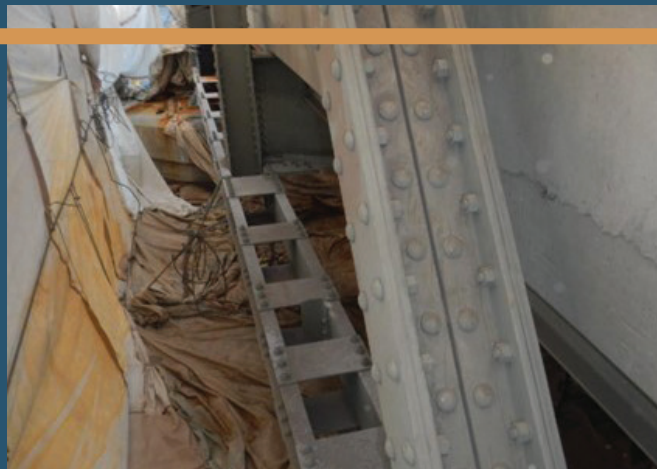
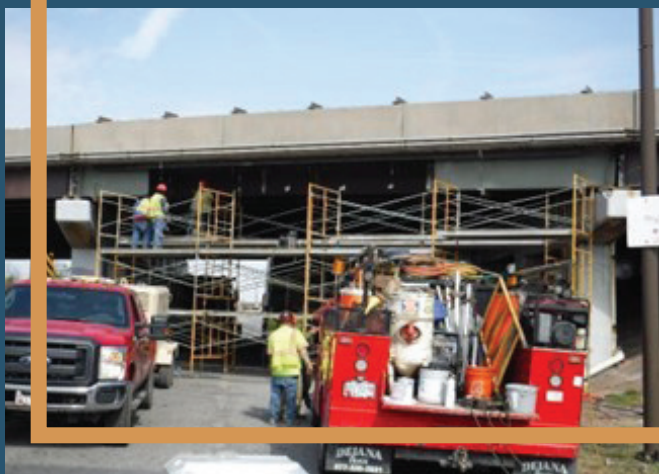




# Guidelines for Field Repairs and Retrofits of Steel Bridges

## G14.2—2023



AMERICAN ASSOCIATION  
OF STATE HIGHWAY AND  
TRANSPORTATION OFFICIALS  
**AASHTO**



### AASHTO/NSBA STEEL BRIDGE COLLABORATION

American Association of State Highway  
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## Preface

This document is a guideline developed by the AASHTO/NSBA Steel Bridge Collaboration. The primary goal of the Collaboration is to achieve steel bridge design and construction of the highest quality and value through standardization of the design, fabrication, construction, inspection, and long-term maintenance. Each standard represents the consensus of a diverse group of professionals.

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## DEFINITIONS AND ABBREVIATIONS

