indiscriminately by designers as a crutch 'to be safe' and to circumvent good engineering practice. Fracture critical classification...is only intended to be for those members whose failure would be expected to result in collapse of the bridge [by fracture]."*

Improper designation of members as FC is not detrimental to the bridge; however, it is poor engineering practice.

7.2.2. Tension versus Compression Members

A fundamental premise of the FCP is that cracks can only extend under tensile load. The Bridge Welding Code specifically states that, "welds to compression members or compression area of bending members shall not be defined as fracture critical" (clause 12.2.2.2). The AASHTO BDS direction is clear that only members or components subject to tension from Strength I load combinations (not other combinations, such as from wind or seismic loads) are to be designated as FCMs. As with misidentifying redundant members as FC, designation of compression zones (or entire members, including compression zones) as FC is poor engineering practice.

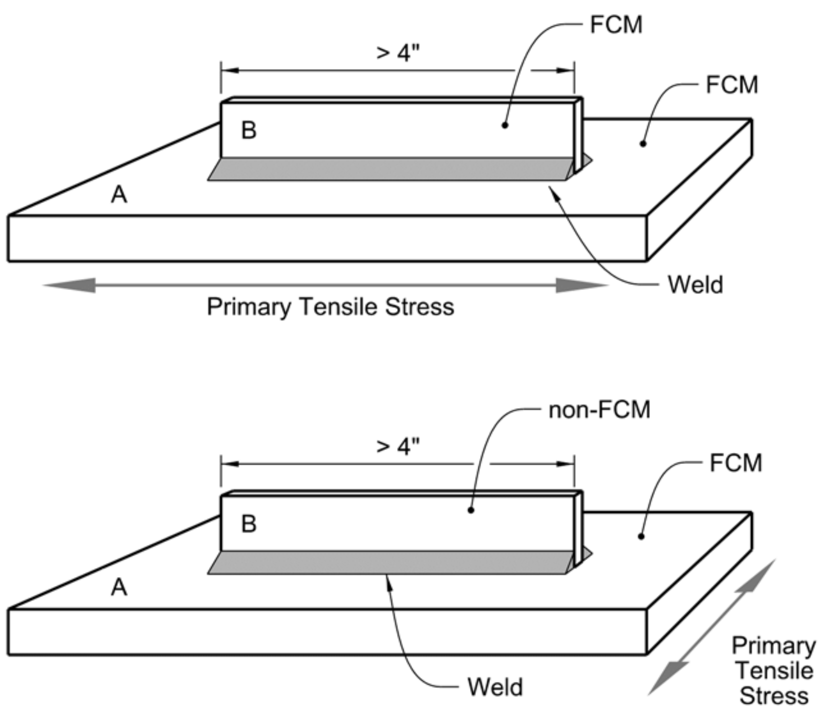
If the fabricator does not clearly understand which components are FC, the fabricator should request clarification. If a plan designates a component that is not in tension as FC, the fabricator may also request a clarification. If this status remains FC, the fabricator will treat the component and associated welds as FC per the engineer's instructions, even though the stress will not extend cracks (see section 7.2.5 for the proper use of FC designation).

7.2.3. Attachments to FCMs

Section 6.6.2.2 of the AASHTO BDS addresses attachments to components designated as FC. It states:

> *"Any attachment, except for bearing sole plates, having a length in the direction of the tension stress greater than 4.0 in[ches] that is welded to a tension area of a component of an FCM shall be considered part of the tension component and shall be considered fracture critical."*

This condition is illustrated in figure 117. The attachment in the top half of the figure is an FC component because it is attached to an FCM, is longer than 4 inches in the direction that is parallel to the primary stress, and is in a tension zone. In the lower half of the figure, the attachment is not FC because, although it is over 4 inches long, it is not longer than 4 inches in the direction that is parallel to the direction of primary stress. However, in both images, the weld itself is FC because at least one of the elements joined by the weld is FC (see section 6.2.4). The rule regarding 4 inches in length is based on the fatigue performance of longitudinal attachments. The longer the attachment is, the more stress the attachment draws and the poorer the attachment performs in fatigue. Attachments that are 4 inches or longer introduce category E or E' fatigue conditions (depending upon the thickness of the attachment); shorter attachments are C or D (reference AASHTO BDS table 6.6.1.2.3-1).



Source: FHWA

Figure 117. Illustration. FC attachments over 4 inches long.

7.2.4. Design Detailing Practice

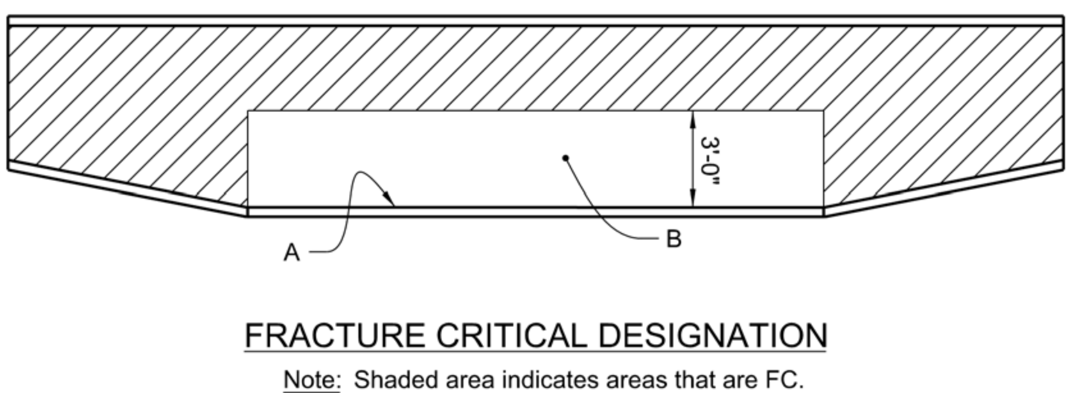
Fabricators need clear and correct information on design drawings about what components are to be fracture critical. Fabricators use this information in two stages:

1. **Ordering material** - FC information is used to order steel. On both FC and non-FC projects, steel is usually ordered before shop drawings are completed so that material will be available when fabrication is ready to begin. It usually takes two to three months to get main member material once it is ordered; these times can be even longer for HPS or during periods when demand is high.

2. **Shop drawings -** FC information is used to provide instructions on shop drawings for welding, inspection, and repairs.

Practices for identifying FC members vary among designating which members are FC, which parts of members are FC, which materials are FC, or some combination of these. Whichever approach is used, it is important that the FC limits are clear. Only members loaded in tension can be FC, but in bridges, it is common for members, and therefore the base metals and welds that the members comprise, to be only partially in tension. Therefore, FC members often comprise of base metals and welds that are only partly FC. It is important to clearly identify which parts of the members are actually FC and which parts are not so that base metals and welds can be handled properly.

1. **Sketches to indicate FC members –** Use of sketches is the clearest means of indicating what members are fracture critical. This is particularly so because fracture critical members are so often only partially FC (since they are only partially in tension). For example, see figure 118. The cross-hatching in this sketch indicate which parts of the web are FC. Because not all of the web is FC, the web-to-flange weld at “A” will not need to be FC in the white zone between the angle points, and any inspection, repairs, or welding to the white zone “B” will also not need to be FC. The web in the white zone does not need to be FC, but if the entire web is made from one plate, then FC material will be used for the entire web.

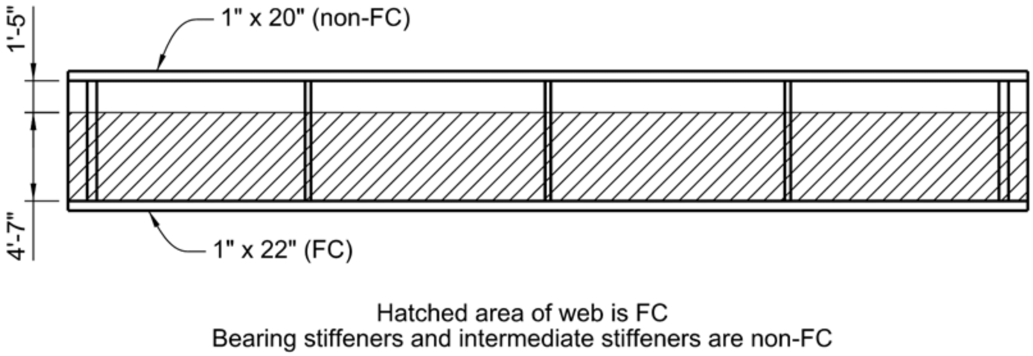


Source: FHWA

Figure 118. Illustration. Indicating FC zones on a partially FC member.

Designers often show a line along the middle of the web to indicate what is FC or non-FC (or tension or non-tension). If the location of the neutral axis is not in the middle of the girder, this should be properly indicated or explained otherwise.

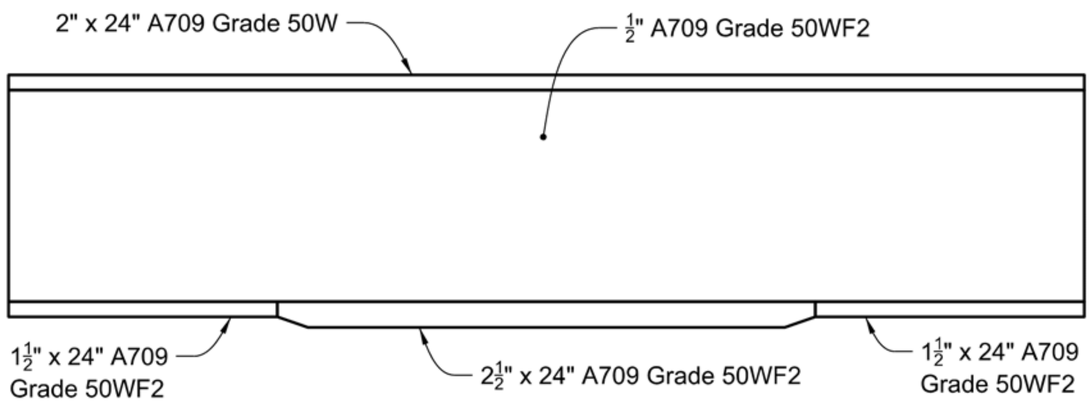
If sketches are used, care should be taken regarding the FC nature of non-FC components that might be in the sketch. For example, see figure 119. The intent of this figure is to indicate which flange parts and which part of the web are FC, but there are also stiffeners that do not need to be FC in the sketch. One possibility is to include a note indicating that the stiffeners are not FC. Perhaps the better choice is to use a simpler sketch that only shows the flanges and web and does not shows the stiffeners at all.



Source: FHWA

Figure 119. Illustration. Components in FC zones.

2. **FC component designations –** One means of FC identification is to designate which components are FC on the member elevation. For example, see figure 120. If this method is used but the components are not in tension in their entirety, then the limits of tension and FC must be designated by some other means.



Source: FHWA

Figure 120. Illustration. FC Designation by component callout.

3. **RFI clarifications -** If a component is shown which the fabricator feels should only be partially FC but is not shown as partially FC, the fabrication may question this with a Request for Information (RFI). For example, if a web is called out as FC but no zones are shown, the fabricator may issue an RFI to find out what part of the web should be FC.

4. **Tension components as FC –** Some designers use the approach of designating what components and zones are in tension and then, stating by note that members and components designated as tension are FC. As with direct designation as FC, this approach is suitable provided the components and zones are clearly identified. If the design is ambiguous, the fabricator will issue an RFI to ask which tension zones are fracture critical. Although this is a common practice, it is more straightforward to indicate the FC zone and save the RFI step and time, and therefore this practice is not recommended.

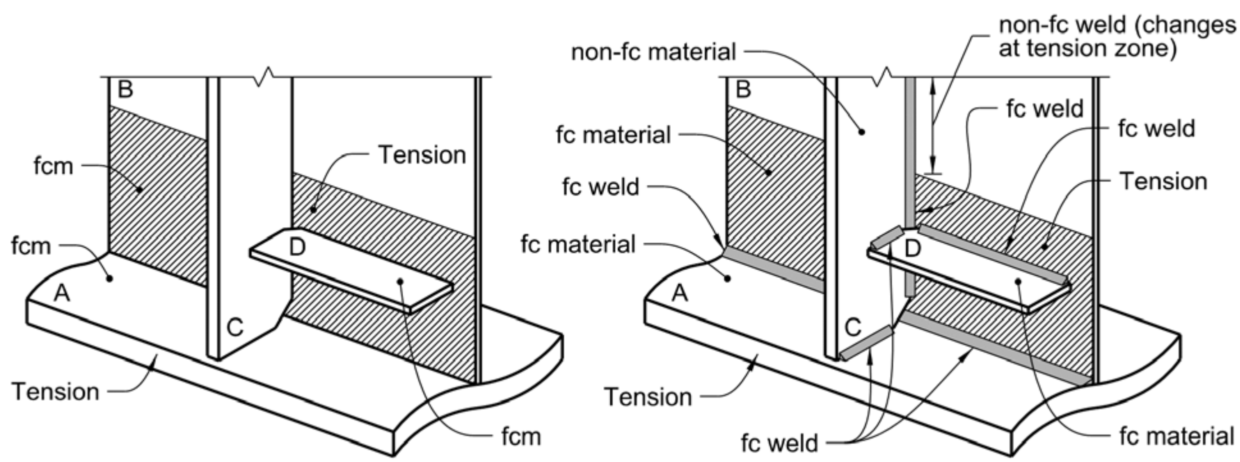
5. **FC designation on bill of material**– Another common approach for indicating what is FC is to designate which materials are FC in a bill of materials instead of drawing callouts. Generally, this is less effective than drawing callouts because this can result in ambiguities about how much of the material is in an FC zone. Using this approach, unless a given material is wholly in tension and is FC, some means is needed to convey the limits of FC.

6. **Attachments –** As described in section 7.2.3, the FC status of attachments is dependent upon their orientation, size and the component the attachment is welded to. Fabricators can only make assumptions about attachments because they do not necessarily know the extent of tension or the direction of tensile stress. Therefore, engineers should designate the FC status of attachments.

7. **Welds -** The design does not need to indicate which welds are to be FC. Fabricators and detailers can establish which welds are to be FC based on what components and materials are FC and D1.5 rules for how to weld FC materials. This will include:

    - All welds in FC zones
    - All welds of non-FC components to FC zones

    See figure 121 as an example. In this figure, the design detailing is shown on the left, and the implementation by the fabricator is shown on the right (note that this figure demonstrates the fabricator's action but is not necessarily a recommended shop drawing detail practice). Because the girder is nonredundant, specific components of the girder (but not the entire girder) are FC, and these are designated "FC", including the bottom flange, "A"; the bottom of the web, "B"; and attachment "D". Indicating the tension zones clarifies which portions of the girder are actually FC—for example, only the tension zone of the web is FC. Based on AASHTO BDS requirements regarding attachments, attachment "D" is fracture-critical, but attachment "C" is not.



Source: FHWA

Figure 121. Illustration. Elements identified as FC on plans (left) and as implemented on the shop drawings (right).

7.2.5. Material Designations

The following describes the best practices for specifying FC materials on plans:

Do:

- Specify the steel grade and that the component is FC (designate as described in section 7.2.4 of this manual), and provide the AASHTO temperature zone (AASHTO BDS section 6.6.2.1). For example:
    - "FC material to be ASTM A709 grade 50W, zone 2" or "ASTM A709 grade 50WF2"

Do not:

- Specify the toughness requirements, test temperature, or service temperature. For example:
    - Do not say "25 ft-lb at 40 °F"
    - Do not say "Service temperature 0 to −30 °F"

7.2.6. Shop Drawings

Based on the designation of members or components as FC in the design plans, and as supplemented by RFI answers if necessary, the fabricator specifically designates which materials and welds are to be FC on the shop drawings. Generally, fabricators will use both notes on drawings and designation in the bill of materials to indicate which components are FC. However, designating a plate as FCM in the bill of materials does not necessarily mean that the entire plate will be in an FC zone. A common way of indicating that welds are FC is to put "FCW" in the weld symbol tail. As discussed in section 7.2.4, many nonredundant members are only FC in certain areas (tension zones). In such cases, fabricators will provide specific information to the shop via the shop drawings about which components and welds are FC. Plates that are used in members that are only partially in tension (or FC) zones will be ordered from the mill as FC. Welds that extend through both tension and compression zones along their length will be likely be welded with the same FC WPS, but they will be treated differently for inspection and repair.

7.3. FRACTURE-CRITICAL MATERIAL REQUIREMENTS

The FC base metal requirements for AASHTO bridges are listed in ASTM A709 (ASTM, 2018a) and the AASHTO BDS. The differences between FC material and non-FC material include the following:

1. Charpy V-notch (CVN) toughness requirements are more stringent for most grades, with higher toughness expected for FC material than for non-FC material. For example, 2-inch thick non-FC grade 50 material in temperature zone 2 must satisfy 15 ft-lb at 40 °F, and FC grade 50 of the same thickness and in the same service zone must satisfy 25 ft-lb at 40 °F.

2. Weld repair of base metal by the material manufacturer or supplier is not permitted for FC material (ASTM A709 paragraph 6.8).

3. All FC material is tested at the "P" frequency, or per plate (ASTM A709 table 12 note "A"), whereas non-FC material (except HPS) is tested at the "H" frequency, or per heat (ASTM A709 table 11 notes "A" and "C").

A comparison between the non-FC and FC CVN tables in ASTM A709 is also instructive. There is nothing that makes an individual member of a nonredundant structure any more fracture-sensitive than an identical member in a redundant structure, if all conditions are identical. The higher CVN requirements for FC material in ASTM A709, typically shown as an increase in the required absorbed energy level, do not mean that there is an increase on the demand side of the equation, but rather reflect risk associated with nonredundant (fracture critical) structures; the consequences of fracture in a nonredundant structure are greater and thus an increase in the material resistance to fracture has been specified for these structures. A member under service stress is able to tolerate a larger crack before fracturing if made with higher-toughness FC material, and that larger crack size is easier to find during in-service inspection.

7.4. FRACTURE-CRITICAL WELD METAL

Weld metal CVN toughness requirements are listed in D1.5 table 12.1. The CVN toughness requirements in D1.5 table 12.1 are not filler metal classification values but are for welding procedure qualification test plates (see section 4.3).

The FCP does not include any requirements for HAZ CVN toughness testing. The assumption is that the base metal requirements for the limited number of steels that are permitted, within the heat input ranges used for typical bridge fabrication, will be such that the HAZ will have acceptable properties. Further, fatigue cracking that could create the initial flaw of concern would typically occur at weld toes; while the fatigue crack may initially be located within the HAZ, it would typically exit the HAZ and enter the unaffected base metal before it is of a critical size that would initiate brittle fracture. Thus, the base metal, not the HAZ, is generally of more concern in terms of fracture resistance (Miller, 2018).

7.5. SPECIAL WELDING REQUIREMENTS FOR FCMs

7.5.1. Welding Process Restrictions

Solid-wire GMAW (section 3.5) is not allowed for FCMs without approval of the engineer (clause 12.5.2); however, GMAW with metal-cored electrodes is allowed without restriction (clause 12.5.1). Generally, use of GMAW has not been allowed due to concerns about short-circuiting transfer mode (section 3.5.7), even though short-circuiting transfer is already prohibited by the Code. The unrestricted use of cored electrodes relates to the reclassification of these electrodes in the 1990s. At that time bridge fabricators were using metal-cored electrodes for FC welding without restriction because, at the time, the use of metal-cored electrodes was classified by AWS as an FCAW process. However, later in the 1990s, AWS changed the classification of these electrodes to GMAW. Originally, metal-cored wire was classified as FCAW because, like FCAW, it has a cored wire. Later GMAW was reclassified because metal-cored wire does not have flux in its core. Given the successful history of the use of metal-cored electrodes on FC materials as an FCAW product, the Code was modified to allow GMAW with metal-cored electrodes. GMAW using solid electrodes is still prohibited without engineer approval. Provided short-circuit transfer is avoided, there are no technical reasons that solid wire GMAW is unsuitable for FC welding. The decades-long successful history in the use of solid wire GMAW on non-FC members also support its suitability for FC welding.

ESW and EGW (section 3.7) are not allowed for use on FC members (clause 12.5.2). In the case of ESW, this is because when ESW was reintroduced in the Code (section 3.7.2), there were concerns about HAZ toughness. FC materials and associated toughness requirements were not the primary focus of the research that developed the new ESW process. EGW was not considered for inclusion as an FCM welding process due to a lack of bridge welding history with this process.

7.5.2. Extra Hydrogen Controls for Welding Consumables

As described in section 5.3 of this manual, hydrogen (in conjunction with other factors) can lead to cracking in welded connections if it is not controlled properly. Many of the consumable and welding practice requirements in the Bridge Welding Code relate to hydrogen controls. Although the hydrogen controls for non-FC welding are have been proven to be very effective, these controls are more stringent in the FCP and include the following:

- **Diffusible hydrogen consumable requirements –** Diffusible hydrogen levels can be determined from laboratory tests performed on deposited weld metal. The test is required to be performed on lots of consumables to be used for welding (clause 12.6.1.1), but the test is not required on production welds; it is impossible to test production welds for diffusible hydrogen. Results of such testing serve to establish an "H" level for the consumable. The Code provides specific requirements based on material and welding process. See section 5.3.2 for a discussion of H designators and their application for hydrogen control.
- **SMAW hydrogen controls -** Clause 12.6.4 describes a number of special hydrogen controls for SMAW electrodes used to weld FCMs. These include special requirements for packaging, storage, exposure limits, and drying. These precautions are specific to SMAW electrodes because SMAW coatings are susceptible to moisture pick-up from the atmosphere, and such moisture is a source of hydrogen. However, the SMAW electrode classifications that the Code allows on bridges for FC use are those with specific hydrogen designators H4, H8 or H16 (clause 12.6.1; see section 5.3.2 for a discussion about low-hydrogen electrodes).
- **SAW flux controls -** Like SMAW electrode coatings, SAW flux can pick up moisture and thereby introduce hydrogen into welds. The Code already has special requirements for flux packaging, handling, drying, reuse, and storage (section 3.3.6); these controls are tightened for FC welding, including moisture-resistant packaging, higher drying temperatures, storage at elevated temperatures, and requirements for delivery and recovery equipment (clause 12.6.5).
- **Wire electrode packaging, storage, handling, and exposur**e **-** There are further precautions in the FCP for SAW, GMAW, and FCAW electrode hydrogen control. These

include special requirements for packaging that is moisture-resistant, undamaged, and sealed; protection from damage during use; lubricant restrictions; and storage controls (various clauses in 12.6).

7.5.3. Welding Procedure Specification Qualification

WPS qualification for welding FCMs is the same as it is for non-FCM welding with these exceptions:

- **Prequalified SMAW electrodes** - Prequalification of WPSs using SMAW is limited to fewer electrode classifications (clause 12.7.1). The prequalified electrodes are lower- and moderate-strength electrodes; procedures using higher-strength electrodes are qualified by test as an extra measure of conservatism for higher-strength welds
- **Weld metal toughness** - The higher weld metal toughness values required for FC welds reflect the importance of toughness for weld resistance to fracture. Decades of history demonstrate that weld metal toughness for non-FC provides sufficient toughness for robust bridge weld performance; the tighter requirements for FC welds are an extra measure of conservatism and are commensurate with the differential increase in base metal toughness from non-FC to FC.
- **Period of effectiveness** - Unlike qualification tests for non-FCM welding, which are valid indefinitely, WPS qualification tests for FC welding are only effective for five years. After five years, qualification tests must be conducted again. As discussed in section 4.2, time limits of validity for WPS qualification tests have changed over time for both FC and non-FC welding.

Fabricators may choose to qualify WPSs for FC use and then use these same WPS qualification tests for non-FC welding. Because all the requirements for non-FC welding are incorporated into or are exceeded by the requirements for FC welding, this practice is implicitly allowed by the Code.

7.5.4. Certification

In addition to the AISC certification required by the Bridge Welding Code for all fabrication (clause 1.4), fabricators must also meet the FC supplemental requirements of the certification program to perform FC work (clause 12.8). Certification to these requirements indicates that the fabricator has knowledge and understanding of FC requirements.

Details about what constitutes the capability to perform FC work are found in chapter 4F of the AISC *Certification Standard for Steel Fabrication and Erection, and Manufacturing of Metal Components* (AISC, 2017). The Code explicitly states that the engineer may accept an alternative equivalent program. While the option of the use of an alternate program is permitted, in practice the AISC program is used.

7.5.5. Welder Qualification

Welder qualification requirements for FC welding are found in clause 12.8.2 of the Bridge Welding Code. Qualification for FC welding is similar to qualification for non-FC welding, and clause 12.8.2 incorporates clause 5, part B, of the Code. Differences for FC welder qualification include the following:

- Qualification for the welding process to be used must be:
    - by test within six months of the start of welding on the FC project; or
    - by annual FC requalification
- Initial qualification for FC groove welding must be by both RT test and bend tests
- Annual requalification for FC groove welding must be by RT. This RT can be by groove welding qualification test or by production RT; the production groove welds do not need to be FC groove welds

Because qualification for FC welding is similar to but more stringent than that for non-FC welding, qualification for FC welding also satisfies requirements for non-FC welding.

7.5.6. Tack and Temporary Welds

Special requirements apply to tack welds on FC materials. Tack welds can introduce local areas of high hardness and an associated concern for cracking. Therefore, it is better practice to tack within the welded joint, such that the tack weld is remelted and any local excessively hard metal is tempered by final welding. D1.5 table 12.2 mandates preheat for tack welds that will not be remelted; the minimum preheat is higher for smaller tack welds that might otherwise cool relatively quickly and result in excessive hardness. For non-FC joints, tack welds can indeed be applied outside of the joint but must be removed (clause 3.3.7.3); however, for FC welding, tacks must be inside the joint unless otherwise approved by the engineer (clause 12.13.1.1). While local zones of high hardness may be more brittle and less tough than unaffected material, such local hard zones are not actually known to be a cause of performance concerns in steel bridges. However, this measure of conservatism is prudent for FC members.

Temporary welds on FC materials are treated approximately the same as they are on non-FC materials: if they are not shown on approved drawings, they must be removed. However, the FCP is more prescriptive regarding removal of temporary welds on FC material than the Code is for non-FC materials: clause 12.13.3 requires removal of an additional ⅛ inch of base material such that the heat-affected zone of the weld is removed. This is a conservative step intended to ensure that no local hard zones will remain from the temporary weld and potentially cause performance problems. However, such removal would leave a considerable excavation in the base metal Therefore, in practice, fabricators avoid temporary welds, and their use on FC materials is unlikely.

7.5.7. Preheat and Interpass Temperatures

Required minimum preheat and interpass temperatures are higher for FC materials than for non-FC materials. Clause 12.14 of the Code refers to D1.5 preheat tables 12.3 through 12.7. These tables are based on the grade of the base metal, with minimum preheat and interpass temperature requirements generally increasing with base metal nominal strength. However, preheat temperatures are also based on the welding procedure heat input (minimum temperatures decrease as heat input increases, and vice versa) and on the diffusible hydrogen limit of the consumable (minimum temperature increases with H number). The commentary of the Code provides a thorough explanation of the rationale associated with preheat and interpass temperatures dependent on these two factors and the history of their adoption.

7.5.8. Postweld Thermal Treatments

Clause 12.15 of the FCP addresses two distinct types of heating treatments that may be applied after welding: hydrogen diffusion postheat and postweld heat treatment (PWHT). There are important distinctions between these two treatments, and they are sometimes confused with each other.

Hydrogen diffusion postheat is described in section 5.3.1 of this manual. Under this type of treatment, the welded steel is kept at an elevated temperature for a period of time immediately after welding is complete to provide extra time for hydrogen to diffuse out before the weld cools. By contrast, PWHT is a type of stress relief; this is described in section 5.7.2.4 of this manual. As described there, stress relieving is sometimes introduced to provide dimensional stability for machining after welding is complete. Use of hydrogen diffusion postheat is common, particularly for repairs, but PWHT is not common. Use of postheat is required for many FC groove weld repairs but typically not used for fillet weld repairs. In addition to when it is required by the Code, fabricators may apply hydrogen diffusion postheat when they observe it is necessary to address any extraordinary concerns about the presence of hydrogen or occurrences of hydrogen assisted cracking (HAC—see section 5.4.2.5). PWHT may only be used with approval of the engineer (clause 12.15.2).

7.5.9. Nondestructive Examination

Most NDE requirements for FC material are the same as those for non-FC materials with a few exceptions:

- **Examination of tension butt joints by both RT and UT** - Unlike non-FC tension butt joints which only require NDE by RT (clause 6.7), the Code requires both RT and UT of FC tension splices (clause 12.16.2.1). The use of the word "tension" in this section of the Code is redundant in that, by definition, only tension splices are FC in the first place. The addition of UT for butt joints is an added precaution given that RT and UT each have their own strengths as examination methods (see section 6.5). In particular, UT is stronger at discovering planar defects like cracks and incomplete fusion.
- NDE is more extensive for FC butt joints than for non-FC butt joints. Although for non-FC web butt joints, examination is not required for the full tension component of web. For FC web butt joints, the entire tension component of the web must be tested, just as all FC butt joints in tension (clause 12.16.2.1).
- **Cooling times** - The Code imposes delay periods before UT, MT and visual inspection as a precaution for detecting hydrogen cracking that could occur after the weld cools. The Code commentary discusses this; also see section 5.3 of this manual regarding hydrogen concerns in welds.

7.5.10. Repairs

The Code divides FC weld repairs into “noncritical” and “critical” repairs. As discussed in the commentary of the Code, noncritical repairs “usually entail limited difficulty: increasing weld size for undersized welds, removing minor edge gouges, excavations less than 65 percent of the weld size in depth, repairing undercut, and base metal surface repairs” (clause C-12.17.2).

Critical repairs “include fabrication errors such as mislocated holes, repair of deep laminar discontinuities, repair of cracks in base metal and weld metal, and repair of discontinuities that require gouging more than 65 percent of the weld depth” (clause C-12.17.2). The non-critical status of some repairs only applies the first time the repair is made (clause 12.17.2(4)). Included in this category are some repairs made to groove welds that have discontinuities that do not pass NDE: if the discontinuity is small enough to be “non-critical” but the repair is unsuccessful, the repair becomes critical. Critical repairs may involve welding under conditions that are more sensitive to cracking than production welding conditions, particularly with regard to restraint (see section 5.3 of this manual).

The Code has more specific requirements for FC repairs than for non-FC repairs, including requirements related to procedure submittal and the use of sketches or drawings to show the location and description of the defects to be repaired. Preheat requirements for FC repairs are higher than for FC production welding (clause 12.17.6(8)) in consideration of the restraint conditions often associated with repairs, Further, postheat is required for repairs that involve excavations deeper than ½ inch in groove welds (clauses 12.17.2(8) and 12.17.6(11)) (see 7.5.8). The Code has an option for preapproved procedures for noncritical repairs (clause 12.17.1.1). Putting such preapproved procedures in place facilitates progress of the work in the shop.

CHAPTER 8 - FABRICATION CONSIDERATIONS IN DESIGN OF WELDED STRUCTURES

This section presents recommended welding design and detailing practices for achieving a constructable welded steel bridge project.

8.1. WELD SPECIFICATION IN DESIGN

When specifying welds, the best and customary practice is for the engineer to provide only general weld requirements in design drawings and leave the details related to executing the weld to the fabricator. Fabricators will then determine the best details to make the welds in accordance with the Code based on the fabricator's equipment and welding preferences and communicate this information in the shop drawings.

For fillet welds, the engineer's responsibility is to specify the fillet weld size, and for groove welds, the engineer specifies whether the weld is a CJP or PJP, and if a PJP, the weld size required. Engineers usually do not need to call out the joint name, the joint type, or any other details related to preparation of the material to complete the weld. Leaving these details to the fabricator gives them the flexibility to use their best practices for achieving the weld, which in turn facilitates productivity and quality.

8.2. WELDING SYMBOLS

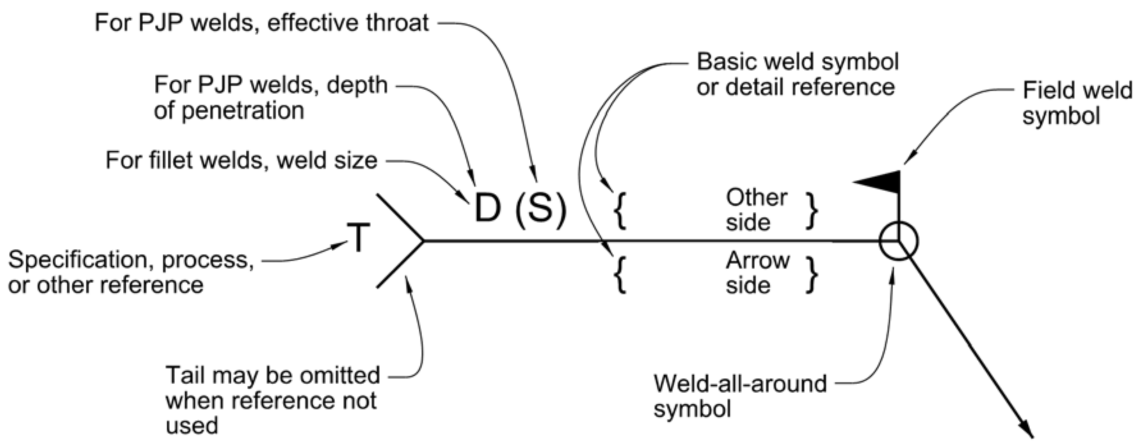
Welding symbols are used as a standard and systematic means of communicating welding instructions in a graphical manner. Welding symbols are miniature, schematic representations of the types of joint geometries and welds to be made. In steel bridge fabrication, welding symbols are placed on design drawings to specify the details of the desired welded joint. In turn, fabricators use welding symbols on the shop drawings to communicate welding information to welders. Further, welding symbols are sometimes written on the actual steel members so that shop and field welders know which joints are to receive the various weld types. Welding symbols are sometimes referred to as "weld callouts".

The ramifications of a welding misunderstanding can range from a small amount of rework, which is undesirable and costly, to rework that causes significant delays, results in claims, and costs in the tens or hundreds of thousands of dollars. The need for the welder to correctly make the weld as the designer intends is paramount, and therefore correct use of welding symbols is paramount. Welding symbols are intended to simply, clearly, and unequivocally communicate the intent of the designer to the welder. However, in practice, this is sometimes not the case.

Errors associated with welding symbols often arise from special conditions on projects that are unusual or not often encountered. Engineers and welders are more likely to correctly use and understand the typical symbols that are found on many bridge projects than the symbols only used occasionally for special situations. For example, most girder projects use the same welding symbol communicating web-to-flange fillet welds (with some variation in the fillet weld size), but projects with complex welded assemblies, perhaps of multiple plates or a mixture plates, angles, and round shapes, with a combination of CJP and fillet welds, provide unique welding solutions and unusual opportunities for communication errors. Engineers should review the weld callouts on shop drawings to confirm their intentions were properly communicated and interpreted. Generally, welding symbols do not need to be reviewed for correctness, but engineers should review higher level details about the welds, such as fillet weld size and use of CJP welds where the design indicates welds are to be CJP.

The requirements for the use of welding symbols are contained in AWS A2.4, *Standard Symbols for Welding, Brazing and Nondestructive Examination* (AWS, 2012b). This standard defines practices for the use of welding symbols, and the welding symbol information in other AWS standards, such as the Bridge Welding Code, which incorporate the requirements by reference. Table 8-2 of the AISC Steel Construction Manual (AISC, 2017a) is based on A2.4 and is also a good reference.

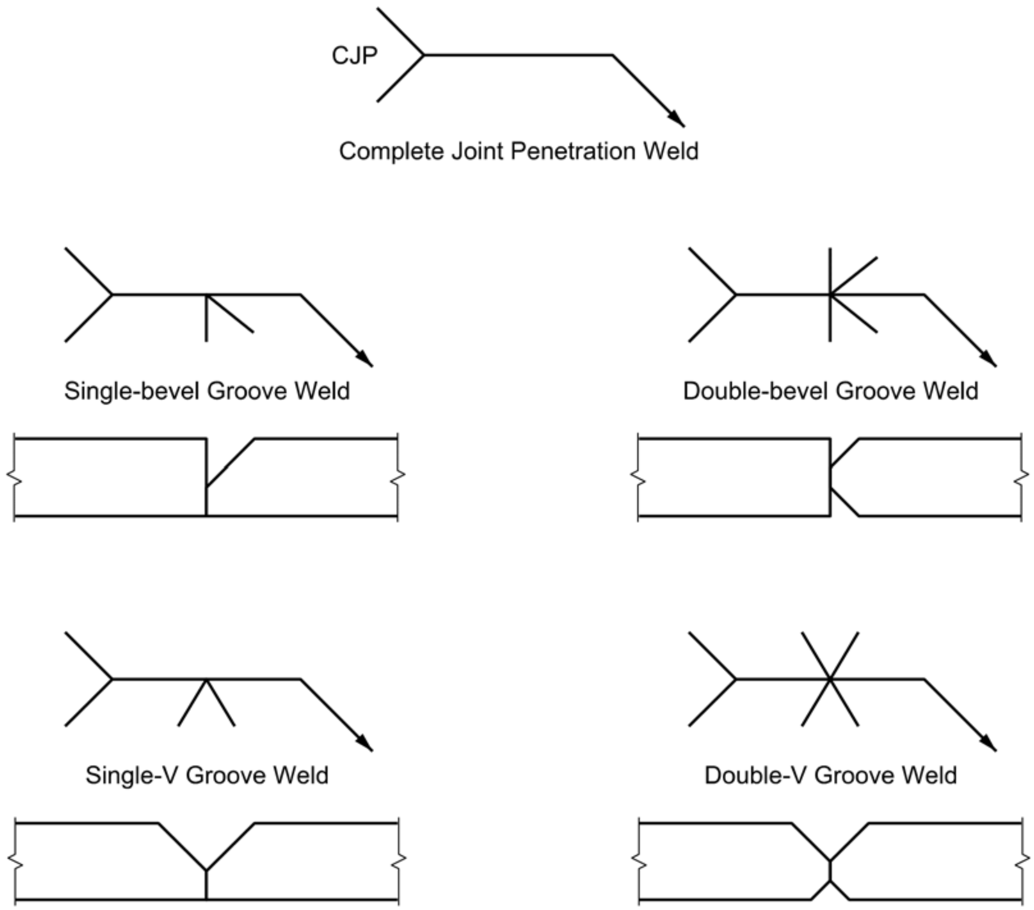
Welding symbols take the format shown in figure 122, which shows typical elements used for bridge welds.



Source: FHWA

Figure 122. Illustration. Basic welding symbol information for typical bridge welds.

Common groove welding symbols are shown in figure 123. The top symbol can be used on design drawing at splices to show that a CJP weld is required. Fabricators will then use the lower symbols to communicate with the shop whether to use a single-V or double-V joint; In design, use of the top symbol is better than use of the lower symbols because this gives the fabricator the flexibility to choose the type of joint that is best suited to the shop.



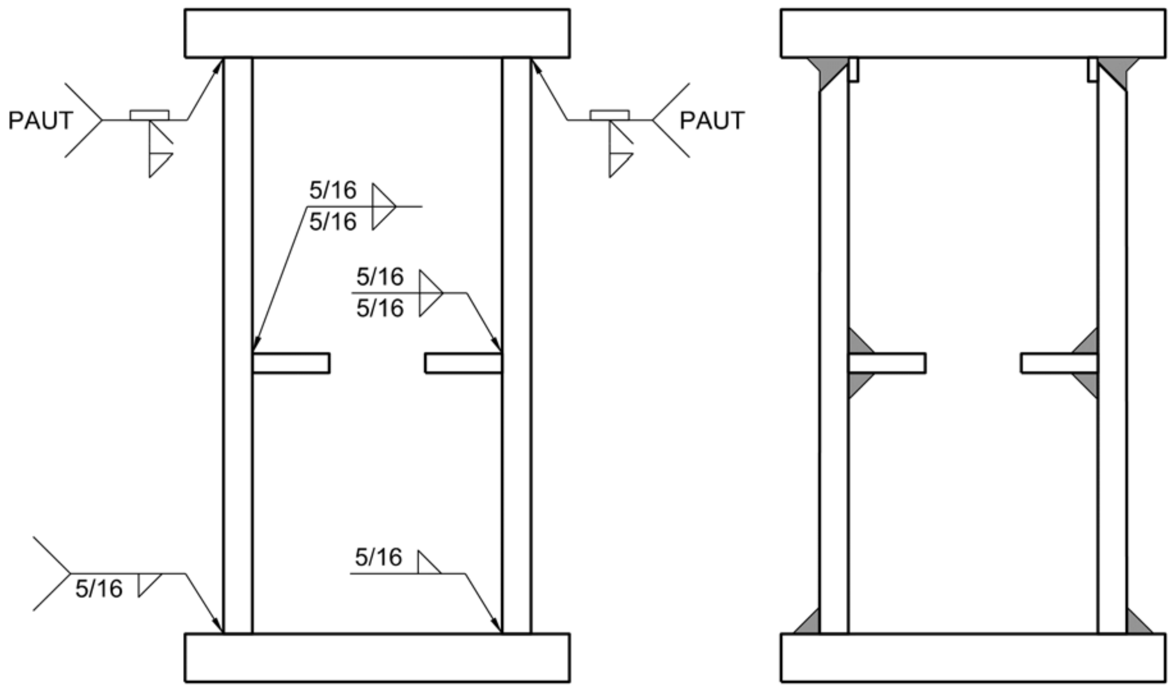
Source: FHWA

Figure 123. Illustration. Common groove weld symbols.

The following are key features of welding symbols:

- **Reference line and arrow -** The symbol includes, at a minimum, a reference line and an arrow. The arrow points to the joint. Optionally, there may be a tail at the end of the reference line opposite the arrow end.
- **Weld symbol** - The weld symbol indicates the type of weld (e.g., fillet or various types of groove welds). The correction orientation for the fillet weld symbol is with the vertical leg on the left side; see the examples in figure 124.
- **Reading left to right** - Welding symbols are always read left to right. This is true for welding symbols with arrows that leave the left or right side of the reference line. A common error is to create a welding symbol with the assumption that the direction of reading is from the arrow to the tail, but symbols are properly created to be read left to right, regardless of the arrow position.
- **Arrow side and other side of joint** -Weld symbols below the reference line refer to the “arrow side.” Weld symbols shown above the reference line indicate that the weld is to be applied to the “other side”, that is, to the side opposite of the one to which the arrow

points. The arrow may point up (as compared to the reference line) or down as needed to clearly point to the weld location. Regardless of the direction in which the arrow points, the significance of “arrow side” and “other side” remains unchanged. There is no specific meaning ascribed to the orientation of the arrow (see figure 122).



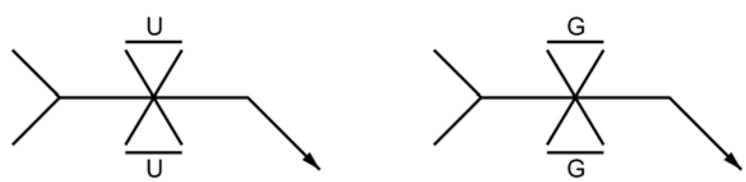
Source: FHWA

Figure 124. Illustration. Example welding symbols; as detailed (in design) at left and as desired at right.

- **Groove weld details:**
  - **CJP weld details-** When CJP groove welds are required, the contract drawings need only specify “CJP” in the tail of the welding symbol with nothing shown on the reference line (see symbol at top of figure 123). This leaves the fabricator or erector with the option of selecting the type of groove weld (bevel, V, U or J), as well as the specific groove dimensions. The top corners in figure 124 are an example of where a designer may want to include groove weld details in the design to convey that a single-sided groove weld with backing that can stay in place is desired. Depending upon loading conditions, Code provisions may require backing removal (section 8.6.1); a note in the design stating that the backing may remain in place would make requirements clear (see also “backing allowances” in Section 10.2).
  - **PJP weld details -** For PJP groove welds, contract document drawings need only specify the weld size, “S”, that is required. Shop drawings must show the weld groove depth “D” that is required to achieve the “S” dimension, based upon the included angle, process and position of welding (see figure 122). With respect to

PJP groove welds, a common error is including only one number without parentheses intending this to be the desired weld size, but causing confusion because, without the parentheses, the number indicates a preparation and not a weld size. The “S” and “D” nomenclature is a recent change in AWS A2.4 from “E” and “S”; other documents, including the AISC Steel Construction Manual, are still in the process of making this change.

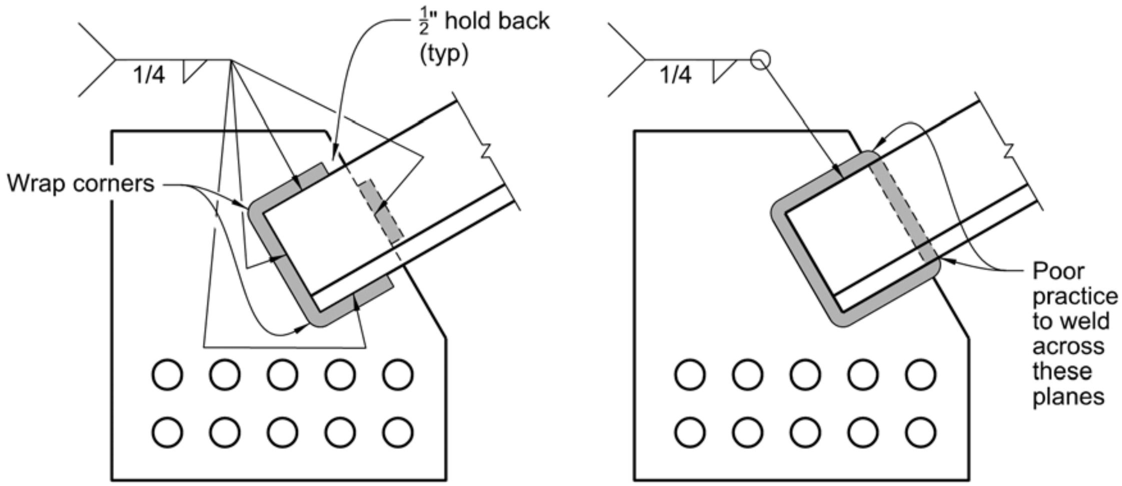
- **Finishing –** Special finishing requirements are indicated as additions to groove weld symbols. The most common example is grinding or otherwise finishing welds flush to remove reinforcing. Such finishing does not need to be called out for flange splices because the Code addresses these (clause 3.6.2.2). For special situations, flush is indicated by a line above the groove (see figure 125). A letter above the line indicates the method of making the weld flush. The best practice in design is not to specify a finishing method; this is done by using a “U” in the symbol, signifying that the finishing method is “unspecified”, shown to the left in figure 125. The most common method used in bridge fabrication for making a weld flush is grinding; grinding is a cost-effective choice and is indicated with a “G”, shown to the right in figure 125). The fabricator may choose to use this symbol to direct the shop to finish by grinding.