|        |                                                                    |
|--------|--------------------------------------------------------------------|
| AASHTO | American Association of State Highway and Transportation Officials |
| API    | American Petroleum Institute                                       |
| ASD    | Allowable Stress Design                                            |
| ASNT   | American Society of Nondestructive Testing                         |
| AWS    | American Welding Society                                           |
| BDS    | <i>(AASHTO LRFD) Bridge Design Specifications</i>                  |
| BFRPs  | Basalt-Fiber-Reinforced Polymers                                   |
| CAFT   | Constant Amplitude Fatigue Threshold                               |
| CJP    | Complete Joint Penetration                                         |
| CFRP   | Carbon-Fiber-Reinforced Polymer                                    |
| CVN    | Charpy V-notch                                                     |
| CWI    | Certified Welding Inspector                                        |
| DOT    | Department of Transportation                                       |
| EOR    | Engineer of Record                                                 |
| FCAW-S | Flux-Cored Arc Welding, Self-Shielded                              |
| FCP    | Fracture Control Plan                                              |
| FFS    | Fitness for Service                                                |
| FHWA   | Federal Highway Administration                                     |
| FRP    | Fiber-Reinforced Polymer                                           |
| HLMR   | High-Load Multi-Rotational (Bearing)                               |
| IDOT   | Illinois Department of Transportation                              |
| LRFD   | Load and Resistance Factor Design                                  |
| LRFR   | Load and Resistance Factor Rating                                  |
| LTB    | Lateral-Torsional Buckling                                         |
| MBE    | <i>Manual for Bridge Evaluation</i>                                |
| MT     | Magnetic Particle Testing                                          |
| NCHRP  | National Cooperative Highway Research Program                      |
| NDE    | Nondestructive Evaluation                                          |
| NDT    | Nondestructive Testing                                             |
| NSBA   | National Steel Bridge Alliance                                     |
| NSTM   | Nonredundant Steel Tension Member                                  |
| OI     | Owner Inspector                                                    |
| PAUT   | Phased-Array Ultrasonic Testing                                    |
| PJP    | Partial Joint Penetration                                          |
| PT     | Liquid Penetrant Testing                                           |
| PTFE   | Polytetrafluoroethylene                                            |
| QC     | Quality Control                                                    |
| QMS    | Quality Management System                                          |
| RCSC   | Research Council on Structural Connections                         |
| RFI    | Request for Information                                            |

|      |                                 |
|------|---------------------------------|
| RT   | Radiographic Testing            |
| SMAW | Shielded Metal Arc Welding      |
| SRM  | System Redundant Member         |
| UHPC | Ultra High-Performance Concrete |
| UT   | Ultrasonic Testing              |
| VT   | Visual Testing                  |
| WPS  | Welding Procedure Specification |

# SECTION 1: INTRODUCTION AND FUNDAMENTALS

The necessity for field repairs of U.S. bridges is urgent. A review of the National Bridge Inventory shows that the average age of all in-service steel bridges is currently 50 years, and that there are over 92,000 steel bridges in service that are older. This is significant when considering that many of those bridges have already reached or exceeded their intended design lives. The railroad industry has an even more impressive aging steel bridge inventory, with 70 percent of their deck plate girder bridge inventory exceeding 80 years of service, 50 percent of them over 100 years old, and roughly two percent exceeding 120 years of service (Otter et al., 2021).

Field repairs that can extend the service life of steel bridges are a necessity for Owners who continue to manage an aging network of bridges to service our communities and economy. These Guidelines provide guidance for Owners and consultants related to the most common forms of damage in steel bridges to help ensure the nation's infrastructure continues safely and dependably into the future. The strategies discussed in these Guidelines are meant to help users understand options that may be available to them so they can evaluate which are appropriate for their particular application. Not all of the strategies discussed in these Guidelines are appropriate for every application. In fact, some are newer strategies that may not have been in service long enough to evaluate long-term performance.

## 1.1—GENERAL CONSIDERATIONS FOR EXECUTION OF FIELD REPAIRS

Repairs and rehabilitation are undertaken to facilitate long-term performance of bridges and optimize service life, including—in some cases—the need to restore safety. There are historical examples of bridge collapses due to loss of section from corrosion, impact damage, and overloading. When these deficiencies are identified, the bridge must be evaluated to determine if traffic needs to be restricted or removed, or if temporary stabilization is required before resuming normal traffic patterns. Repairs and retrofits should only commence after any necessary stabilization or traffic restrictions have been implemented.

Many larger agencies have staff dedicated to repair and rehabilitation work. Their experience often allows them to make field decisions to accelerate repair initiatives. By contrast, some smaller agencies may not employ in-house staff with the expertise to make field decisions and are more likely to rely on a consultant to support them in their decision making.

Similarly, Contractor experience can vary dramatically when the Contractor is selected on a low-bid basis. Many larger agencies use qualification-based selections to hire on-call Contractors to perform maintenance and repairs under a force account (or time and material) arrangement. In these cases, the Contractor and agency staff often develop an understanding of when additional discussions are necessary. For contracts that are procured on a design-bid-build basis, it may be beneficial to review the agency's standard specifications and plan notes and discuss whether additional notes are warranted to foster collaboration between the Contractor and Designer.

Repair contracts generally require more field decisions to be made than new construction. Therefore, repair plans need to be formulated in such a way that the goals of the repairs are clearly communicated while allowing some flexibility in the execution.

## 1.2—REPAIRABILITY, ADAPTABILITY, AND DURABILITY OF STEEL BRIDGES

Repairability, adaptability, and durability are important characteristics of our infrastructure (e.g., to protect structures against environmental factors, to achieve relatively maintenance-free structures, and to have the ability to adapt to changes in usage over the service life promptly). Many Owners now target a 100-year design life (Azizinamini et al., 2014). It is difficult to predict the conditions, load combinations, traffic demands, or other maintenance and durability challenges a 100-year service life could potentially pose for Owners. Bridges with repairability, adaptability, and durability, therefore, are desirable. Structural steel bridge components can be easily strengthened and adapted as future needs arise to address new and changing demands without requiring complete replacement. When necessary, steel bridges have been

efficiently and rapidly repaired following vehicle or barge impact, fire, or other unforeseen conditions. Consider a few more notable examples of steel bridges that experienced some form of damage and were quickly restored to full capacity:

- The Hernando De Soto bridge carries I-40 across the Mississippi River. When a fracture of the tie girder was discovered in 2021, an emergency repair was designed and fabrication commenced within one week, and the emergency stabilization was completed within two weeks of the discovery of the fracture. In less than eight weeks, a comprehensive repair of the fractured girder was completed.
- The Brent Spence Bridge, a through-truss carrying I-75/I-71 traffic across the Ohio River, was subjected to severe fire damage when a box truck crashed and caught fire on the lower of two decks in 2020. The bridge was closed, inspected, repaired, and reopened to traffic 41 days later (which included surgically removing and replacing a section of the floor system and casting a new concrete deck section).
- In 2016, during a rehabilitation project for the Liberty Bridge, a construction fire accidentally started below one of the primary compression members of the deck-truss bridge. The compression member weakened in the heat and buckled under load. The bridge was closed, repaired, and reopened within 27 days.
- The Mathews Bridge in Florida experienced the complete severing of a tension chord member by an overheight barge in 2013. The broken tension chord of the through-truss was repaired, and the bridge reopened within 34 days.
- The Diefenbaker Bridge crosses the North Saskatchewan River in Canada. In 2011, a person below the bridge discovered that one of the two girders had a full-depth fracture—the bridge continued to carry service loads, with the fracture unnoticed by travelers. The cause was found to be a constraint-induced fracture, a result of poor detailing that is no longer a risk factor for new designs. The bridge was immediately closed to traffic, repaired, and reopened three months and 21 days later.

Simple and efficient field repairs and ease of inspection of steel bridge members make steel bridges ideal candidates for the extended service lives desired by Owners. Dependable and rapid repairs are needed for the traveling public and businesses who rely on safe and accessible infrastructure to commute to work and school and to move goods and services. Some Owners enhance their bridge repair capabilities by using standard drawings and authorizing the bridge office to execute emergency contracts.

### 1.3—HISTORICAL MATERIAL PROPERTIES

Field repair of steel bridges may require an understanding of the steel material properties, such as yield and tensile strength, hardness, toughness, or weldability. However, the original design plans are typically not available to identify the type of steel from which the bridge was built, nor are the material properties characteristic of a particular steel. Those properties can play an important role in the planning phase of a field repair such that an appropriate repair procedure can be designed, taking into account any material limitations.