Source: FHWA

Figure 125. Illustration. Example symbols for finishing flush; unspecified at left and ground smooth at right.

- **Weld all-around symbol -** A commonly misused portion of a welding symbol is the weld-all-around symbol. The preferred alternative is to point to each joint that is to be welded. In many cases, the intent of the designer is *not* for a continuous weld all around the member, despite what the symbol indicates. Consider, for example, the crossframe gusset to angle connection as shown in figure 126. The desired configuration is shown at left with the correct symbol; if the weld-all-around symbol is used as shown at right, then the symbol conveys an undesired welding configuration.



Source: FHWA

Figure 126. Illustration. Correct weld symbol (left) and incorrect use of well-all-around symbol (right).

- **Tail Entry -** A variety of instruction can be conveyed by notes in the tail of the welding symbol. For example, in figure 123, the "CJP" communicates the need for a complete joint penetration weld. As another example, the "PAUT" in figure 124 communicates the need for the special process of phased array ultrasonic testing (see section 6.5). Fabricators may choose to provide NDE direction to the shop using welding symbol tail notes or by other means such as general notes or special callouts; on design drawings, however, such specification is typically not used for NDE that is already required by the Code. Many other special instructions may appear in the tail, such as special backgouging instructions, fracture-critical designations, or directions to other locations in the drawings where special instructions may be found. There is no limit to the amount of information that can be included in the tail. It is advisable to use the tail to convey unique requirements for a specific joint or weld. A few extra details included in the weld tail can prevent problems on the shop floor and in the fabricated girder.

Welding symbols offer a convenient shorthand for welding instruction communication, but their use is not mandatory for every situation. In fact, there may be situations for which no defined AWS symbol exists for which the standard symbols do not adequately describe the configuration, or for which the engineer is not sure how to communicate the weld requirements correctly with a weld symbol. In such cases, note and sketches used as supplements to welding symbols are recommended. A sketch that uses shading to communicate weld dimensions is particularly useful. Figures 124 and 126 are examples. However, such sketches should not be substituted where a weld symbol can readily be used effectively.

8.3. TACK WELDS

A tack weld is defined in AWS 3.0 as "a weld made to hold the parts of a weldment in proper alignment until the final welds are made" (AWS, 2010c). For example, see the tacked web-to-flange connection in figure 127. This definition does not require that tack welds be small or intermittent; tack welds may be large or continuous. Tack welds are generally not shown on shop drawings unless they are needed to provide special instructions to welders. Tack welds are not the same as temporary welds, which are separately discussed in section 8.4 of this manual.

Source: FHWA

Figure 127. Photo. Shallow girder with lower flange tacked.

Tack welds are usually placed in the weld joint. Tack welds in the weld joint may be totally remelted by subsequent welds. In other cases, the tack weld remains behind and becomes part of the final weld; these may be called “incorporated” tack welds. Practices and Code requirements differ between remelted and incorporated tack welds; these are discussion in sections 8.3.2 and 8.3.3. Tack welds may also be placed external to the joint; however, if so, the Code requires that such external tacks welds be made continuous or be removed, “in such a manner that the base metal is not nicked or undercut” (clause 3.7.1), unless otherwise approved by the engineer. The Code commentary explains that the terminations of tack welds outside of the joint may act as fracture initiation points (clause C-3.3.7.3).

Although for tack welds there are exemptions from some requirements that apply for permanent welds, for some tack welds, the requirements for tack welds by the Code must be followed to ensure quality. All tack welds, whether incorporated, remelted, or external to the joint, must be made by qualified welders and in accordance with an approved WPS.

8.3.1. Tack Weld Strength

A tack weld must be sufficiently strong to resist the loads that will be transmitted through the tack weld during handling, preheating, and welding of the component or structure. Some weldments have individual components that are massive, and the weight of such parts may be transferred through tack welds while the weldment is handled during fabrication. Tack welds are required to hold parts in alignment while assemblies are being preheated for final welding. Thermal expansion and its corresponding strains and resultant stresses may necessitate tack welds of significant strength.

The strength of tack welds, like other welds, is proportional to the throat size and the length. Thus, a tack weld may be made stronger by making it with a larger throat, longer length, or both. Generally, fitters and welders on the shop floor size tack welds in accordance with shop standards and practices.

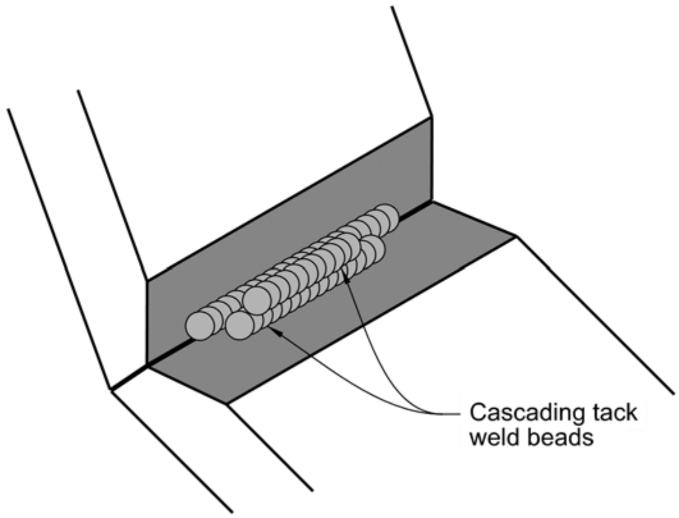
8.3.2. Remelted Tack Welds

A remelted tack weld is one in which all of the original tack weld metal is completely remelted by subsequent weld passes. Preheat is not required for tack welds that will be remelted by subsequent SAW welds (clause 3.3.7.1(1)), and qualification testing is also not required for the WPS used to make the tack weld (section 4.2.1). When the intent is to remelt the tack weld, the tack weld should be sized accordingly; the weld throat should be made small. When more tack weld strength is needed, fabricators increase tack weld length while keeping the weld size down to facilitate remelt. Keeping the tack weld size down also minimizes the tendency to disrupt the surface appearance of the final weld.

8.3.3. Incorporated Tack Welds

A tack weld that is not completely remelted is known as an "incorporated tack weld". Incorporated tack welds are treated like the root pass of a final weld, including WPS qualification and workmanship requirements, since they become part of the final weld. Tack welds must be cleaned before final welding (clause 3.3.7.2). Incorporated tack welds should be made with a size and heat input level that will ensure soundness and sufficient strength.

The configuration of tack welds that will be incorporated into final welds must be conducive to good fusion with the subsequent weld pass. Incomplete fusion at the start of the tack weld will become an incomplete fusion indication in the final weld. Further, fusion problems can occur where the final weld transitions from the weld root to the tack weld if the tack weld terminations (starts and stops) are abrupt. Welders may grind the tack terminations so there is a smooth transition to the untacked weld root to avoid potential fusion issues. Similarly, multipass tacks welds must have cascaded ends (clause 3.3.7.2) to provide a weld geometry that will facilitate fusion between the tack weld and the final weld. See figure 128 for an example of cascaded tack weld ends, in which tack weld beads are stair-stepped to help avoid abrupt tack weld terminations.



Source: FHWA

Figure 128. Illustration. Cascading tack weld ends.

Welding continuously over large, intermittent tack welds may disrupt the arc or affect the appearance of the subsequent final weld. In such cases, to facilitate workmanship, it may be necessary to weld the unwelded space between the tack welds before subsequent weld layers are made.

8.3.4. Cracked Tack Welds and Broken Tack Welds

When tack welds crack, they must be removed before final welding (section 5.4.2). The Code states, "Tack welds shall be subject to the same quality requirements as the final welds" (clause 3.3.7.1), and cracked welds are not allowed. Cracked tack welds cannot be addressed merely by welding over them, because the cracks may extend beyond the tack weld and into the base metal, and if such cracked tack welds are welded over, the crack may remain.

When making longer fillet welds, such as web-to-flange fillet welds, tack welds sometimes break ahead of the point of welding due to thermal expansion from welding and preheating. In such cases, if welding is progressing at the time the tack weld breaks, and the geometry of the part is still maintained, it may be preferable to continue welding over the broken tack weld (provided it is a tack weld that will remelt) to avoid a weld termination. Broken tack welds should be considered distinct from cracked tack welds; broken tack welds break all the way through the throat of the weld, with the break running parallel to the joint as shown in figure 129. Unlike the broken tack weld shown in this figure, a cracked tack weld would have a crack with an orientation that is less linear and with a crack tip that possibly extends into the base metal. Weld terminations (starts and stops) along the length of the weld are permitted, but welding over broken tack welds to avoid stops and restarts is better practice. The 2015 edition of the Code does not distinguish between broken and cracked tack welds, but new language about the practice of welding over broken tacks has been adopted by AASHTO and AWS, which will be published in the next edition of the Code.


Source: FHWA

Figure 129. Photo. A broken tack weld, distinct from a cracked tack weld.

8.3.5. Tack Welds Outside of Joints

When tack welds are placed outside the weld joint, the same criteria that would apply to a final weld are applied to tack welds. Tack welds used to attach steel backing to a joint (refer to section 8.6.1) are an example of the use of tack welds outside the weld joint. Such tack welds may be left in place if they are made continuous (clause 3.3.7.6); otherwise, they must be removed. When tack welds made outside the weld joint are removed, they are usually removed by grinding.

8.3.6. Welding Process and Filler Metals for Tack Welding

Tack welds may be made with any welding process that is capable of meeting the requirements of the Code. SMAW (section 3.2) is often used because of its flexibility. GMAW (section 3.4) is also popular for tack welding since it does not have a slag covering that requires removal before subsequent welding.

The filler metals used for tack welding must meet the same property requirements of final welds (clause 3.3.7.2). If FCAW-S is used for tack welding and another welding process for the final weld, or vice versa, the compatibility of the various processes should be investigated (section 3.6.6).

8.4. TEMPORARY WELDS

A temporary weld is defined in AWS 3.0 as "a weld made to attach a piece or pieces to a weldment for temporary use in handling, shipping, or working on the weldment." (AWS, 2010c). As the term "temporary" implies, these welds have a limited life. Temporary welds are used to facilitate fabrication as well as construction; for example, they are common for attaching erection aids like lifting lugs.

The Code addresses requirements for temporary welds in clause 3.3.8. Temporary welds are to be shown on shop drawings. Though they are not permanent, temporary welds are subject to the technique and quality requirements of the Code—they must be made in accordance with an approved WPS by qualified welders and satisfy profile requirements. NDE is not required for temporary welds.

The Code requires removal of temporary welds once their service is complete; removal will be in the field for erection aids. Generally, temporary welds are removed by grinding or with a combination of gouging and grinding. The Code does not prescribe removal methods, and removal procedures do not require approval. Once the welds are removed, the base metal must be finished flush. If the base metal is in a tension or stress-reversal zone, it must be checked with MT, and hardness testing of the HAZ is recommended by the Code (clause 3.3.7.4, referenced from 3.3.8). The Code commentary to clause 3.3.7.4 provides a good discussion of HAZ hardness testing. If removal damages the base metal, the base metal must be repaired in accordance with clause 3.7 of the Code.

8.5. SEAL WELDS

A seal weld is defined in AWS 3.0 as: “any weld designed primarily to provide a specific degree of tightness against leakage” (AWS, 2010c). The purpose of a seal weld may be to contain a fluid, either gaseous or liquid. In steel bridges, seal welds are used most often not to prevent leakage out of a container, but to prevent entry of a fluid into a space where some type of harm, often corrosion, is expected to occur.

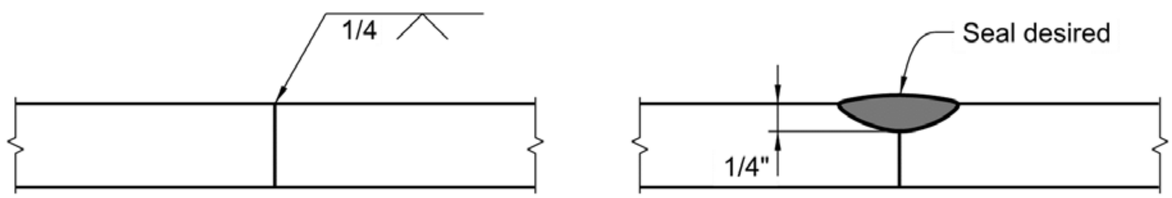
The Code does not mention seal welds. There are no special additional requirements nor special releases from requirements for seal welds. Hence, welds may be included in the design for sealing and may be designated as “seal welds”, but they must be produced with the same practice and quality requirements as other welds. Use of the term “seal” is sometimes misinterpreted to mean “not important” or “code provisions don’t apply” because the seal weld may not be intended for loads, and the Code does not mention “seal” welds. For this reason, when including welds just for sealing, it may be better not to label them as seal welds. See “Use Caution When Specifying ‘Seal Welds’” (Miller, 1999). Seal welds are occasionally specified to keep rust from bleeding out of small gaps and onto paint, as at the ends of stiffeners or connection plates.

Caution should be exercised when seal welds are specified. In some cases, the application of a seal weld may result in a conflict of specification or code requirements, or violate what is typically deemed good practice. For example, weld termination requirements may be violated by a requirement to apply seal welds (see section 8.8.2), and sealing cross frame cross member-to-gusset connections violates Code clause 2.8.8. Seal welds may perform structural functions that were unintended, resulting in undesirable load paths or unintended stress raisers: although a weld might be placed just for sealing purposes, this does not keep the weld from carrying load, and load paths through seal welds must be considered in design. Seal welds may affect inspection practices, particularly the interpretation of ultrasonic inspection results, if the seal weld introduces an alternate and unintended sound path.

8.5.1. Seal Weld Detailing

If a seal weld is to be a fillet weld, the size of the fillet weld must be specified so that the fabricator will have complete instructions. If no particular strength and associated effective throat is needed, the fillet weld can be sized to Code minimums (D1.5 table 2.1).

Situations where seal welds are desired but fillet welds will not work are not common but do occur. When they occur, clear details and instructions should be provided. For example, the condition shown on the left in figure 130 is not a fillet weld but a PJP weld and should be detailed on the design drawings as shown on the right (and satisfy minimum size requirements of D1.5 table 2.2). The WPS for this weld would need to be qualified as a groove weld (section 4.2.2 of this manual).



Source: FHWA

Figure 130. Illustration. Example special seal weld detail, as detailed at left and as desired at right.

8.5.2. Sealing for Hot-Dip Galvanizing

Seal welds may be needed on parts to be hot-dip galvanized to prohibit pickling acids or liquid zinc from entering into a specific region. However, it is difficult to completely seal joints by welding. Small fusion discontinuities, particularly at points where welds intersect, are common. Any discontinuity in a perimeter seal weld may allow cleaning solution to enter into the overlapped region, and these liquids can flash to steam and damage the parts being galvanized or cause localized uncoated surfaces. Further, sealing should not be used where it will impede the flow of degreasers, pickling solutions, rinse agents, flux, and molten zinc that are used at different stages of the galvanizing process. For this reason, the best practice for achieving prudent sealing for galvanizing is to consult with a galvanizer or the American Galvanizing Association and show this sealing in the plans.

8.6. WELD BACKING

Backing is defined as in AWS 3.0 as "[a] material or device placed against the back side of the joint adjacent to the joint root….to support and shield molten weld metal. The material may be partially fused or remain unfused during welding and may be either metal or nonmetal." (AWS, 2010c.) Examples of backing configurations are shown in figure 131. Backing is typically associated with CJP groove welds made from one side. Although usually made of steel, backing may be made of other materials such as ceramic or copper. While the proper terminology according to AWS A3.0 is simply "backing," various colloquial terms are used, including "weld backing," "backing bars," "backing strips," and "backup bars".

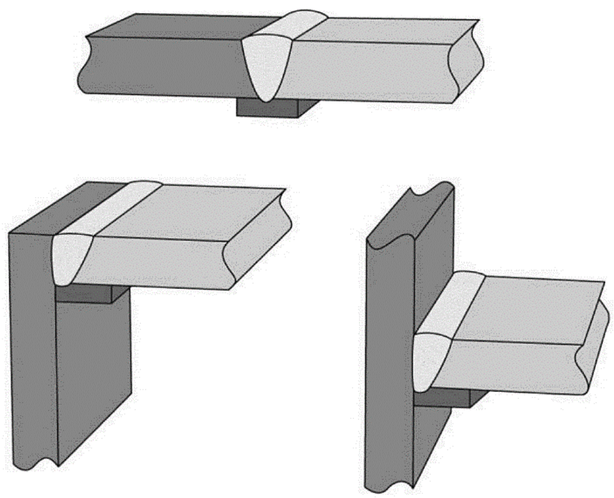


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Figure 131. Illustration. Examples of weld backing.

8.6.1. Steel Backing

As shown in D1.5 figure 2.4 of the Code, the standard joints (section 4.4) in the Code that are CJP groove welds welded from one side use steel backing. Steel backing is fusible backing, and it is intended that the weld will fuse with the backing material and become part of the welded connection. While backing might be casually viewed as merely part of the fabricator's means and methods, it is important to consider the potential influence of the backing on the performance of the connection. Left-in-place steel backing may, in some cases but not in all, introduce unintended load paths, or may create unanticipated and unacceptable stress raisers and even change the fatigue category of the joint. The Code allows steel backing that is parallel to the direction of stress or not subject to computed stress to remain in place. Because steel backing transverse to stress is a stress raiser, it must be removed; removal is by gouging and grinding smooth (clause 3.13.3). However, in situations where backing is transverse and the backing cannot be accessed after welding, the engineer may want to consider allowing the backing to be left in place, with due consideration to fatigue life of the joint.

Acceptable steels for weld backing are defined in clause 3.13.1 of the Code. To perform effectively, backing must be of sufficient thickness to support the weld puddle. The Code requires backing to be "of sufficient thickness to prevent melting through"; it does not have a minimum requirement, but it does provide recommended minimum thicknesses (clause 3.13.4). These thicknesses are dependent upon the welding process being used. In bridge fabrication, use of ⅜ inch backing is very common; this is generally an effective thickness for use with SAW. Further, when backing is ⅜ inch thick or less, it is exempt from CVN testing (clause 3.13.1(3)).

The fit-up of backing to the back side of the joint is held to a maximum of $^{1}/_{16}$ inch (clause 3.3.1.1). Close fit-up helps ensure soundness at the weld root. However, achieving a $^{1}/_{16}$-inch gap can be a challenge depending upon the flatness and stiffness of sections being joined. Poor fit-up of the backing to the back side of a joint is not itself a problem as long as an acceptable root pass can be made; extreme gaps between the backing and the steel being joined can make this task difficult. Perhaps the greatest problem with poorly fit backing comes when the weld is inspected with radiographic testing (section 4.2.2). Inevitably, slag will collect in the gap between the back side of the steel and the surface of the backing. While this slag is not in the weld itself and is of no consequence to the performance of the connection, the slag will be observed on the radiograph. Resolution of such inspection problems may involve removal of the backing (and the slag) and reinspection. Section 6.4.2 has further discussion about NDE of joints with backing.

Permanent steel backing is required to be continuous for the length of the joint (clause 3.13.2). Consider steel backing that is parallel to the stress field in a longitudinal member, such as a box section (figure 132-A. If segments of backing are used in a single joint, the weld bridges the unfused butt joint between the two backing bars (figure 132-B), effectively creating a notch at the root of the weld that can initiate fatigue crack growth. A continuous length of backing may be made by splicing shorter pieces; soundness of the spliced bar must be verified by either RT or UT (clause 3.12.2(2)). Further, on FC welds, backing parallel to primary stress is an FC attachment, and therefore subject to both RT and UT (section 7.2.3).

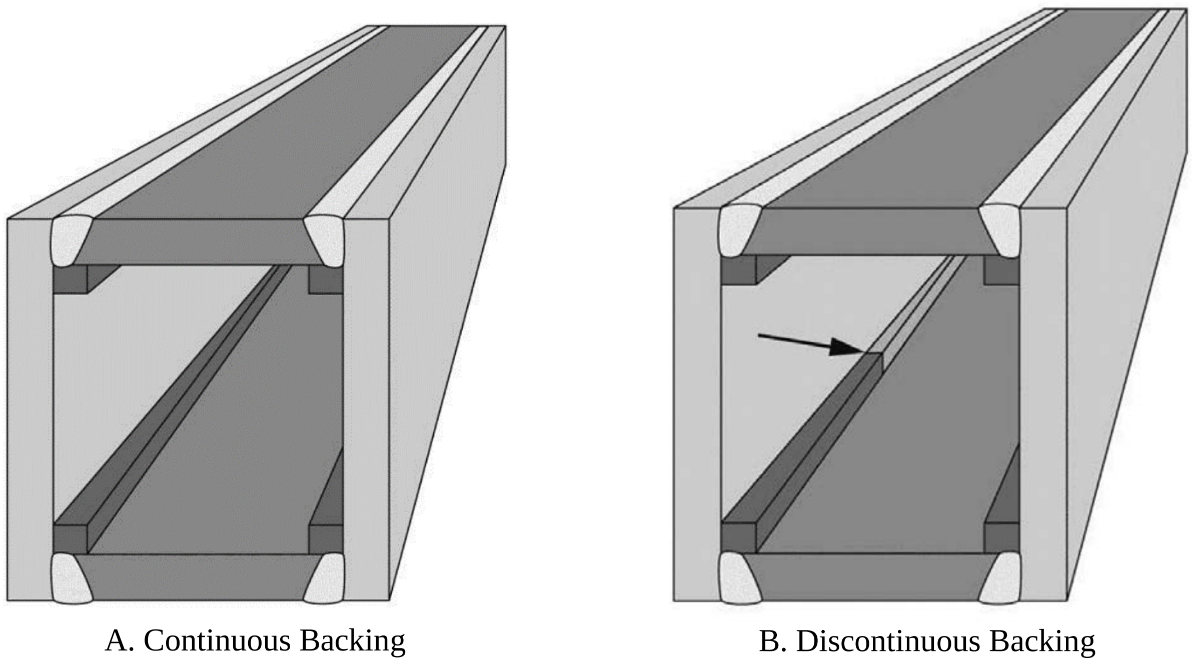
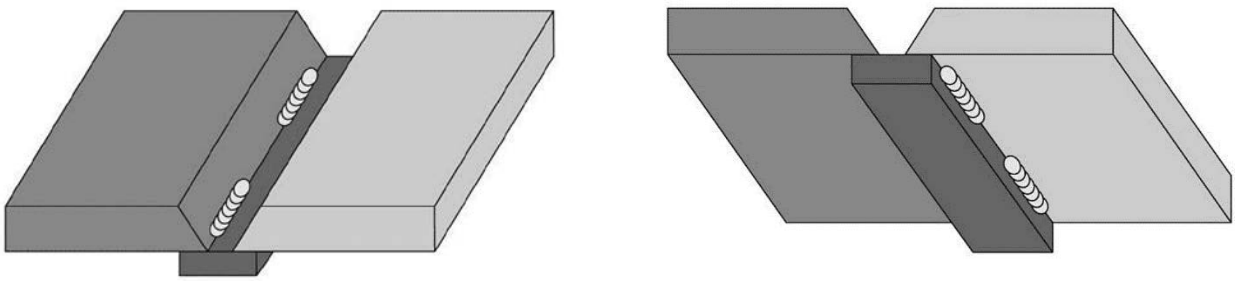


Figure 132. Illustration. Continuous and discontinuous backing.

It is typical for tack welds to be used to hold steel backing in place prior to the weld being deposited. As discussed in section 8.3 of this manual, tack welds are usually placed inside the joint as shown in figure 133-A. Backing can be tacked outside of the joint as shown in figure 133-B, but such tack welds must be removed or made continuous (clause 3.3.7.6).



A. Tack welds in the joint B. Tack welds outside the joint

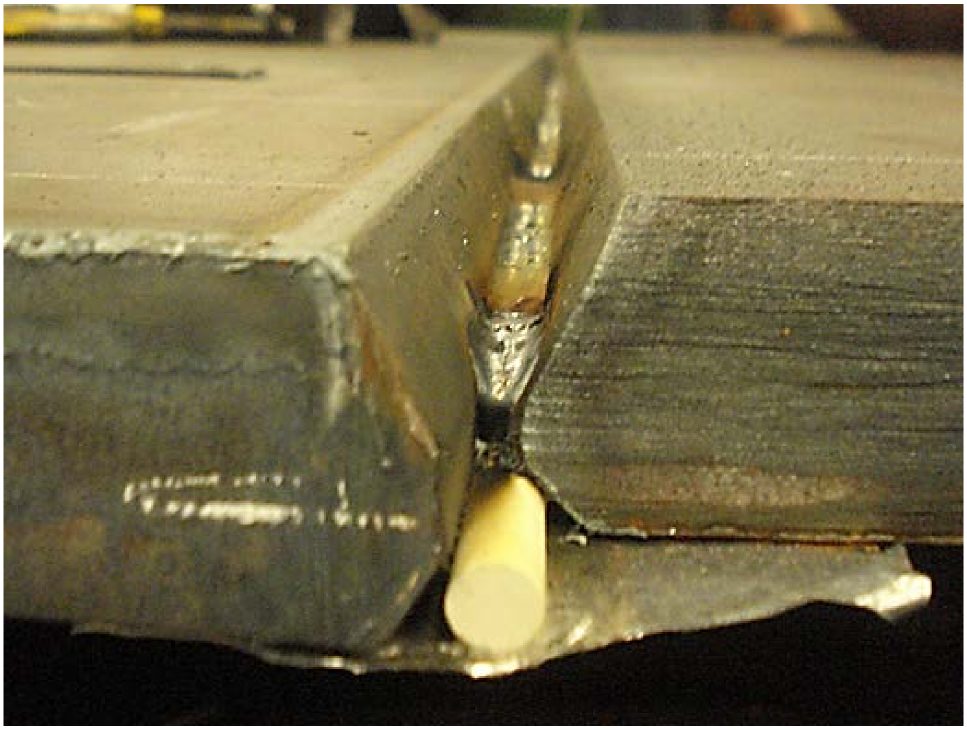
Figure 133. Illustration. Backing with tack welds inside versus outside the joint.

8.6.2. Ceramic Backing

Ceramic backing is a non-fusible backing that consists of a series of ceramic tiles or rods affixed to an adhesive tape that can be attached to the root side of a joint. The high melting point of the ceramic permits this material to contain the molten pool of weld metal during welding and then later be readily removed.

Examples of how bridge fabricators occasionally use ceramic backing in CJP groove welds include the following:

- **CJP groove welds welded from one side –** Ceramic backing is sometimes used to where making CJP groove welds from one side is desired but permanent backing is not allowed or is undesirable. In these situations, the ceramic backing provides a dam for the initially deposited weld metal.
- **At the root of CJP double-V groove welds –** Fabricators may choose to use ceramic backing in special situations where they feel it will help them achieve soundness. For example, in figure 134 ceramic backing is used to help prevent blow-through in the root pass of the joint shown. The Code standard joints (section 4.4) that feature backing are based on steel backing, and CJP joints welded from both sides require backgouging. Therefore, joints where ceramic backing is used instead of steel backing and joints where ceramic backing is used to avoid backgouging are nonstandard and must be qualified by test (section 4.4.4).



Source: FHWA

Figure 134. Photo. Ceramic backing in a CJP groove weld.

Regarding the use of ceramic backing for one-sided welding, caution is warranted because use of ceramic backing in this way introduces challenges:

- Ceramic backing cannot take the pressure of clamping, including the clamping that is normally used for fitting up members. Therefore, member connections must be fabricated to tighter fit-up tolerances than normal.
- Special welding techniques are needed to deposit weld metal onto the ceramic in two respects:
    - The weld puddle must be large enough to fuse to both sides of the weld. To build a large enough puddle, the arc is held in the front third of the puddle until enough molten metal builds up to fall forward and progress the weld pass. If proper care is not taken, an undersized bead can result in lack of fusion, and oversized beads are susceptible to width-to-depth ratio cracking.
    - Ceramic has low thermal conductivity properties. Therefore, with ceramic backing, heat flow is only into the surrounding steel, and there is virtually no heat transfer to the ceramic backing. As a result, when viewed from the joint cross-section, the weld grains grow nearly horizontally from each side of the groove, meeting in the middle. If the grains do not fuse across the midline, a crack could potentially result. By contrast, steel backing facilitates cooling in the additional direction towards the backing due to its high thermal conductivity. Therefore, ceramic backing may not give good results on otherwise identical welding procedures that are already known to work well with steel backing. This is why

procedure and joint qualification testing is required. Both qualification testing and NDE of production welds will demonstrate soundness of welds made with ceramic backing.

- Some dressing of the back side of the weld will be needed once the ceramic is removed to achieve reinforcing with a satisfactory profile. Access to the back side of the weld is needed for this work. If there will be no access to the back side of the weld, it may be prudent to allow steel backing that remains in place or use a one-sided joint which requires special qualification of the joint detail and the welder.

8.6.3. Copper Backing

The most common use of copper backing in bridge welding is the use of copper shoes for ESW; these are required by the Code to be water-cooled against the heat of the large volume of molten metal in an ESW weld; otherwise, it is rarely if ever used. When copper backing is properly used, the steel weld does not fuse to the copper backing. When the weld is complete, the copper backing is removed and can be reused. Typically, copper backing is held in place with mechanical clamps and brackets. However, unlike steel backing that can be used to help facilitate fit-up as it is clamped and tacked in place, copper is less stiff than steel and generally requires closer fit-up of the parts to be joined.

Copper has a lower melting point than steel but a much higher rate of thermal conductivity (see discussion of copper shoes for ESW in section 3.7.2). When the molten steel weld puddle comes into contact with the copper, the thermal energy is conducted away, raising the temperature of the copper but not melting it. This is a delicate balance, and it is easy to inadvertently melt the copper, causing several problems. If melted, copper is introduced into the weld metal and may cause cracking in the weld. Further, melted copper backing will no longer be easy to remove, and the backing may be so damaged as to prevent its reuse. For these reasons, the Code prohibits the use of copper backing “when there is any possibility that the welding arc may strike the copper” (clause 3.13.6).

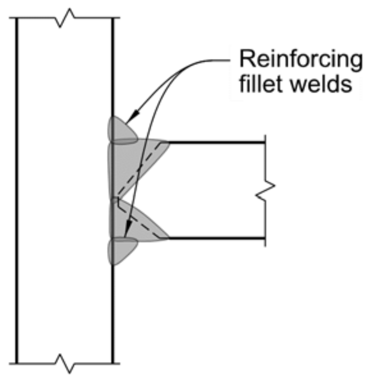
Another concern associated with copper backing is that the higher thermal conductivity rate associated with copper compared to steel may result in root pass cooling rates that cause unacceptable mechanical properties or cracking. The Code allows copper backing, but requires that its use be qualified by testing as a non-standard joint (clause 3.13.6).

8.6.4. Other Backing

In addition to copper, the Code refers to other materials that may be used for backing, including, “...flux, glass tape, iron powder, or similar materials...” (clause 3.13.6). As for copper and ceramic, their use requires qualification of the joint to be used as a non-standard joint. However, the Code provides a specific exception to qualification for the use of flux “that fills gaps not exceeding $^{3}/_{16}$ inch between adjacent parts” (clause 3.13.6). In this practice, flux is placed into such gaps before welding, helping protect the arc from exposure at the root. This flux placement amounts to the same enveloping of the arc by the flux that occurs during welding, and therefore no special joint qualification testing is necessary.

8.7. REINFORCING FILLET WELDS

A reinforcing fillet weld (as shown in figure 135; also figures 3-B, 44, and 124) is a fillet weld added to PJP or CJP weld T-joints or to a PJP or CJP weld inside corner joint (unless there is backing at these locations) to provide a contoured transition and avoid what would otherwise be an abrupt corner. Reinforcing fillet welds are required by the Code (clause 2.11.3) and their size is prescribed, so engineers do not need to specify them. "Reinforcing" is a misnomer—the weld is not intended to reinforce the connection in the sense of strengthening it. However, as part of the weld, the reinforcing fillet will indeed draw load. Like other welds, reinforcing fillet welds are to be made in accordance with an approved WPS. The fabricator may choose to use the same WPS for the reinforcing fillet weld that is used for the original CJP or PJP weld (this is sometimes referred to as making the reinforcing fillet integral with the groove weld), or the fabricator might choose to use a different process and approved WPS. Either approach is permitted by the Code.



Source: FHWA

Figure 135. Illustration. Reinforcing fillet welds on a CJP weld T-joint.

8.8. WELD TERMINATIONS

Where welds terminate (start or stop), care must be taken to satisfy the quality requirements of the Code. The Code states, "[w]elds shall be terminated at the end of a joint in a manner that will ensure sound welds. Whenever possible, this shall be done by use of weld tabs (extension bars and run-off plates) placed in a manner that will duplicate the joint detail being welded" (clause 3.12.1). Termination practices vary depending upon the type of weld (fillet weld or groove weld) and how local conditions provide opportunities to use weld tabs.

8.8.1. Weld Tabs

Weld tabs (see figure 136) are often used to facilitate good workmanship in weld terminations. AWS 3.0 defines a weld tab as "[a]dditional material extending beyond either end of the joint on which the weld is initiated or terminated" (AWS, 2010c). If the weld is started on the tab, it is called a starting weld tab; if a weld ends on the tab, it is called a runoff weld tab. These devices are colloquially called "runoff tabs", regardless of whether they are used for starting or ending the weld. In bridge welding, weld tabs are most often associated with CJP groove welds (figure

136) and web-to-flange fillet welds (figure 137). Weld tabs are on a CJP weld T-joint are illustrated in figure 138.

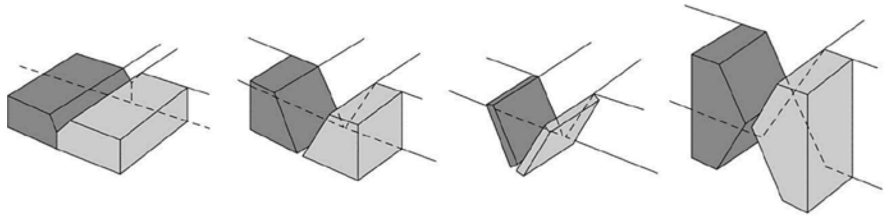


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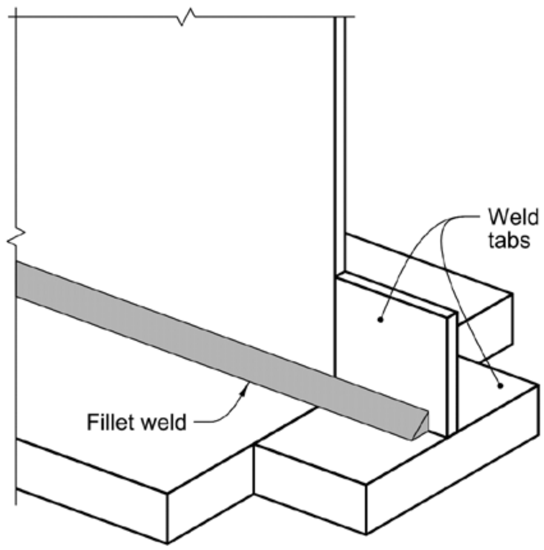
Figure 136. Illustration. Examples of weld tabs.



Source: FHWA

Figure 137. Illustration. Girder web-to-flange weld tab.

Source: FHWA

Figure 138. Photo. Weld tabs on a CJP weld T-joint.

A minimum length for weld tabs is not prescribed in the Code. A common rule of thumb is that weld tabs should be at least as long as the thickness (throat) of the groove weld, but this is neither required nor practical in some situations. In some cases, the weld tab length is dependent upon the equipment used by the fabricator to produce the weld because the tab is used to support the equipment. Acceptable steels for weld tabs are defined in clause 3.12.2 of the Code. Weld tabs must be removed after welding is complete, and the surface from which the tabs are removed must be "made smooth and flush with the edges of the abutting parts".

When groove welds are used in confined spaces where there is no room for weld tabs, welders will use a technique of terminating weld passes beyond the joint to help ensure soundness, as shown in the access hole in figure 139. In this picture, inside of the access holes has been ground to a smooth transition after welding. However, it is preferable to avoid this situation if possible. For example, at bearing stiffener terminations, use of a mill-to-bear (or finish-to-bear) condition and fillet welds is superior to use of CJP weld T-joints (see section 9.3.2.1).

Source: FHWA

Figure 139. Photo. Beads in a CJP weld T-joint terminated in access hole.

8.8.2. Fillet Weld Terminations

The general topic of fillet weld termination deals with the issue of the length of the fillet weld compared to the length of the joint. Three options exist: the fillet weld may stop short of the end of the part, may extend to the end of the part, or may wrap around the end of the part. All three options are illustrated in figure 140.

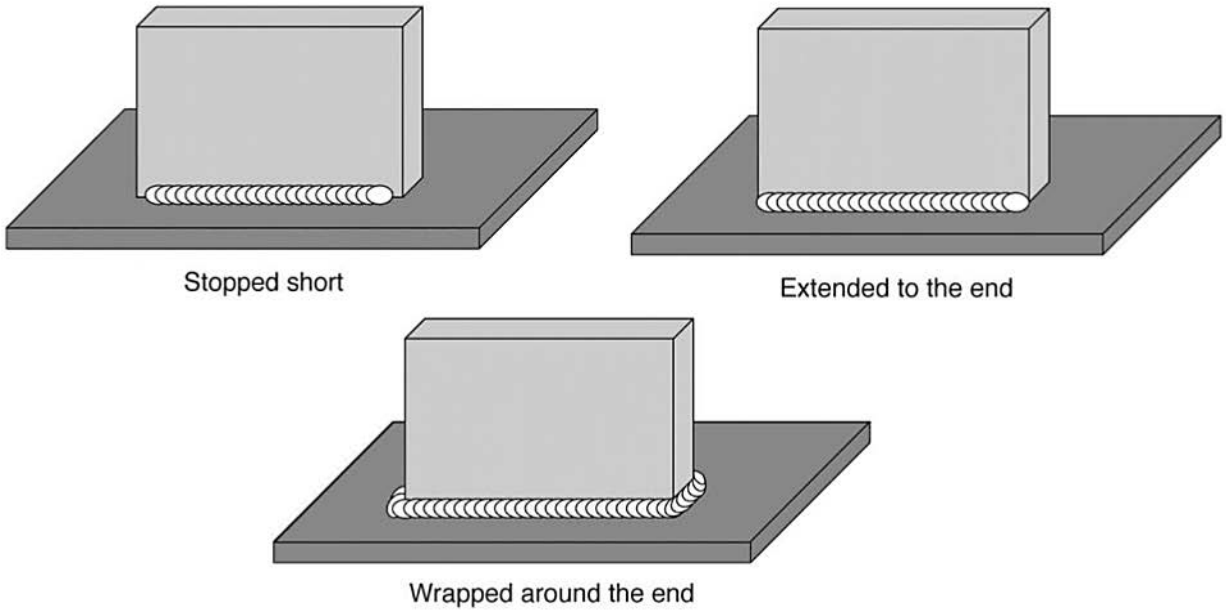


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Figure 140. Illustration. Fillet weld termination options.

While it may seem desirable for fillet welds to extend to the ends of the parts or wrap around the ends of parts, these approaches overlook the reality of production welding. It is difficult for welders to initiate an arc at the exact end of a part, and it is difficult to then make a full-sized weld all the way to the end of the part, complete with a filled crater. The issue is not one of skill but rather of thermal and electrical considerations. At locations along the weld other than at the ends of the part, the thermal energy introduced into the weld metal is conducted away in three dimensions. As the end of the part is reached, there is a buildup of thermal energy. During crater filling, the extra-hot base metal may lead to unacceptable undercutting and other weld discontinuities at the end. Figure 141 provides an example of a fillet weld with good holdback practice.

Source: FHWA

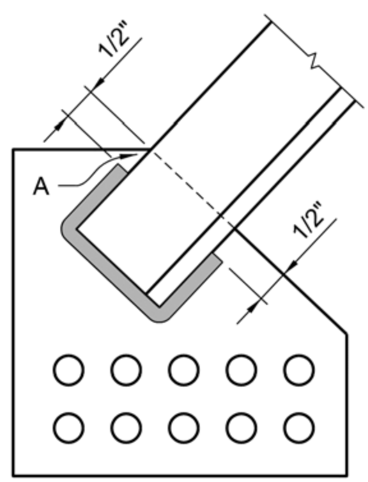
Figure 141. Photo. Fillet weld with holdbacks.

When welding with direct current (as is typical), the electrical current creates a magnetic field that surrounds the arc. Away from the ends of the joint, the magnetic field is uniformly distributed in the steel around the arc. When the end of the part is approached, the magnetic field becomes concentrated and may deflect the arc and make it difficult or even impossible to deposit a quality weld; this condition is known as "arc blow".

For these two reasons, unless there is a specific design motivation to do otherwise, it is generally preferable to start and terminate fillet welds a short distance before the end of the joint. The AASHTO BDS does not address weld terminations, but AISC 360 states in section J2.2b(g), "Fillet weld terminations shall be detailed in a manner that does not result in a notch in the base metal subject to applied tension loads" (AISC, 2016). A user note then provides this guidance: "Fillet weld terminations should be detailed in a manner that does not result in a notch in the base metal transverse to applied tension loads that can occur as a result of normal fabrication. An accepted practice to avoid notches in base metal is to stop fillet welds short of the edge of the base metal by a length approximately equal to the size of the weld. In most welds the effect of stopping short can be neglected in strength calculations." However, the designer may need to consider the effect on shorter welds, such as those between crossframe connection plates and girder flanges, where owner-specified holdbacks or the values suggested below reduce the total length of the weld by a significant percentage.

The Bridge Welding Code does not address fillet weld terminations specifically. In practice, many owners have adopted standard holdback dimensions for fillet weld terminations. A reasonable holdback for longer fillet welds is ½ inch ± ¼ inch, and ¼ inch ± ⅛ inch for shorter welds such as those joining stiffeners to flanges. These holdbacks and tolerances allow the welder to target a distance where it is not likely that the fillet will actually reach the edge of the plate and cause an undercut. The need for a fillet weld to reach the edge of the plate is uncommon. If this is the case, then special attention should be drawn to this condition in the design plans. This condition should only be specified if there is room for a weld tab that can be removed.

The Code requires that "[f]illet welds deposited on opposite sides of a common plane of contact between two parts shall be interrupted at a corner common to both welds" (clause 2.8.8). This situation is frequently encountered on crossframes where rolled angle shapes are joined to gusset plates. Figure 142 shows a ½-inch holdback. This distance is prudent given the variety of angles that can be encountered between crossframe members and gussets. Given the acute angle at point "A" in figure 142 between the rolled angle shape and the edge of the gusset plate, the ½-inch holdback provides enough clearance between the end of the fillet weld and the edge of the gusset plate, preventing interference with the end of the fillet weld and the edge of the gusset plate.



Source: FHWA

Figure 142. Illustration. Crossframe holdbacks.

8.9. INTERSECTING WELDS

Unhelpful rules of thumb have been developed and promulgated over the years based on concerns associated with "intersecting welds", such as "welds should not cross each other" and "welds should not intersect each other". Such statements can be misleading. This section provides clarifications about intersecting weld conditions that actually are or are not of concern.

There is no definition of "intersecting welds" in AWS A3.0. In typical usage, the term refers to two or more welds with different axial orientations meeting at a common point. Sometimes, the intersection point involves the end of one or more welds; in other cases, one weld may cross another. Sometimes, "intersecting welds" are described in terms of "touching welds." Individual weld passes in groove welds are not considered "intersecting welds".

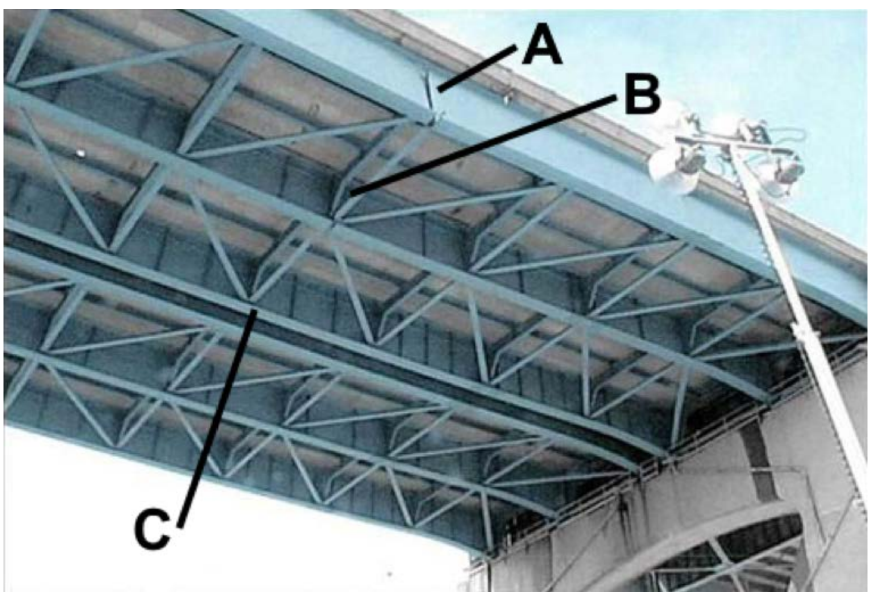
The topic "intersecting welds" is used in practice to address several specific topics. Some of these topics represent significant concerns while others are innocuous. The broad use of the term has caused some confusion about what concerns are important and what perceived concerns are non-issues. The intersecting welds topic can be categorized in two ways:

- **Design Concerns –** There are special design and loading configurations that should be avoided to help preclude constraint-induced fracture.
- **Fabrication –** In fabrication, some intersections should be avoided to ensure weld quality, but other intersections are of no concern.