Fortunately, there are references available that help to identify what type of steel was most likely used based on the year of construction. The American Association of State Highway and Transportation Officials' (AASHTO's) *Manual for Bridge Evaluation* (MBE) provides estimated steel properties and concrete compressive strength based on the year of construction. Other resources include AISC's *Design Guide 15* (AISC, 2018), Ocel (2021), and the NSBA *Steel Bridge Design Handbook*, Chapter 1. The NSBA *Steel Bridge Design Handbook*, Chapter 1 provides a general overview of the properties of bridge steels, including hollow structural section specifications, wires and cables, bolts and rivets, castings, and also how processes used in fabrication, such as thermal cutting or cold bending, might affect them. Ocel (2021) is recommended for determining material properties of historical bridge steels based on the date of construction. That publication explains the advancements made over the years in the steelmaking processes, changes to material specifications and properties, and the evolution of the design methodology for steel highway bridges.

## 1.4—EFFECTS OF FIRE DAMAGE ON MATERIAL PROPERTIES

Steel material properties can be altered by severe fire damage through changes in the microstructure that are possible under extended periods of exposure to high temperatures. These cases are relatively rare but do occur. Research conducted at Purdue University and followed up by the Pennsylvania Department of Transportation resulted in a better understanding of how fire damage might affect material properties (Brandt et al., 2011). They produced a visual guide for post-fire inspection of steel bridges to help identify how the steel material properties might have been altered during a fire based on the appearance of the paint coatings. In cases where material testing may be required to determine the effects of fire, such as for weathering steel bridges where a coating is not typically present, material samples acquired from fire-exposed and unexposed portions of the member can be compared following the testing protocols, provided in ASTM E23 (for Charpy V-notch [CVN] impact testing correlating to toughness) or ASTM E8/E8M (for tensile strength and ductility properties).

## 1.5—IDENTIFICATION OF HISTORICAL STEEL SHAPES

Along with the material, welding, and design advancements over the many decades, the shapes of standard rolled sections have also changed. Many of the original rolling mills, such as Carnegie Steel and Bethlehem Steel, are no longer in operation. However, the American Institute of Steel Construction (AISC) has compiled many of the original rolled beam cross-sectional property catalogs from those mills. These resources for historical rolled shapes are available online from the AISC website at <https://aisc.org>. Figure 1-1 shows two examples from early 20<sup>th</sup>-century Carnegie (left) and Bethlehem (right) rolled shape catalogs.