8.9.1. Design Concerns

8.9.1.1. Hoan Bridge Fractures

Concerns about triaxially intersecting welded elements in service were brought to light with the Daniel Hoan Memorial Bridge (Hoan Bridge) fractures on December 13, 2000. The Hoan Bridge carries I-794 over the Milwaukee River in Milwaukee, Wisconsin. The events and mechanisms associated with the fractures are thoroughly discussed in the July 10, 2001, FHWA memo about the bridge, with the subject, "ACTION Hoan Bridge Investigation" (FHWA, 2001). As seen in figure 143, the girders at locations "A" and "B" fractured full-depth, and at location "C", the web had three feet of fracture.

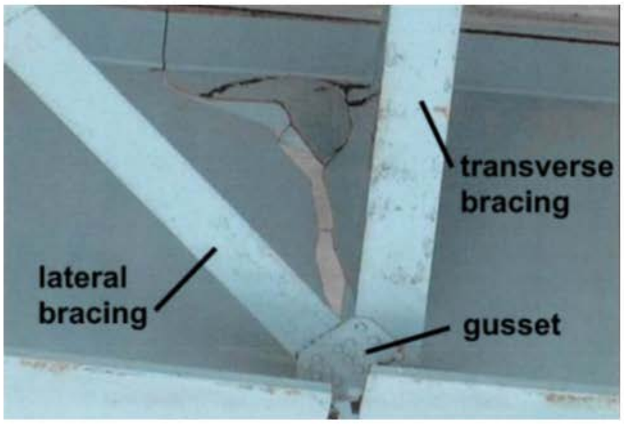


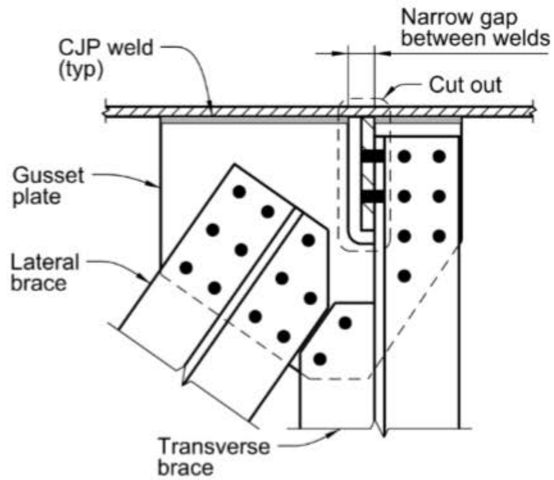
Source: FHWA

Figure 143. Photo. The Hoan Bridge fracture locations.

The Hoan Bridge fractured at a complex joint on the bridge. At the nodes where the fractures occurred, lateral and transverse bracing were joined to girders. The lateral bracing was bolted to gusset plates, which were welded using CJP welds to the girder webs. The gusset plates had cutouts so that they could clear the transverse stiffeners. Thus, there was a small gap between the two welds that joined each gusset to the girder webs. See figure 144 for the detail at the exterior girders. At the interior girder, the detail was similar, except that the gusset plate was larger and had lateral bracing connecting to it from both sides.

Since the gaps between the various elements were so small, the girder web was subject to a high degree of triaxial constraint. The gaps were small enough for the welds to touch in some cases, leading to misconceptions about “intersecting welds”.





Source: FHWA

Figure 144. Photo and Illustration. Hoan Bridge bracing node connection detail, exterior girder.

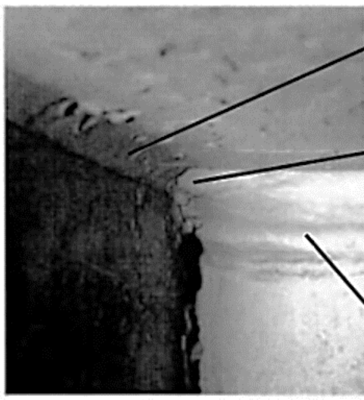
8.9.1.2. Hoan Memo Weld Cautions

The FHWA memo (FHWA, 2001) about the fracture provided this instruction:

> *“Structures that are known to have narrow web gaps in tension zones should be inspected closely with a hands-on visual inspection…. If intersecting or touching welds are identified or suspected, steps should be taken to further evaluate the connection and consider possible retrofit options.”*

Thus, the memo described a very specific condition that was to be inspected, one where triaxial constraint conditions were created by narrow web gaps (distance between the toes of the welds connecting the constraining elements to the constrained element). However, in practice, designers and inspectors have not fully understood the conditions that resulted in critical constraint concerns, and instead have mistakenly believed that intersecting welds were, in and of themselves, the source of the problems.

In the case of the Hoan Bridge, because of normal fabrication tolerances the actual size of the narrow gap between the two CJP welds varied, including some locations where welds touched or intersected (see figure 145). Normal shop tolerances had an effect on the actual gap between the CJP welds and other welds in the connection. The connection may also have been affected by tolerances for the location of the transverse stiffeners on the web; the location of the gusset plates about the stiffeners; the actual as-cut dimensions of the gusset plate; and the actual size of the welds. Where the web gap was too small, especially in locations where the gusset plate welds touched the stiffener welds, the web was constrained, introducing a triaxial restraint condition. Analysis showed that under this triaxial constraint, the web was unable to yield. As a result, the web was able to develop tensile stresses significantly higher than the yield stress of the steel without exhibiting any plastic deformation (i.e., without yielding). In fact, at the location where the fracture initiated, the web achieved a tensile stress greater than the ultimate strength of the steel without exhibiting any yielding. The resulting fracture was then sudden and brittle in nature.



Source: FHWA

Figure 145. Photo. Example of unintended weld intersection on the Hoan Bridge.

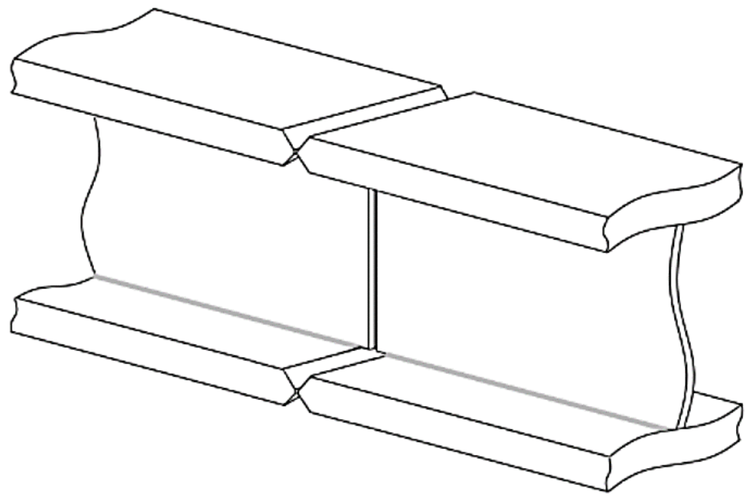
Addressing steel bridge design and detailing concepts to avoid constraint-induced fracture is beyond the scope of this manual. However, the Hoan Bridge example is included here to clarify historical misconceptions about intersecting weld concerns. Designers are cautioned to avoid details which may result in high degrees of triaxial constraint and elevated risk of constraint-induced fracture

8.9.2. Intersecting Welds in Fabrication

8.9.2.1. Girder Fabrication and Splicing

The building of plate girders provides examples of very common intersecting welds that are not a concern. Plate girders are typically built as follows: web and flange welded butt splices are made before the web and flanges are assembled into the girder configuration. Then, flanges are assembled to webs with longitudinal web-to-flange welds. The intersection of the longitudinal web-to-flange weld with the welded splices in the web and flanges constitutes a weld intersection. There is nothing wrong with these "intersecting welds," which do not create triaxial constraint, and there is no need for weld access holes at these intersections.

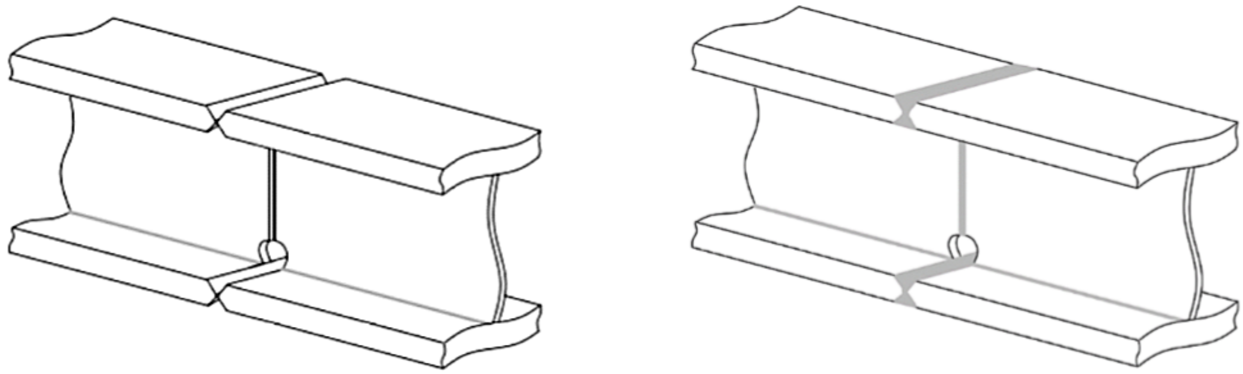
There are situations where the adage to "avoid intersecting welds" has some legitimate application. Figure 146 shows the planned splicing of two complete girders or rolled beams, where flanges and webs are to be joined to each other by CJP welds, such as for field splicing (section 11.3.1.1), without access holes. Note that this is distinct from the previously described typical shop practice of splicing flanges and webs before building girders. If welded in the configuration shown in figure 146, the flange and web welds would intersect. It is not possible to complete these welds in a sound manner. The flange CJP welds cannot be made across the full width of the flange because of interference from the web, and there is not room for proper termination of the web welds next to the flange welds. In addition, it would not be possible to perform NDE on the welds in the corner. Therefore, this situation constitutes an unacceptable use of “intersecting welds”.



Source: FHWA

Figure 146. Illustration. Girder or beam splices not suitably prepared for splicing.

Figure 147 illustrates how the two components can be suitably prepared and spliced properly. Weld access holes interrupt the otherwise intersecting welds and permit access for depositing quality welds (see also section 8.11). The access holes allow for weld tabs at the end of the web splice welds and the use of backing or backgouging in the flange splice welds. The fatigue performance of the weld access hole needs to be considered (per AASHTO BDS Table 6.6.1.2.3-1 [AASHTO, 2017a], it is a Category C detail in a rolled beam and a Category D in a built-up plate girder), and the access hole must be made in conformance with clause 3.2.5 of the Code, which requires a radius and smooth transition (see detail in section 8.11). The Code does not include access hole details, but both D1.1 (Figure 5.2) and D1.8 (Figure 6.2) have details that may be considered, with a radius and a smooth transition. The term “access hole” is appropriate; the holes are provided for proper welding access (both deposition and termination), not for the sole purpose of avoiding intersection.



Source: FHWA

Figure 147. Illustrations. Girder properly prepared and joined.

From the previous plate girder splicing example, it should be clear that, in some cases, it is helpful and even essential to interrupt what would ordinarily be intersecting welds, and weld access holes provide the needed space to make the necessary welds. This principle can be simply stated as follows: welds may need to be interrupted to provide access for depositing quality welds.

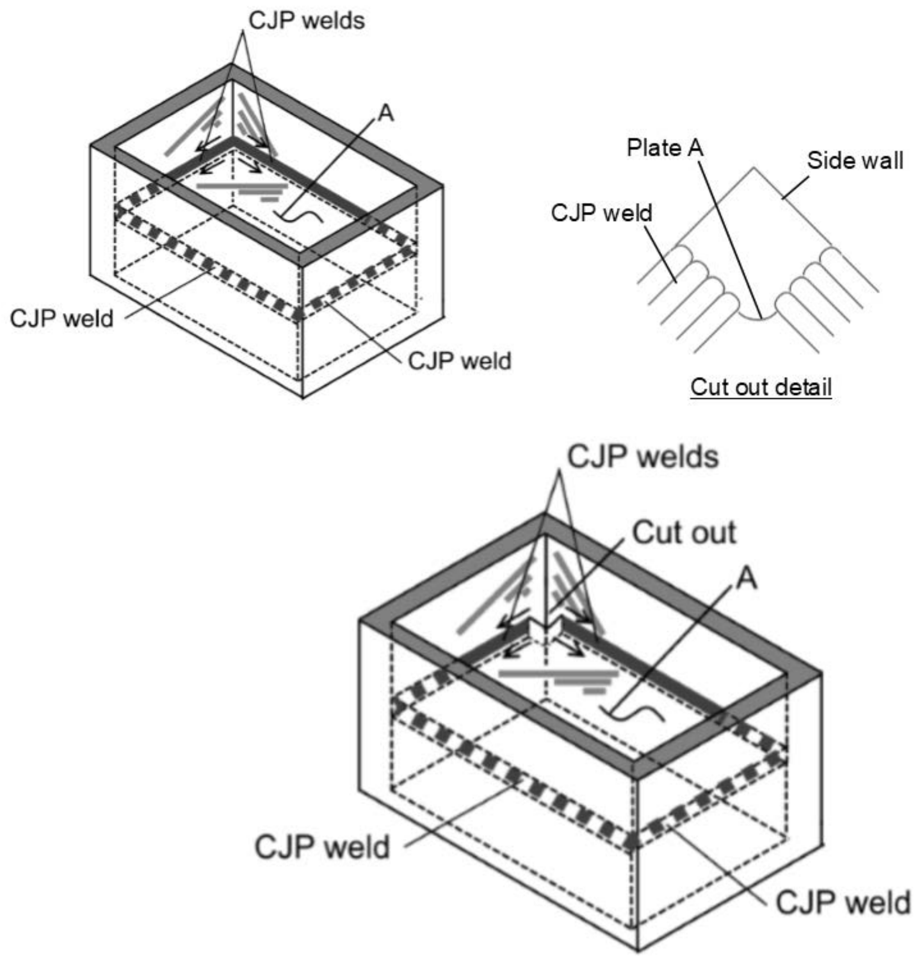
8.9.2.2. Cracking Control during Fabrication

Welds may be interrupted to avoid intersections for another reason: to control cracking during fabrication. When welds intersect at a point from two or three orthogonal directions, the residual stresses from welding may cause cracking in the weld or base metal at that location. Normally, it is the longitudinal shrinkage stress along the direction of a weld that is the primary driver of such cracking. Small, short welds on thinner plates commonly have intersecting welds without experiencing cracking, but longer, larger welds on thicker plates (higher constraint or resistance to shrinkage) can be problematic. However, quantification of the differences between the first and second conditions is difficult, if not impossible. In general, the concern of cracking during fabrication is not a concern with normal plate girder bridges.

For highly constrained (or highly restrained) connections, the residual stresses from large, long, intersecting welds create a condition in which localized shear stresses, necessary for yielding and deformation, are reduced. This in turn results in an apparent increase in the yield strength of the steel, and a decrease in ductility (Blodgett, 1995). Avoiding the intersection of welds at this location will increase the localized ductility and reduce cracking tendencies.

For example, consider the conceptualized assembly in figure 148. Plate “A” (an internal diaphragm) is intended to be welded by CJP to four sides of a box. If plate A is thick, perhaps over 1 ½ inch thick, many weld passes will be needed to make the CJP welds to each side wall of the assembly. At the corners the welds will shrink away from each other, as indicated by the arrows, introducing a cracking tendency at the corner in the upper left of the figure. To relieve this tendency, cut-outs can be introduced in the corners, as shown on the lower right. The cut-out should be at all four corners. A detail of the cut outs is shown in the upper right of the figure.

This example was provided to illustrate a legitimate concern regarding intersecting welds. In reality, the best solution to preclude cracking in the assembly shown in figure 148 would be to replace the CJP groove welds with minimum-sized fillet welds that in most cases would be adequate for the application.



Source: FHWA

Figure 148. Illustration. Access holes to relieve constraint.

Also, it may be difficult to obtain good, discontinuity-free fusion at the intersection of two welds made at different times, particularly for CJP welds that intersect at a point. For example, in the assembly in figure 148, this situation would exist at each corner of plate "A" in the CJP welds if cut-outs are not included. If the cut-out is not included, and the CJP weld between plate A and the short side wall is completed, then the passes of this first weld would terminate against the corner. Then, in making the CJP weld between plate A and the long side wall, the presence of the first weld would restrict access to the corner and make it challenging to tie the second weld into the first.

While, the Code does not define where special detailing is required to avoid the fabrication-related cracking, a cut-out that is at least equal to the thickness of the intersecting steel members is a good general recommended size.

In contrast to the aforementioned situation, there are many cases where intersecting welds will be deposited at different times. The intersection of web-to-flange fillet welds with flange and web butt splices and, potentially, with stiffeners are just two examples where welds may touch without concern. There are many others.

8.9.3. Summary

"Intersecting welds" are not a problem when complex assemblies are built-up, one piece at a time, where welds are deposited and cooled before additional members are added. Examples include:

1. Web and flange splicing performed first, then assembled into a girder: the longitudinal weld intersects the web or flange splice, and this is not a problem issue.
2. Where longitudinal stiffener welds intersect a transverse web splice, or transverse stiffener welds intersect a longitudinal web splice.

However, "intersecting welds" may be problematic when the geometric configuration of the assembly precludes deposition of quality weld metal and inhibits NDE. Examples include:

1. Welded splicing of fabricated plate girders without access holes.
2. Welded splicing of rolled sections without access holes.
3. CJP welds required to be continuous around corners or terminating at corners where there is no room for weld tabs (section 8.8.1) or other runoff provisions.

Furthermore, "intersecting welds" may be present in details which are otherwise problematic because they represent highly restrained conditions where large, multiple pass welds intersect at corners. Examples: Internal stiffeners of highly restrained assemblies, joined with large welds.

In addition, "intersecting welds" may be present in details which are otherwise problematic due to the intersection of structural elements converging from three orthogonal directions in configurations which impose a high degree of triaxial constraint and thus an elevated risk of constraint-induced fracture.

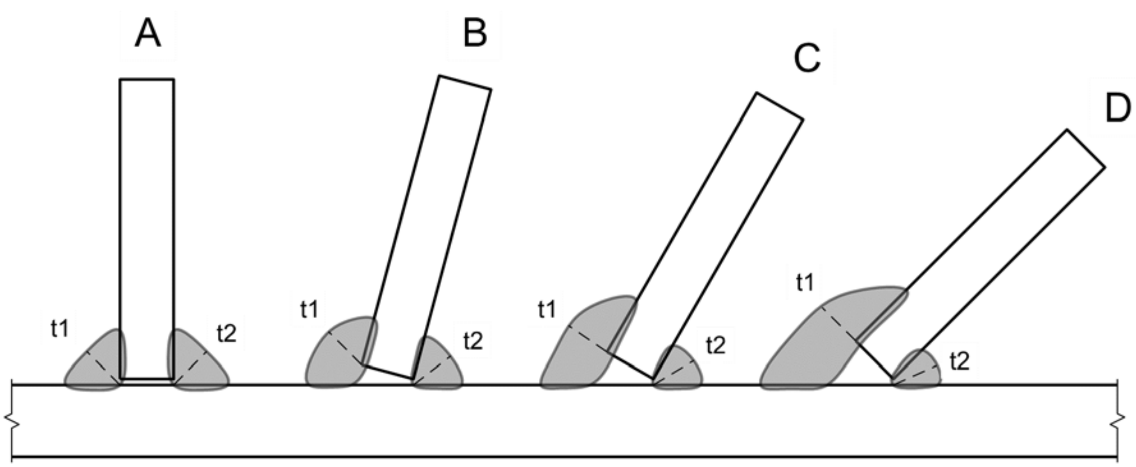
The concerns in both of the above situations are better described as cases of "intersecting elements" than "intersecting welds".

8.10. SKEWED T-JOINTS

Most T-joints intersect at 90-degree angles, but skewed T-joints are also common in steel bridges. Examples include web-to-flange joints in tub girders and skewed connection plates in I-girder bridges (section 9.3.2.4). When the T-joint is skewed, special consideration must be given to both the acute and the obtuse sides of the joint.

Consider figure 149. Image "A" shows a typical stiffener with fillet welds that require throats of t1 and t2. At image "B", the stiffener is slightly skewed, and the fillet welds sizes are adjusted to provide the same effective t1 and t2 throats. On the obtuse side, to achieve t1, the leg of the fillet weld needed becomes a bit larger, and on the acute side the leg of the fillet weld needed to achieve t2 becomes a bit smaller. Shown in "C" and "D", this effect exaggerates as the skew grows larger. Further, as the skew grows larger, the fillet welds become more challenging to make and eventually become impractical. If the fillet welds in “A” are $^{5}/_{16}$-inch fillet welds, then these welds can reasonably be made in a single pass. At “B”, the fillet weld on the obtuse side can reasonably be made in two or three passes, and the weld on the acute side can reasonably be made in one pass. At “C”, making the weld on the obtuse is doable but will require many passes, perhaps four to six, and once again, the weld on the acute side can reasonably be made in one pass.

At “D”, making fillet welds on both sides of the joint becomes challenging. The idealize fillet shown on the obtuse side shows a large gap under the fillet weld, but the weld cannot be accomplished this way; rather, weld must be deposited to provide a base for subsequent passes. This weld will require many passes, perhaps eight or more. As an example, the stiffener shown in figure 150 is laid over at a 27-degree angle, measured between the planes of the web and stiffener, and required nine passes to make the weld on the obtuse side. On the acute side, depending upon the actual skew angle, it may not be possible to reach the throat of the connection; rather, the weld will only be near the throat.



Source: FHWA

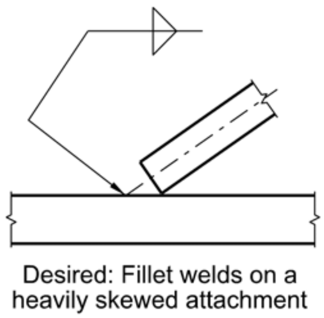
Figure 149. Illustration. Fillet welding skewed plates.

Source: FHWA

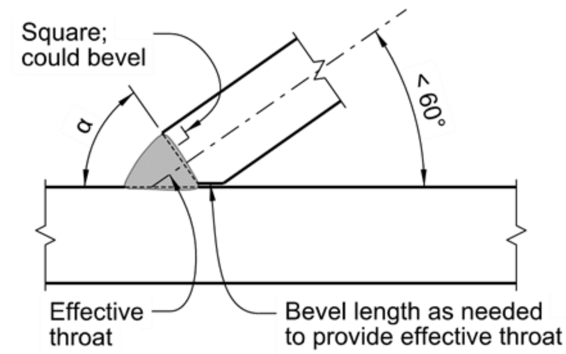
Figure 150. Photos. Very skewed stiffener with fillet welds.

As figure 149 illustrates, there are practical limits for use of fillet welds on skewed joints. On the acute side of the stiffener, the Code allows fillets welds for skews up to 60 degrees (clause 2.8.4), and on the obtuse side, the Code allows a skew angle of up to 135 degrees (clause 2.8.4), measured from the plane of the web to the stiffener. In both cases, the actual size of skewed fillet welds must be adjusted to provide the desired effective throat. Annex B of the Code describes how this is done.

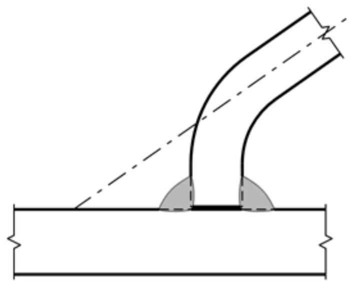
For joints skewed more than 60 degrees, the desired option of fillet welds (as shown in figure 151-A) is not permitted. One option is to use a PJP weld as shown in figure 151-B. The key detail in this figure is the effective throat. The stiffener is shown with a square end, but this could be beveled to adjust the groove angle, α. The fabricator might propose a bevel to achieve a 60-degree or 45-degree bevel because those angles would make prequalified PJP weld joints for SAW and FCAW, respectively, in accordance with D1.5 figure 2.5 of the Code. For highly skewed joints, another option is to use a bent plate stiffener as shown in figure 151-C; however, the design may need to consider the reduced stiffness of this configuration.



A. Fillet welds not permitted.



B. Option for less than 60-degree skew.

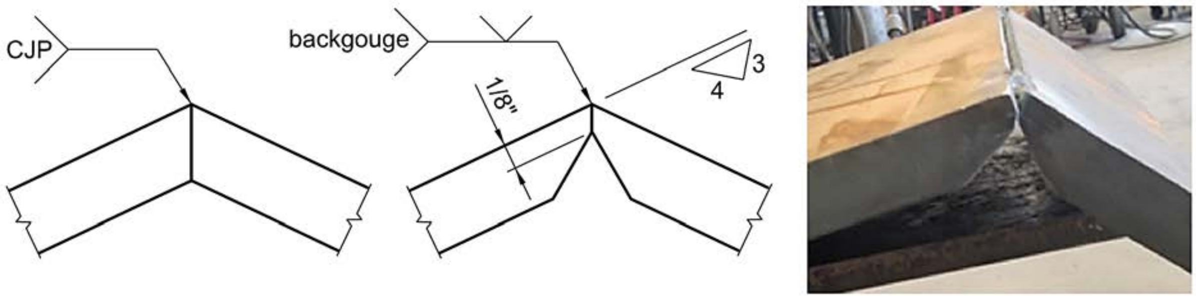


C. Bent plate option.

Source: FHWA

Figure 151. Illustrations. Weld details for highly skewed joints.

In cases where skewed butt joints are needed, the best practice is to simply show the plates coming together in the design and not prescribe welding details except for whether a CJP or PJP weld is required and, for PJP welds, the required weld size. Fabricators can adapt the Code standard butt joints of D1.5 figures 2.4 and 2.5 to the skewed condition as needed. An example of such a joint is shown in figure 152. Note “h” of D1.5 figures 2.4 and 2.5 states that “joints may vary from 135 degrees to 180 degrees”. For practical purposes, the joint becomes a corner joint beyond this variation.



Illustrations: FHWA / Photo Source: FHWA

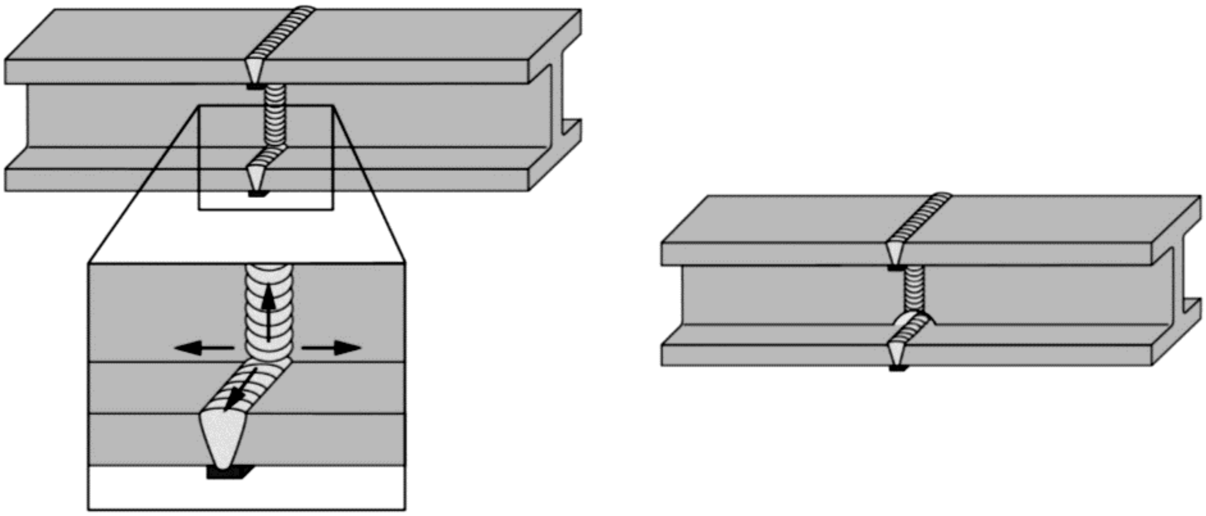
Figure 152. Illustrations and Photo. Skewed CJP groove weld joint, as designed at left, as detailed center, and as fit up for welding at right.

8.11. WELD ACCESS HOLES

Weld access holes are openings that permit access for welding, backgouging, or for insertion of backing. Example are shown in figures 138 and 139. Additionally, in some connection details, weld tabs may need to be installed in such openings. Weld access holes are known by various slang terms, including “rat holes” and “apple holes.”

Weld access holes must be large enough to permit the welder to insert the welding electrode or welding gun into position, to see the weld pool while welding, and to permit the weld to be cleaned and visually inspected between weld passes.

In addition to the practical function of providing access to the weld joint being made, weld access holes also limit the interaction of the various residual stress fields. Consider a beam splice for which no weld-access hole has been provided, as illustrated on the left side of figure 153. Furthermore, for this example, assume the vertical web weld is tied into the flange-to-flange weld. In addition to the problem that the flange welds cannot be made under the web and the backing would interfere with the web splice, the longitudinal and transverse shrinkage of these two welds will cause triaxial residual tensile stresses that all meet at a point. For such highly constrained (or highly restrained) connections, the residual stresses from large, long, intersecting welds (see discussion in 7.9) create a condition where localized shear stresses, necessary for yielding and deformation, are reduced. This in turn results in an increase in the yield strength of the steel, and a decrease in ductility (Blodgett, 1995). Use of access holes to preclude the intersection of welds at this location will increase the localized ductility and reduce cracking tendencies.



Original figure: © 2018 Lincoln Electric

Figure 153. Illustration. Beam splice without access hole, left, and with access hole, right (modified by authors).

When a weld access hole is provided as shown in the right of figure 153, the stresses caused by the longitudinal and transverse shrinkage of the vertical weld are physically prevented from interacting with those of the flange weld, since there is no material at this juncture through which to transfer stress. Triaxial stresses are reduced, ductility is increased, and cracking tendencies are reduced.

Weld access holes must be made properly, with the proper size (height and width) and with smooth surfaces that are free of notches and gouges. The Code does not have specific access hole details; however, AWS D1.1 provides effective guidance for access holes in clause 5.16.

Weld access holes are prepared by thermal cutting or by a combination of drilling and thermal cutting. Use of computer numerically controlled (CNC) equipment in modern fabrication allow use of custom shaped access holes (figure 154). Often fabricators will propose preferred details by RFI based on their cutting capabilities. Surface roughness criteria apply to the thermally cut edges. Clause 3.2.5 provides quality requirements for access holes. Grinding of weld access holes is not normally required if thermally cut surfaces meet acceptance criteria. As with other cutting under the Code, freehand cutting of access holes is not permitted (clause 3.2.2).

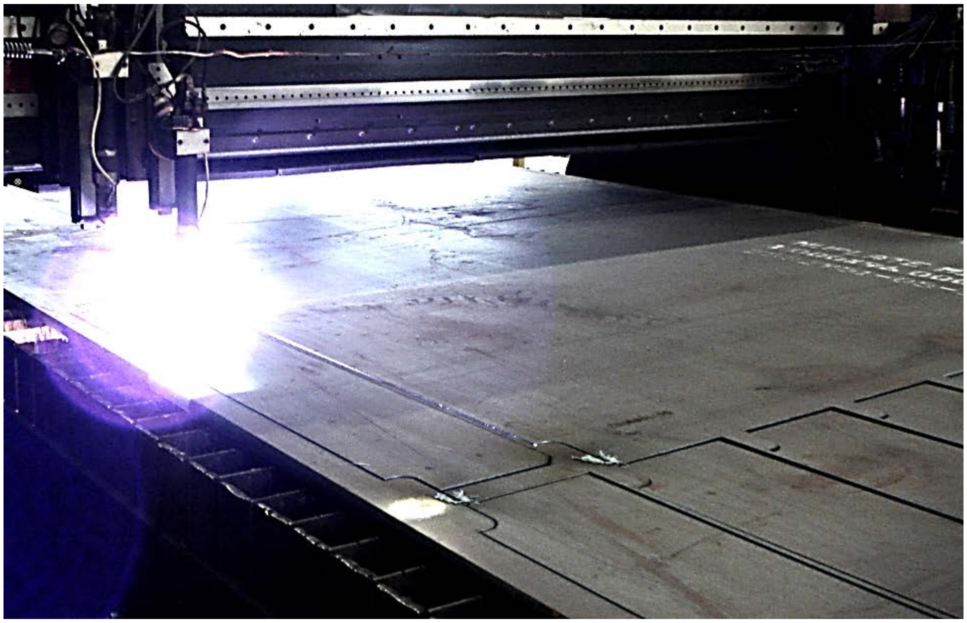

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Figure 154. Photo. Plasma cutting stiffeners, including access holes.

CHAPTER 9 - DETAILS FOR WELDED BRIDGES

This chapter provides best practice recommendations for welded bridge design constructability. The term "constructability" as used in the bridge fabrication industry refers to the relative ease with which bridge members can be fabricated. These fabrication activities include cutting and beveling of steel, fitting of steel components and depositing quality welds in a safe manner. Good constructability leads to reduced fabrication costs, quicker delivery of members, and improved quality. The emphasis on constructability is an attempt to provide coordination among bridge design, detailing, and fabrication. Well-designed bridge members with good details may include factors that make the member difficult to fabricate; such members have poor constructability.

In steel bridge fabrication, constructability is a function of the following values:

- **Safety** - The design should be such that the bridge elements can be produced safely. As an example, in the case of large boxes, it must be possible to work safely inside the box, with due consideration of confined space concerns, which may add costs and thus reduce constructability (see section 9.8.1).
- **Quality** - The design should best facilitate the achievement of quality in accordance with the Code and other project requirements.
- **Cost and time** - The design should facilitate the optimal use of fabrication equipment and technology so that the project can be cost-competitive and completed in a timely fashion.

The constructability recommendations of this chapter are about achieving these values.

9.1. FABRICATOR INPUT

Often the best constructability is achieved when fabricators provide comments about the bridge details while the design is still being developed. This is particularly important for unusual and complex designs. Although fabricators are generally not part of design teams, many fabricators are willing to review plans or design sketches and providing constructability comments. Fabrication input can be obtained through the NSBA, who can provide expertise and reach out to its membership to collect feedback from member fabricators. Fabricators can also be contacted directly. A third option is to contact subcontract detailers who provide shop drawings to fabricators; experienced bridge detailers have extensive knowledge about constructability.

9.2. FACTORS AFFECTING WELD CONSTRUCTABILITY

The following subsections address factors that relate to weld connection constructability.

9.2.1. Preferred Weld Types

The type of weld chosen for the connection has a significant effect on the constructability of the design based on the effort required to make each type of weld. Generally, the relative constructability of the four most common types of welds used in bridge fabrication is as follows, listed from highest constructability to lowest:

- Single-pass fillet welds
- Multipass fillet welds (up to the size limit discussed in 9.2.2)
- Partial joint penetration (PJP) welds
- Complete joint penetration (CJP) welds

The factors that contribute to the constructability of these weld types are explained in the next section. The engineer should consider these factors and recognize that this order is a general trend and not an absolute ranking.

From a design standpoint, when a fillet weld size is specified, there is no distinction between a single-pass fillet weld and a multipass fillet weld; engineers do not call out “multipass fillets” or “single-pass fillets”. Similarly, for PJP or CJP groove welds, engineers do not specify whether the weld is to be made in a single pass or multiple passes. Rather, in the case of fillet welds and PJP welds, engineers call out the weld size and the fabricator will determine how the weld will be made. Hence, when an engineer specifies a fillet weld of a certain size, the engineer may be requiring a de facto multipass weld.

9.2.2. Fillet Welds versus PJP Groove Welds

Fillet welds and PJP groove welds can both be used in T-joints and corner joints. For welds with equal throat dimensions, a PJP groove weld in a 90-degree T-joint requires roughly one-half the volume of weld metal compared to a fillet weld of similar strength (see figure 155).

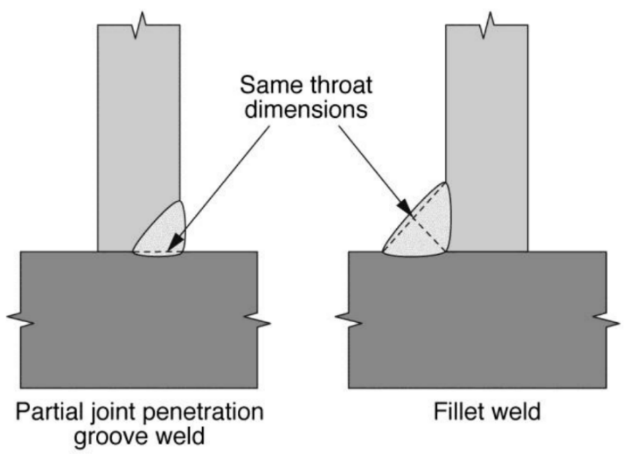


Figure 155. Illustration. Fillet and PJP groove weld throats.

PJP groove welds require that a bevel be applied to create the groove, adding to the overall cost. When a choice is made between large multipass fillet welds and PJP groove welds, a useful rule of thumb is to use fillet welds whenever the required fillet weld size is 1 inch or less, and use PJP groove welds instead when larger fillet welds would be required. Most fillet welds are not required to have sizes greater than 1 inch, so fillet welds are typically the most economical choice.

9.2.3. Combination PJP and Fillet Weld Option

When the required weld size justifies a PJP groove weld rather than a fillet weld (see section 9.2.2), a PJP/fillet weld combination as shown in figure 156 may be the most economical option.

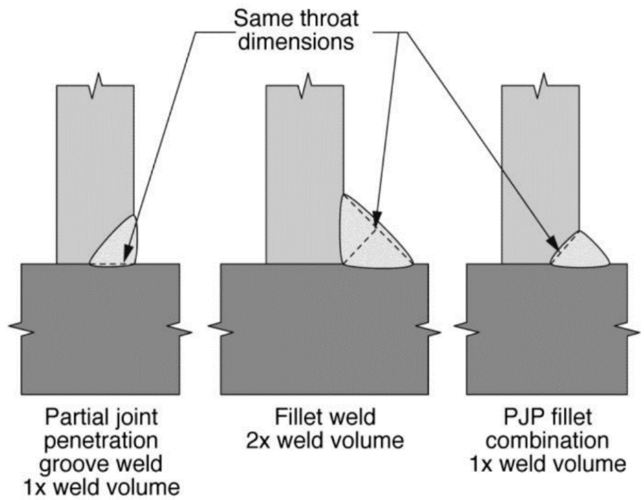


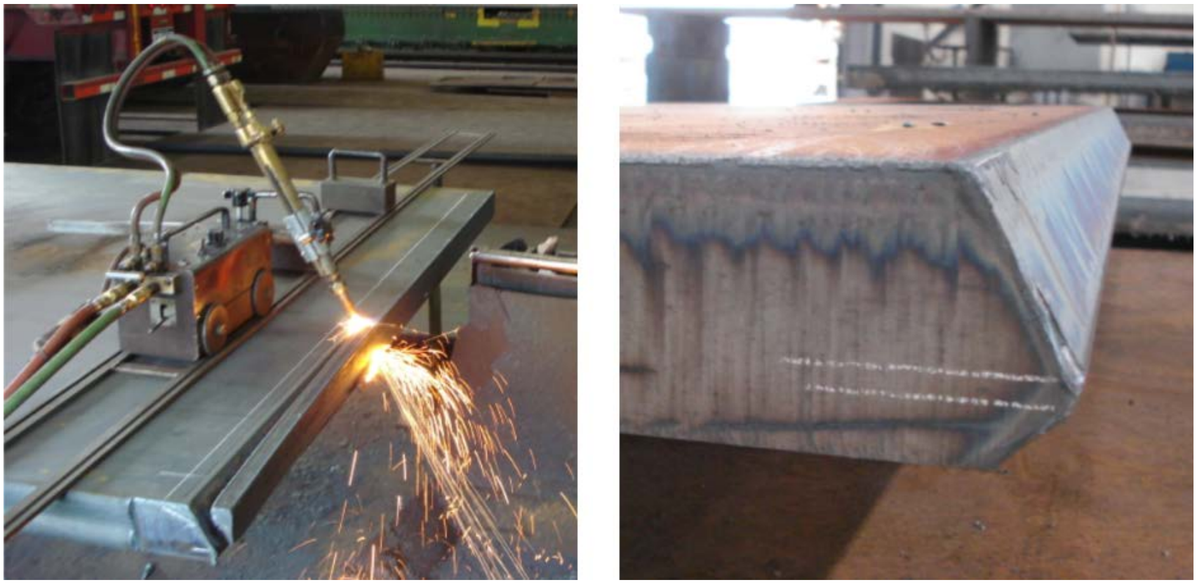
Figure 156. Illustration. T-joint fillet weld and PJP groove weld options.

When a 45-degree included angle is used for the PJP groove portion of the weld, and the fillet weld leg size is equal to the depth of groove weld preparation as shown on the right of figure 156, this combination PJP/fillet weld requires no more weld metal than does a PJP weld on its own (see left of figure 156). This will be one-half as much as would be required for a fillet weld of equal strength (center of figure 156). The fact that the Code requires a reinforcing fillet weld (see section 8.7) on inside T- and corner joints for contouring purposes makes the case for combination PJP/fillet welds instead of just a PJP groove weld even more compelling, as long as there is room for the fillet. Even if the detail on the left side of figure 156 is called for in design, the reinforcing fillet will nonetheless need to be added.

9.2.4. Weld Type Factors

The constructability of the types of weld is based on the following factors that affect the effort required to make the weld:

- **Bevel preparation** - Bevels require cost and time to prepare (see figure 157). Fillet welds do not require a bevel. Most groove welds require bevels. The amount of effort needed to make bevels depends on the thickness of the material and the length of the joint. The fabricator’s equipment is also a factor in the effort required to produce bevels. Bevels are most often produced by cutting torch (as shown in figure 157), but bevels can also be made with special beveling tools or by machining. Machining is not common but may be used by fabricators who choose to use U- or J-joints, such as joints B-U6 and B-U8 (see D1.5 figure 2.4). When making bevels, care must be taken to produce bevels that will result in good fit-up to help avoid workmanship problems, such as melt-through (or blow-through) at the joint root.



Source: FHWA

Figure 157. Photo. Making bevels for groove welds.

- **Number of weld passes –** The number of weld passes is a factor in constructability. Generally, the fewer passes the better. Given the shape of fillet welds and the beveled configuration of most groove welds, the number of passes required is typically proportional to the square of the weld size.
- **Single-pass fillet welds –** Good constructability for fillet welds is best achieved by using fillet welds that can be made in a single pass. The size of fillet weld that can be made in a single-pass depends upon the welding process, the welding equipment used, and welding position. Positions and their effect on fillet welding are discussed below. The most common welding position used in bridge fillet welding is horizontal, and fabricators readily make $^5/_{16}$-inch fillet welds in this position. Single pass $^3/_8$-inch fillet welds are possible to make in the horizontal position with SAW, but can be a challenge to make consistently. Therefore, a good rule of thumb is to limit fillet welds to $^5/_{16}$ inch whenever possible (i.e., do not arbitrarily increase weld size). In some cases, fabricators can rotate assemblies to weld in the flat position and make larger single-pass fillet welds, but this is not always possible or effective.
- **Effect of position on the number of weld passes –** Welding position affects the size of weld passes that can be used and therefore the number of passes needed to make the weld. Weld pass size is affected by the effect of gravity on the weld puddle. Although this effect is applicable to both groove welds and fillet welds, in bridge fabrication it is more relevant to fillet welds. The effect of each position on fillet welds is explained in this section. Consider figure 158, where various positions are shown (see section 2.3) and desired fillet weld sizes are represented by dashed lines:
  - **Flat** - As shown in figure 158-A, when welding is performed in the flat position, the work forms a “trough” that fully contains the weld puddle; in fact, “trough” is a non-standard term for the flat position. In this position, single pass welds of ½ inch, ¾ inch, or even greater size are attainable. The flat position also reduces the

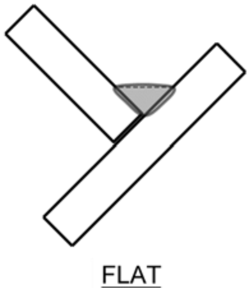
number of passes for large multiple-pass fillets. However, positioning work for fillet welding in the flat position typically requires special fixturing to support the work and keep it stable during welding, so fabricators often prefer horizontal to flat. Flat position fillet welding is particularly challenging when fabricating deep girders.

- **Horizontal** - When welding is performed in the horizontal position, gravity tends to pull the weld puddle down, away from the vertical leg, and this limits the amount of weld metal that can be deposited in a single pass. In the left weldment of figure 158-B, the smaller weld is satisfactory, but the larger weld, which was too large to be made effectively in a single pass due to flow, has a poor profile, with undercut at the top toe and incomplete fusion and overlap at the bottom toe. Shown in the right weldment of figure 158-B is the same size fillet weld made with multiple passes. Three passes were needed to make this weld effectively.

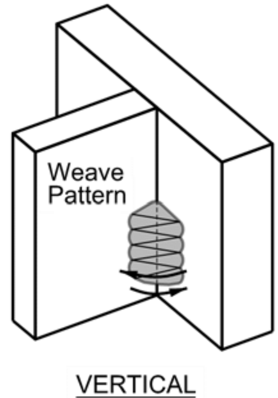
  The horizontal position is by far the most common position used in bridge fabrication. In the horizontal position, single-pass welds of $^{5}/_{16}$ inch are readily achievable and are the most common size fillet weld used in bridge fabrication, due to the fact that clause 2.8.1 and table 2.1 of the Code require a minimum $^{5}/_{16}$-inch fillet weld for plates ¾ inch and greater in thickness. Using SAW, ⅜-inch fillet welds can also be made in a single pass, although this is more challenging, particularly on longer joints.

- **Vertical -** Figure 158-C shows welding vertically up, which is the progression required by the Code when welding in the vertical position. The upward progression is effective because when progressing upward with a weave pattern, each pass solidifies and provides support for subsequent weld metal. Using the vertical-up progression, single-pass fillet welds of ⅜ inch can readily be made.

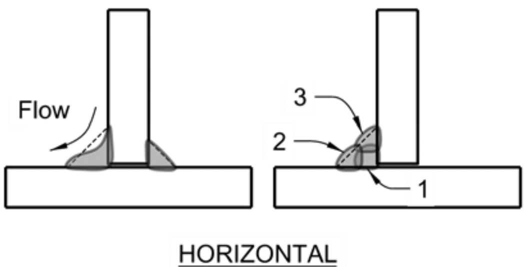
- **Overhead** - With overhead welding as shown in figure 158-D, gravity pulls the weld puddle away from the joint, and the size of weld that can be made in a single pass is dependent upon how much weld metal can solidify before it starts to drop out of the joint. Generally, the largest fillet weld that can be readily made in the overhead position is ¼ inch.



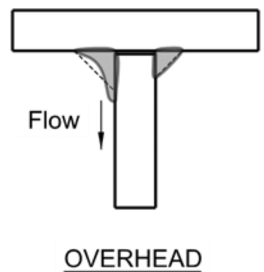
A. Welding in flat position.



C. Welding in vertical position



B. Welding in horizontal position.



D. Welding in overhead position.

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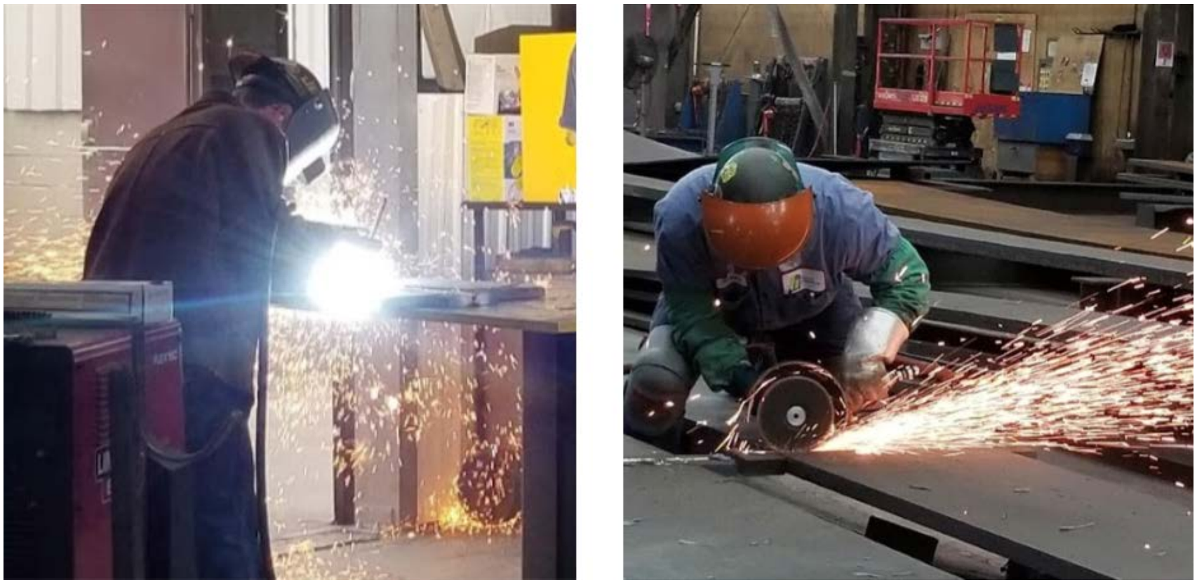
Source: FHWA

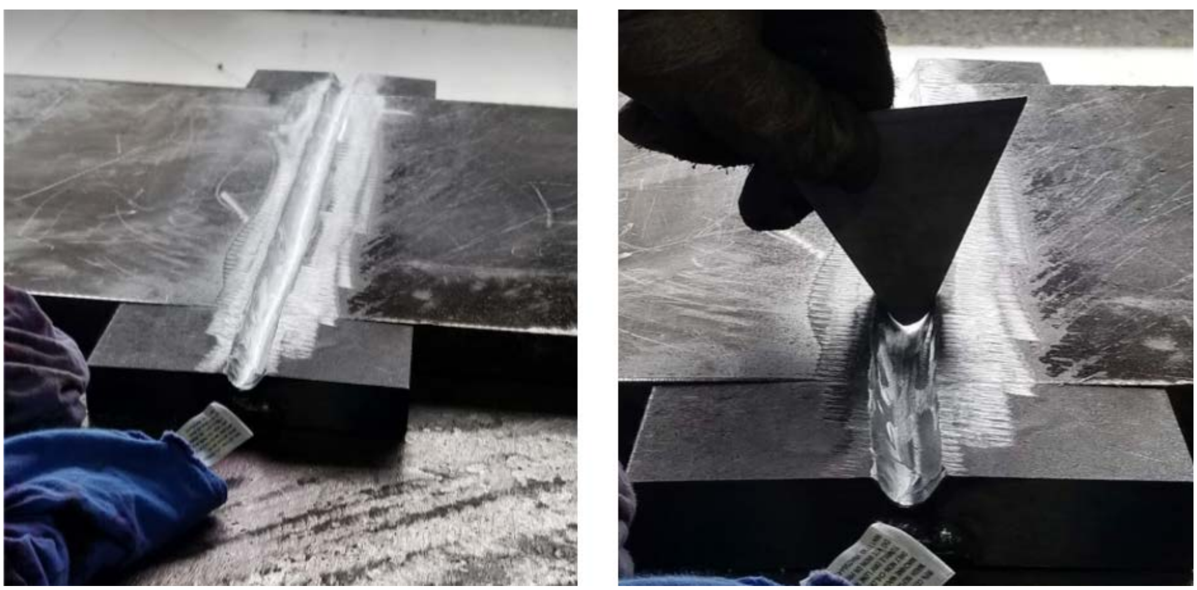
Figure 158. Illustrations. Weld passes associated with fillet weld position.

- **Root soundness / backgouging** - Compared to PJP welds, CJP welds require extra steps to achieve soundness at the root of groove welds. For the standard CJP welds in the Code (D1.5 figure 2.4), backgouging (figure 159) is required to help ensure soundness (except for joints with backing). Following backgouging, grinding may be used to complete the joint preparation. A joint that has been backgouged and ground is shown on the left in figure 160. The picture on the right shows a check of the backgouged joint to ensure the desired groove angle is achieved.
- **Joints with backing –** CJP groove welds made with backing (see section 8.6) and welded from one side are an alternative to CJP groove welds made from both sides. Such joints do not require backgouging or turning work over to reach the back side of the weld. However, joints with backing left in place are not common for butt joints in bridges because of their lower fatigue category. In contrast to butt joints, backing is routinely used for corner joints where a CJP weld is needed but where double-sided welds have reduced constructability, such as in box sections. Additionally, the backing in this type of joint does not lower the fatigue category because the backing and the CJP weld are parallel to the direction or primary tensile stress.



Source: FHWA

Figure 159. Photos. Backgouging (left) and grinding a backgouged joint (right).



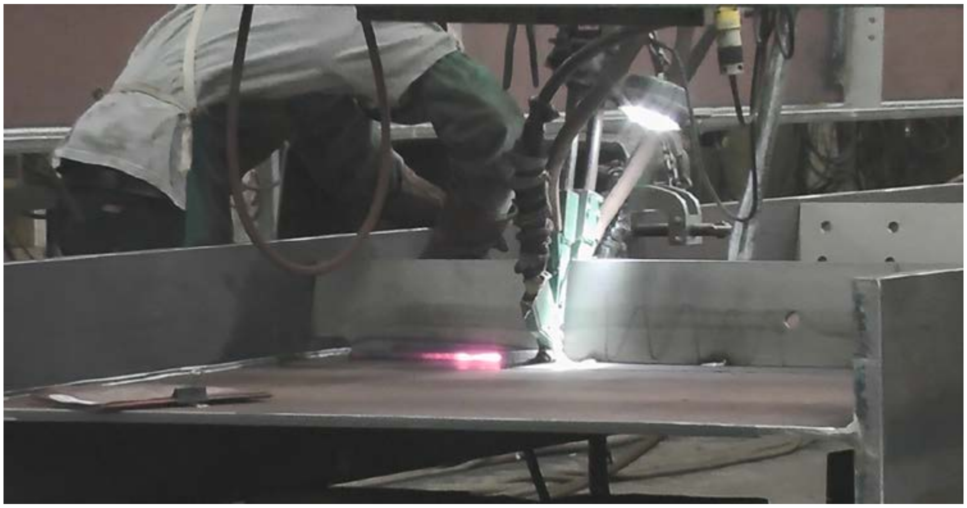
Source: FHWA

Figure 160. Photos. A backgouged and ground joint.

9.2.5. Welding Equipment

Certain equipment has become typical for welding steel bridges and can be found in most fabrication shops. Fabricators use mechanized and automatic equipment to make work easier; therefore, facilitating use of this equipment in design improves constructability.

Stiffener-to-web welding (including welding connection plates to webs) provides a good example of where consideration should be made for the use of common equipment. Many fabricators use mechanized equipment that permits the use of submerged arc welding (section 3.3) to make these welds of both sides of the stiffeners at once (figure 161).

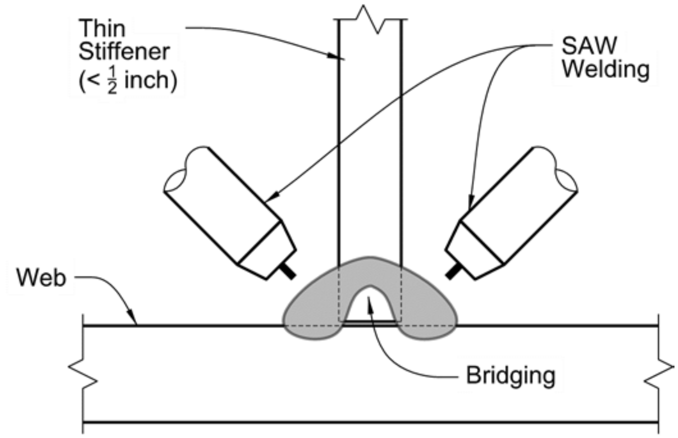


Source: FHWA

Figure 161. Photo. Two-sided mechanized stiffener welding.

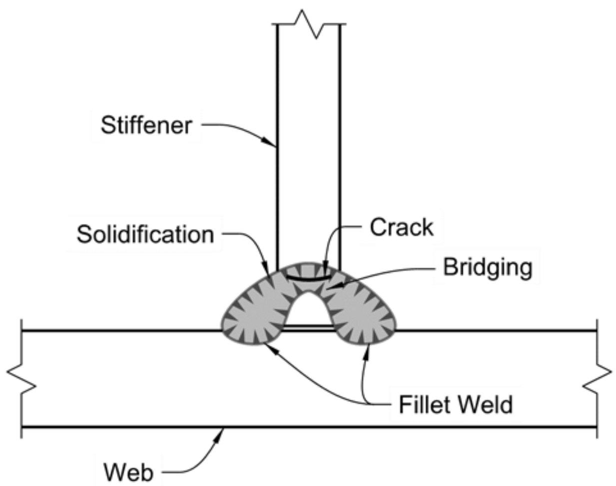
Making simultaneous welds on both sides of the stiffener saves the time associated with performing the two stiffener welds in sequence. Also, the mechanization facilitates quality by providing a more consistent weld than can be made by hand. Therefore, a design with good constructability is one that allows the fabricator to use this equipment. The designer should consider the following:

1. **Stiffener spacing** - Stiffeners should not be so close to each other that the equipment will not fit between the stiffeners. The minimum amount of space needed varies with the width of the stiffener; one foot is usually sufficient.
2. **Stiffener thickness** - Stiffeners should be at least ½ inch thick so that the welds on either side of the stiffener do not "bridge" beneath the stiffener (figure 162) when welding is performed from both sides simultaneously. Such bridging will likely result in a crack in the weld, slightly below the upper weld toe. It is important to note that such cracks result not simply because the welds touch each other but rather due to the solidification pattern of the weld puddle. As shown in figure 163, a crack is likely in the slender solidification "bridge". This is related to a bead-shape-induced crack, such as that described in section 5.4.2.3. For a further explanation, see Miller, 1997.



Source: FHWA

Figure 162. Illustration. Welds bridging beneath a stiffener.



Source: FHWA

Figure 163. Illustration. Solidification crack from bridging fillet welds.

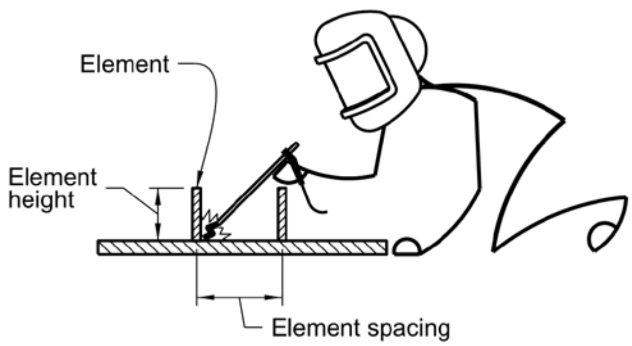
It is not prudent to specify the type of equipment to be used in fabrication. Doing so may unnecessarily hamper fabricators who do not have particular equipment and potentially stifle innovation. Rather, the best course for achieving good constructability is to design with consideration of what fabricators are likely to use (if this is known), but then allow the fabricator to use whatever equipment the fabricator prefers, provided project specifications are met.

9.2.6. Access

To improve constructability, the designer should ensure sufficient access for welding to be performed. Access may be divided into these factors:

- **Global access** - The welder must be able to get to the joint with welding equipment and work at the weld location safely. As an example, see the discussion in section 9.8.1 regarding access to welds inside of boxes.
- **Sight and reach** - Welders need to be able to reach the joint and see the weld puddle in order for a quality weld to be made. This does not mean that a person can simply snake their arm though the work and touch the joint to be welded. Rather, the welder needs to be able to weld in a comfortable position, with a clear line of sight and access for head and shoulders. Inspectors also need to be able to see the completed weld in order to perform visual inspection.

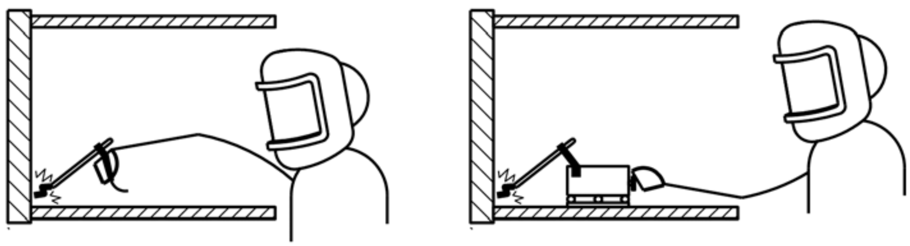
The size and spacing of members are factors associated with providing sufficient access to the joint. As shown in figure 164, when components near to each other are welded, welders can see over short components, and the welding equipment can clear them provided they are not too close to each other. As a general rule, it is good to space parallel elements at least three inches farther apart than the element height (figure 164). However, this guideline may not apply when mechanized equipment is used. For example, as described in section 9.2.5, at least one foot spacing between parallel elements is needed for mechanized fillet welding of girder stiffeners to girder webs (figure 161).



Source: FHWA

Figure 164. Illustration. Adequate spacing for welding.

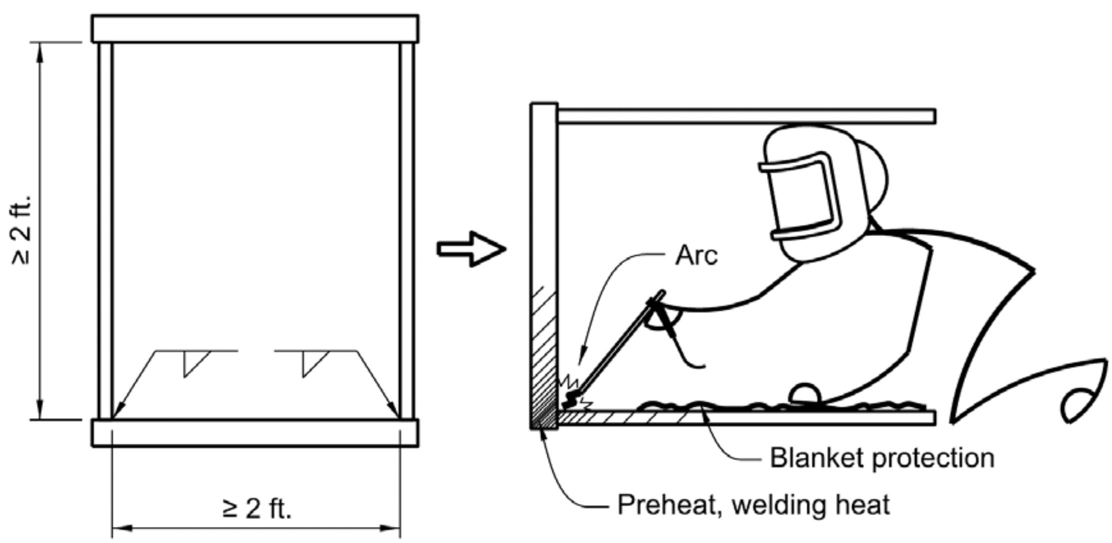
As the element height reaches about a foot (for example, when the element in figure 164 is about a foot or more tall), the welder can no longer weld with equipment held over the element. Instead, the welder reaches between elements, as shown in figure 165. If the welder must reach in, then space is needed for welding equipment (handheld or mechanized), the welder's arm, and visual access for the welder to see the puddle. For this access by reaching, an element spacing of two feet or more is adequate. In figure 165, the assembly is rotated so that the welder can reach in from the side of the assembly. If the assembly cannot be rotated to this position and access must instead be from the top down, welding is more difficult because reaching down from a position directly above the weld is awkward and, from the heat of preheat and welding, uncomfortable.



Source: FHWA

Figure 165. Illustration. Reaching between elements for handheld (left) or mechanized (right) welding.

When elements are greater than two feet in depth, the welder can no longer reach between the elements, and then the welder must crawl into the assembly. For example, the box in figure 166 is rotated, and the welder climbs in before the top flange of the box is added to make the fillet weld required at the bottom flange. If the welder must climb in, space is needed for the welder's body. At a minimum this is two feet, but the more space the better. Not only must the welder be able to comfortably reach and see the weld, but proper ventilation must also be assured, and the welder must be protected from the heat of preheating and of welding. The fabricator may also choose to mechanize this welding if the elements are long enough and there is not interference from other elements; for this, there must be room for the equipment to be set up and operated.



Source: FHWA

Figure 166. Illustration. Climbing in between elements for welding.

9.2.7. Finish-to-Bear and Tight Fit Conditions

Though "finish-to-bear" connections do not necessarily involve welding (unless combined with fillet welds), the finish-to-bear condition has implications for welding constructability. Where a "finish-to-bear" condition can be used instead of welding, it usually has better constructability. "Finish-to-bear", which is also known as "mill-to-bear", refers to preparing the end of a plate and bringing it within sufficient contact to another plate such that the compression loads effectively transfer through the contact surface (see section 9.3.2.1). The industry is moving away from the term "mill-to-bear" to "finish-to-bear" because milling is not necessary to achieve the required condition: the condition can also be achieved through other means, such as sawing, cutting and grinding. Further, grinding can be very important for helping make adjustments on the shop floor to properly achieve fit when the mating surface is not otherwise sufficiently flat.

"Tight fit" is another term used to describe a fit condition. For intermediate stiffener ends, tight fit is described in clause 3.5.1.10. No contact is required; rather, there must be no more than a $^{1}/_{16}$-inch gap in the connection.

9.3. GENERAL DETAILS

9.3.1. Fillet Welds

9.3.1.1. Sizes

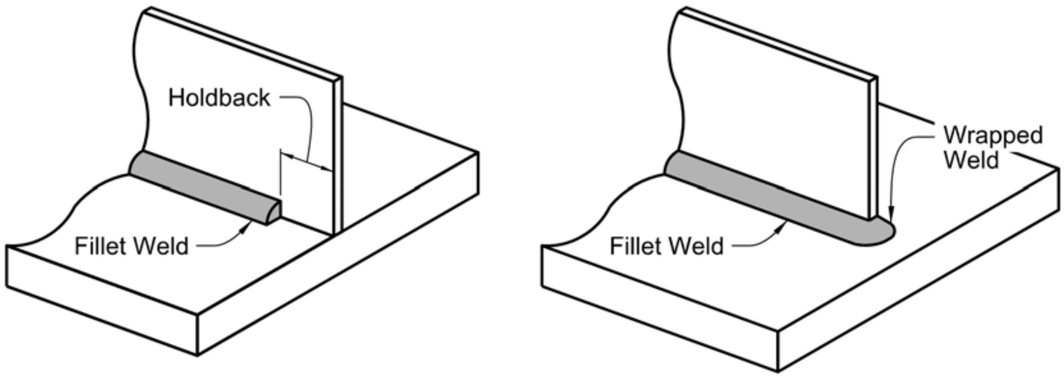
Plate girders have a significant amount of fillet welding, typically including web-to-flange connections and connections between stiffeners (and connection plates) to webs and flanges. For good constructability, welds for these connections should be kept to sizes that can be made in a single pass; i.e., if design requirements permit, they should be kept to a maximum of $^{5}/_{16}$ inch (see discussion of single-pass fillet welds in section 9.2.4).

9.3.1.2. Terminations

There are three common options for terminating fillet welds:

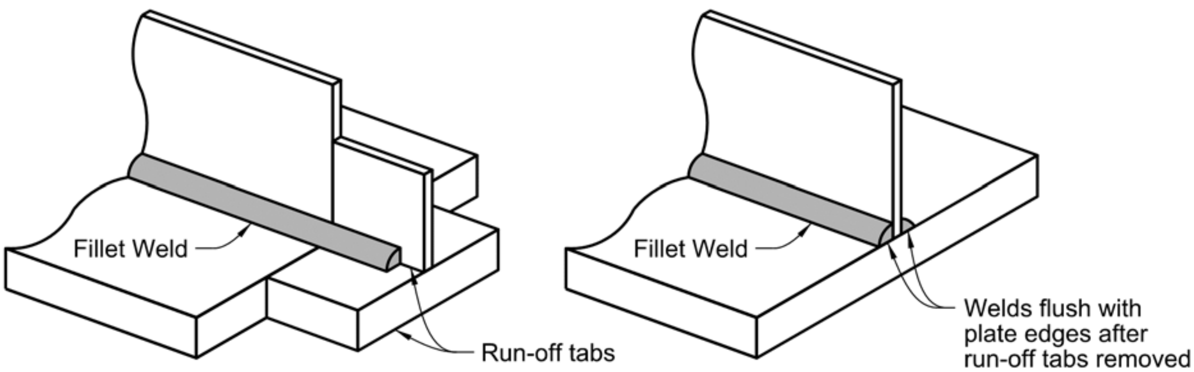
- Held back from plate edges (figure 167, left; known also as a "holdback")
- Wrapped around the edge (figure 167, right)
- Taken to the edge of plates with the use of weld tabs (figure 168)

In clause 3.12, the Code requires the use of weld tabs "whenever possible". Generally, the possibility for using fillet weld tabs relates to the feasibility of attaching them and removing them after welding is complete. Therefore, on girders, weld tabs are common where there is space beyond the joint, such as web-to-flange fillet welds, but are not used where fillet welds terminate in corners, such as stiffener-to-web welds. Tabs are not used on stiffener-to-flange welds; it is not possible to use a tab on the inside termination, where space is not available, and on the outside termination, stiffeners do not usually run to the edge of the flange, which means that continuing the weld past the edge of the stiffener would involve removing weld from the flange. See also the discussion in section 8.8.



Source: FHWA

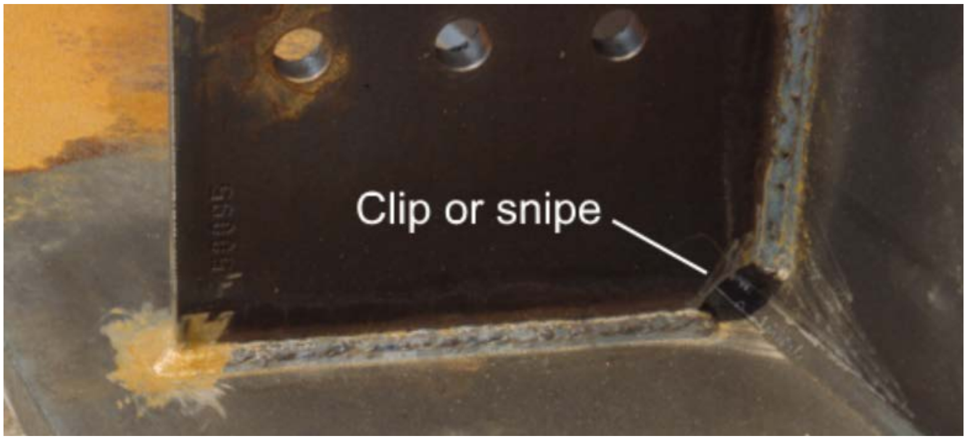
Figure 167. Illustration. Fillet weld holdback, left, and wrapped fillet weld, right.



Source: FHWA

Figure 168. Illustration. Use of weld tabs for fillet weld terminations.

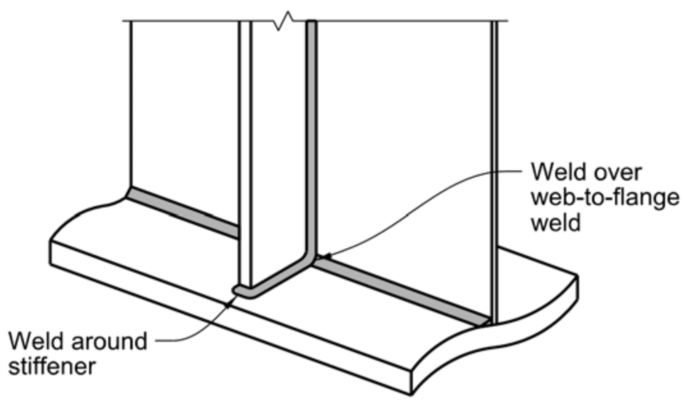
While it is customary to use holdbacks on stiffener-to-web and stiffener-to-flange welds, it is also possible to wrap these fillet welds around the edge of the stiffener, as shown on the connection plate in figure 169. Note that the connection plate has a small cut-out on the inside corner where that connection plate clears the girder web-to-flange fillet weld. These cut-outs are commonly called “clips” or “snipes”. A simple diagonal cut is shown here, but other shapes are possible, depending upon owner standards and fabricator preferences.



Source: FHWA

Figure 169. Photo. Connection plate with wrapped welds.

It is also possible to connect the stiffener-to-flange weld to the stiffener-to-web weld such that the two make a continuous weld over the top of the web-to-flange weld (see figure 170). In this practice, the stiffener would be clipped just enough to clear the web-to-flange weld and facilitate the eventual weld. In practice, the weld would probably not be made continuously by the fabricator because two different welding positions and associated welding procedures are needed for the two different welds.



Source: FHWA

Figure 170. Illustration. Continuous stiffener-to-flange and stiffener-to-web weld.

All three practices mentioned in this section are suitable for bridge performance. For improved constructability, when run-off tabs are not used for fillet welds, the best practice is not to call for wrapped welds but rather call for a holdback termination (see section 8.8.2)

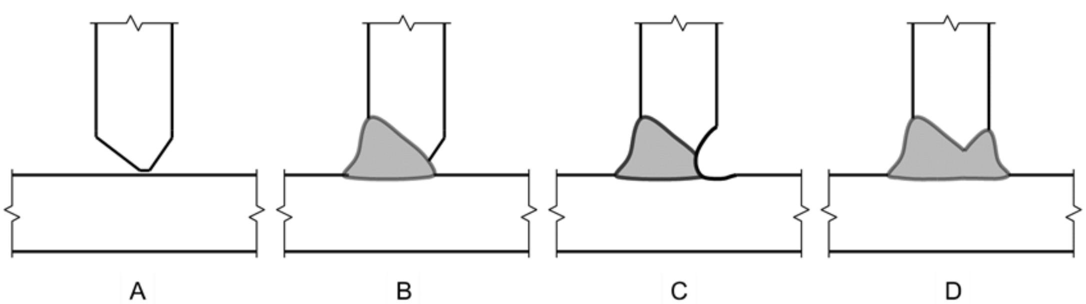
Wrapping welds is not common and generally not recommended because making the welds around the front of the stiffener and over the web-to-flange welds is extra work. However, these welds can be used if sealing the corners and stiffener fronts is desired. If wrapping is considered, be aware that some owners may prohibit wrapping in their standard practices. Research conducted by the University of Texas (Spadea and Frank, 2004) studied the undercut that can result at the top fillet weld toe when welding around a corner such as done in wrapping. The research found that such undercut does not compromise fatigue performance.

9.3.2. Stiffeners and Connection Plates

9.3.2.1. Bearing Stiffener Connections to Flanges

At bearing stiffener-to-flange connections, it is preferable to avoid a CJP weld and use either a finish-to-bear condition alone or a combination of finish-to-bear condition with fillet welds. The finish-to-bear condition transmits compression loads from the bearing stiffener directly to the flange. The use of additional fillet welds is a matter of preference among fabricators; some fabricators prefer to use fillet welds with the finish-to-bear condition to help maintain the bearing condition as fabrication proceeds. Without fillet welds, the bearing stiffener can separate from the flange as a result of other fabrication operations, such as installing bearing stiffeners on the other side of the web. Other fabricators prefer to use the finish-to-bear condition alone, without the additional fillet weld. Therefore, the best practice is to design the connection with a finish-to-bear condition with an optional fillet weld. The fillet welds should be the minimum size required by the Code. The connection at the non-bearing end of the stiffener (typically the top flange) should not require special fit conditions.

With or without an additional fillet weld, use of a finish-to-bear condition offers better constructability than using a CJP weld. Making a CJP weld T-joint takes considerable effort, including the steps shown in figure 171. As shown in “A”, the stiffener is beveled for CJP groove welding. The stiffener shown is beveled on both sides; thinner stiffeners may be beveled only on one side. Then, as shown in “B”, the first side of the stiffener is welded. Next, as shown in “C”, the second-side of the weld is prepared for welding, first with back-gouging and then finishing with grinding. Finally, as shown in “D”, the weld is completed. After welding, the joint is tested with ultrasonic testing per clause 6.7.1, and if defects are discovered, they are repaired. If CJP welds are specified at both ends of the stiffener, shrinkage and restraint at the two stiffener ends create additional problems.



Source: FHWA

Figure 171. Illustration. Making a CJP weld T-joint.

9.3.2.2. Multiple Stiffeners at Bearings

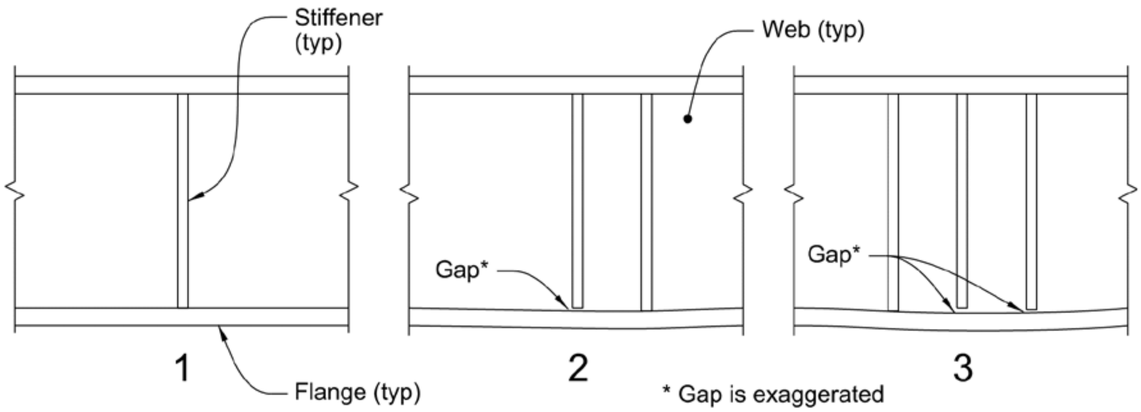
Fitting multiple stiffeners near each other when the stiffeners are required to have a finish-to-bear condition is problematic. Making the finish-to-bear condition requires careful fitting, often including multiple cycles of setting the stiffener in place, checking the condition, grinding or other finishing to adjust the condition, and reinstallation until the fit is correct. Further, stiffeners (unless they are not full-depth) are usually installed under some pressure: they are cut to such length that they fit tightly between flanges, and often jacking of flanges is needed to fit them (see figure 172).

Source: FHWA

Figure 172. Photo. Girder flanges are commonly jacked to fit stiffeners.

If there are multiple bearing stiffeners, installing subsequent stiffeners can compromise the fit of the previous ones. The closer the stiffeners are to each other, the more difficult it is to achieve the finish-to-bear condition for all of the stiffeners. See figure 173 for this example and consider the steps described here. Note that the steps in this example are provided to illustrate the challenges and are only one possible approach a fabricator might use to install the stiffeners.

- At image 1, the center stiffener is installed. The flanges have been jacked as needed to facilitate the installation, the stiffener is tight, and the finish-to-bear condition is achieved.
- At image 2, the right stiffener is installed. As with the center stiffener, the flanges are jacked as needed to achieve fit. In this process, a gap opens at the center stiffener. In the figure the gap is exaggerated to make it apparent, but it is probably very small. However, even a $^{1}/_{32}$-inch gap is enough to compromise the required finish-to-bear condition. Some combination of heating, grinding, and refitting is applied to achieve fit of both stiffeners.
- At image 3, the left stiffener is installed. Once again, flanges may be jacked to install the stiffener, raising challenges at the first two stiffeners that need to be addressed.



Source: FHWA

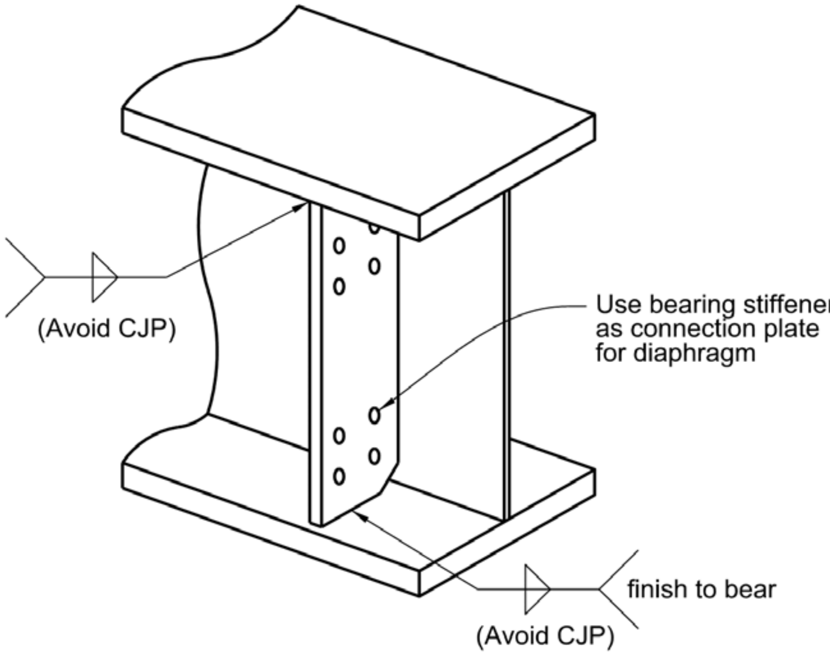
Figure 173. Illustration. Installing multiple bearing stiffeners.

Compounding the problem in this situation is that the underside of bottom flanges at bearing locations must satisfy the tolerances of clause 3.5.1.9. Depending upon the heating, jacking, welding activities, and thickness of the bottom flange, some remediation may be needed after stiffener installation to achieve the required flatness. On thinner flanges, heat can be used to achieve the desired flatness. For thicker flanges, it may be necessary to mill the flange in the bearing area.

Considering the challenge of fitting multiple stiffeners with finish-to-bear requirements, for good constructability, it is best to avoid using multiple stiffeners close together. If multiple stiffeners are needed near bearings because jacking stiffeners are specified (e.g., for use during bearing replacement later in the life of the structure), then it is best not to use full-depth jacking stiffeners with a finish-to-bear condition, if design and owner requirements permit. Rather, it is better to use a normal fit with fillet welds for the jacking stiffeners. Further, the jacking stiffeners can be made partial-depth to avoid the shop jacking that is typically needed to fit full-depth stiffeners (see also 9.3.2.5). Fillet welds introduce much less heat and distortion than CJP welds.

9.3.2.3. Bearing Stiffeners as Connection Plates

At bearings where diaphragms are required, it is not necessary to use separate plates for the bearing stiffener and the connection plate. Often, when a separate plate is used for the diaphragm, the plates are very close, making them difficult to weld. Instead, to avoid unnecessary plate duplication as well as congestion at bearings, a single plate can be used as both the bearing stiffener and the connection plate (see figure 174).

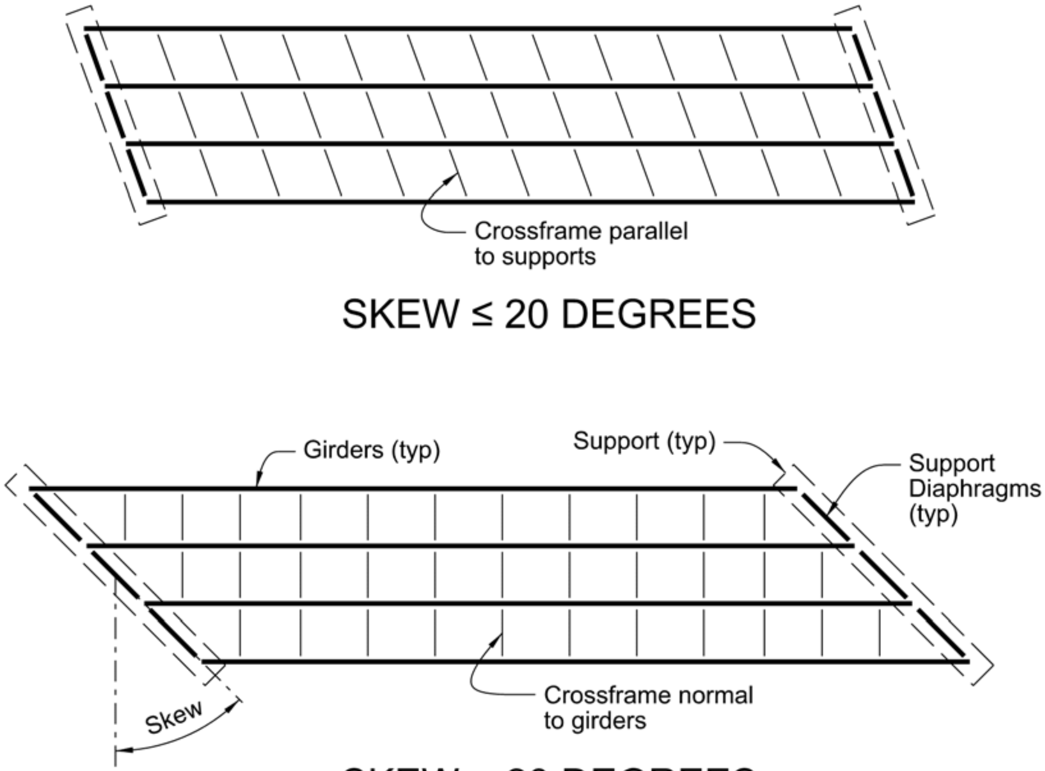


Source: FHWA

Figure 174. Illustration. Bearing plate details.

9.3.2.4. Skewed Connection Plates

On skewed bridges, there is a choice between arranging crossframes such that they are skewed (i.e., parallel to the skewed supports) or instead arranging the crossframes perpendicular to girders. If the crossframes are skewed, then the connection plates should be skewed as well, or bent crossframe gusset plates or bent connection plates should be used (see figure 175).



Source: FHWA

Figure 175. Illustration. Crossframe arrangements on skewed bridges.

When skews are small (less than 20 degrees), then skewed connection plates can be welded to girder webs readily. However, for greater skews, welding skewed connection plates becomes problematic (see discussion in section 8.10). Also, if crossframes are skewed over 20 degrees, the connections associated with keeping the crossframes on the skew angle grow more complicated and crossframes get longer. Therefore, on skewed bridges, the following framing arrangements are recommended:

- For skews less than 20 degrees, arrange crossframes such that they are skewed (i.e., parallel to supports), and provide the option to use a skewed connection plate or a bent connection plate; or use bent gusset plates on the crossframes and keep the connection plate normal to girders.

- For skews 20 degrees and greater, arrange crossframes perpendicular to girders, with connection plates normal to the girders.

For more discussion, see G12.1, *Guidelines to Design for Constructability*, section 2.1.2.6, “Connection of Skewed Cross Frames or Diaphragms”, and section 1.6.1, “Fit and Differential Deflections”.

There are some framing members, such as diaphragms at bearings, which are framed on the skew regardless of the skew of the bridge. In such cases, special attention is needed for the skewed connection between the diaphragm and girder. For recommendations, see section 8.10.

9.3.2.5. Full Depth versus Partial Depth Stiffeners and Tight Fit Stiffeners

Transverse stiffeners are not required to be welded to flanges. Connection plates, in contrast, are required to be positively attached to flanges, per the AASHTO BDS (AASHTO, 2017a). Fabricators differ in their preferences between stiffener connections that are tight-fit with no weld and connections that are fillet-welded (with normal fit). For tight-fit stiffeners with no weld (figure 176), there are savings in not welding and in not repositioning girders for welding. However, achieving tight fit takes effort in the preparation of the end of the stiffener (although not as much as achieving the finish-to-bear condition). Therefore, the best practice in design is to allow the option for both conditions.

Source: FHWA

Figure 176. Photo. Stiffener not welded, full depth.

Stiffeners are sometimes designed to be partial depth. This practice avoids fitting to one of the flanges entirely. However, full depth stiffeners help keep flanges square during fabrication and during shipping, and therefore use of partial depth stiffeners is not recommended.

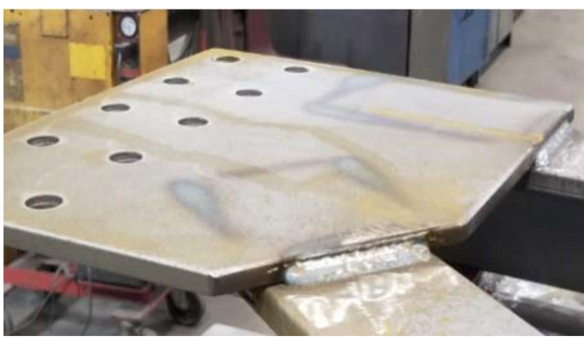
9.4. LONGITUDINAL STIFFENERS

The use of longitudinal stiffeners adds welding labor as well as material cost. For most girder depths, longitudinal stiffness can be achieved by using a thicker web, and this offers better constructability than the use of longitudinal stiffeners. However, longitudinal stiffeners are necessary on some very deep girders, and they are common on edge girders for cable-stayed bridges.

The AASHTO BDS provides some constructability guidance, including that longitudinal stiffeners should be placed on only one side of the web, and the side chosen should be the side with the fewest transverse stiffeners. This guidance reflects the complications and concerns about constraint-induced fracture (see section 6.6.1.2.4 of the AASHTO BDS) that may arise at the welded connection where longitudinal stiffeners intersect with transverse stiffeners. At these intersections, only one of the two elements can be continuous, and the AASHTO BDS recommends that the longitudinal stiffeners be continuous, except at bearing stiffeners. This means the transverse stiffeners will be interrupted at the longitudinal stiffener. Fillet welds should be used for these connections.

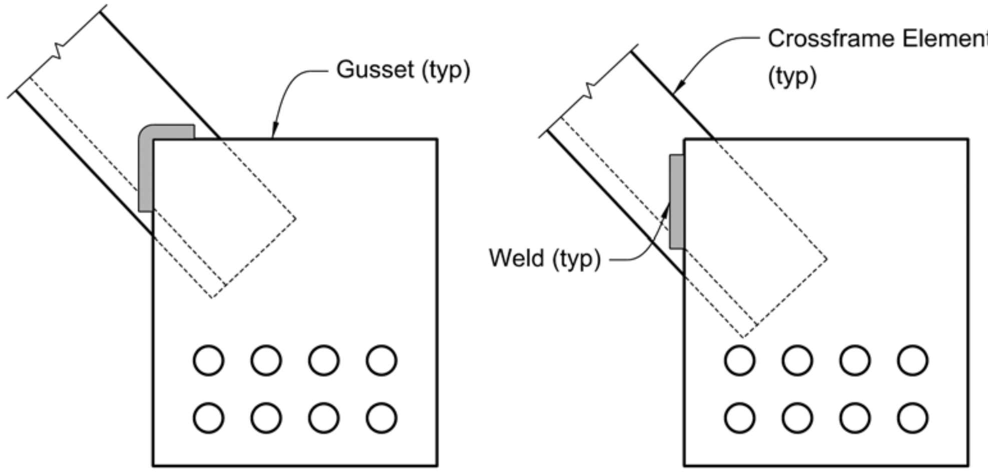
9.5. CROSSFRAMES

For good constructability, gusset plates should be detailed with clipped corners so that in the plane of welding, shapes are square with the gussets, as shown in figure 177. This avoids complications associated with attaching shapes at gusset plate corners, which is shown in figure 178. When gussets are not clipped, the resulting corner requires a weld that goes in two directions, which is like making two welds, as shown on the left in figure 178. Further, as shown on the right in figure 178, crossframe elements can fall very close to corners or at corners, making it more challenging or impossible to provide the intended weld if the intent is to weld on both sides of the corner.