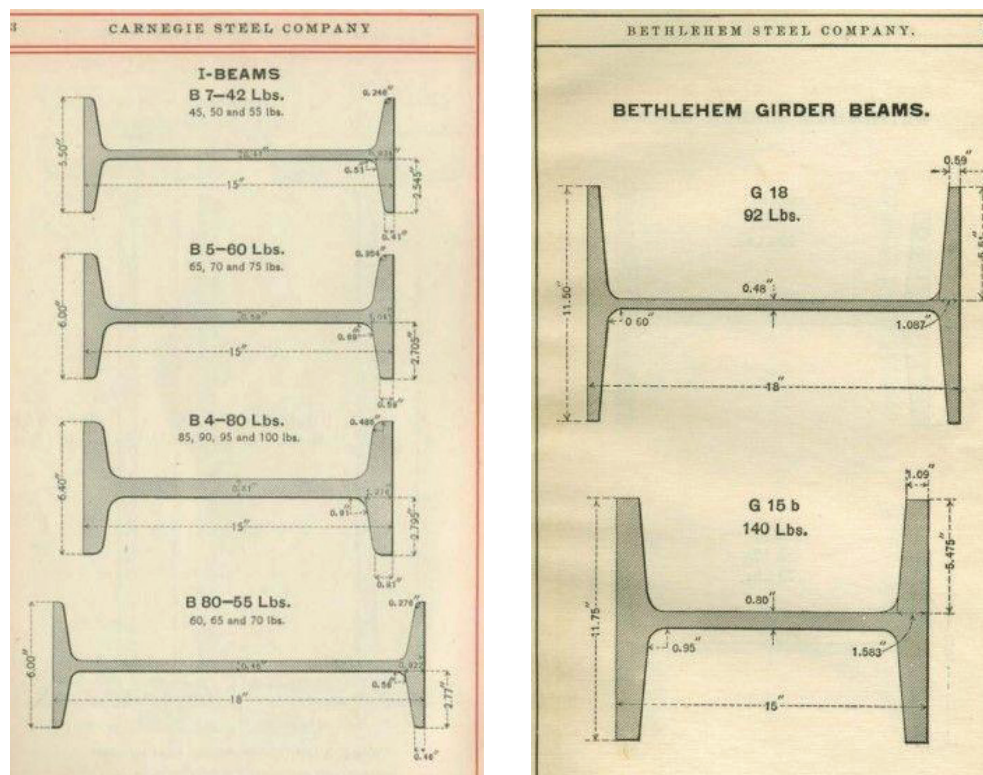


**Figure 1-1—Examples taken from Carnegie and Bethlehem historical steel shape reference catalogs**

In addition to these online catalogs, AISC's *Design Guide 15: Rehabilitation and Retrofit* (AISC, 2018) provides several tables of section properties for rolled sections produced between 1873 and 2000. The tables can be useful to identify a rolled section based on known dimensions or identify dimensions and properties based on known rolled section. It is possible that section properties may have evolved over the decades for the same rolled section, but typically these changes are minor and will not have a significant effect on flexural or axial capacities.

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## SECTION 2: CONSIDERATIONS FOR REPAIR PLANNING

The design must produce an achievable repair plan considering the method of attaching the repair components. Both bolting and welding are addressed in these Guidelines. Further consideration must be made for repair and remediation of coatings, if present.

New steel may replace existing steel (e.g., damaged flange and web) or supplement particular areas (e.g., web and flange reinforcement, cross-frame connections). High-strength bolting or welding are the preferred attachment methods, depending on the situation.

Highly congested areas (e.g., converging diagonal braces and a floor beam between gusset plates) may limit both welding and bolting solutions, and alternatives should be evaluated before construction. Stopping traffic-induced joint distortion during welding may not be possible.

### 2.1—GENERAL CONSIDERATIONS FOR FIELD REPAIR DESIGN

The following should be addressed:

#### 2.1.1—Performance Requirements

It is important for the Owner to establish performance requirements for the repair. This can be in terms of final acceptance criteria (e.g., welding process, bolt tightening verification, surface finish, results of nondestructive evaluation [NDE], and structure geometry), functional requirements (e.g., traffic clearance, reattachment of utilities, appearance, behavior under live load), or performance guarantee.

#### 2.1.2—Remaining Service Life

The repair needs to function properly for the remaining life of the bridge. This should be reflected in the performance requirements and the repair selected.

#### 2.1.3—Available Time for Completion of Repair

The length of time to complete a repair is a function of the type and details of the repair, the access permitted (e.g., lane closures above and below the repair), permitted work hours, and aesthetic requirements.

#### 2.1.4—Owner Inspection

Owner inspection is an essential consideration to support conformance at various points throughout the entire construction process. (Owner inspection is sometimes called “Quality Assurance” or “QA,” but that term can also refer to higher-level quality management activities on the part of the Contractor or Fabricator.)

##### 2.1.4.1—Owner Inspection Role

The Owner Inspector (OI) verifies that Contractor quality control (QC) is conducted as required. The Owner may choose to contract out inspection services or use internal personnel. Owner inspection activities include review and oversight of prefabrication submittals, in-process and final weld quality, weld repair or bolt testing and installation procedures, NDE, and coating application. The OI does not typically perform NDE unless specified by the Contract for routine verification or when the Contractor’s results are questioned. The OI reviews all NDE results and certifications. Owner inspections may include oversight of QC activities or a random sampling of members to verify the Contractor is meeting the Contract requirements.