Source: FHWA

Figure 177. Photos. Gussets with clipped corners are preferred.



Source: FHWA

Figure 178. Illustrations. Corners on gussets complicate welding and are not preferred.

9.6. I-GIRDER BRIDGES

Most steel bridges built today are beam or girder bridges, and the majority of these are welded, built-up plate girder bridges (I-girders). Recommendations provided in this section are based on decades of production of I-girder bridges.

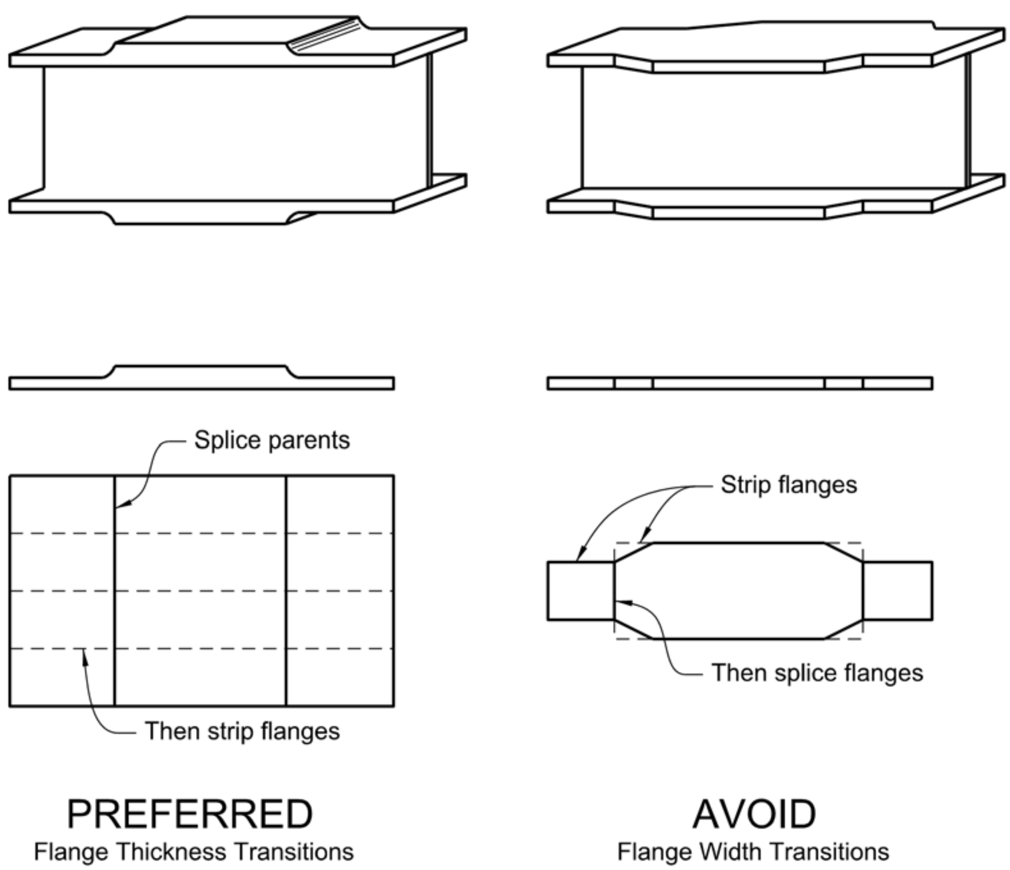
Built-up I-girder bridges can span hundreds of feet and be continuous for over a thousand feet. The length of field pieces (girders as fabricated) for these long bridges are primarily dependent on reasonable dimensions for shipping and erection. Field pieces are usually over 100 feet long, and 130 to 140 feet in length is not unusual. Although the type of plate used to build girders is available in lengths well over 100 feet, shorter lengths are usually spliced to make flanges and webs because of section changes along the girder length and the cost of shipping raw plate. Most steel plate is shipped by rail, with a small percentage shipped by truck. Rail cars are 90 feet long, and have a useful shipping length of 83 feet (longer plate can be shipped by rail but requires additional cars). Therefore, the maximum length of plate typically used by fabricators is 83 feet. Accordingly, only field pieces less than 83 feet long and without thickness transitions are typically fabricated without welded butt splices.

The following are best practices regarding flange and web butt splices in I-girder bridges:

- **Web shop splice locations –** The general practice is to leave shop web splice locations up to the fabricator. It is also a good practice to include a note in the plans that allows optional shop splices. Even if this is not noted, the fabricator will request permission to add the shop splices that are needed. From the standpoint of girder behavior, shop splices are category C details or category B if the weld reinforcement is removed, which is often not the controlling fatigue detail for the girder.
- **Flange splice locations** - In design, shop splices of flanges are typically located where cross-section changes are needed or, less commonly, when the material changes grade (such as girder flanges that are partially grade 50W material and partially grade HPS

70W). However, if flange segments longer than the maximum shipping length have the same cross-section, then shop splices will likely be needed within these segments. If additional shop splices are needed, the fabricator will request them.

- **Flange section transitions** - When changing the cross-section of a girder flange, to the extent possible, a thickness transition is preferred to a width transition. Width transitions may be preferred over thickness transition when a flat top flange is needed (discussed later in this section). In particular, transitions that change both width and thickness should be avoided because if both transition types are used, then the cost of making the transition doubles to include the cost for making both. If possible, it is best to keep the middle flange section the same length from girder to girder. As shown in figure 179, the use of thickness transitions with middle sections of the same length allows flange plates for multiple girders to be spliced before individual girder flanges are cut. This saves time in welding and avoids unnecessary material waste. See further discussion of this in section 1.5.3, "Flange Plate Width," of G12.1 (AASHTO/NSBA Steel Bridge Collaboration, 2016a). Also, for good constructability, avoid over-optimizing flange thickness and keep transitions to a minimum because changes in section may not be worth the cost of adding the splice. For a discussion about good optimizing practices, see sections 1.5.1 and 1.5.2 of G12.1.



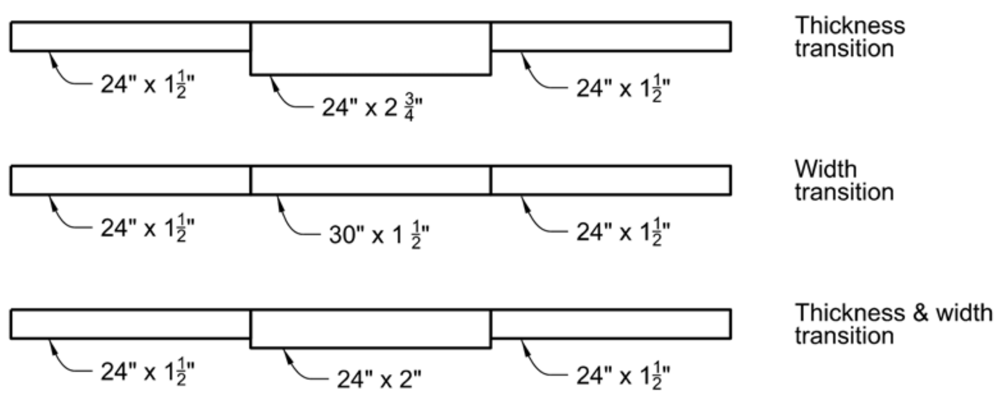
Source: FHWA

Figure 179. Illustration. Flange transitions.

- **Optional shop splice approval and documentation** - Flange and web shop splices added by the fabricator must be approved to ensure proper documentation of the splices. This can be handled through RFI, with the splices then shown on the shop drawings, or it can be handled through the shop drawing approval process.
- **Flange transition options** - Specific weld details are required at flange transitions to facilitate good fatigue performance. Standard flange transition details are found in D1.5 figures 2.7 and 2.8 and are required in clause 2.17.5. Therefore, the best approach for designs in accordance with the Code is to be silent about flange transition details. Figure 180 shows the best practice for indicating flange thicknesses and associated transitions; the special treatments of the Code do not need to be shown on design drawings. However, they will be shown on shop drawings and can be reviewed during shop drawing review and approval.

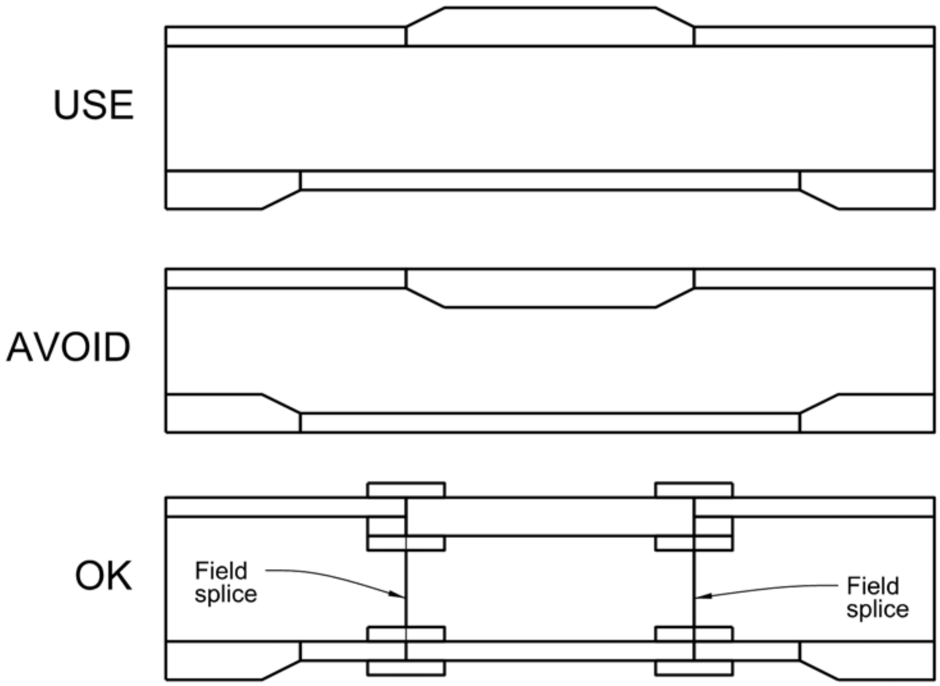
  Splices that involve increases in flange thickness should be oriented away from the web, with the two sections aligned flush to the web as shown in figure 181; the web should not be cut to accommodate the thicker plate. This simplifies web cutting, web-to-flange fitting, and subsequent welding; it facilitates the use of mechanized equipment for the long, continuous web-to-flange welds (see figure 182). Additionally, the use of such linear fillet welds facilitates better quality than the interrupted fillet welds that would be required if the thickness transitions were cut into the web.

  If a continuous top-of-flange elevation is needed, section changes can be made with width transitions. If width transitions are insufficient, and thickness transitions are needed, these can be used by locating them at field splices as shown in the third image in figure 181, which allows for linear web-to-flange welds on field pieces.



Source: FHWA

Figure 180. Illustration. Types of flange transitions.



Source: FHWA

Figure 181. Illustration. Flange thickness transitions.

Source: FHWA

Figure 182. Photo. Long, continuous web-to-flange fillet welds.

- **Flange Bends** - Girder flanges sometimes have bends or corners, such as for haunched girders. When flanges have corners, or bends that are like corners, they may be bent or welds may be used to produce the corners (figure 183). Generally, fabricators prefer to bend the corners. However, if the bend radius is small, welding may be better, or may be required because of limits on bending. Flange corners with an inside bend radius tighter than the AASHTO design limit of 5*t* (AASHTO, 2017a) are not likely to be needed, and usually a 5*t* bend is readily achievable on flanges. However, in addition to the radius, the fabricator's preference will be based on the flange thickness and the equipment available for bending. When there is a question about whether or not a weld is needed to make a bend or corner, it is preferred that an optional welded splice be shown in the plans.

Source: FHWA

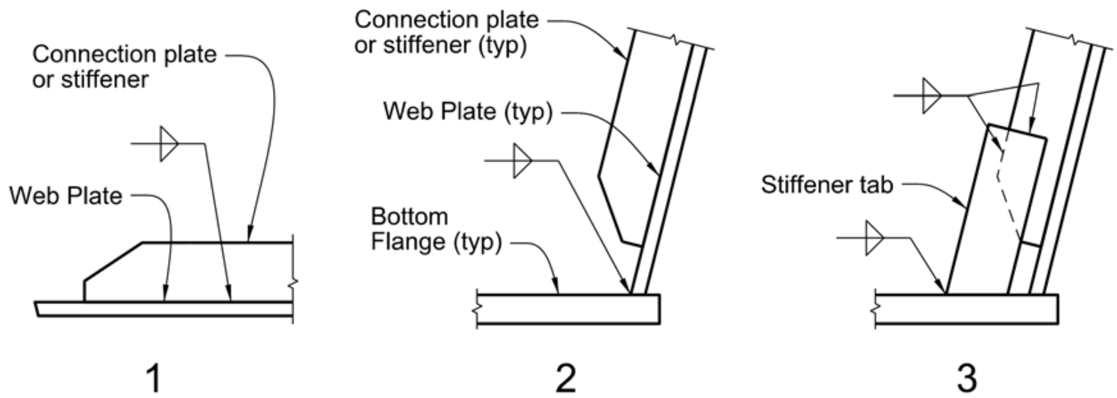
Figure 183. Photos. Bent flanges (left) and welded corners (right).

9.7. TUB GIRDER BRIDGES

Many of the I-girder recommendations also apply to tub girders; those are not repeated here. Recommendations that are specific to tub girders are addressed in this section.

9.7.1. Stiffener to Bottom Flange

In tub girders, stiffeners are usually attached to webs before the webs are joined to the bottom, to avoid out-of-position welding when attaching the stiffeners, or rolling the tub to improve the welding position. However, this means that at the bottom flanges, the stiffeners interfere with making the web-to-flange welds. Therefore, it is preferable to leave the stiffeners short and then attach them to the bottom flange with a tab after the web-to-bottom flange weld has been made, as seen in figure 184, which is based on the detail in G12.1 (AASHTO/NSBA Steel Bridge Collaboration, 2016a). Figure 184 illustrates the sequence of fabrication for the tab. In "1", stiffeners are shown attached to tub webs. In "2", the web-stiffener assemblies are fillet-welded to bottom flanges. In "3", after web-to-flange welding is complete, the tab is attached to the stiffener and to the flange.



Source: FHWA

Figure 184. Illustration. Tub girder stiffener tab attachment.

9.7.2. Bottom Flange Stiffening

Because tub girder flanges are rather wide, the plate may not need to be very thick to meet design requirements. However, thin tub girder bottom flanges will warp during fabrication, requiring additional effort to flatten them after fabrication is complete. It is possible to add longitudinal elements to the bottom flanges to stiffen them, but this also adds additional cost. The best practice is to use a bottom flange that is at least 1 inch thick to avoid warping without the need for longitudinal stiffening.

9.8. CLOSED BOXES

Distinct from open boxes used as girders, such as in tub girder bridges, closed boxes offer their own unique challenges. Common applications for box sections include pier caps, tie beams, and truss and arch members. Recommended practices are discussed below.

9.8.1. Design Considerations

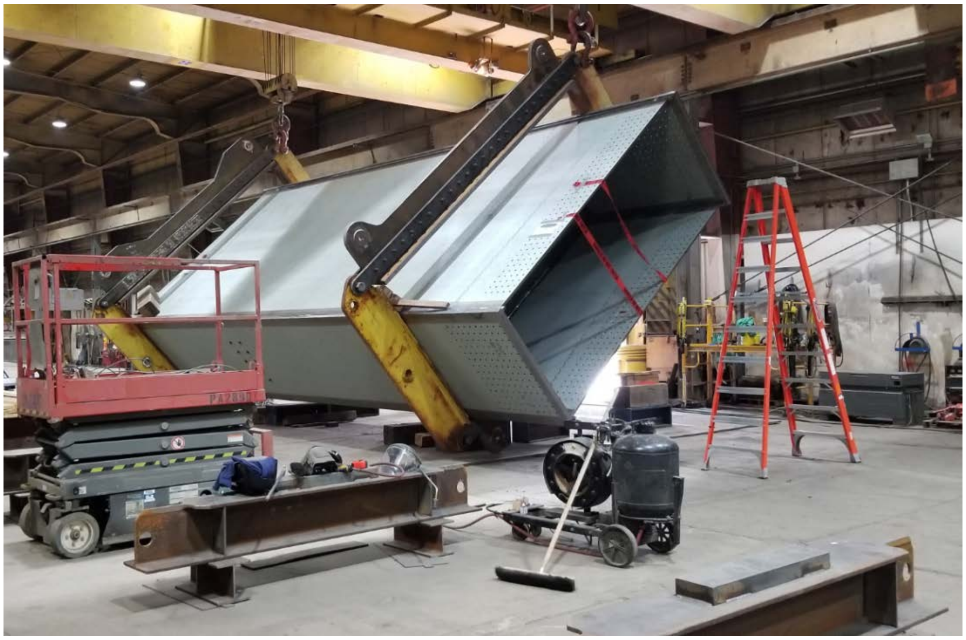
Special considerations for box fabrication and associated welding details include the following:

- **Accessibility –** The accessibility constraints discussed in section 9.2.6 should be considered when designing boxes. Welders must be able to bring equipment into the box to the location to be welded. Within the box, welders must be able to reach the connection to be welded, and see the puddle when welding.
- **Confined space safety** - For large boxes that require work inside the box, fabricators must ensure the safety of their welders and other personnel when they work in confined spaces. OSHA publishes safety standards that address welding and other fabrication activities in confined spaces (Code of Federal Regulations, 19 CFR, Part 1910, Subpart Q, “Welding, Cutting, and Brazing”). Considerations include environmental conditions, welding fumes, equipment installation, temperature, electrical (voltage) limits, moisture (including perspiration), and electrical shock. Furthermore, the fabricator must be able to

extract workers from confined spaces in the event the worker is injured or has a health emergency.

- **Box movement** - Because they are generally heavier and more bulky than I-girders, boxes require special care and more effort to move and to rotate. Rotating boxes is particularly problematic when they are curved due to the way the box center-of-gravity shifts as the box is rotated. Typically, special fixtures are used to secure large boxes for rotation, as shown in figure 185.

For both safety and efficiency reasons, it is better to keep as much welding outside of the box as possible. When considering welds that are to be made inside boxes, the ideas discussed in this section should be considered.



Source: FHWA

Figure 185. Photo. Rotating a large box using custom handling fixtures.

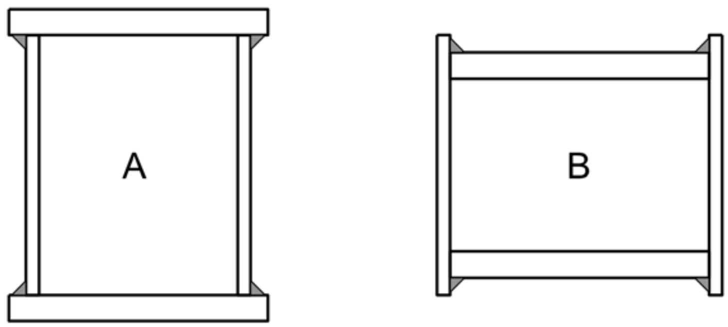
9.8.2. Welded Box Corners

The engineer has many options in the design of welded corner connections on boxes. As described in section 9.2.1, good constructability for the corner welds of boxes is best achieved with fillet welds, then PJP welds, and then CJP welds. Further, constructability is better when welding can take place outside of the box rather than inside. Therefore, the constructability of various box corner choices is ranked as follows below, with best constructability listed first.

- **Outside fillet welds only** - The way to make box corners with the best constructability is to use outside fillet welds only, as shown in figure 186. Note these features:
  - All welding is performed outside of the box.
  - In "A", the flanges extend beyond the webs, creating a small lip; similarly, in "B", the web extends beyond the flanges. This lip provides room for the fillet weld and

should be at least twice the size of the fillet weld leg to help support the weld deposit and, when SAW is used, to support the flux.

- No bevel is required for either plate.

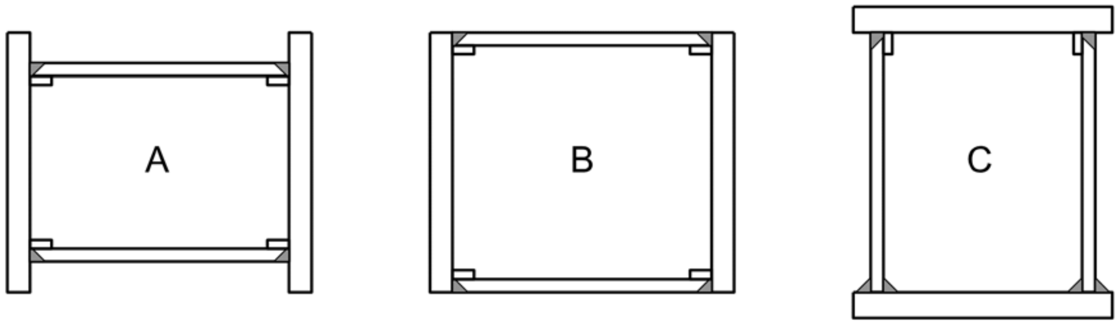


Source: FHWA

Figure 186. Illustration. Box corners with only outside fillet welds.

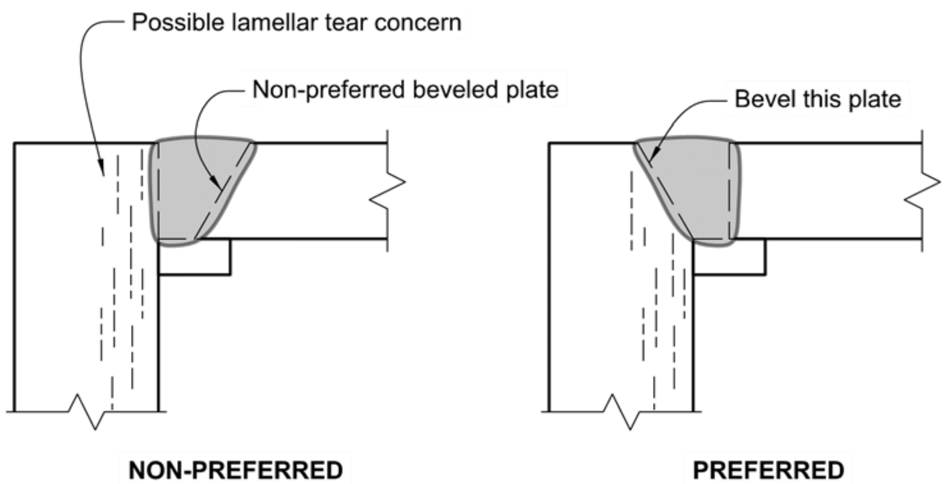
- **Single-sided CJP welds** - If CJP welds at the box corners are necessary, then they should be made from one side with backing as shown in figure 187. Note these features:
  - Welding of the CJP welds from one side allows all longitudinal welding to be performed from outside of the box.
  - One plate at each corner must be beveled. The bevels are shown in this sketch but do not need to be shown on design drawings. Rather, the welding symbol should call for a CJP weld; the fabricator will then prepare the bevel in accordance with the details associated with the joint that the fabricator chooses from the Code. At a corner, if for some reason lamellar tearing is a particular concern in a particular plate (see section 5.4.1.5), then that should be the plate that is beveled. See figure 188.
  - Backing is required for the standard single-sided CJP groove welds listed in the Code. The backing will remain in place once the box is closed and welding is complete. While a longitudinal CJP weld with backing left in place is a fatigue category B' detail, this rarely if ever is a limiting factor. See section 8.6 for a discussion about permanent backing and section 8.3.5 regarding tack welds outside of joints.
  - Webs can either extend beyond the flanges, as in “A”, or be flush to the flanges, as shown in “B”.
  - As shown in “C”, double-sided fillet welds can be used at one side of the box. Generally, fillet welds are preferable to CJP welds, but making fillet welds inside boxes is not desirable once diaphragms are in place because diaphragms interfere with the use of equipment to make the internal web-to-flange fillet welds continuously. In option “C”, fillet welds on one side of the box can be made before diaphragms are added.

- Double-sided fillet welds may not be used effectively at both flanges because once the box is closed, the internal web-to-flange fillet welds may not be readily made (depending on the size of the box).



Source: FHWA

Figure 187. Illustration. CJP groove weld box corner details.



Source: FHWA

Figure 188. Illustration. Bevel location for lamellar tear concern.

9.8.3. Diaphragms in Boxes

In addition to serving their purpose in the performance of the box in service, internal diaphragms facilitate building boxes by helping maintain the shape of the box during fabrication. Fitting and welding the diaphragms to the box web and flanges becomes increasingly difficult with the addition of both flanges and webs. If the box is large enough for a welder to get into the box (and there is access), then diaphragms can be welded to both webs and both flanges. If the box is too small for a welder to get inside, then the diaphragms can be welded to the first three sides of the box, but the final side should be left unwelded if possible, depending on design requirements. The preferred approach is to avoid finish-to-bear conditions inside boxes, if possible. Achieving a finish-to-bear condition often requires many cycles of installing plates, checking the condition, and then pulling the plate out for grinding to improve the bearing condition.

CHAPTER 10 - THE ENGINEER'S ROLE IN WELDED FABRICATION

The engineer, who may be the owner or a consultant acting on behalf of the owner, has distinct roles to play in the design and fabrication of welded steel bridges. How effectively the engineer fulfills these roles affects the cost, quality, and delivery of the structure. These roles are performed in three stages, and may involve different individuals or the same individual:

- **Design** - In the first stage, the goal of the engineer is to establish a design that is safe, durable, and constructable, as well as to clearly and effectively communicate the intent of the welded design in the contract drawings and specifications. Strength and fatigue design of bridge members is an important part of this role but not within the scope of this manual; refer to the AASHTO BDS for details (AASHTO, 2017a). However, the design of welded bridges also includes proper use of weld symbols, arranging plates and shapes to facilitate fabrication, and selecting weld details to facilitate welding constructability, which are within the scope of this manual.
- **Prefabrication -** In the prefabrication stage, the engineer answers detailer, fabricator or contractor RFIs, providing replies that support shop drawing development. The engineer reviews and approves shop drawings and WPSs.
- **Fabrication -** Once shop drawings and WPSs are complete and approved, fabrication can begin. In this stage, the engineer may still need to address questions that come up on the shop floor from fabricators or inspectors regarding welding details, and approve of fabrication and repair procedures.

On many projects these engineer roles are played by different individuals. In particular, the engineer responsible for design is usually different from the engineer responsible for welding procedure approval. If so, good communication among the individual engineers is important.

10.1. WELDING CODE SELECTION

For bridge welding, the question of which welding code to specify generally does not need much consideration—the Bridge Welding Code (AASHTO/AWS, 2015) should be specified, and this is generally a complete solution unless there are special situations where materials are used that are not covered by the Code. When such materials are used, the engineer may specify other welding codes for those materials or provide direction for use of the unlisted materials under the Code in accordance with clause 5.4.3. Clause 5.4.3 provides an overview of what to consider and address; this information is similar to section 10.1.3, which provides items to consider when welding outside of the Code or of other D1 codes.

When specifying a welding code, it is the engineer's responsibility to ensure the code is appropriate for the item to be welded. In some respects, the code titles make their application self-evident: e.g., the D1.9 *Structural Welding Code—Titanium* is intended for titanium structures (AWS, 2015a). However, not every code title sufficiently addresses its scope and associated applicability. Rather, the scope of each code is published in the first clause of the document.

10.1.1. Use of the Bridge Welding Code for Structures Other Than Vehicular Bridges

It is important to recognize that, as described in the Code's scope, the Bridge Welding Code is specifically written for highway bridges. It is also well suited for welding railroad bridges. The practices in the Code are adopted on the basis of helping ensure the long life of these large structures under the significant cyclic loads they endure. Further, the geometric tolerances in the Code are suited to highway and railroad bridges. Other transportation-related structures, such as sign supports and bridge rails, are not within the Code's scope. The Code does address some non-bridge "ancillary" products in clause 1.3.7; the fundamental difference under this clause is that WPSs for the products listed do not need qualification tests. However, the information within clause 1.3.7 is not complete for fabrication of these products in the same way that the Code applies to bridges.

Specifying the Code on other types of structures may be a contract error and lead to ambiguities. For example, the Code specifically addresses materials typically used in bridges (clause 1.2.2). If the Code is specified for a product that contains other materials and no other contract direction is provided about what welding consumables to use, such specification is a contract error. Further, the Code is specific to materials equal to or greater than ⅛ inch thick (clause 1.2.3) and provides dimensional tolerance information, such as camber and sweep, which apply to bridge members; these tolerances may be unsuitable or not even possible to apply to other products.

Under the AASHTO pedestrian bridge design specification, AWS D1.1 (AWS, 2015) is to be used for pedestrian bridges, and this approach is sound. The Bridge Welding Code was not developed with pedestrian bridges in mind. The loads associated with pedestrians on bridges do not create the stress ranges that are associated with highway fatigue loading; occasional vehicular traffic on a pedestrian bridge will not constitute a sufficient number of loading cycles to create a fatigue-sensitive situation.

In addition, many pedestrian bridges incorporate AESS (architecturally exposed structural steel) members, a condition not addressed in the Code. Tubular steel, a material form not currently addressed in the Code, is often incorporated into pedestrian bridges. The thickness of materials, grades of steel and other factors associated with pedestrian bridges may not be covered by the Code.

10.1.2. Use of other D1 Codes

For non-bridge structures, the use of other applicable D1 codes is appropriate (section 1.4.1.2). For steel structural products, such as sign, signal, and light supports; joints; and railing, D1.1 is applicable. Refer to Appendix B for a comparison of provisions in AWS D1.1 and AASHTO/AWS D1.5. Similarly, D1.2 and D1.6 are applicable for aluminum and stainless-steel structures, respectively. The decision of which other code or codes to specify should include the following considerations:

- **Material** - Is the base metal to be welded found in the code being considered? Primarily this is important because the applicable code will identify the welding electrodes that are suitable for use with the material as well as aspects of welding that are specific to that material for achieving quality, such as preheat requirements.

- **Consumables -** Are the desired welding consumables to be used found in the code being considered? Use of the appropriate welding consumables with the project materials is crucial. Generally, these go hand in hand with the materials: if a code includes a material, the code will also include consumables that are appropriate for use with that material.
- **Connections -** Are the types of connections to be welded found in the code being specified? This is important because the code requirements apply to preparation of the connection details and, in the case of bridges, the applicable non-destructive testing based on the type of connection. A common mistake in the application of the Bridge Welding Code is specification of the Code for tubular fabrication. The 2015 edition of the Code does not include tubular materials, nor does it address tubular connections.
- **Nondestructive Examination** - Is specific nondestructive examination required, and if so, is this testing and associated acceptance criteria found in the code? For bridges, the Bridge Welding Code mandates required methods, including frequency of testing and acceptance criteria. By contrast, D1.1 includes the same methods but does not mandate their application. Rather, if D1.1 is specified on a project and NDE is desired, the contract documents must explicitly specify the method and frequency of testing. Acceptance criteria for traditional NDE methods are found in D1.1; if such methods are specified, the associated acceptance criteria will apply unless otherwise addressed in the contract documents.
- **Thickness –** Is the Code applicable to thickness of material being used? Like D1.5, the welding code in consideration may have specific thickness limitations.
- **Welding Processes –** Are the appropriate welding processes for the material in question found in the Code being considered?
- **Loading Condition –** Does the Code in consideration address any specific applicable loading condition? For example, D1.1 addresses both static and cyclic loading conditions, and D1.8 addresses seismic loading.

If the Code is partially suitable for the component to be fabricated, the Code can be specified in conjunction with provisions from other standards. For example, if a steel bridge is to be partially fabricated using tubular components, the Code may be applied for most of the project with supplemental requirements, such as use of D1.1 for the tubular welding, in the contract documents. However, use of this approach requires diligence in preparation of thorough specification language that clearly delineates the specific application of each code and avoids conflicting requirements.

10.1.3. Welding Outside of the D1 Codes

If a D1 code is not available for use on the product to be welded, additional criteria specific to the intended application, in addition to those listed in section 10.1.2, must be addressed, and the following questions should be considered in developing the necessary criteria:

- If the material is not listed in an AWS welding code, why not? Is the reason something that makes the material inappropriate for the application (e.g., bridge or sign structure)? Are there adequate controls on the mechanical properties of the material? The properties of interest certainly include minimum yield and tensile strength, and elongation, but may also extend to maximum yield and tensile strengths, and minimum fracture toughness levels.

- Are there adequate controls on the chemical composition of the steel from a welding perspective? This should include minimum and maximum levels for carbon, sulfur, phosphorus, and other chemical elements. Compositional limits may need to be placed on elements that are not listed in the material specification (e.g., ASTM specification) as well.
- Are there adequate controls on the physical dimensions of the steel material? This is particularly important for non-plate products.
- How is the material processed? Is it hot-rolled, quenched and tempered, or made using some other process? How would any heat treatment of the steel be affected by welding and cutting processes?
- Is weldability data available?
- What other projects have used this material? Does it have a history of successful implementation and service? If so, is there anything to be learned for the welding experience on those projects?
- What are the steel producer's recommendations regarding the welding of the material? Considerations include:
    - Minimum preheat temperature levels
    - Maximum interpass temperature limits
    - Filler metal requirements
    - Any restriction on thermal treatment of the steel
    - Heat shrinking temperature limits
    - Susceptibility to getting defects during welding
    - Use of the material for the application intended

The preceding discussion provides general criteria that should be considered if the material to be used is not found in the D1 codes, but it may not be comprehensive. This information is similar to how the Code addresses the use of unlisted materials (clause 5.4.3); namely, the engineer must consider these issues. The information in clause 5.4.3 may also be helpful when considering use of an unlisted steel and a non-D1 code.

WPS qualification testing will be required for most WPS, and the PQR will provide some data regarding the weldability of the steel. However, the WPS qualification tests do not constitute true weldability tests. In general, the procedure qualification test plates prescribed in D1.5 will not be as restrained as some production weldments. WPS qualification test plate are not required to be made from the same thickness of steel, nor from the same heat of steel that will be used in production. For these reasons and others, qualification tests may not have sufficient restraint to predict cracking that could occur in actual applications. Accordingly, a thorough examination of weldability may be necessary for the use of an alternate material. In other cases, the alternative steel may be sufficiently similar to a listed steel as to preclude the need for specialized testing. As an example, when high-performance steel was developed and introduced for bridges in the 1990s, the FHWA partnered with AISI, and the US Navy, to conduct sponsored research that included weldability studies. The welding tests performed in the course of this research exceeded the current requirements of the Code. Welding guidelines were developed from the research and published in a welding guide specification (AISI, 2003); later these were adopted into the Code.

10.2. CONTRACT DOCUMENT REQUIREMENTS

There are a number of aspects of the Bridge Welding Code that require proper attention by the engineer in design and creation of the contract documents. These include:

- Designation of tension zones: NDE and the need for CVN toughness depend upon the state of stress of the weld and the material being joined (chapter 6).
- Designation of fracture critical (FC) members and FC zones (section 7.2.4).
- Designation of primary and secondary members: there are fabrication limitations (such as surface finish requirements) and inspection requirements that depend upon these designations.
- Restrictions for optional shop splices: given the length of typical steel bridge members, fabricators commonly add welded shop splices to produce long or odd-shaped plate components. If there are any restrictions on where shop splices can be located, these should be clear in the plans.
- Backing allowances: the Code has requirements for when backing can stay in place and when backing must be removed (section 8.6). If it is necessary to leave backing in place because of some access restrictions when it is otherwise required to be removed, indicate this allowance definitively in the plans (section 8.6.1). When backing is shown in the plans without a removal note, this implies that backing is to be left in place, but to ensure understanding, it is good practice to also include a note that says backing is to remain in place.
- Special joint details: provide details if a special CJP or PJP weld is required.

Any special welding-related requirements that are not addressed by the Code or are different from Code requirements must be included. For example, include any special NDE requirements, such as special NDE techniques, testing frequencies, or alternate acceptance criteria. More comprehensive details about information needed on welded steel bridge plans can be found in AASHTO/NSBA Steel Bridge Collaboration G1.2, *Design Drawing Presentation Guidelines* (AASHTO/NSBA Steel Bridge Collaboration, 2003).

10.3. SHOP DRAWINGS

The purpose of shop drawings is to provide complete fabrication instructions to the shop before work begins, including clear welding instructions. The preparation, review, approval, and release of shop drawings is generally on the project critical path. Once work does begin, interruptions to production must be avoided, so information provided in the shop drawings must be complete and correct before the shop drawings are released to the shop.

While preparing shop drawings, the fabricator (or the fabricator's detailer) will interpret the design and work with the shop to make decisions about how to produce the welds and then present this information on the shop drawings. Later, for most owners, shop drawings will be submitted for review and approval. AASHTO/NSBA Steel Bridge Collaboration G1.1, *Shop Detail Drawing Review/Approval Guidelines* (AASHTO/NSBA Steel Bridge Collaboration, 2000), provides useful recommendations for the review and approval process. Given that shop drawings are on the project critical path, the engineer should be expeditious when addressing RFIs and reviewing shop drawings. In both cases, if something in the fabricator's submittals is not clear, it can be helpful to discuss this by phone rather than requesting clarifications by paper or email and waiting for a written response. Further, if there are no major concerns with shop drawings but there are some minor adjustments that can be specified by the engineer, it is helpful to approve the drawings "as noted", indicating the necessary minor adjustments to help keep the project on schedule.

10.4. WELDING PROCEDURE SPECIFICATION (WPS) REVIEW AND APPROVAL

Under the Code, welding may only be performed in accordance with an approved WPS (clause 1.9). Some owners require WPSs to be approved on a per-project basis; if so, or if there are unique welding requirements on a project for which the fabricator does not already have an approved procedure, then approval of WPSs is on the project critical path.

WPS review and approval is addressed in section 4.5 of this manual. As with shop drawings, the engineer should provide expeditious review and approval of the WPSs, and to facilitate this, it is beneficial for the engineer to communicate with the fabricator by phone if any questions come up during review of the WPSs. Unless it is required by the owner, it is not necessary for the engineer to approve WPSs in the context of specific welds on the project, except for unusual details. Rather, WPSs can be reviewed in and of themselves for conformance with the Code and later, on the shop floor, inspectors can verify that approved WPSs are used in appropriate applications.

The fabricator may choose to put WPS identifications in welding symbol tails on shop drawings as the means of communicating to the shop which WPSs are to be used in each application. Regardless of how the choice of WPS is communicated to welders, the fabricator is responsible for ensuring that all bridge welding is performed in accordance with an approved WPS.

10.5. TOLERANCES

The Code has tolerances for many aspects of welded bridge fabrication. Tolerances provide helpful flexibility in the fabrication of the design; this flexibility is necessary for effective project execution given the scale of bridge elements, the allowable variations in steel materials, the imprecise predictability associated with welding shrinkage and distortion, and the inexact science of heat for fabrication and correction.

When the Code is specified on a design, the engineer should ensure the Code is suitable for the design. The Code primarily covers the situations that are encountered on typical I-girder bridges. If a project has details or component arrangements for which Code tolerances are unsuitable or that are not addressed by the Code, the engineer should address these situations with special tolerances in the contract documents. If these situations are not recognized until after the project begins, appropriate tolerances should be worked out between the engineer and the fabricator.

Tolerances are contractual and therefore enforceable and set useful acceptance boundaries for inspectors. However, during fabrication, issues may arise for which tolerances do not exist or may not be applicable. Further, in some cases, repairs may be more deleterious to a given situation than the nonconformance itself. For example, specific fillet weld holdbacks (section 8.8.2) are intended to facilitate weld quality by preventing undercut. Usually, holdback dimensions are provided with a tolerance. If a holdback tolerance is violated but the fillet weld is of good quality, then it is probably better to leave the fillet weld "as is" rather than grind away the end of the weld. Therefore, some judgment is prudent in the application of tolerances. This philosophy is embodied in clause 1.1.2 of the Code, which states:

> *The fundamental premise of the code is to provide general stipulations applicable to any routine bridge situation. Acceptance criteria for production welds different from those described in the code may be used for a particular application, provided they are suitably documented by the proposer and approved by the Engineer.*

In pursuit of the best resolution for a tolerance violation, the engineer should be mindful of the following:

- Code tolerances are primarily based on I-girder bridge fabrication (stringer bridges composed of I-shaped plate girders) and may not be applicable to other types of bridge components.
- Tolerances are developed generally, intended to cover a broad range of welding conditions on a broad range of bridge weld details.
- The tolerances in the Code are developed and adopted by committee, with discussion about what allowance will be suitable for most conditions, providing enough allowance for the product to perform as intended while also allowing the work to be performed with minimal disruption and avoiding unnecessary corrections.
- Like the other quality requirements in the code, tolerances are generally based on workmanship and not fitness for purpose (see discussion in section 5.1)—i.e., acceptance of a product that is outside of tolerance does not necessarily compromise the performance of the product. However, this also does not mean that there is always further latitude in the application of tolerances.
- The lack of a tolerance does not mean the tolerance is zero; the lack of a tolerance indicates that a tolerance has not been established.

10.6. NONCONFORMANCES

The existence of a nonconformance does not necessarily mean the work must be remediated. Rather, in some cases the engineer may determine that the nonconformance is better left "as is". If a design configuration is unusual and results in recurring nonconformances, new tolerances should be established and agreed upon between the fabricator and the engineer. Cable anchorage assemblies can be a good example of this: such anchorage assemblies tend to be complicated combination of plates joined with a mixture of fillet and full penetration welds, and once the anchorage assemblies are complete, they must properly fit to cable stay edge girders. In design, the engineer does not have knowledge of the fabricator's equipment, methods, and sequence of assembly, all of which will affect how the assembly will distort during fabrication. Even the fabricator may not be able to predict how the assembly will distort; further, this distortion will vary along the bridge and anchor assembly lengths change. The best practice is for the engineer to get as much input as possible from industry during design, and later, once fabrication begins, the fabricator and engineer should work together to establish and refine tolerances that will both facilitate fabrication and also achieve the desired design. Further, if fabricators encounter trouble achieving project tolerances on complex work, they should discuss the issues with the engineer early in the project rather than continue errant work without feedback from the engineer.

In some cases, as fabricator learn how members behave under fabrication, the fabricator may propose changes or exceptions to project tolerances. The engineer may use past experience, experimental evidence, or engineering analysis as the basis for the alternative acceptance criteria. Fabricator proposals for alternative processes or to address non-conformances must provide adequate information to allow the engineer to thoroughly evaluate the proposal.

Handling of nonconformances depends on the severity of the issue and on owner practices. Many welding nonconformances, such as minor profiles nonconformances or defects discovered by NDE, will be corrected by the fabricator without engaging the engineer. Other nonconformances, such as proposals to leave mislocated or miscut parts "as is", require engineer approval. The engineer must be involved in the resolution of any condition that results in a change to the geometry of the bridge Examples of where the engineers should be notified include addressing errant bolt holes, mislocated parts, and miscut members. The engineer should review both the geometric appropriateness of the repair, the process for making the repair, and any special NDE required to verify that the repair is adequate. Section 5.8 of this manual provides additional useful details about welded repairs.

Though preferably minimized and avoided, nonconformances are a normal part of fabrication. When nonconformances reach the level of engineer engagement, the general practice is for the fabricator to submit a proposal to the engineer for approval. When repairs involve welding, there are many items that should be addressed, including preparation for welding, the WPS to be used, and any NDE to be applied to help verify that a sound repair is achieved. Comprehensive repair guidance is outside the scope of this manual, but a good resource is AASHTO/NSBA Steel Bridge Collaboration G2.2, *Guidelines for Resolutions of Steel Bridge Fabrication Errors* (AASHTO/NSBA Steel Bridge Collaboration, 2016). With the engineer's concurrence, some fabricators prefer to develop standard repair procedures that the engineer can pre-approve for repetitive issues. Such preapproved procedures may include a conditional provision that the fabricator may proceed with the repair once the inspector verifies that the preapproved procedure is appropriate for the repair in question. This practice is encouraged to help expedite nonconformance approvals.

10.7. PROPOSED ALTERNATIVES

Proposed alternatives are common in bridge fabrication, including requests from fabricators to use one of the many alternatives that are explicitly permissible by the Code when approved by the engineer, and also requests to use new processes, practices, or, less commonly, alternative materials. Generally it is good practice to be open to the use of alternatives. As Omer Blodgett, structural welding pioneer and author of *Design of Welded Structures*, (Blodgett, 1966) was noted for saying, “Codes always lag industry.” It is typically through the industry’s development and implementation of new practices and technology that the state of the art advances, and Code changes follow later.

However, practices and processes that are not listed in the Code may be absent for a very good reason, and there are also reasons why some practices are only allowed by the Code with the engineer’s approval. Thus, the engineer must research the situation and approve departures from the Code with a full understanding of any implications.

As an example of a technology that can be very helpful, consider phased array ultrasonic testing (PAUT, section 6.6). The Code prescribes radiographic testing (RT) for butt joints, but if a butt splice is skewed, it may not be possible to test the splices effectively by RT depending upon the amount of skew. For such joints, PAUT is an excellent alternative. This particular example is a case of using a process that is already in the Code (PAUT) as a substitute for another required process (RT).

10.7.1. Welding Processes

The Code specifies welding processes that are approved for use in bridge fabrication and does not provide a prescribed path for the approval of alternate welding processes. Alternate welding processes could involve a variety of new or different controls that need to be monitored, and the evaluation of such variables should be part of the alternate process approval activity. If alternate welding processes are to be used, there should be thorough research conducted or an established use history of the process that demonstrates the suitability of the process for bridges. As an example, when ESW-NG (section 3.7) was developed for bridges, the FHWA funded research that studied the use of the process to achieve the properties needed for bridge welds (FHWA, 1987, 1987a, 1987b, 1987c and 1996). The research findings were used to develop the provisions adopted by the Code for the new ESW process.

An example of a process that is almost always inappropriate for conventional bridge structural members is short-circuit transfer (GMAW-S, section 3.5.7). Clause 1.3.6 of the Code says that GMAW-S may only be used with the engineer’s permission. Given its very low heat and the thick materials that are typical in bridges, use of GMAW-S is inappropriate for bridge use except for special circumstances (such as for the open root joint of a tubular connection). If GMAW-S is proposed, the engineer should carefully consider the components to be welded and impose strict controls to ensure the process is used properly and is only used on specific materials and components. This example also relates to specifying the correct welding Code on the project: there may be a non-bridge product for which use of GMAW-S is normal or even preferred, but if the Bridge Welding Code is specified for the product, GMAW-S would not be allowed.

10.7.2. Alternative Practices

Various alternatives to standard welding practices may be used when approved by the engineer. The following sections describe alternatives and issues that should be considered.

10.7.2.1. WPSs Qualified to Other Standards

Most welding codes and specifications require similar determination of weld soundness and mechanical properties as the Bridge Welding Code, although CVN testing and all-weld-metal tensile testing are less common. If the required tests and criteria are the same as those required by the Code, acceptance of the alternate WPS qualification testing for bridges may be considered. However, if the Code qualification requirements are not satisfied, which is likely given the bridge-specific nature of the Code qualification testing, acceptance of alternative qualification testing is not recommended.

10.7.2.2. Post Weld Heat Treatment

Post weld heat treatment (PWHT) is used to adjust residual stresses or change weldment microstructures. The Code only permits the use of PWHT when it is specified on the plans (clause 4.4) or otherwise approved by the Engineer. Most welded structural steel is used in the as-welded condition, that is, without any PWHT. When considering the use of PWHT, caution is warranted because PWHT may affect the mechanical properties of the weld, the heat-affected zone or the base metal.

PWHT is distinct from postheat for hydrogen control (section 5.3.1), which does not require Engineer approval. Postheat temperatures are much lower than those for PWHT and will not affect microstructures.

While rarely used for steel bridge fabrication, the most common reason for PWHT in welded structures is stress relief, in which case the more specific term “thermal stress relief” may be used instead of “PWHT”. Stress relief per the Code involves heating the steel to 1,100 °F (clause 4.4.2.3) and holding it there for at least one hour per inch of steel thickness. Thermal stress relief may be specified to reduce residual stresses that are causing ongoing fabrication-related cracking problems; more frequently, thermal stress relief is used to achieve dimensional stability for post-welding machining operations (section 5.7.2.4).

In some situations, the need for stress relief may be anticipated before a project begins. More commonly, a contractor may request permission to stress relieve a product after problems are encountered during construction—e.g., cracking that cannot be resolved using standard means, or the part distorts excessively during machining.

When considering approval of PWHT, the effect on the base metal, weld metal, and heat-affected zone must be considered. The steel producer can be contacted for information on the steel, and the filler metal supplier for information on the weld deposit. Some filler metals are classified in the as-welded condition, others in the PWHT condition, and sometimes the same filler metal will be classified both ways. The Code provides details for stress relief heat treatment in clause 4.4.

10.7.2.3. Alternative Preheat

Generally, preheat for bridge welding is controlled by clause 4.2 or, for fracture-critical welding, clause 12.14. However, annex G of the Code contains an alternate method for determination of required preheat that is based upon factors that include the chemical composition of the actual steel, the heat input of welding, the hydrogen content of the deposited weld metal, and the anticipated degree of restraint. When preheat temperatures are calculated in accordance with annex G, the resulting values may be higher than or lower than those prescribed under clause 4.2.

Use of the annex G methods for establishing preheat is not common in bridges. Largely, this is because the preheat requirements prescribed in clauses 4 and 12 has been proven effective for bridge welding. Further, the prescribed preheat temperatures are minimums and therefore fabricators may increase preheat when they think it necessary for better hydrogen control (provided maximum interpass temperatures are not exceeded) without formally calculating new preheat temperatures using annex G.

CHAPTER 11 - ADDITIONAL TOPICS

This section addresses a selection of topics related to welding of bridges and other transportation structures that do fit elsewhere in the manual. The topics include welding to metals other than materials found in the Code, stud welding, field welding (including both new construction and welding to older structures), items that are not yet in the Code but planned for inclusion soon, and welding-related innovations that may come to steel bridge fabrication in the future.

11.1. WELDING OTHER METALS

This section discusses materials other than steel that are sometimes used in highway and bridge construction. The Bridge Welding Code does not include these materials, and therefore, designating that the Code be used for welding these materials, even for bridge applications, is inappropriate. Among other things, a key purpose of a welding code is to define the allowable combinations of base metals with welding consumables, and also to provide requirements for achieving the weld quality needed for satisfactory weld performance. Specifying the Code for use on materials not found in the Code is inadequate, confusing, and incorrect.

11.1.1. Aluminum

Some vehicular and pedestrian bridges have been fabricated from aluminum. Aluminum is sometimes used for light supports and bridge railing. Perhaps the most attractive feature of aluminum for such applications is the natural resistance that uncoated aluminum offers to corrosion, although it is incorrect to say that aluminum does not corrode, and the corrosion resistance of some alloys in some environments is poor.

The light density of aluminum (approximately one third that of steel) makes it attractive in many industries, but this characteristic is not normally important in bridge construction. Along with the lower density comes a characteristic that is very important in bridge design and can make the use of aluminum challenging: a lower modulus of elasticity, which is also approximately one third that of steel. The lower modulus of elasticity is also responsible for a lower allowable stress range that must be considered when evaluating fatigue.

Just as there are many grades of steel with different chemical compositions and methods of processing, so there are many grades of aluminum. Some grades are readily weldable while others are considered unweldable. Even for weldable grades of aluminum, many arc-welded connections will have softened HAZs which limit the strength of the connections. This is particularly limiting when splices are made.

While this is a welding-related manual, it is noteworthy to mention that bolted connections are not free from problems either, and lapped bolted connections are sensitive to unique corrosion problems. Additionally, aluminum-to-steel bolted connections can be problematic because of galvanic corrosion.

For any transportation structure application, use of AWS D1.2, *Structural Welding Code — Aluminum*, is appropriate (AWS, 2014b). Chapter 7 of the AASHTO BDS covers the design of aluminum structures (AASHTO, 2017a). In addition, the *Aluminum Design Manual*, published by the Aluminum Association, is a good reference for design (Aluminum Association, 2015).

11.1.1.1. Surface Aluminum Oxide

Aluminum is highly durable because it nearly immediately forms a passive and protective aluminum oxide surface layer when in contact with the atmosphere. The nearly instantaneous nature of this oxide formation has a significant effect on the welding techniques that are needed to achieve sound welds.

The melting point of aluminum is in the range of 1100 to 1200 °F, but aluminum oxide melts at a much higher temperature of 3600 °F. Therefore, the welding arc can melt the aluminum but will leave a thin oxide layer that float to the surface of the molten weld pool. If this oxide contains moisture or other sources of hydrogen, the hydrogen will form porosity. It is impossible to completely remove the aluminum oxide layer before welding, but it is best to keep it as thin as possible. Therefore, proper cleaning of the aluminum surfaces to be welded is needed to ensure quality welds.

Aluminum oxide builds up quickly, so cleaning just prior to welding is best. Chemical cleaning is possible on smaller parts, but the chemicals are very strong acids or bases (metal hydroxides), which require special handling. The best preparation for larger aluminum structures is a vigorous brushing using a stainless-steel brush that has only been used to clean aluminum. Power brushing with rotating stainless-steel brushes can be efficient but must be carefully monitored to ensure that the heat from brushing does not overheat the aluminum surface or that mechanical pressure from brushing does not drive oxides deeper into the metal instead of removing them.

11.1.1.2. Welding Practices

The primary welding process for aluminum in large structures is GMAW (section 3.4). Thinner sections of aluminum are commonly welded with GTAW. On thinner sections, GTAW more readily produces a visually pleasing weld, and therefore it is commonly used on items such as handrails. However, GTAW is slower and more labor intensive that GMAW, and therefore is more costly than GMAW.

When welding aluminum, GMAW uses DC positive polarity (section 2.5.9) such that positive ions are attracted to the negative pole of the base metal. The bombardment by the positive ions helps clean the base metal during welding. Modern GMAW power supplies can also provide waveforms (section 3.5.7) that helps break up oxides during welding.

To avoid contamination, welding gun liners and anything else that touches the welding wire from the spool to the welding arc must not be used for steel welding. Otherwise, iron pickup from chafing of the welding wire in the gun will become embedded in the aluminum welding wire and be transferred through the arc and contaminate the weld.

Particular welding techniques are needed when welding heat-treated aluminum to maintain strength. During welding, heat from the arc will affect the strength of the heat-affected zone (section 2.2), and it is desirable to minimize this impact. Heat-treated aluminum is usually found in extrusions.

AWS D1.2 has recommended electrode classifications for different aluminum base metals (clause 4.4.1 of AWS D1.2). Filler metals for GMAW welding of aluminum are available in a variety of alloys, which are generally classed as 4XXX alloy filler metals used for heat-treated alloys and 5XXX alloy filler metals used for non–heat-treated alloys. Electrodes are further categorized by F numbers, which are based on the usability of the electrode. In AWS D1.2, F numbers are an essential variable for welding procedure qualification, but otherwise electrode alloy is not an essential variable, and strength varies by alloy. Therefore, in addition to the requirements of AWS D1.2, it is prudent to require welding procedure qualification using the filler metal alloy to be used in production.

Aluminum structures such as railing and signage are sometimes anodized, which is a chemical process used to thicken the aluminum oxide layer. Generally this process is used to improve durability, such as for use in wet, salty environments, and it can also be used to change color. Welding should be done before anodizing, and care should be taken to facilitate color matching in the final product because different filler metals anodize to different colors.

11.1.2. Stainless Steel

Stainless steel is known for its excellent durability. Use of stainless steel for structural bridge elements is not common. However, stainless steel is sometimes used for bearings; this is discussed later in this section. There is also a relatively new stainless steel grade available to the bridge market. This material, ASTM A709 50CR, is addressed in section 11.4.1.

Stainless steels are available in a broad variety and are characterized by their chromium content. By definition, stainless steel must have a minimum of 10.5 percent chromium, and high-chromium stainless steels are those with over 18.5 percent chromium. Based on its strength, toughness, and durability, stainless steel can be suitable for bridges. However, as a base metal alloy constituent, compared to iron, chromium is quite expensive, and so the use of stainless steel in a bridge primary structural member will more than double the cost of the bridge. However, considering the life-cycle cost and reduced maintenance of stainless steel, its cost becomes more competitive.

Stainless steels are generally available in austenitic, martensitic, and ferritic microstructures as well as two-phase austenitic and ferritic microstructures known as “duplex” stainless steel. The microstructures affect material properties, and therefore, when considering the use of stainless steel, the grade must be chosen that will provide the desired performance in service. In structures, use of 300 series chromium-nickel austenitic material is common. AWS D1.6, *Structural Welding Code—Stainless Steel*, should be specified for structural stainless steel welding applications (AWS, 2017a). AWS D1.6 addresses welding but not necessarily all special fabrication needs when using stainless steel. AISC’s *Design Guide 27: Structural Stainless Steel* is a useful resource for stainless steel structural design and also fabrication recommendations (Baddoo, 2013).

Austenitic stainless steels are readily weldable, but there are some key differences compared to welding carbon (i.e., non-stainless steel):

- Stainless steels are susceptible to carbide precipitation when welded, so care must be taken to avoid this phenomenon. When stainless steel is held between 800 °F and 1400 °F, carbon combines with chromium and form chromium carbide precipitates at grain boundaries. This phenomenon is also known as sensitization. With the chromium thus bound by carbon, it is not available for corrosion resistance. During welding, maximum interpass temperatures are limited to help avoid this temperature range. Further, care must be taken to avoid carbon contamination of the weld. This means using different sets of tools for stainless steel and carbon steel if those tools, such as brushes and chipping hammers, will come in contact with the weld or with surfaces to be welded, and not using carbon-arc gouging. Fabricators who work with both stainless steel and carbon steel may establish separate work zones and tool supplies to help ensure contamination is avoided.

- The high alloy content of stainless steel reduces the thermal conductivity of the material compared to carbon steel, and the weld pool stays molten longer compared to carbon steel welding. Particularly for this reason, when welding stainless steel, welding techniques must be adapted to the base metal and associated filler metals. AWS D1.6 includes welder qualification requirements, so specifying AWS D1.6 addresses helps ensure that welders are skilled in the proper techniques for welding stainless steel.
- Special safety rules apply for welding stainless steel. Hexavalent chrome is a constituent of stainless steel welding fumes, and OSHA has specific thresholds for this. Enhanced ventilation or even ventilated welding face shields may be required.

Using special techniques, stainless steel can be welded to carbon or low-alloy steels. Use of stainless steel at bearing locations is one example, such as shown in figure 189. Such bearing applications typically consist of a thin sheet of stainless steel attached to guide plates. A useful welding process for this application is GTAW. Welder qualification and requirements for welding procedures for this application are addressed in D1.6.

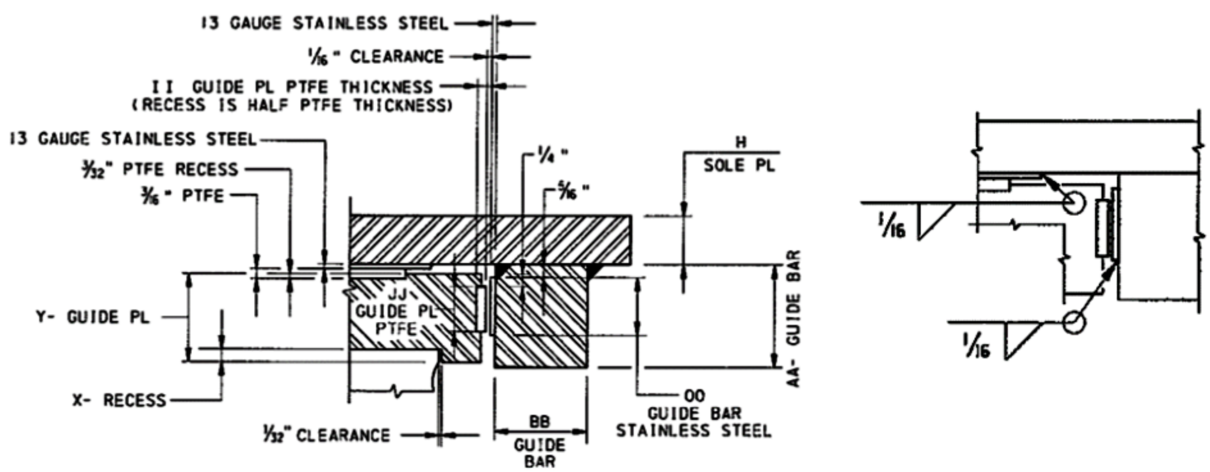


Figure 189. Illustration. Typical detail using GTAW to join a stainless steel sheet to carbon steel (pot bearing guide bar).

11.1.3. Reinforcing Steel

Use of reinforcing steel is synonymous with use of reinforced concrete, and applications in transportation structures abound. Many welding applications facilitate this use, including welding reinforcing steel to other reinforcing steel and welding reinforcing steel to plate or shapes (see figure 190). Welding is useful and common for joining reinforcing steel to make assemblies for cast-in-place and precast concrete elements. In particular, when used with proper joints, welding offers an advantage over the use of tied connections in assemblies that are moved because welded joints provide a more rigid assembly. Welding can provide continuity for long reinforcing steel through use of welded lap joints to join smaller segments.

Source: FHWA

Figure 190. Photo. Welding reinforcing steel to structural plate.

AWS D1.4, *Structural Welding Code—Reinforcing Steel*, is the appropriate standard to specify in contracts for control of welding practices for reinforcing steel (AWS, 2011a). It is not appropriate to specify the Bridge Welding Code for welding of reinforcing steel, including reinforcing steel used in bridges. D1.4 also provides useful guidance regarding welded design and structural joints for reinforcing steel.

As with welding structural steel, effectively welding reinforcing steel depends on the use of proper base metals, consumables, and welding practices. Some reinforcing steel has good weldability; other reinforcing steel may be welded depending upon its carbon equivalency (C.E.) and may require special qualification (see section 11.3.3 for a discussion on carbon equivalency). Weldable reinforcing steel is identified with a “W” stamp (see figure 191). Steel meeting the requirements of ASTM A706, “Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement”, is intended to be weldable (ASTM, 2016).

Source: FHWA

Figure 191. Photo. Weldable reinforcing steel with a "W" stamp.

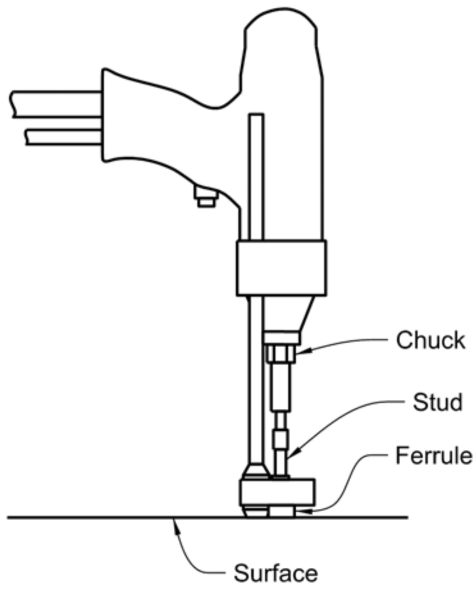
As addressed by AWS D1.4, other reinforcing steel that is not specifically formulated for welding may actually be weldable, depending upon the C.E. of the material to be used. D1.4 provides recommendations for C.E. in clause 1.5.4. As carbon content or C.E. value increases, the HAZ in the steel can become harder and more brittle and therefore higher preheat is needed, as well as higher interpass temperatures for hydrogen control (see also section 11.3.3). In Table 7.2, D1.4 provides minimum preheat temperatures for various C.E. values.

AWS D1.4 does not address welding of stainless steel reinforcing steel. Use of AWS D1.4 is useful for reinforcing bar splice details, but AWS D1.6 is a better choice for the actual welding.

11.2. STUD WELDING

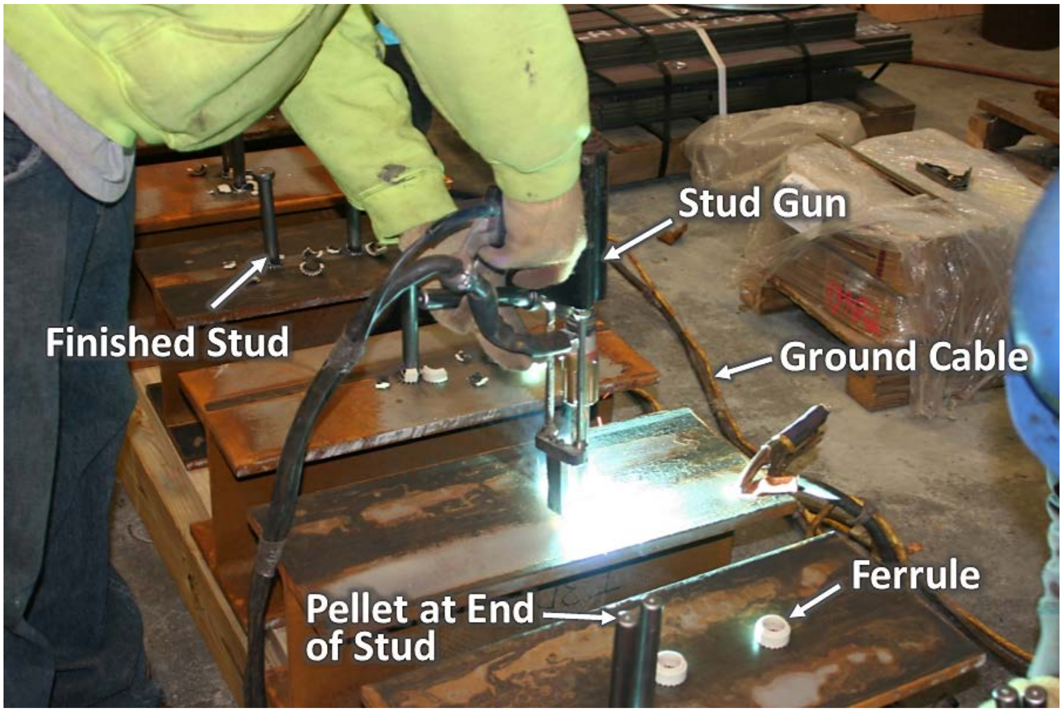
The process used for welding shear studs is unique and relatively simple as compared to other welding on steel bridge members. As such, all discussion of shear stud welding, including process control and quality requirements, are addressed in this section below and not in other parts of this manual.

Shear studs are welded by arc stud welding (SW), which is "[a]n arc welding process using an arc between a metal stud, or similar part, and the other workpiece. The process is used without filler metal, with or without shielding gas or flux, or with or without partial shielding from a ceramic or graphite ferrule surrounding the stud, and with the application of pressure after the faying surfaces are sufficiently heated" (AWS, 2010c). The equipment used for this process, commonly known as a stud gun, is shown in figures 192 and 193. The stud gun operator places the stud and a ceramic ferrule at the intended location and then pulls the trigger to initiate the arc, simultaneously pressing the stud into the weld pool that is created. Much of the molten metal and any contamination are expelled from the weld area as the stud is mechanically forced into the weld pool. A small pellet on the end of the base end of the stud provides some deoxidizers. Arc stud welding is frequently referred to as "stud welding" or "shear stud welding", and is used to attach headed shear stud connectors to beams.



Source: FHWA

Figure 192. Illustration. Stud welding gun.



Source: FHWA

Figure 193. Photo. Stud welding in process.

Arc stud welding is a semi-automated process and fairly simple to use. The keys to obtaining a quality weld are to weld on relatively clean materials, use studs that are clean, and obtain the proper balance between welding current and arcing time. The Bridge Welding Code addresses cleanliness requirements for both the studs and the work surface. The welding operation takes only a few seconds. Afterwards, welded studs are visually inspected to ensure that weld flash surrounds the perimeter. When current is too low, or time too short, the flash typically will not extend around the whole stud. Conversely, when the current is too high, or the time too long, the flash may extend a long way beyond the ferrule, or may undercut (see chapter 5 for a discussion about undercut) the stud itself. The Bridge Welding Code has detailed language about what is and is not allowed regarding this flash; notably, the Code is explicit that this flash is not a fillet weld and quality requirements for fillet welds do not apply.

To ensure that proper procedures are used, the Bridge Welding Code requires that at the beginning of a production shift, or before welding with a given equipment setup, the first two studs are tested by mechanically bending them over to an approximate 30-degree angle from the original stud axis. This testing is performed by striking the stud with a hammer or by inserting a pipe or other hollow device around the stud and bending it. A good weld will allow for such deformation and will not break. Poor procedures will typically result in the stud breaking from the beam in the weld region. Successfully tested studs are not straightened afterwards because the studs will be encased in concrete, and they perform effectively as bent.

Studs that do not have flash for the full 360-degree perimeter may be repaired by using SMAW to apply a fillet weld around the perimeter. Then the repaired studs are bent 15 degrees, not 30 degrees, to prove their adequacy. Generally, welded repair is optional, if the fabricator is willing to risk the bend test without the repair. Further, if a contractor prefers, the studs may be attached by fillet weld using SMAW instead of the SW process. The Bridge Welding Code describes the procedure to be followed for this, including the requirement that "[t]he stud base shall be prepared so that the base of the stud fits against the base metal", which means that the small pellet and point at the base of the stud must be removed if studs are to be attached by SMAW. The Bridge Welding Code provides minimum sizes for these fillet welds, and the fillet weld must otherwise satisfy quality criteria required for other bridge fillet welds.

Stud welding is addressed in its own clause of the Bridge Welding Code, separate from other welding processes. The previously mentioned testing of production welds, as well as the manufacturer's stud base testing, provides assurance that quality welds are made when proper procedures are used and welding procedures (which the Bridge Welding Code refers to as "stud application" procedures) do not require qualification testing (see chapter 4), with some exceptions. The exceptions include welding to non-planar surfaces, welding in the vertical or overhead positions, welding through decking, or welding to steels other than those listed in the Bridge Welding Code. The stud welding clause of the Code provides a qualification method for procedures that are not prequalified.

11.3. FIELD WELDING

"Field Welding" generally means welding on a jobsite in a construction operation as opposed to welding in a shop in a fabrication operation. Field welding out of doors at a fabricator's facility, such as in an assembly yard, it generally not considered to be "field welding" although the environmental controls needed for field welding (section 11.3.2) apply as conditions warrant. The Code is applicable to bridge welding in the field just as it is in the shop.

Welding in the field facilitates versatility in structural design and construction and is very common. Field welding can be used to attach any bridge components that can be joined with welding in the shop. Most requirements for field welding are the same as shop welding requirements, but in practice special consideration must be given to the welding environment. As with shop welding, diligence and adherence to Code requirements are essential to achieving sound welds, but concerns that field welding is inherently not good and to be avoided are unfounded.

11.3.1. Practices and Advantages

There are many examples of field welding used in bridges, both domestically and outside of the United States. The reasons for use of field welding include constructability in both new construction and in retrofits, aesthetics, and cost advantages.

11.3.1.1. New Construction

The use of field welding has a long history in joining main member I-girder field splices (see figure 194). When girder flanges and webs are joined in the field by CJP welds, the resultant joints provide connections that are as effective as field bolted connections or flange and web shop splices. Field welded girder splices offer some economy compared to bolted connections because splice plates, hole drilling, and fasteners are not required. However, to facilitate welding in the field, all three joints (the web and both flanges) of the girder ends must be in close alignment when the girders are erected. Therefore, fabricators may need to bring the mating ends of girders together during shop assembly to achieve and demonstrate this close alignment, including verification that both sides of each joint are in the same plane and root openings are within acceptable tolerances. When field-welded splices are used, the contract should specify the details to be used for these connections.

© 2019 Lewis Gamboa (left) and John Holt (right)

Figure 194. Photos. Bridge girder welded field splices in progress. Girder elevation (left, courtesy of Lewis Gamboa, LJA Engineering) and flange splice (right, courtesy of John Holt).

Use of field-welded girder splices has become less common in the U.S. over time, which may be due to contractor preferences, as TxDOT experience demonstrates. Until the 1980s, all I-girder splices for TxDOT were field welded, including those on long-span curved girder units. However, in the 1980s contractors began requesting bolted field splices as an option on bridge plans. The bolted splices increased the cost of fabricated steel, but this was offset as contractors found they preferred the bolted option.

Although field-welded splices fell into disuse in Texas, field welding is still popular there for making crossframe to girder connections in the field (a practice unique to Texas bridges). The standard TxDOT crossframe detail (figure 195) includes just one bolt hole in each gusset plate, which is used for fit-up during erection. After the erected steel is in place, typically when an entire unit has been erected, the crossframe-to-girder connections are completed with fillet welding. This approach provides considerable flexibility for handling differential deflections, particularly on curved bridges.

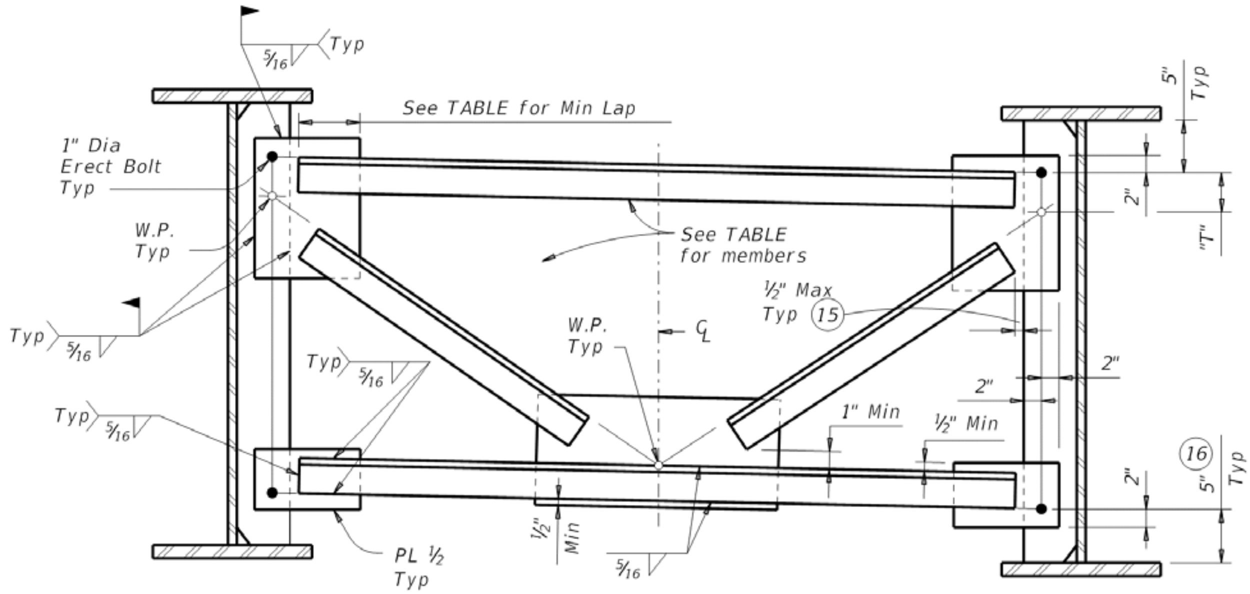


Figure 195. Illustration. TxDOT crossframe detail for field welding (TxDOT, 2015).

Special consideration is needed for detailing field-welded connections, particularly those with CJP groove welds. For example, at plate intersections, such as where the girder flange is joined to the web, it is a better practice to allow the CJP groove weld in each plate to terminate outside the design length of the weld; a weld access hole provides both room for termination of the web weld and access to make the flange weld (hence the name “access hole”). Box sections, with two web and more restricted access inside the box, are more complicated.

Many effective field-welded designs have been accomplished. Details are not provided in this manual, but benchmarking with others, including the NSBA, is prudent.

11.3.1.2. Aesthetics

Field welding is a useful option when a smooth surface is desired instead of a bolted connection in the final structure. For example, the field-welded I-girders in figure 196 provide smooth uninterrupted surfaces compared to bolted connections.

Another example is the Minto Island Bridge, which was a 2018 NSBA Prize Bridge winner. The designer used CJP welds to make main member connections and provide a smooth surface (AISC, 2018). Similarly, TxDOT used CJP welds on seven tied-arch bridges over US 59 in downtown Houston, the first four of which won an NSBA prize bridge award in 2003. The arches, shown in figure 197, were field-spliced at two locations in the arch. The splices are not visible in the photo and are barely detectable to the passersby.

© 2019 John Holt

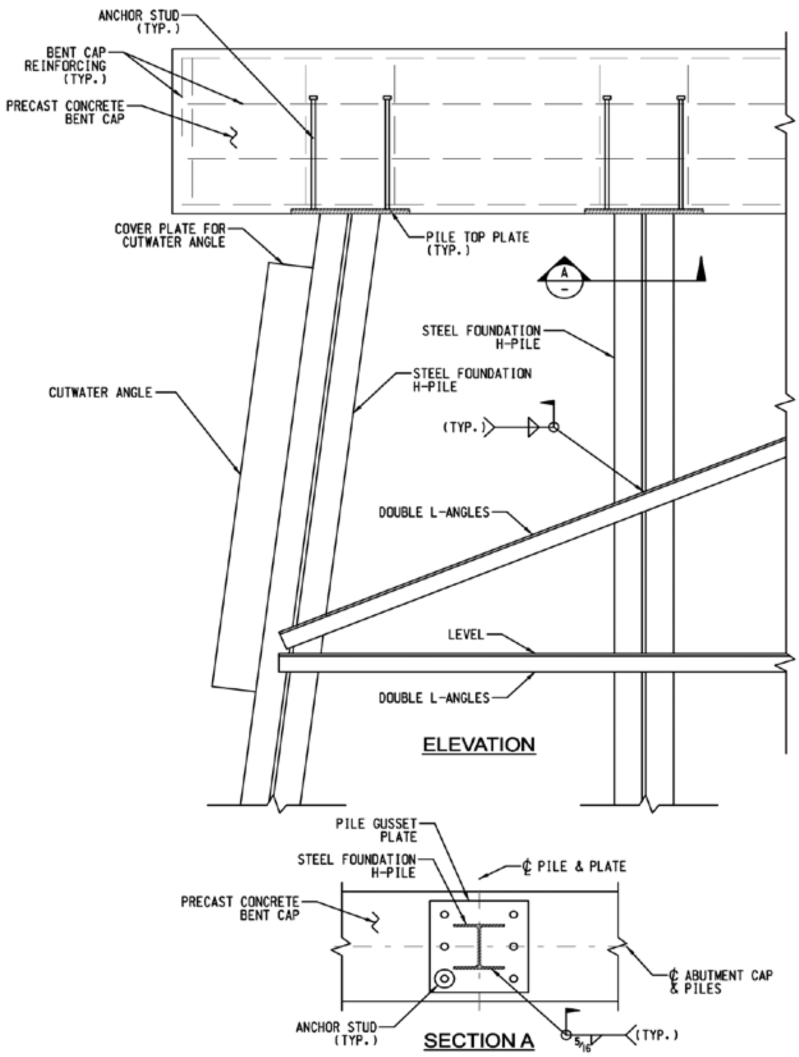
Figure 196. Photo. Use of field welding provides a pleasing uniform finish free of bolted splices (courtesy of John Holt).

© 2019 John Vogel

Figure 197. Photo. Houston tied arch bridges of US 59 with field welded arch rib splices (courtesy John Vogel).

11.3.1.3. Use of Embedments

Field welding is used to join concrete elements to steel elements and concrete elements to each other through the use of steel plates embedded in concrete. A typical example is the use of embedded plates and field welding to attach concrete pier caps to steel piling in pier construction. An example is shown in figure 198. The construction method of such piers illustrates the flexibility and versatility of field welding. Piling is driven to the intended depth of resistance, and then the piling is trimmed to the proper elevation. The pier cap is then set in place on the piling, with the embedded plates resting on the piling, and then welds are made to join the caps to the piling. Hence, field welding allows the piling to be shipped to the field with no connection preparation and without cutting to any particular length, facilitating very fast construction. Another example is the use of field-welded embedded plates for longitudinal joining of deck slab beams (figure 199).



Source: FHWA

Figure 198. Illustration. An example use of embedded plates for field construction (FHWA, 2009).

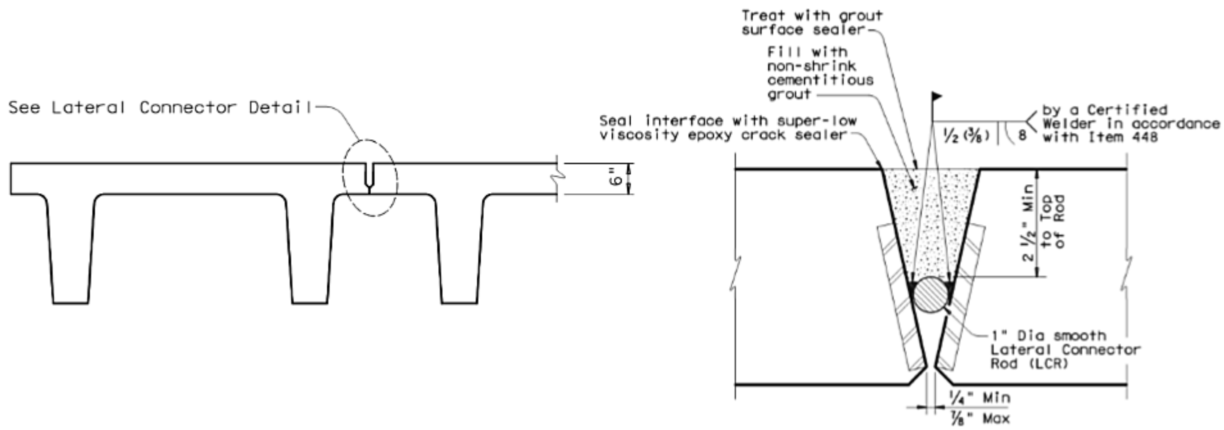


Figure 199. Illustrations. Field welded lateral connection detail (Ralls et al., 2004).

11.3.1.4. Issues Associated with Welding Embedments

Challenges exist in the use of embedments that should be considered when using them:

- **Differences in response to heat –** One challenge exists in welding on steel embedments that are in concrete are due to the heat associated with arc welding and the differences in the response to the changes in temperature between steel and concrete. With differences in the properties of thermal conductivity and thermal expansion, the result is differential straining between the materials. The thermal conductivity of concrete is only about 2 percent of that of steel, and therefore significant heat buildup can occur in the concrete. Due to the difference in thermal conductivity, heat is conducted through the steel and away from the joint faster than the heat being pulled away by the concrete. Concerns have been expressed about uncured (“wet”) concrete “exploding” during welding on embed plates. The integrity of the anchorage, distortion of the embed plate and cracking of the concrete surrounding the embedments are all issues that have been raised in association with welding embedments.
- The use of thicker embedment plates reduces the peak temperatures that are experienced by the concrete. However, thicker steel of certain grades may require preheat, negating the benefits of thicker material. Selecting steel of a grade and thickness that do not require preheat is advisable. The use of controlled hydrogen welding processes may preclude the need for preheat in some situations. If multiple passes are required, or when welds are localized in one area of the embedment, allowing the plate to cool between welds will avoid unnecessary accumulation of localized heat. Allowing the welds to cool to near ambient temperature is desirable (unless the steel is required to have been preheated for welding).
- **Embedment expansion –** Embedment plates are often set into concrete with the front face of the embed plate flush with the concrete surface. When this is done, thermal expansion of the width and height of the embed plate (as compared to the thickness) will apply pressure to the surround concrete and may cause the concrete to crack. To

mitigate this problem, a gap can be provided between the edge of the embedment and the surrounding concrete as shown in Figure 14-13. If filling of the gap is necessary, this can be done after welding is completed.

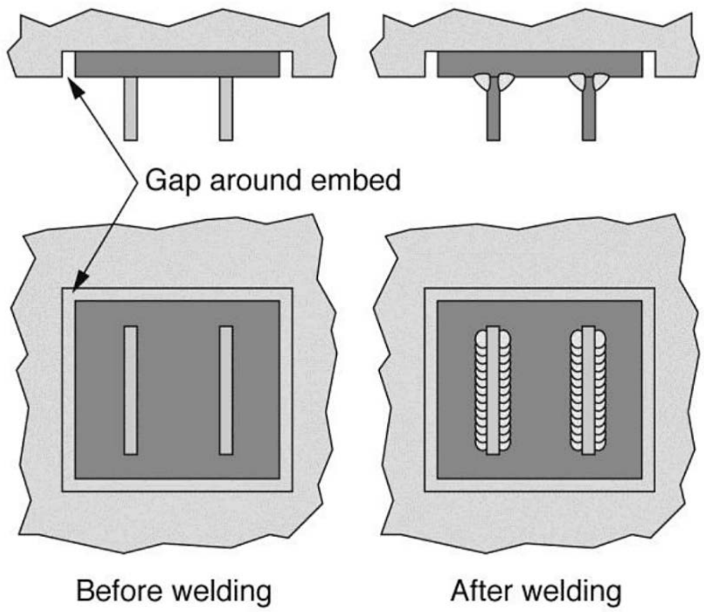


Figure 200. Illustration. Use of a gap with embedments that are flush with concrete.

- **Distortion -** Welding can cause embedments to distort, which may cause the surrounding concrete to crack, or may cause the anchorage to pull away from the concrete. Designs and techniques that minimize distortion should be used. Methods of reducing distortion may include using a thicker embed plate, reducing heat input, and avoiding over-welding. Thicker embedments allow for more heat absorption and dissipation. Reducing heat input decreases the thermal energy available to cause distortion. Over-welding will require more heat than necessary to complete the welded joint.
- **Damage to concrete -** Welding will result in high temperature, short duration temperature excursions; the effect of this temperature rise on the properties of the concrete are not known to these authors. For critical applications, tests that measure the effects of these temperature excursions should be performed.

11.3.1.5. Retrofits and Rehabilitation

Field welding is useful for repairing, strengthening, retrofitting, or otherwise rehabilitating existing steel bridges. Attaching cover plates for strengthening is a very common example. When this is done, the resulting fatigue details must be considered during the design phase. Some early uses of flange cover plates terminated them in the tension zone, and over time fatigue cracks resulted in the toes of the transverse fillet welds that attach the cover plates to the flanges. It is now known that when cover plates are used, the best practice is to extend the cover plate beyond the tension zone (Fisher et al., 1978). A bolt group at the termination of the cover plate significantly increases the fatigue resistance (AASHTO, 2017a).

Field welding is an ideal way to retrofit older I-girder bridges where connection plates were not attached to flanges, resulting in out-of-plane bending. A good example of this is the I-80 bridge over the Mississippi River, in La Claire, Iowa. Details about this retrofit are well documented in NCHRP Synthesis 489 *Extending Bridge Service Life through Field Welded Repairs and Retrofits* (National Academies of Sciences, Engineering, and Medicine, 2016). The bridge was built in 1966, when floor beam brackets were not typically attached to flanges, and in 2009, cracks from out-of-plane bending were discovered on brackets at stiffeners near the girder top flange. A connection between the stiffeners and top flange was needed. A bolted connection was considered, but there was no room for additional bolted brackets. Instead, the field-welded retrofit shown in figure 201 was used successfully. Shown left, the existing stiffener is cleaned in preparation for welding. Shown right, the stiffener is welded.

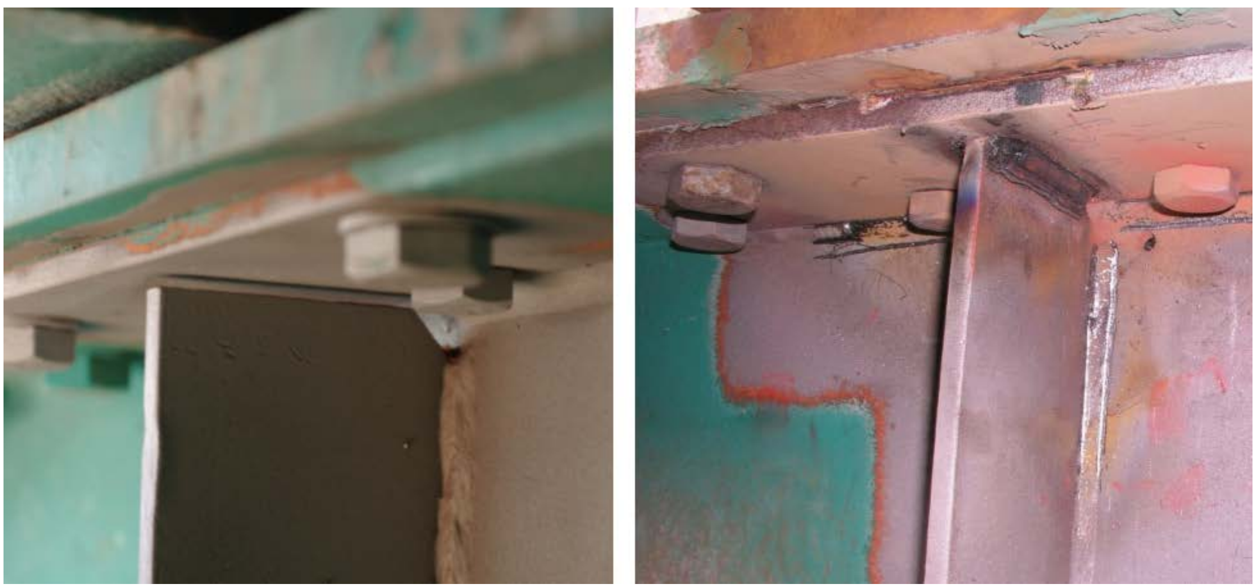


Figure 201. Photos. I-80 bridge field weld retrofit, before and after welding (courtesy Illinois Department of Transportation).

The NCHRP study provides five other examples of successfully executed field welding retrofits. These demonstrate the effectiveness of such field welding and also demonstrate that field welding is a mature and robust technology.

11.3.2. Challenges with Field Welding

Though welding in the field is common, field welding does come with special challenges. Control of welding is always more easily achieved in the shop, but this does not mean that quality and economical welding cannot be accomplished in the field. Field challenges must be acknowledged and managed.

Many of the technical challenges associated with field welding are related to the environment. Wind, rain, cold temperatures and dealing with materials that have been exposed to field storage conditions are all concerns related to field welding. Field welding typically requires more out-of-position welding (section 2.3), and access to the joint is often more awkward. Yet, all of these conditions can be mitigated by good planning and use of proper equipment. For example, the use of SMAW (section 3.2) or FCAW-S (section 3.6) can help avoid the problems associated with using gas-shielded processes such as GMAW (section 3.5) and FCAW-G (section 3.6) in windy conditions. Tents or other enclosures can be used to protect the welder and the joint from the elements. Out-of-position welds can be made easily by a skilled and qualified welder using the proper electrode and procedures. Storing staged material on timbers rather than on the ground helps protect the material.

Another challenge associated with field welding is not technical but managerial: it is easier to manage shop work than field work. Shop personnel may be better known to the foreman and may be more familiar with company practices and expectations.

Field welding is subject to the same Code provisions as shop welding, including requirements for welder qualification and qualification of WPSs. Welds are also subject to the same visual and NDE requirements. Similar to the process of welding, performing NDE in the field is more challenging than in the shop, but it can certainly be performed. As an example, use of radiography is standard for the TxDOT field welded splices described in section 11.3.1.1.

Organization certification is one exception where quality requirements are different in the field than in the shop. In the shop, certification is required by the Code (clause 1.4), but the Code is silent about organization certification for bridge erectors and other field contractors. However, AISC does have a certification program for structural erection. This certification is not necessarily appropriate for all contractors who perform field welding, but it is worth considering requiring this certification for those who perform bridge erection.