In-process weld inspection by the OI may include verifying that joint details conform with the design, fit-up meets tolerances, the welding procedure specification (WPS) is followed (e.g., voltage, amperage, travel speed, preheat, interpass, and postheat requirements are met), and proper weld cleaning is performed

between passes. In-process bolting inspection by the OI may include verification that fasteners are of the correct grade and rotational capacity lots, monitoring of testing and installation procedures, and—where applicable—verifying tensioning indications such as twist-off splines, load-indicating washers, or turn-of-nut markings.

#### **2.1.4.2—OI Qualifications**

The OI needs to be qualified for their inspection activities. When welding is involved, the OI should be an American Welding Society (AWS) Certified Welding Inspector (CWI). The OI should be familiar with the welding and NDE processes, fastener installation methods, and procedures being used for the repair. The OI must anticipate potential problems associated with the processes and procedures, their cause, and how they can be mitigated or corrected.

#### **2.1.4.3—Owner Inspection Documentation**

All Owner inspection activities should be documented.

#### **2.1.4.4—Hold Points**

Hold points are specific points in the progression of the repair where the work is stopped until the OI is available to verify project requirements are being met. Defining these points is particularly important if there is a critical condition that needs verification by the OI or a person with specific expertise (e.g., NDT Level II). Hold points should be used prudently, as they can be disruptive and significantly affect the project schedule. If used, they should be identified in the Contract documents.

#### **2.1.4.5—Final Acceptance of Shop-Fabricated Material**

Welded assemblies produced by a Fabricator require final acceptance by the OI before incorporation at the field location. That acceptance may be based on reports from the OI at the fabrication shop, or at the field site based on shop documentation and visual inspection. Shop acceptance may be after welding is complete or after coating.

#### **2.1.5—Quality Control**

QC is the element of quality management conducted by the stakeholder performing the work to confirm that requirements are met. Quality controls include dimensional tolerance limits, welding procedures, weld quality acceptance criteria, material property requirements, bolt tensioning, and fracture control plan (as applicable). Quality is ensured through timely monitoring, inspection, and correction of nonconformances.

The Contractor's quality plan should identify how quality will be maintained and the QC activities to be performed. Roles and responsibilities of each member of the QC organization should also be identified. Weld inspection should be performed by AWS CWIs. AISC addresses quality through their certification programs for quality management systems (QMSes) and establishes minimum QMS requirements for certified Fabricators and erectors.

#### **2.1.6—Plans and Specifications**

Design plans can range from simple sketches to a complete set of drawings, depending on the scope of the repair. Design drawings and details should clearly indicate the Contract requirements (e.g., permitted work schedule, incentives/disincentives, obstructions/limitations, sizes, locations, final dimensional tolerances, materials, testing, submittals). Specifications should reference all material requirements, including but not limited to applicable AWS, AISC, AASHTO, ASTM, AMPP/SSPC/NACE, state, and local codes and standards; define project-specific acceptance standards, performance requirements, NDE, and qualification for welders and inspectors; and list any prohibited activities or practices. Detailed instructions (means and methods) may be included based on the contract type and experience of local Contractors.

### 2.1.7—Engineering During Construction

Engineering during construction can be divided into pre-construction and construction activities. Pre-construction activities include review of submittals and requests for information (RFIs). A pre-construction meeting between agency and Contractor representatives is helpful in establishing lines of communication, providing background information on the repair, clarifying uncertainties, and establishing expectations. During construction, the Engineer of Record (EOR) should maintain involvement including review of RFIs, change orders, and inspection reports. Scheduled hold points may be necessary to promote appropriate involvement of the EOR.