11.3.3. Welding Older Structures

Often there is a desire to weld onto older structures for repairs or retrofits. The key to welding older structures is to establish the weldability of the material. “Weldability” is a term that is used to describe the relative ease with which a material can be welded. “Weldability” is not the same as “weldable”; many materials are weldable (that is, they can be welded), but not all weldable materials have good weldability. Materials with good weldability can be successfully welded without unusual precautions. Conversely, when materials require special techniques to be successfully welded such as careful control of preheat and interpass temperature, they are said to have poor weldability.

The chemical composition of the steel controls its weldability. By definition, steel is iron processed with the addition of carbon and other alloys, and it is especially carbon that gives steel its strength and hardness. However, carbon is not good for welding because as the amount of carbon increases the steel becomes more brittle, increasing the propensity for cracking during welding. To a lesser degree, other alloys such as manganese, silicon, and chromium also impact hardness. Therefore, the general weldability of a particular steel is based on its content of carbon and such other alloys. Collectively, the weldability of steel based on carbon and other alloys may be measured as a “carbon equivalency”, or C.E. Various C.E. formulae and recommended limits exist and are applicable depending upon the use of the steel. C.E. values are useful in helping establish weldability, but only partly so. Alloy and carbon content must be measured to establish the C.E. of the steel. Lab analysis can be conducted on a small amount of material, such as turnings from a drilled hole.

In older steels, other constituents that are not included in the C.E. formulae may be present and can have a negative impact on weldability. Prior to the 1930s, acidic bricks used for insulation in Bessemer kilns resulted in high phosphorus content in some steel; high phosphorus may cause cracking and porosity in welds. High quantities of nitrogen in steel are also problematic.

ASTM A36 was established as a weldable structural steel, with proper chemistry controls to ensure good weldability, and came into the market in 1960. Prior to use of A36, ASTM A7 was used for plate and ASTM A9 was used for structural shapes. These materials did have controlled chemistries, including carbon controls, but the chemistries were not specifically controlled for welding, and the ranges of acceptability included steels that have good weldability as well as others that do not. Therefore, when working with A7 and A9, weldability must be established. If welds are present on the structure in question, then the structure is known to be weldable. If welds are not present, further investigation is needed.

Typically, special mitigation is required when C.E. values reach over 0.50 percent. Such mitigation can include higher preheat, continuous heating (i.e., once preheat is established, not letting the temperature of the steel drop below the minimum preheat temperature until welding is completed), postheat (section 5.3.1) or use of low-hydrogen electrodes (section 5.3.2). If high sulfur is present, it may be possible to butter the steel with one or two layers of weld metal under tight controls and then proceed normally from that point on (as defined by AWS 3.0, “buttering” is defined as “a surface variation depositing surfacing metal on one or more surfaces to provide metallurgically compatible weld metal for the subsequent completion of the weld”).

Knowing the chemical composition of the steel is useful, but AISC Design Guide 21 (Miller, 2018) emphasizes that chemistry is only an indirect indication of weldability. Design Guide 21 section 5.4.5 describes a simple mechanical test that can be used to determine weldability, based on fillet-welding a flat bar to the steel and then breaking it off. Such a test may also be useful in helping demonstrate the suitability of the WPS that will be used for welding because, without samples of the actual material, it will not be possible to perform normal qualification testing.

The presence of non-metallic inclusions in high volume can also be problematic when welding older structures. When present, inclusions will generally be at the center of the thickness of the steel. Such inclusions can introduce discontinuities if welding is performed on that section of steel or can cause delaminations (section 5.4.1.4) if loading or shrinkage pull in the through-thickness direction.

When existing structures are coated, it is generally preferable to remove the coating for welding. However, it is possible to weld over some thin coatings. See the discussion in section 11.5.

A recommended reference for establishing the weldability of older structures is *Weldability of Steels* by Robert D. Stout and W. D’Orville Doty (Stout et al., 1971). AWS D1.7, *Guide for*

Strengthening and Repairing Existing Structures is a useful guide for general retrofits and repairs (AWS, 2010).

11.4. BRIDGE WELDING CODE CHANGES ON THE HORIZON

This section discusses two topics that as of the writing of this manual are under development and planned for a future edition of the Code, probably in a 2022 addendum. Because Code language does not exist as of this writing, the information in this section is presented as recommendations and not requirements.

11.4.1. ASTM A709 Grade 50CR

11.4.1.1. General Description and Use of 50CR

ASTM A709 grade 50CR (50CR) is a structural stainless steel with a mixed lean ferritic/martensitic microstructure and good weldability. Stainless steels are defined as steels that have more than 10.5 percent chromium, and grade 50CR is required to have 10.5–12.5 percent chromium. 50CR has a dual-phase microstructure of martensite plus some ferrite. This base metal microstructure provides relatively tough HAZs because the ferrite restricts austenite grain growth, making the HAZ relatively finer than other common stainless grades with 12% chromium, such as the fully ferritic T409 and the fully martensitic T410.

Grade 50CR was originally adopted by ASTM as standard ASTM A1010 (ASTM, 2018a). When this material was incorporated into A709, it was designated as grade 50CR. Therefore, A1010 and grade 50CR are the same material, except that ASTM A709 includes mandatory CVN testing for primary members in tension.

Steel mills developed this material with the goal of providing an economical material with good weldability that could satisfy the goal of a 100-year life in challenging environments without coating. Characteristics of this material include the following:

- Unlike high-chromium classes of stainless steels (more than 18 percent chromium) which develop a chromium oxide passive layer, 50CR is not intended to be a “shiny” stainless steel; rather, iron oxyhydroxides (rust) develop over time, and it has the same red/brown appearance as traditional uncoated weathering steel.
- Because the final appearance of 50CR material will be like that of weathering steel (and not a gray or chrome-like appearance), the following guidelines apply:
    - Generally, the same tools that fabricators use with traditional steels can be used with 50CR without concern for risk of contamination that would mar the finished appearance of the steel. For example, this includes hammers, clamps, and grinding wheels. Special restrictions do apply for welding to avoid carbon contamination; see section 11.1.2.
    - 50CR may be blast-cleaned with ferrous media including traditional shot. Because the steel is not painted, blast-cleaning is not necessary, but fabricators may prefer to preblast some material to facilitate fabrication, and blast-cleaning is typically required for slip-critical bolted connections.

- Strength, ductility, and toughness of 50CR are similar to other 50-ksi grades of A709.
- Modulus of elasticity for 50CR is the same as for carbon and low-alloy steels.
- 50CR has a higher coefficient of thermal expansion and lower thermal conductivity than carbon and low-alloy steels.
- As with most steels used for bridges, heat shrinking can be used in fabrication provided a maximum temperature limit of 1200 °F is observed.

50CR is not available in shapes. Two options for cross frames include use of carbon or low-alloy steels and then galvanizing the cross frames, or fabricating shapes from plate, such as by bending into angles or fabricating T-shaped sections.

As a stainless steel, 50CR is not suitable for cutting by oxy-acetylene and must be cut with plasma torches or mechanical methods. Depending on the equipment being used, plasma becomes impractical when cutting material over two inches thick. As of this writing, ASTM A709 lists 50CR up to two inches thick, and given that plasma cutting capabilities are common among fabricators, this two-inch limit aligns well with fabrication capabilities. However, mills report that 50CR can be produced up to three inches thick by using quenched-and-tempered processing (quenched and tempered 50CR is not currently found in A709).

11.4.1.2. Welding

Where Buy America requirements are applied to welding electrodes on Federal Aid projects (23 CFR § 635.410), advance consideration should be given to the availability of stainless steel welding consumables. Electrodes made with steel that is melted and manufactured in the United States are generally available, but not necessarily in all electrode classifications that might be preferred by fabricators. In some cases, electrode manufacturers can adjust procurement practices to meet these requirements, but they need advance notice to do this.

The minimum preheats in the Code for A709 grades 36, 50, and 50W are suitable for welding 50CR. However, a maximum interpass temperature of 450 °F is appropriate because when stainless steel is held an elevated temperature for an extended period of time, carbide precipitates may develop. New Code language is anticipated to provide a maximum interpass temperature that will apply unless the fabricator demonstrates that a higher temperature is suitable through welding procedure qualification testing.

Care should be taken to avoid introduction of carbon into welds to avoid the formation of carbides which can lead to intergranular corrosion. Carbon-arc gouging should be avoided, and use of tools that have been used for carbon steel welding, such as brushes and grinding wheels, should be avoided.

Weld on recent 50CR projects have been made with austenitic stainless steel consumables. Austenitic electrodes classified as “L” and “LSi” are appropriate for welding 50CR because the “L” identifies the electrode as low-carbon. (Seradj, 2010).

For hybrid welding of 50CR to traditional carbon steels, consumables should be chosen and welding procedures designed to achieve the desired mechanical properties and facilitate the compatibility of the two base metals being joined in the weld puddle as the two metals mix. Also, special care may be needed to avoid cracking associated with restraint due to differences in thermal properties. 50CR expands more due to the heat from welding than carbon and low-alloy steels, and due to its lower thermal conductivity, more heat remains concentrated at the joint, and therefore it will not cool as quickly as carbon and low-alloy steels. If not managed, and depending upon restraint conditions (see section 5.7), these differences can cause warping or cracking.

Volumetric NDE methods (section 6.9) are appropriate for inspection of CJP welds of grade 50CR material, although some technique adjustments may be needed. Grade 50CR is ferromagnetic, so MT (section 6.7) may be used to evaluate 50CR base metal, but welding consumables used for 50CR may not be ferromagnetic. If not, for surface weld inspection, it is appropriate to use PT (section 6.8) instead of MT.

11.4.2. Tubular Structures

As of the 2015 edition of the Code, it is incorrect to specify the Code for use on tubular structures, including for bridges. The Code includes neither tubular materials nor tubular joints. Therefore specifying the 2015 edition of the Code for a tubular structure fails to provide the appropriate information for welding.

However, provisions for the inclusion of tubular materials in the Code is under way and is anticipated for a future edition of the Code. Notes and highlights include the following:

- The provisions will be applicable to vehicular bridges designed to AASHTO LRFD Bridge Design Specifications. However, the provisions are not intended for pedestrian bridges (since pedestrian bridge do not have the loading conditions of highway structures).
- The provisions are not applicable to non-bridge tubular structural components, such as handrail, bridge rail, light supports, and signal supports.
- The provisions are largely based on current AWS D1.1 provisions, including use of T, K, and Y connection joint details that are found in that Code. However, AWS D1.1 provisions for tubular design provisions are not included since AASHTO specifications apply for design instead.
- NDE will include current Code provisions along with additional techniques needed for tubular members.
- The provisions will be found in a new clause. All other provisions of the Code will apply to the structure in question unless superseded by the requirements of the new tubular clause.
- A variety of tubular base metals will be included.
- Welding procedure qualification will be modified to accommodate the new shapes.

11.5. WELDING ON COATED SURFACES

In bridge fabrication and rehabilitation, welding on coating surfaces is not typical. The Code does not allow welding on coated surfaces. Clause 3.2.1 states that "[s]urfaces to be welded and surfaces adjacent to a weld shall also be free from loose or thick scale, slag, rust, moisture, grease, and other foreign material that would prevent proper welding or produce objectionable fumes". However, welding through certain coatings is successfully performed in other industries. It is discussed here for consideration.

11.5.1. Applications (Non-Bridge)

Coatings applied to steel surfaces range from a light coat of primer to various final coatings of multiple coat paint systems. For new work, final paint coatings are normally applied after welding since the heat of welding will locally destroy the paint. For repair and rehabilitation work, welding on previously painted surfaces may be required. Removal of primers and paint is expensive, so the ability to weld on coated steels is desirable, but weld quality may suffer. Additionally, welding on coated surfaces may create problematic fumes.

Not all coatings are weldable; some primers are. Critical variables to consider when determining the suitability of welding on various coatings involve the coating type, the thickness of the coating, and the welding process and procedure. Generally, when "weldable" coatings are thin, welding can be performed without any harmful effect on the weld, but when the coatings are thick, weld quality may suffer. The welding-related problems that can occur when welding on thick coatings include porosity, cracking, and incomplete fusion.

Many paints contain hydrocarbons that are a source of hydrogen, which can lead to cracking. Porosity will typically, but not always, extend to the weld surface when welding on heavily coated surfaces, making it easy to detect such problems in production.

In extreme cases, very thick coatings may preclude proper fusion. However, heavy coatings of this nature often interfere with the flow of electrical current as well, and thus it is difficult to establish and maintain an electric arc under such conditions.

The potential influence of the material coating on weld quality depends on the joint and weld type. For example, plate may be primer coated, then thermally cut and beveled: the thermal cutting operations will burn away some of the primer, leaving a cut edge that is free of primer. Fillet welds on the surface of primed plate will be more affected by the primer, whereas groove welds made on the thermally cut bevel faces will be essentially immune from the effects of primer applied before cutting. On the other hand, if the plate is primer-coated after beveling, the presence of the primer may be highly problematic for groove welds.

When various primers and paints are heated by welding, special precautions may be necessary to protect the welder and others in the workplace from such fumes. AWS Safety and Health Fact Sheet No. 34 is a useful resource regarding welding safety concerns for coated steels. (AWS, 2014c).

11.5.2. Requirements (Non-Bridge)

AISC 360 section M3.5 (AISC, 2016) addresses the issue of field welding through coating by requiring that surfaces within 2 inches of any field weld be free of materials that would prevent proper welding or produce objectionable fumes during welding. AISC 360 section M4.6 requires shop paint to be wire-brushed from surfaces adjacent to joints to be field welded, if necessary to assure weld quality. AWS D1.1 allows welds to be made on surfaces with “surface protective coatings” providing the required weld quality is achieved (see AWS D1.1 clause 5.14.4.2).

Primers can be classified in accordance with AWS D3.9, *Specification for Classification of Weld-Through Paint Primers* (AWS, 2010a). This allows various primers to be compared, and provides a quantifiable means to assess the paint before production welding takes place.

11.6. WELDING INNOVATIONS

This section provides information about newer welding and welding-related technologies that may be coming to bridge fabrication in the near future. These innovations are not found in the Code. However, that does not mean they are not suitable for bridge fabrication. As Omer Blodgett, a welding pioneer who developed many of the welding design practices codified today, was fond of noting: “Code always lag industry”. In other words, codes inherently deal with current, established practices; innovation will routinely precede codification. To advance the state of the art and realize fundamental improvements that benefit everyone, fabricators should be encouraged to innovate, and owners should be open to innovation.

When use of a particular innovation is desired, the fabricator should establish the benefits of the innovation and also establish what concerns may exist with the innovation. The fabricator should then have a discussion with the owner, requesting feedback and the identification of any concerns the owner has. Then research should be conducted, as necessary, that demonstrate concerns are effectively addressed, and finally, the fabricator should establish a practice that will be followed to keep the process under control in the shop.

The use of digital radiography provides a good example of the use of this process. There are strong benefits for both owners and fabricators using this process. Fabricators gain from the immediacy of test results, lower RT costs, and the ease of storing digital files. Owners gain from ease of electronic transfer, receipt, and storage of files instead of handling and storage of film. Owners also gain over time as lower NDE costs result in lower fabrication costs. As of the 2015 edition of the Code, digital radiography is not yet included, but it is expected to be in the 2020 edition. Fabricators who use digital radiography can develop in-house procedures for the process that can be submitted to owners for approval. The procedures should address the controls that the fabricator puts in place to ensure radiograph quality and integrity of the process. Controls should address such items as digital artifacts, digital image sensitivity, file control, and, to facilitate discontinuity measurement on the monitor, linear reference comparators. This cooperation not only facilitates early use of innovations, but it also helps strengthen Code language because when Code language is developed, it is done so with the experience of the fabricators and owners who have used the innovation.

11.6.1. Homopolar Welding

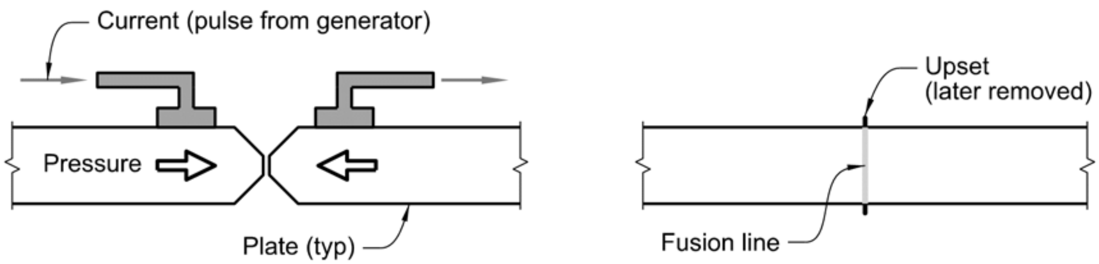
Homopolar welding is process that uses a homopolar generator. It is considered to be a form of welding that combines features of forge welding and resistance welding, and it is best understood in the context of these two processes. With homopolar welding, a pulse of electricity from the homopolar generator heats the surfaces of the steel to be joined, through resistance, to the forging temperature of steel (2250 °F), and then the surfaces are forced together.

Homopolar generators in one form or another have existed since the 19^{th} century, having first been proposed by Michael Faraday (Tesla, 1891). The principle of the generator is to create kinetic energy by rotating a magnetically conductive mass and then instantly stopping the mass and converting the kinetic energy to a pulse of electricity by contacting the mass with conductive brushes. Homopolar generators for bridge welding will have drums that weigh about five tons and rotate at 5000 to 6000 rpm, and they will reach this speed in approximately seven minutes. In a shop, then, the only energy required for homopolar welding is the energy need to power a motor to bring the drum to this speed. Typical bridge shop power can provide this energy.

In conjunction with the pulse of electricity from the generator, the parts to be joined must be under sufficient pressure to achieve the forge weld, which is 20 ksi. As an example, a 2-inch by 24-inch flange has a cross-sectional area of 48 square inches and therefore requires $20 \times 48 = 960$ kips. If a 2-inch by 24-inch flange represents a typical large flange, then to use homopolar welding, fabricators need a means of bringing flanges together under about a million pounds of pressure perpendicular to the welded joint. Plates of different width and thickness can be joined by homopolar welding, but as with traditional welding processes, transitions must be introduced such that plates are the same width and thickness at the joint. Homopolar welding could be used for either flanges or webs but, given the fixtures that would be necessary to deliver the pulse to the joint and introduce the pressure, the process may be more practical for flanges.

In summary, the practice of joining plates by homopolar welding may be summarized as follows (see figure 202):

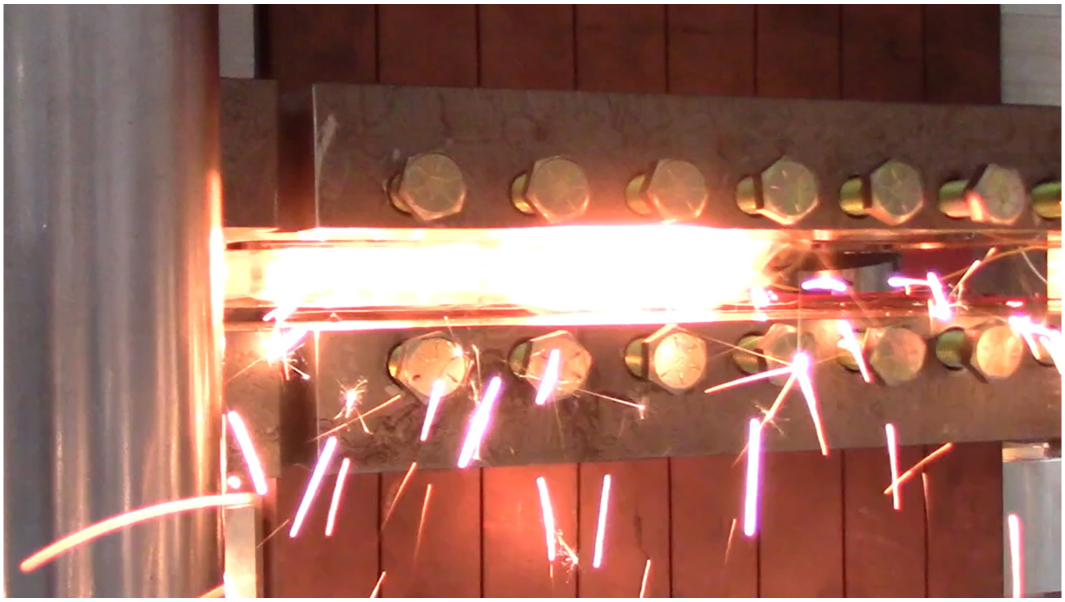
- The surfaces of the flanges to be joined are prepared for joining.
- The flanges are brought together with pressure on the joint of approximately 20 ksi.
- Over about seven minutes, the homopolar generators are brought up to speed, perhaps about 5000 rpm.
- When the proper speed is reached, brushes are dropped onto the rotating drum, instantly converting the kinetic energy of the rotating drum to an electric pulse.
- The pulse is delivered to the joint instantly heating the flange through resistance.
- Now heated, the flange ends soften and, being under pressure, collapse and forge, with a slight amount of material upsetting above the flange surface.



Source: FHWA

Figure 202. Illustrations. Welds made by homopolar welding.

As of the publication of this manual, homopolar welding is not available on the market. However, research sponsored by the FHWA at the University of Texas, Austin, has demonstrated the viability of this process to produce sound bridge welds (see figure 203), and it is being developed further (Zowarka and Rech, 2015). Use of homopolar welding will require significant investment in generators and also fixtures that will provide pressure, but given the speed of joining by homopolar welding, the cost of this equipment may prove to be worthwhile.

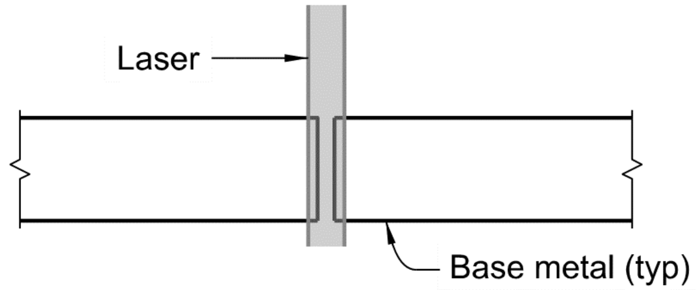


© 2019 Roy Pena (University of Texas Center for Electromechancis)

Figure 203. Photo. Homopolar welding during pulse.

11.6.2. Hybrid Laser Arc Welding (HLAW)

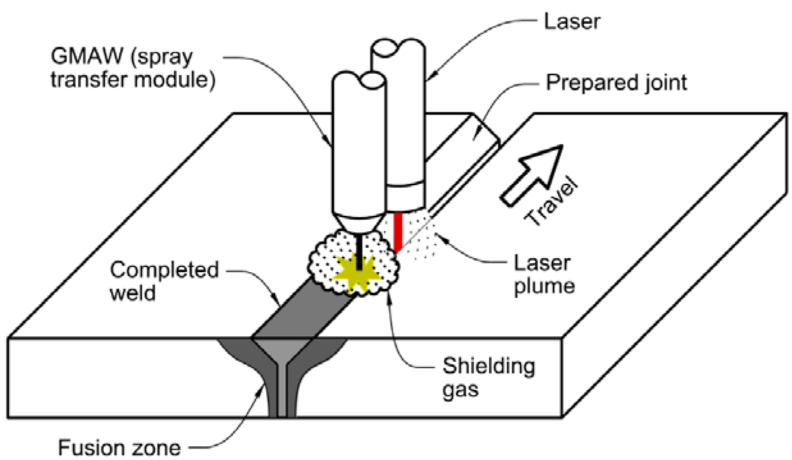
Hybrid laser arc welding (HLAW) is a hybrid process of autogenous (no filler metal) laser welding and GMAW (section 3.5). Laser welding is very fast, operating as quickly as 200 inches- per- minute. Laser welding works by melting the edges to be joined (see figure 204), which then form a CJP weld when the base metal cools and solidifies. The thickness of material that can be welded with a laser is dependent upon the strength of the power supply. As of the publication of this manual, material up to 1 inch thick can be welded by laser. Because it must melt base metal edges, the prepared weld joint must be narrower than the laser beam. Laser beams are about 0.01 inch to 0.02 inch wide, which means that joints must be no more than 0.01 inch wide. Achieving this narrowness requires machining, which is considerably more costly than traditional weld joint preparation. Lasers cannot operate over tack welds, which means that some other method, probably involving large equipment, is needed to hold parts in alignment for welding.



Source: FHWA

Figure 204. Illustration. Autogenous laser welding.

The tight joint tolerances needed for laser welding, the inability to weld over tack welds, and the thickness limitations probably make laser welding impracticable for bridge welding. However, hybrid welding (figure 205), has stronger potential. With travel speeds of up to 80 inches- per -minute, it is not as fast as autogenous laser welding, but it can accommodate joint widths of up to about $^{1}/_{16}$ inch, which means that joints could be prepared with normal shop cutting and grinding instead of machining. HLAW can tolerate tack welds, also making fit-up easier. The thicknesses that can be completely welded with HLAW are still limited to about 1 to 1 ½ inches; in thicker joints, HLAW could be used for root passes and then traditional processes used for fill passes to complete joints, however, the fact that two different processes and associated equipment would be needed to make a joint probably erases the advantages to be gained from a faster HLAW root pass.



Souce: FHWA

Figure 205. Illustration. Hybrid laser arc welding, combining laser welding and GMAW.

HLAW's primary advantage is speed. Its speed is indeed impressive, but the cost of the equipment coupled with the fact that other processes would still be needed to complete joints started with HLAW at the root means that it will probably be some time before it is used in bridge fabrication, if ever.

HLAW has been introduced on a limited scale for use on poles for light support structures. Their process is shown in figure 206, including the fixture used to shape the pole and then hold it for welding, left, and then welding is process, right.

Figure 206. Photos Hybrid laser arc welding of light poles.

11.6.3. Multiple-electrode GMAW

Use of multiple electrodes (see section 3.3.2) when welding with SAW is common and is addressed by the Code. Use of multiple electrodes for GMAW is not discussed in the Code but might be of interest in bridge fabrication.

Welding using two electrodes increases productivity by increasing the deposition of weld metal per pass. With advancements in power supplies, which facilitate stability of the two arcs in the same puddle, multiple-electrode GMAW became feasible and first came into use in the 1990s. As with multiple-wire SAW, multiple-wire GMAW is operated by feeding two independent welding wires into the same welding puddle, with each wire having its own power supply. Generally, the lead wire is larger and primarily serves to drive penetration, while the trailing wire controls bead contour and wetting. The two wires can also be the same diameter.

While SAW is the workhorse process in a bridge shop, GMAW has gained popularity for smaller welds, particularly because it is very clean and to some extent because it facilitates automation. Multiple-wire GMAW does not approach the deposition rates of multiple-wire SAW and its productivity for large CJP welds, but multiple-wire GMAW could supplant SAW for fillet welds and smaller CJP welds or for welds that must be made out of position.

11.6.4. Full Matrix Capture / Total Focus Method (FMC/TFM) PAUT

Full matrix capture/total focus method (FMC/TFM) PAUT is a form of PAUT that offers far greater resolution than traditional PAUT. For example, see figure 207. The top image is an aluminum block with notches at five angles; the bottom left image is a traditional PAUT scan, and the bottom right image is an FMC/TFM PAUT scan.

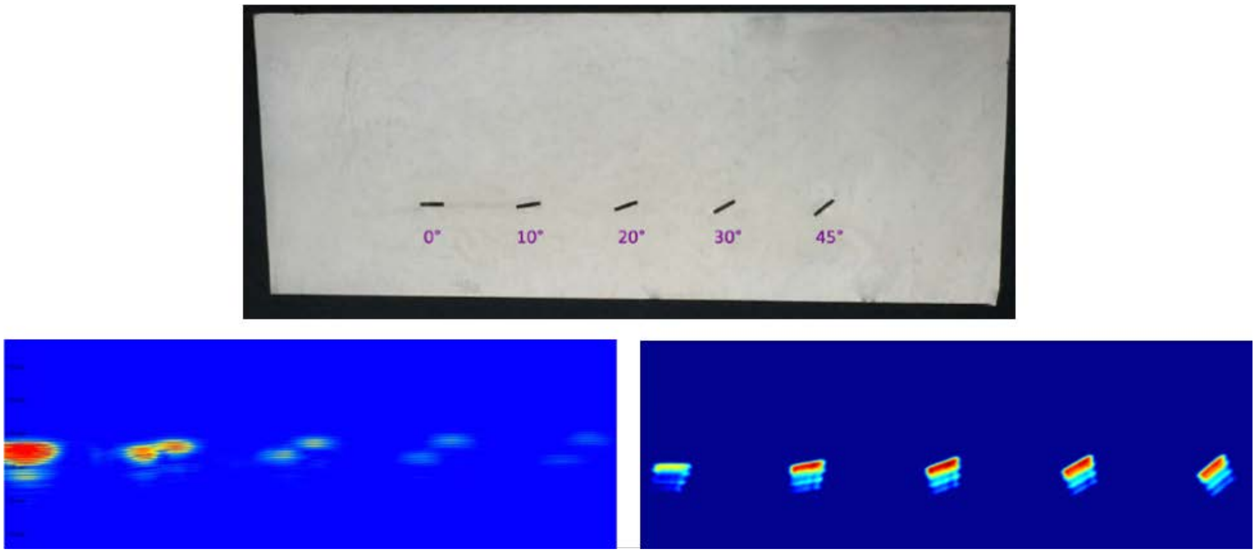
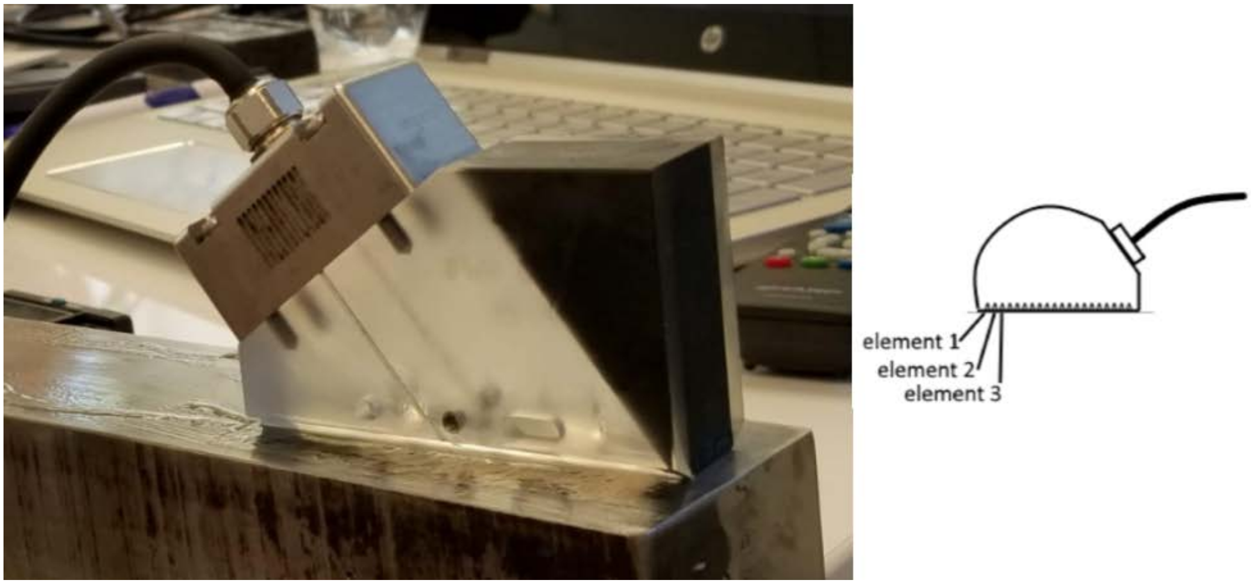


Figure 207. Photo and Images. Comparing FMC/TF PAUR (lower right) with traditional PAUT (lower left).

The improved resolution is achieved by dramatically increasing the data captured during scanning (FMC) and through advanced techniques in post processing (TFM). The data increase comes from having a pulse/echo sound transmission between every pair of elements in the transducer. Every element transmits to each other, creating a full matrix of responses (see figure 208).



Source: FHWA

Figure 208. Photo and Illustration. Photo of transducer (left) and illustration of elements (right).

Total focusing is the compilation and processing of the captured information to provide images of discontinuities.

The resolution of defects using FMC/TFM will improve discovery and remediation of defects. Further, use of FMC/TFM might facilitate acceptance and rejection of discontinuities by measuring their size, as is done with RT, and this direct approach might prove to be advantageous over the current UT and traditional PAUT method of using dB ratings to reject defects.

11.6.5. Split Pipe Stiffener

The split pipe stiffener was developed by the University of Texas and the Texas Department of Transportation to facilitate the connection of heavily skewed diaphragms to girders (Quadrato et al., 2010). Regardless of the skew angle of the diaphragm, the diaphragm can be connected squarely to the pipe. Further, the pipe increases the torsional stiffness of the girder where it is used. Recommendations regarding the design of the half-pipe stiffener are found in the research report.

The split pipe stiffener is suitable for intermediate, pier, and end diaphragms, although in the case of intermediate diaphragms (cross frames), on bridges with skew angle over 20 degrees, it is better to design the cross frames square to the girders instead of on the skew. Therefore split pipe stiffeners are more appropriate for connecting pier diaphragms and end diaphragms.

Proper detailing for use of the split pipe stiffener is shown in figure 209. As indicated by the pipe-to-web and pipe-to-flange welding symbol, the pipe is welded all-around, including welding over the web-to-flange fillet weld. This seals the pipe against moisture and durability concerns. The figure shows a connection plate welded to the half pipe.

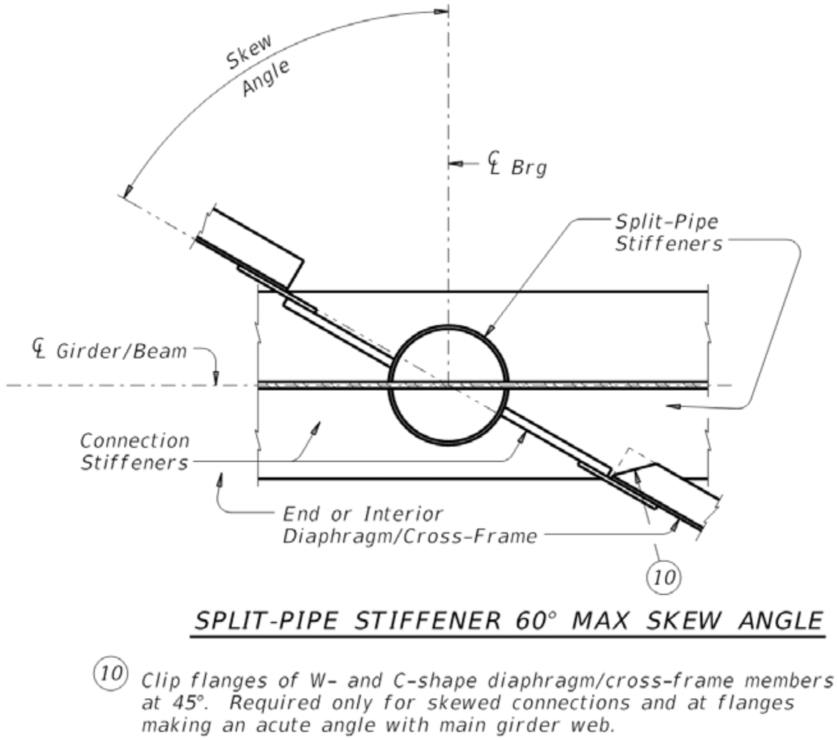


Figure 209. Illustration. Split pipe stiffener connection detail (TxDOT, 2015).

Figure 210 shows the pipe stiffener fit for welding. The pipe stiffener could be welded under the 2015 edition of the Code with some complication: the 2015 edition does not list the material from which the stiffener would be cut. Until the Code includes tubular members, including the pipe material, one could specify the use of a WPS qualified for A709 grades of similar metallurgy, or refer to AWS D1.1. Welding the connection plate to the flanges is optional. Although typical connection plates must be welded to flanges to avoid out-of-plane bending distortion and associated fatigue damage to the web, in this case the half pipe provides the stiffness needed to avoid distortion. Further, as discovered by the study, the split pipe is much stiffer than a bent plate and thereby improves resistance to torsional buckling and increases the elastic buckling strength of the girder.

Figure 210. Photo. Split pipe stiffener fitted for welding.

11.6.6. A709 QST grades (A913)

Based on ASTM A913 Grade 65 and Grade 70, Grade QST65 and QST70 material is new to A709 but not included in the Code as of the 2015 edition. These grades are quenched and self-tempered steel used for rolled beams. High strength, with yield strengths of 65 or 70 ksi, the steel is also tough, ductile, and has good weldability. The QST process consists of heating and cooling the material in-line, with tight controls. The process results in fine grains and thus high toughness.

From a strength standpoint, these grades provide a cost-effective higher yield material for large rolled shapes that somewhat complements Grade HPS 70W, which is not available in shapes; however, these materials do not have weathering capabilities. It is anticipated that the QST grades will be referenced in the Code in future editions.

11.6.7. Castings

Steel castings are formed by pouring molten steel into a mold that is typically made from sand. In the structural (non-bridge) field, steel castings are often used to create nodes or member end connectors to which conventional steel shapes are joined by welding. Steel castings have become popular in architecturally exposed structural steel (AESS) applications. For complex connections where the geometry would cause congestion or weld access issues in conventionally fabricated connections, steel castings may be a practical alternative. For highly loaded connections, steel castings have been used to create strong and stiff nodes while retaining lighter weight and more slender rolled steel elements away from the connection. In fatigue sensitive connections, steel castings can be used to move the locations of welded joints away from geometric stress raisers, thus improving the fatigue performance of the system.

The casting process permits complex, 3-dimensional shapes to be produced, including hollow shapes and configurations that involve different thicknesses of material. The part to be cast must be designed to allow for patterning, molding, feeding of the liquid metal during casting, and proper grain growth during solidification. The shape of the casting must together satisfy the requirements of the engineer, the foundry and the fabricator. As is often the case in steel construction, cooperative collaboration between the involved parties is necessary to achieve an effective solution.

Due to the steel casting process, steel castings will always exhibit some non-metallic inclusions and porosity. The casting designer should establish NDE criteria that result in a product capable of safely addressing the structural performance requirements for the application. While modern foundries use sophisticated solidification modelling software to predict the internal quality of cast products, the first casting produced to a new geometry (termed the “first article”) should be closely scrutinized to ensure the part and casting design process result in the appropriate quality. In addition to visual inspection, appropriate NDE methods include UT, MT, and RT. RT is typically reserved for first article components in the structural field.

Several ASTM specifications govern steel castings. Commonly specified grades for structural applications include the following:

- ASTM A216 Grades WCA, WCB, WCC – These three grades each have different minimum specified yield strength, ranging from 30 to 40 ksi, along with corresponding changes to the chemical requirements. Castings supplied to ASTM A216 are the most readily welded that are also suitable for structural uses but have limited strength. A supplementary specification can be applied to control the maximum carbon equivalency for better weldability.
- ASTM A352 Grades LCA, LCB, LCC – These three grades are commonly used for structural applications. The LCA, LCB and LCC grades are like those of ASTM A216, but A352 requires that the cast steel material be capable of delivering specified Charpy V-notch toughness. A supplementary specification can be used to control the maximum carbon equivalency for better weldability.
- ASTM A958 Grade SC8620 Class 80/50 – This grade is the most common of the A958 grades used for structural applications. The primary benefit of using ASTM A958 Grade SC8620 Class 80/50 is the higher minimum specified yield strength of 50 ksi (345 MPa). However, the strength is gained by the addition of Cr, Mo and Ni, and the alloy additions diminish weldability.

The Code does not include steel castings, and therefore if steel castings are included a bridge project and welded, the contract documents must provide welding instructions. As of the 2015 edition, AWS D1.1 also does not include castings, but castings may be included in future editions.

For castings to be used on a bridge project, many code modifications and additions would be required. Because a casting is an unlisted steel, WPS qualification testing would require tests made on castings. Note that obtaining cast material suitable for the qualification test can be a challenge. The complex geometric configurations which might have justified the use of a steel casting may not provide a configuration suitable for the required qualification tests. It may, therefore, be necessary to have special flat plate-like slabs cast to make the WPS qualification test plates. However, there may be a significant difference in thickness of the casting to be used in production and the plate-like slab used for WPS qualification. The differences between production casting and specially made slab-like plates may result in differences in mechanical properties, or the producing foundry may use a different chemical composition or heat treatment to attain the required properties on the thicker material as compared to the thinner test plates. Thus, it may be difficult to directly apply the data gained from the WPS qualification test to welding on production castings. Special caution is justified when the casting used in the qualification test is of a significantly different chemical composition than that used for the production castings (even though the same casting grade may apply).

While the welding of steel castings is slightly more complex than welding on hot rolled sections, it is routinely done. Casting suppliers often have helpful information on welding that can assist in the design of projects utilizing these materials.

CHAPTER 12 - RESOURCES

The documents listed in this chapter are useful references for engineers and others seeking information about welding and weld design.

12.1. AASHTO DOCUMENTS

LRFD Bridge Design Specifications
These specifications are required by the FHWA for the design, evaluation, and rehabilitation of highway bridges. Chapter 6 covers steel bridge design, including provisions for welded details (AASHTO, 2017a).

LRFD Bridge Construction Specifications
These specifications are not adopted directly by many states but are used as a reference for many state standard specifications. They represent a conservative consensus among all states. Chapter 11 addresses steel fabrication and erection, and refers to AASHTO/AWS D1.5 (AASHTO/AWS, 2015) for welding. As of this writing, AASHTO is in the process of creating a standalone steel fabrication specification that will supersede the steel bridge fabrication portions of chapter 11 (AASHTO, 2017). The *LRFD Bridge Construction Specifications* are discussed in section 1.4.3 of this manual.

LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals
These specifications address design, fabrication, and erection of highway signs, luminaires, and traffic supports (AASHTO, 2015).

LRFD Guide Specifications for the Design of Pedestrian Bridges
These guide specifications govern the design and construction of common pedestrian bridge types (AASHTO, 2009).

12.2. AWS DOCUMENTS

12.2.1. AWS D1 Documents

The AWS D1 documents are introduced in section 1.4.1. All include some weld-related design provisions, inspection and acceptance requirements, workmanship requirements and tolerances, and requirements for qualification of procedures and personnel.

AASHTO/AWS D1.5, *Bridge Welding Code*
This code, which is the main focus of this manual, is specified by almost every bridge-owning agency in the United States and used in some other countries as well. It is a joint publication of AASHTO and AWS. The commentary is helpful for understanding the background and intention of many of the code’s provisions. Many of its details are discussed throughout this manual. Designers should be particularly aware of the design provisions of clause 2 and the fabrication tolerances in clause 3 (AASHTO/AWS, 2015). The 2015 edition does not include

provisions for tubular members or steel other than carbon or low-alloy steel listed in ASTM A709 (ASTM, 2018a).

AWS D1.1, *Structural Welding Code—Steel*
This code is used for many agency-owned steel structures not subject to highway loading, such as pedestrian bridges and overhead sign structures. It is discussed in several locations within this manual. The scope of AWS D1.1 includes both cyclically and statically loaded carbon and low-alloy members, both tubular and non-tubular, for steels up to a minimum specified yield strength of 100 ksi and with a minimum thickness of $^{1}/_{8}$ inch. D1.1 is also discussed in section 10.1.2. Appendix B of this manual outlines some key differences between AWS D1.1 and AASHTO/AWS D1.5 (AWS, 2015).

***AWS D1.2,* Structural Welding Code—Aluminum**
This code is used for many agency-owned aluminum structures such as sign structures and pedestrian bridges (AWS, 2014a). D1.2 is discussed in section 11.1.1.

***AWS D1.4,* Structural Welding Code—Reinforcing Steel**
This code is specified by many owners for the welding of steel reinforcement in concrete (AWS, 2011a). D1.4 is discussed in section 11.1.3.

AWS D1.6, *Structural Welding Code—Stainless Steel*
This code is specified by many owners for the welding of stainless steel components. It also governs the welding of stainless steel to carbon or low-alloy structural steel (AWS, 2017a). D1.6 is discussed in section 11.1.2.

AWS D1.7, *Guide for Strengthening and Repairing Existing Structures*
This document contains useful information for repairs and retrofits on older structures. At the time of publication of this manual, it was undergoing a major revision to be published in 2020 (AWS, 2010).

12.2.2. Other AWS Documents

AWS A2.4, *Standard Symbols for Welding, Brazing, and Nondestructive Examination*
This standard is the definitive reference for proper use of welding symbols (AWS, 2012b). The summary chart from the end of the standard is available separately as AWS A2.1. AWS A2.4 is discussed in section 8.2.

AWS 3.0, *Standard Welding Terms and Definitions*
This standard establishes definitions of terms used in other AWS references. It also defines standard vs. non-standard terminology and standard abbreviations. Some terms are repeated in the glossaries of other AWS documents, but many are defined only here (AWS, 2010c). AWS 3.0 is used extensively in sections 2.2 to 2.4 of this manual.

AWS A5 Documents
These specifications govern the formulation, filler metal and weld metal characteristics, classification testing requirements, and nomenclature for welding consumables (electrodes, flux, and gas), as well as other filler metal related issues. Steel bridge designers typically do not need to use these specifications directly, but the commentary for various designations is informative. The A5 documents are introduced in section 1.4.1.4.

Welding Inspection Technology
This book is most often used as the study guide for the closed-book portion of the CWI exam, but it is a good general reference for welding-related topics, including metallurgy and NDE (AWS, 2008).

12.3. AASHTO/NSBA STEEL BRIDGE COLLABORATION DOCUMENTS

The Collaboration's website states:

> *"The AASHTO/NSBA Steel Bridge Collaboration is a joint effort between the American Association of State Highway and Transportation Officials (AASHTO) and the National Steel Bridge Alliance (NSBA) with representatives from state departments of transportation, the Federal Highway Administration, academia, and various industry groups related to steel bridge design, fabrication and inspection. The mission of the Collaboration is to provide a forum where professionals can work together to improve and achieve the quality and value of steel bridges through standardization of design, fabrication and erection."*

The Collaboration publishes two types of consensus documents that are approved by AASHTO: "S" documents (which are called either Specifications or Guide Specifications), and "G" documents (which are usually called Guidelines). Documents relevant to welding and that are recommended as references are discussed below.

12.3.1. "S" Documents

The "S" documents were first called Guide Specifications by AASHTO but more recently have been permitted to be called Specifications; there is no intended contractual difference between the two types of "S" document. The "S" documents are written in specification-style language so that they may be adopted by reference in their entirety. Some owners have adopted some of the "S" documents by reference; many more use them as a resource in writing their own standard specifications.

S2.1, *Steel Bridge Fabrication Guide Specification*
This document reflects the state of the art in steel bridge fabrication requirements. It will eventually be superseded by the forthcoming AASHTO steel fabrication specification, but the Collaboration will continue to provide input and guidance (AASHTO/NSBA Steel Bridge Collaboration, 2018a). S2.1 is discussed in section 1.4.3.

S10.1, ***Steel Bridge Erection Guide Specification***
This document addresses steel bridge erection, including references to field welding (AASHTO/NSBA Steel Bridge Collaboration, 2014a).

12.3.2. "G" Documents

The "G" documents are intended to provide recommendations and guidance.

G1.1, ***Shop Drawing Approval Review/Approval Guidelines***
This document provides guidelines for use by engineers in their review and approval of shop drawings submitted by the fabricator or contractor. It includes general provisions and lists of specific items that should be checked (AASHTO/NSBA Steel Bridge Collaboration, 2000). G1.1 is discussed in section 10.3.

G1.2, ***Design Drawings Presentation Guidelines***
This document provides guidelines for engineers and their drafting staff for the creation of design drawings, including the minimum information that needs to be provided to the fabricator, along with sample drawings (AASHTO/NSBA Steel Bridge Collaboration, 2003). G1.2 is discussed in section 10.2.

G1.3, ***Shop Detail Drawing Presentation Guidelines***
This document provides guidelines for fabricators and detailers for the creation of shop drawings. It includes lists of what should be shown for clarity on the shop floor and for ease of review, and includes sample drawings. Although the document is written as a guideline with nonmandatory language, it has been specified by a number of owners (AASHTO/NSBA Steel Bridge Collaboration, 2002).

G2.2, ***Guidelines for Resolution of Steel Bridge Fabrication Errors***
This document presents nonconformances frequently encountered in steel bridge fabrication shops, with recommended solutions. It is intended as a guide for both engineers and fabricators (AASHTO/NSBA Steel Bridge Collaboration, 2016). G2.2 is discussed in section 10.6.

G12.1, ***Guidelines to Design for Constructability***
It provides recommendations for design details that are more economical and easier to fabricate, while still providing appropriate structural performance (AASHTO/NSBA Steel Bridge Collaboration, 2016a). This document is mentioned in section 9.3.2.4.

12.4. OTHER DOCUMENTS

AISC Design Guide 21, ***Welded Connections—A Primer for Engineers***
Some of the material presented in Design Guide 21 has been incorporated into this manual, but Design Guide 21 contains more detail on additional topics such as thermal cutting, details of specific welded connections, welding requirements for specific steels, distortion control, seismic welding issues, fatigue

considerations, specialized welding applications, and specific welding problems and their fixes. Design Guide 21 is written for construction of steel buildings, and centers around AWS D1.1, but many of the principles are applicable to bridges as well (Miller, 2018).

AREMA *Manual for Railway Engineering,* Chapter 15, "Steel Structures"
While the AREMA *Manual* is officially a recommended practice, it is adopted as a specification for most railroads. Chapter 15 includes provisions for design and fabrication of steel bridges, and refers to AASHTO/AWS D1.5 (AASHTO/AWS, 2015) for welding (AREMA, 2019).

Procedure Handbook of Arc Welding
Although much of this book is intended more for those who produce the welds than those who design them, there are sections on welded design, and more detailed presentations of various fundamental welding topics than those included in this manual (James F. Lincoln Arc Welding Foundation, 2003).

Design of Welded Structures
Written in 1966 by Omer Blodgett, this book continues to be an important reference for the design of welded connections. Since it was written before the allowable stresses on fillet weld throats were changed in AWS D1 standards in 1969, it is overly conservative with respect to sizing of fillet welds. The principles used in the book remain sound. (Blodgett, 1966).

APPENDIX A - GUIDE TO REVIEW AND APPROVAL OF WELDING PROCEDURE SPECIFICATIONS (WPSs) AND PROCEDURE QUALIFICATION RECORDS (PQRs) UNDER THE BRIDGE WELDING CODE (AASHTO/AWS D1.5)

Overview

This appendix provides a guide to the review and approval of welding procedure specifications (WPSs) and procedure qualification records (PQRs) under the Bridge Welding Code. The directions are based on these basic steps:

- First, conduct a high-level review to establish what the WPSs will be used for (if known), what qualification tests are required and included in the PQRs, and what choices the fabricator may have made for achieving the qualification tests. This review will help ensure that the reviewer does not unnecessarily work through the details of the WPS and PQR before confirming that the WPS is applicable. Note, however, that this step applies for project-specific procedures; since the Code permits WPSs to be used across multiple projects, fabricators may choose to submit procedures that are not necessarily tied to a specific project.
- Second, review the WPSs and qualification tests documented in the PQRs.

This appendix is closely aligned with section 4 of this manual. Section 4 addresses the purpose, development and application of WPSs and qualification tests, and provides a good resource for facilitating WPS and PQR review and approval.

As described in section 4.1.1, all bridge welding under the code must be done in accordance with an approved WPS. As discussed in section 4.2, qualification of WPSs is by qualification tests, which are documented in a PQR. Therefore, review and approval of WPSs includes both the WPS itself and the PQR that supports the WPS.

It is important to note that not all WPSs require the same qualification tests. As described in section 4.2 and summarized in table 3, the qualification testing associated with the WPS varies.

Further, one PQR may support multiple WPSs, and one WPS may be supported by more than one PQR (section 4.3.6).

High-Level Review

Conduct this high-level review before reviewing the specific elements of the WPS, including the WPS itself and any qualification tests:

1. If the WPS is for a project-specific submittal, establish the application of the WPS—i.e., for what welds and what materials does the fabricator intend to use the WPS?

2. Recognize the qualification testing requirements for the WPS depending upon its application, possibly to include:
    a. A groove weld qualification test – applies to groove weld WPSs and multipass fillet weld WPSs

b. A fillet weld soundness test – applies to fillet weld WPSs
c. A nonstandard joint qualification test – applies if a nonstandard joint is to be used

3. If a groove weld qualification test is required, recognize the qualification approach used by the fabricator. This will be one of the following:

   a. The heat input approach (section 4.2.2.1); if so, then either:
   b. The maximum heat input approach, or
   c. The maximum minimum heat input approach
   d. The production approach (section 4.2.2.2)
   e. The pretest/verification approach (section 4.2.2.3) – this is very unlikely based on current practices
   f. The ESW approach (section 4.2.4) – If the WPS is for ESW, the qualification will be the special approach that the code uses for ESW

4. Based on items 1 through 3, establish the elements of the WPS to review. These will include:

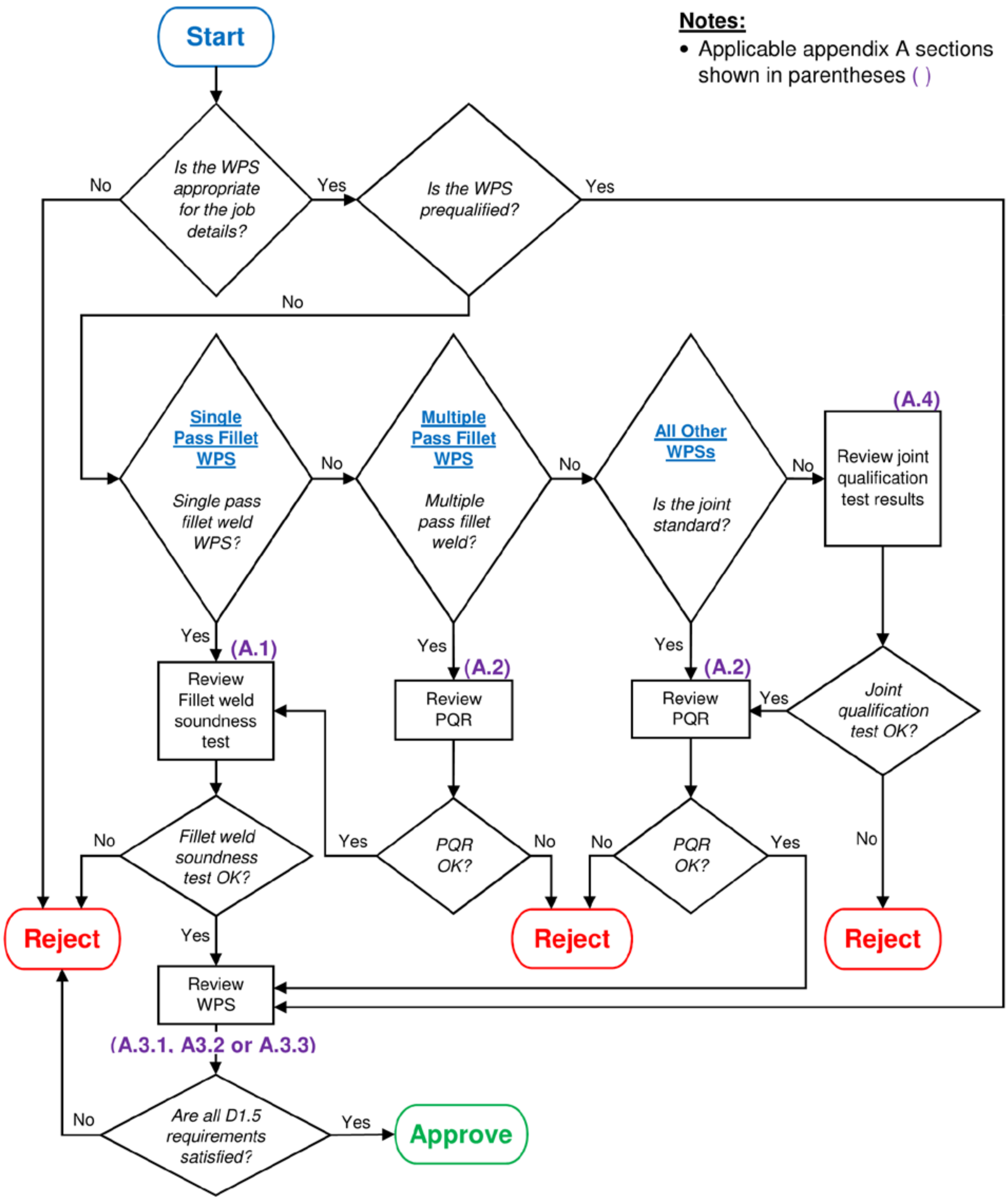
   a. The WPS itself
   b. If the WPS is not prequalified (section 4.2.1), the groove weld qualification test in the PQR(s), if the WPS is for a groove weld or multipass fillet weld
   c. The fillet weld soundness test if the WPS is for a fillet weld.

As discussed in the review section below, it is prudent to check the qualification tests before checking the WPS because the qualification tests support the values in the WPS or WPSs.

Review of the WPSs and PQRs

A WPS approval flow chart shown in figure 49 of chapter 4 is repeated below in figure 211. One approach for review and approval of WPSs (and associated PQRs) is to follow the flow chart logic. The flow chart has labels for the various elements that might be reviewed; items in parentheses, (A.1, A.2, etc.), represent the applicable following sections of this appendix. These elements are not necessarily presented in the order in which they should reviewed, particularly because not all WPSs require all of the same qualification tests.

A basic premise of the flow chart is that, after the high-level review, the qualification test reports are checked before the WPS is checked in conjunction with those test reports. This is because if the qualification tests are not correct, the WPS cannot be correct.



Source: FHWA

Figure 211. Illustration. WPS approval flow chart.

Table of Contents – Review Sections

A.1. FILLET WELD SOUNDNESS TEST (FWST) REVIEW

Steps for review and approval of fillet weld soundness test (FWST):

1. Check that the test plate thicknesses are correct for the fillet weld size being tested.

2. Check for proper preparation of the macroetch test specimens (clause 5.10.3(2) and 5.18.2).

3. Check specimen requirements. Macroetches must satisfy requirements of clause 5.19.3.

4. For multipass fillet welds, ensure that the parameters of the FWST are within the limitation of variables of the PQR. (This means essentially checking the FWST to the PQR as if it were a WPS. See the multipass fillet weld provisions of section A.3.2 below.)

A.2. GROOVE WELD QUALIFICATION TEST REVIEW (PQR REVIEW)

Fabricators may use Forms O-3 and O-4 (or O-8 for ESW) from the Bridge Welding Code or choose their own form, as long as the form presents the same information as required by the code. If the owner has a required form, check that the owner's form is used. Even if the owner has a required form, owners frequently grant an exception for reciprocity—i.e., if the PQR was previously approved by another owner.

There are a number of approaches for conducting the groove weld qualification test (section 4.2.2). The instructions below encompass all of these methods. For any step where there is a difference depending upon the method, each approach is covered.

A.2.1. Steps for review and approval of procedure qualification records (PQRs) except ESW

1. Check the date of the PQR for validity if it is for fracture-critical welding (section 4.2).
    a. Non-fracture critical PQRs do not expire (clause 5.3).

b. Clause 12.7.4 provides expiry provisions for fracture-critical PQRs. There are two limits:

    i. Five years - clause 12.7.4 states that “tests shall be conducted at a frequency that will ensure no groove weld PQR used for preparation of WPSs is more than 60 months old”. Because the date of WPS preparation can be ambiguous, the customary and recommended application of this requirement is that to be valid, the PQR has to be within 60 months of production welding.
    ii. One year - clause 12.7.4 also requires that a fabricator who has not previously performed fracture-critical welding use WPSs based on a PQR that is no more than one year old when beginning fracture-critical welding.

2. Check that the welding consumables and associated materials are of an electrode or electrode/flux combination classification permitted under the Bridge Welding Code in D1.5 table 4.1.

3. Verify that the test plate configuration was in conformance with D1.5 figure 5.1.

    a. For all processes except EGW, verify that the joint used conforms to one of the details listed in note 3 of the figure.
    b. For EGW, verify that the test plate meets clause 5.13.1.

4. Check preheat temperatures:

    a. For maximum heat input qualification plates (clause 5.12.1), a minimum 210 °F, or as required in clause 4.2 (thicker plate, which may be used for EGW, may require a higher temperature).
    b. For minimum heat input qualification plates (clause 5.12.2), 50–100 °F (for both fracture-critical and non–fracture-critical procedure qualification tests).
    c. For production qualification plates (clause 5.12.4), minimum preheat in accordance with clause 4.2.

5. Check interpass temperature:

    a. For a maximum heat input test (clause 5.12.1) or a production procedure qualification test (clause 5.12.4), the Bridge Welding Code does not prescribe a maximum interpass temperature limit, except for Grades HPS 70W and HPS 100W (clause 4.2.2).
    b. For a minimum heat input qualification test (clause 5.12.2), the maximum interpass temperature must be 125 °F.
    c. For a maximum heat input plate, the minimum interpass temperature must be at least 210 °F.
    d. For a production procedure qualification test, the minimum interpass temperature must be in accordance with clause 4.2.

6. Verify that all weld passes are recorded, including the amperage, voltage, and travel speed for each pass.

7. Verify that the heat input presented is the properly calculated heat input based on the amperage, voltage and travel speed.

a. For heat input qualifications, the heat input is the average of the heat input for all weld passes deposited during the test except for the root and cap passes.
b. Except for the root and cap passes, the heat input of each pass deposited during the test must be within ±10 percent of the average.
c. The average amperage, voltage and travel speed, excluding root and cap passes, should be presented to facilitate verification of these parameters on the WPS.

8. Check the amperage. If a heat input method (clause 5.12.1 or 5.12.2) is used, the amperage must fall within the limits of D1.5 table 5.10.

9. Verify that all required tests were conducted and that results were satisfactory:

    a. Required tests are listed in clause 5.15.1, except that for fracture-critical PQRs, special CVN requirements apply per clause 12.7.2, and undermatched procedures are exempted from the reduced-section tensile test and bend test per clause 5.15.1 (7) & (8).
    b. Required results are listed in clause 5.19 and D1.5 table 5.1. Clause 5.19.1 requires that tensile strength in the reduced-section specimen must be no less than the minimum nominal base metal strength of the test plate. This requirement can be found in the base metal specification (usually ASTM A709).
    c. Test results should be reported on the top page of the PQR and supported by test reports from an accredited laboratory, attested and signed by laboratory management.
    d. Test reports should show actual test result values for quantitative tests (e.g., tension, CVN) and pass/fail notations for qualitative tests (e.g., bends and macroetches).

10. The fabricator should attest to the accuracy and truthful representation of the information in the PQR by having an authorized representative sign the report (section 4.3.1). Example form O-3 in the code includes this attestation with a space for signature.

11. Verify that the qualification test welding and machining was witnessed by an owner witness or third-party witness; preferably this will be indicated with a signature attesting to this witnessing.

A.2.2. Steps for review and approval of PQRs for ESW

This section provides review of PQRs for ESW.

1. Check that the welding consumables and associated materials are of an electrode or electrode/flux combination classification permitted under the Bridge Welding Code in D1.5 table 4.1.

2. Verify that the amperage, voltage, and travel speed are recorded.

3. Verify that:

    a. The PQR shows all of the variables in D1.5 table 5.6.
    b. The groove dimension conforms with clause 4.22.1.
    c. The electrode and flux conform with Annex I.
    d. The electrode is tubular, and its diameter conforms with clause 4.22.6.

   e. Consumable guides conform with clause 4.22.2 and clause 4.22.3.
   f. Travel speed conforms with clause 4.22.8.2.

4. Verify that all required tests were conducted and that results were satisfactory:

   a. Required tests are listed in clause 5.15.1, except that undermatched procedures are exempted from the reduced-section tensile test and bend test per clause 5.15.1 (7) & (8).
   b. Required results are listed in clause 5.19 and D1.5 table 5.1. Clause 5.19.1 requires that tensile strength in the reduced-section specimen must be no less than the minimum nominal base metal strength of the test plate. This requirement can be found in the base metal specification (ASTM A709).
   c. Test results should be reported on the top page of the PQR and supported by test reports from an accredited laboratory, attested and signed by laboratory management.
   d. Test reports should show actual test result values for quantitative tests (e.g., tension, CVN) and pass/fail notations for qualitative tests (e.g., bends and macroetches).

5. The fabricator should attest to the accuracy and truthful representation of the information in the PQR by having an authorized representative sign the report (section4.3.1). Although no such attestation is shown on example form O-8, the PQR form for ESW, such an attestation can be seen on example form O-3.

6. Verify that the qualification test welding and machining was witnessed by an owner witness or third-party witness; preferably this will be indicated with a signature attesting to this witnessing.

A.3. WELDING PROCEDURE SPECIFICATIONS (WPSs)

Given their inherent differences, procedures are divided into two types: fillet weld procedures and groove weld procedures.

Fabricators may use Form O-2 or Form O-9 (for ESW) from the Bridge Welding Code or choose their own form, as long as the same information is presented that is on these forms (section 4.2). If the owner has a required form, check that the form is used.

Unless they are prequalified, WPSs must be qualified by testing, and such test results are described in a PQR. The parameters listed on a WPS may vary from those tested and reported on the PQR as allowed in D1.5 tables 5.4, 5.5, and 5.6 and clause 5.12.3 as applicable.

A.3.1. Groove WPS other than ESW

For groove welding procedures that are not prequalified, review as follows:

1. Check that the PQR is referenced on the WPS.

2. Check the welding process for applicability under the Bridge Welding Code as required in clause 1.3.1.

3. Check that the groove weld qualification method is noted (the qualification test itself is reviewed as described in other steps). The production method (clause 5.12.4) must be used:

   a. For EGW.
   b. If the base metal to be welded is not listed in the Bridge Welding Code.
   c. For procedures of more than two passes that use active flux.
   d. For Grade HPS 100W with matching strength electrodes.

   For other cases, choice of qualification method (5.12.1, 5.12.2, 5.12.4) is at the fabricator's option.

4. Check the base metal identified on the WPS for applicability under the Bridge Welding Code. The Bridge Welding Code lists "approved" base metals in clause 1.2.2.

5. Check that the base metal listed on the PQR is acceptable for qualifying the base metal listed on the WPS. Requirements and allowances are described in clauses 5.4.1 and 5.4.2. If other materials are to be used, special considerations apply. Clause 5.4.3 describes qualification of WPSs for unlisted base metals.

6. For hybrid joints, check that the base metal listed on the PQR matches the lower strength base metal listed on the WPS.

7. Check that welding type matches the process:

   a. "Manual" is only SMAW for bridge welding.
   b. "Mechanized", "Semiautomatic", and "Automatic" may apply to SAW, GMAW, and FCAW.
   c. "Tandem" and "parallel" only apply to SAW, and only with more than one wire.

8. Check current type and polarity. The current type (AC or DC) or polarity (DC+ or DC−) must be the same as that listed on the PQR (5.12.1.3 or 5.12.2.1(2) for heat input methods, line 8 of D1.5 table 5.4 for production method).

9. For GMAW qualified under the production method (clause 5.12.4), check that the mode of transfer on the WPS matches the mode of transfer on the PQR.

10. Check that the welding consumables on the WPS match the welding consumables on the PQR as required by D1.5 table 5.2.

11. Check the filler metal specification and classification to be used for compatibility with the base metal to be welded as required in D1.5 table 4.1 and, as applicable, D1.5 table 4.2:

    a. D1.5 table 4.1 applies for all applications.
    b. D1.5 table 4.2 applies for weathering steel when it is to be uncoated. If the WPS material is weathering steel and the filler metal is not in conformance with D1.5 table 4.2 because the weathering steel is to be coated, there should be a note explaining this on the WPS.

c. D1.5 table 4.2 does not cover matching electrodes for ASTM A709 HPS 70W and HPS 100W. For these grades of steel, verify that for uncoated exposed applications, the electrode is formulated for those conditions.

12. If the owner has an approved list of electrodes, check that that electrode brand is on the list (it is not common, but some owners maintain such a list, in accordance with clause 4.1.4.8).

13. For SMAW, check the electrode size for the limitations of clause 4.6.3.

14. Check WPS electrode diameter with respect to the diameter tested:

    a. If a heat input method is used, there is no limit on electrode diameter change.
    b. If the production method (5.12.4) is used, the electrode diameter can increase or decrease by one standard size (line 5 of D1.5 table 5.4) from that shown on the PQR.

15. Check that the number of electrodes is the same as that listed on the PQR (5.12.2.1 for heat input methods, line 6 of D1.5 table 5.4 for production method.

16. If a heat input method is used, check that electrode extension shown on the WPS matches what is listed on the PQR (5.12.1.3 or 5.12.2.1(2)) within:

    a. ¾ inch for SAW.
    b. ¼ inch for GMAW and FCAW.

17. For SAW under the production method, check special filler metal requirements of lines 1 through 4 of D1.5 table 5.4.

18. For SAW using multiple electrodes under the production method, check special multiple electrode requirements of lines 14 through 18 of D1.5 table 5.4.

19. For gas-shielded processes (GMAW, FCAW-G), check the shielding gas for the following:

    a. The electrode and gas combination must be shown on the consumable manufacturer’s certificate of conformance.
    b. The shielding gas must conform with AWS A5.32 (4.12).

20. If a heat input qualification method is used, gas flow rate on the WPS must be at least that on the PQR.

21. Check that welding positions listed on the WPS conform with 5.8.2.

22. Check the joint details:

    For CJPs and PJP welds (except for EGW), the fabricator will usually choose, respectively:

    a. A standard CJP weld joint from D1.5 figure 2.4, or
    b. A standard PJP weld joint from D1.5 figure 2.5.

There should be a sketch on the WPS of the joint to be used; check the sketch of the joint for conformance with D1.5 figure 2.4 or 2.5. Joint details in those figures are specific to the welding process.

If the joint is not standard, the nonstandard joint configuration is required to be qualified by testing to the 5.12.4 (production) qualification method using a D1.5 figure 5.3 test plate and the nonstandard joint to be qualified, in addition to the standard qualification to any 5.12 method using D1.5 figure 5.1 (clause 5.7.5; see section A.4, "Nonstandard Joint Qualification," below). The WPS sketch will show the nonstandard joint details.

23. For CJP weld procedures except for EGW, check the root treatment instructions for the second side of welding; standard CJP weld details require backgouging.

24. For corner and T-joint groove weld procedures, check for the inclusion of a reinforcing fillet weld, and check to ensure the size of the fillet weld is correct; size requirements are found in clause 2.11.

25. Check the preheat for conformance with requirements for the base metal grade and thickness, and fracture-critical status. Since preheats are thickness-dependent, the fabricator may include a table of thickness ranges and associated preheat. For fracture-critical welding, check that preheat conforms with the diffusible hydrogen content of the electrode and the minimum heat input level of the WPS (12.14). For tandem procedures, use the total heat input for all wires to determine the required preheat.

    The fracture-critical preheat tables in clause 12 are silent about preheat for fracture-critical welding when the heat input is under 30 kJ/inch. Common practices about what to do in these situations and which this manual recommends are a) use of the "30 kJ/inch < HI < 50 kJ/inch" column for heat inputs under 30 kJ/inch; b) adding another 25 °F based on the patterns in the tables; or c) for H4 or H8 bump up one category to H8 or H16, respectively.

26. Check maximum interpass temperatures for conformance with clause 4.2, 5.112.2.9, and, for fracture-critical welding, clause 12.14. Clause 5.112.2.9 means that the maximum interpass temperature observed while welding the maximum heat input test must also be listed as the maximum interpass temperature on the WPS.

27. For procedures qualified under the production method (clause 5.12.4):
    a. Ensure that if post-weld heat treatment (PWHT) is shown on the PQR, it is also shown on the WPS, and vice versa (D1.5 table 5.4 line 24).
    b. Check for plate thickness changes from the PQR to the WPS on procedures for Grade HPS 100W (D1.5 table 5.4 line 25).
    c. Although D1.5 table 5.4 line 19 requires the number of passes on the WPS to be within ±25 percent of the number of passes on the PQR, as adjusted for groove cross-section, it is usually not possible to check this because it is not a requirement to list the exact number of passes on the WPS, nor will the number of passes for a large weld be precisely predictable. This parameter is adequately controlled by the limitations on heat input, since heat input controls bead size.

d. Lines 20–23 of D1.5 table 5.4 are listed as applying to qualification using D1.5 figure 5.1. However, the details discussed are not consistent with the required joint details of D1.5 figure 5.1. Logic dictates that these variables apply only when checking a WPS for conformance with a nonstandard joint qualification test in accordance with D1.5 figure 5.3 (see section A.4).

28. Check WPS heat input for conformance with the PQR heat input limits. The amperage, voltage, and travel speed will likely be presented as ranges. Calculate the highest WPS heat input using the highest amperage, highest voltage, and lowest travel speed on the WPS, and calculate the lowest WPS heat input using the lowest amperage, lowest voltage, and highest travel speed on the WPS.

    a. If the maximum heat input method (5.12.1) is used, the heat input must fall within a window bounded by the average heat input of the test plate and 60 percent of this average heat input.
    b. If the maximum–minimum heat input method (5.12.2) is used, the heat input must fall within a window bounded by the average heat input of the maximum heat input test plate and the average heat input of the minimum heat input test plate.
    c. If the production method (5.12.4) is used, the heat input must not vary from the average heat input the PQR by more than the limits allowed in line 13 of D1.5 table 5.4.

29. Check amperage:

    a. If a heat input method (5.12.1 or 5.12.2) is used, the amperage must fall within the limits of D1.5 table 5.10.
    b. If the production method (5.12.4) is used, the amperage must not vary from the average amperage in the PQR by more than the limits allowed in line 7 of D1.5 table 5.4.

30. Check voltage:

    a. If the maximum heat input method is used, the voltage must be within ±10 percent of the average PQR voltage, except that for active or alloy fluxes, the voltage must be within +0 percent and −10 percent of the average PQR voltage (5.12.3.2).
    b. If the maximum–minimum heat input method is used, the voltage must be between 90 percent of the average voltage used on the minimum heat input test and 110 percent of the average voltage used on the maximum heat input test.
    c. If the production method (5.12.4) is used, the voltage must not vary from that shown on the PQR by more than the limits allowed in line 10 of D1.5 table 5.4.

31. Check travel speed:

    a. If a heat input method is used, there is no explicit limit on travel speed, as long as the heat input limits are satisfied.
    b. If the production method (5.12.4) is used, the travel speed must not vary from the average travel speed shown on the PQR by more than the limits allowed in line 12 of D1.5 table 5.4.

32. For EGW, check that the WPS parameters conform to the additional limitation of variables in D1.5 table 5.5.

A.3.2. Groove WPS for ESW

1. Check that the PQR is referenced on the WPS.
2. Check the base metal identified on the WPS for applicability under the Bridge Welding Code. The Bridge Welding Code lists "approved" base metals in clause 1.2.2.
3. For hybrid joints, check that the base metal listed on the PQR matches the lower strength base metal listed on the WPS.
4. Welding type should be "Automatic".
5. Check current type and polarity. The current type (AC or DC) or polarity (DC+ or DC−) must be the same as that listed on the PQR (4.18.2(8)).
6. Check that the electrode and flux conform with Annex I.
7. Check that the electrode is tubular, and its diameter conforms with 4.22.6.
8. If the owner has an approved list of electrodes, check that that electrode brand is on the list (it is not common, but some owners maintain such a list, in accordance with clause 4.1.4.8).
9. Verify that consumable guides conform with 4.22.2 and 4.22.3.
10. Verify that the groove preparation shown on the WPS conforms with 4.22.1.
11. Ensure that if post-weld heat treatment (PWHT) is shown on the PQR, it is also shown on the WPS, and vice versa (D1.5 table 5.9 line 9).
12. Check that voltage conforms with 4.22.8.1.
13. Check that travel speed conforms with 4.22.8.2.
14. Check that the WPS parameters conform to the limitation of variables in D1.5 table 5.6.
15. Verify that all items listed in 4.18.2 are shown on the WPS.

A.3.3. Fillet WPS

The Bridge Welding Code has distinctions between qualification of WPSs for single pass fillet weld procedures and qualification of multipass fillet weld procedures. These are presented together with differences noted as applicable. Unless noted specifically for single pass or multipass, the review step applies to both methods.

Review steps for fillet weld procedures that are not prequalified:

1. Check the base metal identified on the WPS for applicability under the Bridge Welding Code. The Bridge Welding Code lists “approved” base metals in clause 1.2.2. If other materials are to be used, special considerations apply. Clause 5.4.3 describes qualification of WPSs for unlisted base metals.

2. Check the welding process for applicability under the Bridge Welding Code as listed in clause 1.3.1.

3. Check that welding type matches the process:

   a. “Manual” is only SMAW for bridge welding.
   b. “Mechanized”, “Semiautomatic”, and “Automatic” may apply to SAW, GMAW, and FCAW
   c. “Tandem” and “parallel” is only applicable to SAW, and only with more than one wire.

4. Check the filler metal specification and classification to be used for compatibility with the base metal to be welded as shown in D1.5 table 4.1 and, as applicable, D1.5 table 4.2:

   a. D1.5 table 4.1 applies for all applications.
   b. D1.5 table 4.2 applies for weathering steel when it is to be uncoated.
      i. Exceptions are allowed for single-pass fillet WPSs within the limits of clause 4.1.7.
      ii. If the WPS material is weathering steel and the filler metal is not in conformance with D1.5 table 4.2 because the weathering steel is to be coated, there should be a note explaining this on the WPS.
      iii. D1.5 table 4.2 does not cover matching electrodes for ASTM A709 HPS 70W and HPS 100W. For these grades of steel, verify that for uncoated exposed applications, the electrode is formulated for those conditions.

5. Check the electrode brand for conformance with one of the following:

   a. The consumable manufacturer’s Quality Assurance Program is approved by the American Bureau of Shipping (ABS), Lloyd’s Register of Shipping, or the American Society of Mechanical Engineers (clause 4.1.3.1). Most consumables used by bridge fabricators come from suppliers who are approved by one or the other.
   b. Heat or lot testing (clause 4.1.3.2): if the consumable manufacturer is not producing the consumable under a program audited by one of the three agencies in “a”, then heat and lot testing applies.
   c. Present on the owner’s approved list, if applicable. (It is not common, but some owners maintain such a list, in accordance with clause 4.1.4.8).

6. For SMAW, check the electrode size for the limitations of clause 4.6.3.

7. For fracture-critical welding, verify that the certificate of conformance lists diffusible hydrogen or coating moisture content testing (12.6.1.1).

8. For gas-shielded processes (GMAW, FCAW-G), check the shielding gas for the following:
    a. The electrode and gas combination must be listed on the consumable manufacturer's certificate of conformance.
    b. The shielding gas must conform with AWS A5.32 (4.12).

9. For single-pass procedures, verify that the size of the fillet weld and associated position is allowable as a single pass fillet weld. Clause 4.6.6 provides maximum single-pass fillet weld sizes for SMAW fillet welds, and clause 4.13.1.3 provides maximum single-pass fillet weld sizes for GMAW and FCAW. D1.5 table 2.1 provides minimum single-pass fillet weld sizes for all processes.

10. If joint details are provided on the WPS, check them for conformance with the intended scope, including fillet weld size. There may be instructions for the welder about how to sequence weld passes or where to place them; while these may be helpful to the welder, they are not required under the Bridge Welding Code and as such should not be considered as part of the review and approval process.

11. For SMAW, check that there are instructions for conformance with the maximum weld layer thickness limits in clause 4.6.7.

12. Check the preheat for conformance with requirements for the base metal grade and thickness, and fracture-critical status. Since preheats are thickness-dependent, the fabricator may include a table of thickness ranges and associated preheats. For fracture-critical welding, check that preheat conforms with the requirements for the diffusible hydrogen content of the electrode (clause 12.14). For tandem procedures, use the total heat input for all wires to determine the required preheat.

    The fracture-critical preheat tables in clause 12 are silent about preheat for fracture-critical welding when the heat input is under 30 kJ/inch. Common practices about what to do in these situations and which this manual recommends are a) use of the "30 kJ/inch < HI < 50 kJ/inch" column for heat inputs under 30 kJ/inch; b) adding another 25 °F based on the patterns in the tables, or c) for H4 or H8 bump up one category to H8 or H16, respectively.

13. Check maximum interpass temperatures for conformance with clause 4.2 and, for fracture-critical welding, clause 12.14.

14. Verify conformance with a valid fillet weld soundness test (FWST) (5.10.3):
    a. Check that the FWST is referenced on the WPS.
    b. Check that the base metal used for the FWST supports the base metal listed on the WPS in conformance with clause 5.4.1 or 5.4.2.
    c. For hybrid joints, check that the base metal listed on the FWST matches the lower strength base metal listed on the WPS.
    d. Check that if post weld heat treatment (PWHT) is used on the WPS, it is used on the FWST as well and vice versa.
    e. Check that welding positions listed on the WPS conform with clause 5.8.3.

f. Check that the size of the fillet in the FWST corresponds to that listed on the WPS (D1.5 table 5.4, line 26). D1.5 figure 5.8 lists only the fillet weld sizes that need to be tested; unlisted weld sizes can be qualified with an adjacent fillet weld size.
g. For multipass fillet welds, check that the number of passes shown on the WPS is within 25 percent of the number of passes shown on the FWST (D1.5 table 5.4, line 27). For example, a 5-pass soundness test can be used for 4-, 5-, or 6-pass WPSs ($5 \times 1.25 = 6.25$; $5 \times 0.75 = 3.75$), but 2-pass and 3-pass versions of the same size weld would need separate soundness tests. However, for Grade HPS 100W, each number of passes must have its own soundness test (D1.5 table 5.4 note c).
h. For skewed fillet welds, check that the dihedral (skew) angle range of the WPS is within 10 percent of the dihedral angle used in the fillet weld soundness test (D1.5 table 5.4, line 26).

15. For multipass fillet welds, ensure conformance with a groove welding qualification test (section A.3.1).

A.3.4. Prequalified Groove and Fillet Weld WPS

Under the Bridge Welding Code, only SMAW procedures are prequalified, and this only under certain broad restrictions. For such prequalified procedures, review as follows:

1. Check that the WPS uses the SMAW process and meets the further restrictions of clause 1.3.2, and 12.7.1 for fracture-critical welding.

2. Check the base metal identified on the WPS for applicability under the Bridge Welding Code. The Bridge Welding Code lists “approved” base metals in clause 1.2.2. If other materials are to be used, special considerations apply and the WPS cannot be considered to be prequalified. Clause 5.4.3 describes qualification of WPSs for unlisted base metals.

3. Check the filler metal specification and classification to be used for compatibility with the base metal to be welded as required in D1.5 table 4.1 (and D1.5 table 4.2 as applicable):

   a. D1.5 table 4.1 applies for all applications.
   b. D1.5 table 4.2 applies for weathering steel when it is to be uncoated.
      i. Exceptions are allowed for single pass fillet weld WPSs within the limits of 4.1.7.1.
      ii. If the WPS material is weathering steel and the filler metal is not in conformance with D1.5 table 4.2 because the weathering steel is to be coated, there should be a note explaining this on the WPS.
      iii. If the owner has an approved list of electrodes, check that that electrode brand is on the list (it is not common, but some owners maintain such a list, in accordance with clause 4.1.4.8).

4. For fracture-critical welding, verify that the certificate of conformance lists diffusible hydrogen or coating moisture content testing (12.6.1.1).

5. Check the preheat for conformance with requirements for the base metal grade and thickness, and fracture-critical status. Since preheats are thickness-dependent, the fabricator may include a table of thickness ranges and associated preheat. For fracture-critical welding, check that preheat conforms to the requirements for the diffusible hydrogen content of the electrode (clause 12.14).

   For SMAW procedures, fabricators sometimes prefer not to specify voltage or travel speed on the WPS because these are not machine settings and are controlled as the welder manipulates the electrode to maintain a consistent bead. These numbers, however, are required for heat input calculation, which is needed for establishing preheat for fracture-critical WPSs. Voltage can be measured with a voltmeter during welding and travel speed can be established with a stopwatch; however, the reviewer is encouraged to consider the option of simply assuming the worst-case heat input (the lowest range shown in the fracture-critical preheat tables, 30 kJ/inch < HI < 50 kJ/inch), rather than forcing the fabricator to generate numerical values for voltage and travel speed.

   The clause 12 preheat tables are silent about preheat for fracture-critical welding when the heat input is under 30 kJ/inch. Common practices about what to do in these situations are a) use of the "30 kJ/inch < HI < 50 kJ/inch" column for heat inputs under 30 kJ/inch; b) adding another 25 °F based on the patterns in the tables, or c) for H4 or H8 bump up one category to H8 or H16, respectively.

6. Check that amperage, voltage (if available; recommendations for voltage are not always provided) and polarity are within the manufacturer's recommendations.

7. Check electrode diameter for conformance with maximum limits associated with welding position (4.6.3).

Groove Weld Procedures:

8. For groove welding procedures, check the joint details:

   For CJP and PJP welds, the fabricator will usually choose a standard joint detail from D1.5 figure 2.4 or 2.5, respectively.

   There will be a sketch on the procedure of the joint to be used; check the sketch of the joint for conformance with D1.5 figure 2.4 or 2.5. Joint details in those figures are specific to the welding process being used. If the joint is not standard, the joint details are required to be qualified by testing (clause 5.7.5; see section A.4) However, a groove weld qualification test to D1.5 figure 5.1 is not required.

9. For CJP weld procedures, check the root treatment instructions for the second side of welding; standard CJP weld details require backgouging.

10. For corner and T-groove weld procedures, check for the inclusion of a reinforcing fillet weld, and check to ensure the size of the fillet weld is correct; size requirements are found in clause 2.11.

Fillet Welding Procedures:

11. For single-pass procedures:
    a. Check that the size of the fillet weld and associated position is allowable as a single pass fillet weld (4.6.6).
    b. Check that fillet welds satisfy minimum fillet criteria of D1.5 table 2.1.

12. For multipass procedures, ensure conformance with maximum fill layer thickness limits of clause 4.6.7 and 4.9.3.

A.4. NONSTANDARD JOINT QUALIFICATION

Steps for review and approval of nonstandard joint qualification test reports are as follows. See associated discussion in the manual in section 4.4.4.

1. Check that the base metal used for the test conforms with the requirements of 5.4.1 or 5.4.2.

2. Check that the test was conducted on the appropriate configuration.

3. Check that the tests conducted are the tests required in D1.5 figure 5.3

4. Check that the test results conform with the requirements of clause 5.19.

5. Ensure that the welding parameters of the D1.5 figure 5.3 test are within the limitation of variables with respect to the PQR. (This means essentially checking the D1.5 figure 5.3 test to the PQR as if it were a WPS; see section A.3.1 above.)

6. Check that the parameters of the WPS conform with those of the D1.5 figure 5.3 test within the limitation of variables allowed in D1.5 table 5.4, and also remain within the allowance of the PQR (see section A.3.1).

APPENDIX B - DIFFERENCES BETWEEN D1.1 AND D1.5

The following provides a general comparison based on the requirements of AWS D1.1 *"Structural Welding Code—Steel"* and AASHTO/AWS D1.5 *"Bridge Welding Code"* using the 2015 edition of both codes.

Table 4. AWS D1.1 and AASHTO/AWS D1.5 comparison.

Requirement	AWS D1.1	AASHTO/AWS D1.5
Fabricator certification	Not required	Mandated, with certification category "as required by the engineer"
Material thickness	⅛ inch and thicker	⅛ inch and thicker, although ASTM A709 provides a practical limit of 4 inches
Material limitations	100 ksi yield and less Carbon and low alloy steels	100 ksi yield and less Carbon and low alloy steels
Steel varieties addressed	62 Broad variety are preapproved	7 Only ASTM A709 materials are listed
Applicable structures	Steel structures, including tubular structures	Welded highway bridges (and also commonly used for railway bridges). Not appropriate for pedestrian bridges and other non-bridge transportation-related structural components Not pressure vessels or pressure piping Not structural tubing
Design	Includes design and is complementary to any general code or specification for design and construction of steel structures	Includes some design provisions but fundamentally aligns with design under AASHTO or AREMA
Shop or field	Both	Both
Cyclic loading	Has special design, fabrication, welding and inspection requirements for cyclically loaded members Few distinctions between tension and compression	Entire code is based on assumption that bridges are cyclically loaded structures, therefore, no special distinctions between cyclically loaded and statically loaded members Distinctions between tension and compression
Joints	Prequalified joints are similar to D1.5 standard joints	Standard joints are similar to D1.1 prequalified joints

Requirement	**AWS D1.1**	**AASHTO/AWS D1.5**
Welding procedures specifications (WPSs)	SMAW, FCAW, GMAW, and SAW prequalified for approved materials within certain ranges of parameters and joint details	SMAW prequalified with listed materials; all others must be qualified by testing
Qualification testing and procedure qualification records (PQRs) *As described above, welding procedures in D1.1 are generally prequalified; the qualifications here are used when welding outside of the prequalification limits*	Root and face or side bends, reduced-section tensile test, and CVN if required by contract Fillet weld test available, can also qualify fillet weld procedures with a groove test Essential variables less restricted for fillet weld tests in D1.1 than D1.5	Groove welds and multipass fillet welds: Root and face or side bends, reduced-section tensile test, CVN, macroetch tests, and all-weld-metal tensile tests; for multipass fillet welds, add macroetch (soundness) test Single-pass fillet welds: Macroetch (soundness) tests Fracture-critical PQRs: Valid for five years Essential variables more restricted for fillet weld tests in D1.5 than D1.1
Calibration of welding-related equipment	None	Three-month frequency of verification for welding equipment, meters, or other devices that record or display welding variables Annual certification of equipment used for verification
Electrode and flux brand and classification	WPSs are specific to classification. A change in classification requires a new WPS (and associated test, although these are usually prequalified)	WPSs are specific to classification except for cored electrodes. A change in classification requires a new WPS (and associated testing), and for cored electrodes, a change in brand also requires a new WPS. A change in brand and type of flux requires a new WPS.
Low-hydrogen electrodes	SMAW electrodes not required to be low-hydrogen	SMAW electrodes must be low-hydrogen with associated handling requirements and exposure limits
Preheat	Requirements in association with four categories of materials Preheat requirements are generally lower than D1.5	Requirements in two groupings for non–fracture-critical and three groupings for fracture-critical Preheat requirements are generally higher than D1.1
GMAW-S (short circuit transfer)	Allowed but not prequalified	Only allowed with approval of the engineer

Requirement	**AWS D1.1**	**AASHTO/AWS D1.5**
ESW	No specific process prescription	Mandated as the narrow-gap version and associated tighter requirements
Welder qualification testing	Grinding allowed during test	No grinding allowed during test; however, more reinforcement is allowed
Inspection	Visual criteria similar to D1.5 MT, RT, and UT only apply if required by contract No phased array	Visual criteria similar to D1.1 MT, RT, and UT applications are prescribed Phased array allowed as substitute for conventional UT
General inspector qualification	Choice of: 1) CWI 2) CWB 3) Competency by training and experience	Choice of: 1) CWI 2) CWB 3) Competency by training and experience as approved by the engineer
Strengthening and repair of existing structures	Addressed	Not addressed
Fracture-critical members and associated welding	Not applicable	Has a dedicated clause of requirements to be applied when fracture-critical members are identified on the plans
Stud welding repairs	No process limits	Only SMAW allowed
Temporary welds	No special requirements	Must be removed unless approved to stay by the engineer Not allowed in tension zones of quenched and tempered steels Must be shown on drawings Requirements regarding removal

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