### 2.1.8—Use of Mockups

Mockups are useful tools to use where complex conditions exist. Complexity includes odd geometries, thick weldments, or intersecting members with difficult access. The mockups provide an opportunity to demonstrate adequate access for the bolt tensioning or welding and NDE to perform the required work. The Contractor should not practice on the structure. The mockup can be used to qualify the procedure and personnel. Multiple mockups may be needed to qualify different procedures or individuals. Destructive testing of the mockup may be necessary to verify quality. The cost of using mockups is weighed against the risk to the project, the potential for rework, and the confidence in the abilities of Contractor personnel.

### 2.1.9—Evaluating Damaged Condition of Existing Material for Work Required

Even if the evaluation was performed by the Owner and the scope is defined in the advertised Contract, the Contractor should conduct a survey to verify the extent of the repair area and severity of damage. Alternatively, the Owner may have an established contractual relationship for the Contractor to evaluate the damage and jointly define the scope and plan the repairs.

### 2.1.10—Realignment of Existing Material

Whether damaged material can be brought back to its original position is a major consideration in determining reparability. If material has severe deformations, realignment may not be possible without loss of required properties. Refer to *National Cooperative Highway Research Program (NCHRP) Report 271* (Shanafelt and Horn, 1984) for further discussion and insight on deformed material. Alignment tolerances may be based on AASHTO/AWS D1.5M/D1.5 or other criteria. If local or general deflections exceed straightening limits, the Contractor should submit a replacement proposal for specific areas or entire members.

### 2.1.11—Documentation of Repairs

Repair provided by the Contractor should include the following:

- Description of as-built repair details
- Material certifications
- Inspection reports
- Nonconformances and resolutions
- Inspector name and credentials

For welded repairs, record the following:

- ID of the welder
- WPS
- Consumables (manufacturer and trade name)

## 2.2—CONSIDERATIONS FOR RIVETED CONSTRUCTION

Field repairs on steel bridges built using hot-driven rivets are relatively common due to the average age of that structure type. Most of these bridges have exceeded their original design life. The following information is provided to help inform engineers of different techniques, material properties, testing procedures, and other general information that may be useful for rivets. Although the rivet was used for many decades as the primary method to fasten structures and fabricate built-up sections for additional capacity, many present-day structural engineers know little about them.

Field repairs might include removing rivets and replacing them with high-strength bolts. Moreover, in-situ testing might be required to establish material properties to estimate connection capacity for load rating. However, the rivets may need replacing in kind to rehabilitate historically significant bridges if an Owner or Owner-affiliated preservation organization requires new rivets rather than substitution of high-strength bolts. Using rivets is sometimes preferred to retain the integrity of design, materials, workmanship, or original construction methods for historical accuracy for bridges on the U.S. National Park Service's National Register of Historic Places. Guidance for the installation of hot-driven rivets is beyond the scope of these Guidelines. If that type of work is desired, a few specialty Contractors in the U.S. are capable of such work.

One of the most common field repairs for riveted connections is the replacement of rivets with high-strength bolts (all references to high-strength bolts in this Section assume a fully tensioned high-strength bolt). Round-headed bolts (e.g., ASTM F3125/F3125M Grade F1852 or F2280, or ASTM F3148) are often preferred for this application because they resemble rivets from one side of the connection. Replacing rivets with bolts is often done to replace overly corroded rivets, replace rivets where heads have come off due to cyclic loading and prying action, improve fatigue resistance, or increase connection capacity to accommodate larger live load demands. There are fundamental differences in the capacity of these two fastener types, as well as in the behavior of connections having either fastener type.

Consider, for example, the fatigue resistance of a rivet versus a bolt. The *AASHTO LRFD Bridge Design Specifications* (BDS) designate riveted details as Category D with a constant amplitude fatigue threshold (CAFT) of 7 ksi. The high-strength bolt, however, is Category B with CAFT of 16 ksi. These fatigue resistance values would be used for new designs for each respective fastener type. The AASHTO MBE allows Category C for fatigue life evaluation of existing riveted connections due to member-level redundancy, with a CAFT of 10 ksi. As stated in the commentary of the AASHTO MBE (2018), Category C “more accurately represents cracking that has propagated to a critical size.” In other words, the AASHTO MBE recognizes that Category D fatigue resistance is a reasonable lower bound for first crack development (a conservative value for design) and detection in an individual component, but that Category C is a reasonable lower-bound estimate of fatigue resistance for the built-up riveted member due to internal redundancy. This means that Category C represents the actual fatigue life of the member, rather than first cracking of a component of that member, taking into consideration the member damage tolerance inherent in mechanically fastened built-up construction.

Another notable difference between rivets and bolts to consider for replacement projects is the load transfer behavior. Rivets provide load transfer primarily through shear across the rivet shaft, while high-strength bolts generally are designed to provide load transfer through friction between the connection faying surfaces imposed by tension in the bolts. Bolted connections designed using a calculated friction force based on an assumed or specified surface condition are referred to as slip-critical connections. There is theoretically some contribution from friction in riveted connections, but this is highly variable based on grip length, condition of the faying surfaces, and driving technique and quality. Thus, frictional contributions to riveted connections are ignored in capacity calculations. The difference in load transfer behavior between rivets and high-strength bolts is the source for the dissimilar fatigue resistance where primarily frictional load transfer reduces stress concentration at the edge of fastener holes for bolts. When replacing rivets with high-strength bolts, it is important to realize that the clamping force, and therefore frictional load transfer capacity of a single high-strength bolt, may not be a one-for-one replacement for the shear capacity provided by the rivet it replaces. This is particularly true when the faying surfaces have degraded over time between the plies of a riveted connection, or more generally when the slip coefficient is unknown. Therefore, it is not always recommended to design rivet replacement repairs with high-strength bolts intended to behave as slip-critical connections. Generally, they should be designed as

shear connections. This means that the fatigue resistance of the connection does not improve to AASHTO Category B. It also is highly recommended that all high-strength bolts be fully tightened regardless of whether they are designed for slip. This will guard against bolt assemblies coming loose over time and falling from the bridge.

One of the benefits to replacing rivets with high-strength bolts is that the new bolt will provide additional capacity over the replaced rivet due to higher material strength. However, a distinction to keep in mind is that the rivet typically fills the entire fastener hole, whereas high-strength bolts do not. (This idealization may not always be true due to low heating temperature at time of installation, misaligned plies, or long grip lengths, but is generally a reasonable assumption for rivets.) This means that over time there could be some negligible slip in the newly bolted connection where rivets used to be. A final consideration in this repair type is the transfer of dead load. If the entire riveted connection is replaced with high-strength bolts, there are a couple of ways to do this so that the dead load is transferred into the bolts. One way is to jack the connection up so that dead load is carried through the jacking system. Then all rivets can be replaced with bolts. When the jacks are released, the dead load will be carried through the newly installed bolts. A second way is to replace a few rivets at a time with the bolts until all rivets are replaced. This procedure successively transfers dead load back into the bolted connection with the progression of rivet replacement. However, if only a few of the rivets are replaced in a connection, leaving a mix of bolts and rivets, the Engineer should realize that the dead load would be carried entirely by the remaining rivets and the new high-strength bolts installed would primarily contribute to live load capacity.

Existing rivets are typically removed using a pneumatic rivet hammer with chisel bit to shear off the rivet head and pop out the remainder of the rivet using a rounded, blunt bit (see AASHTO/NSBA G14.1, *Maintenance Guidelines for Steel Bridges to Address Fatigue Cracking and Details at Risk of Constraint-Induced Fracture*, for additional information related to rivet removal). The removal time may factor into how much work can be completed under a short-term closure. On average, rivet removal may take 5 to 10 minutes per rivet, with shop-installed rivets generally being easier to remove than field-installed rivets. In some instances, more stubborn rivets through misaligned holes may require drilling or an oxygen lance and can take several hours to remove.

### 2.2.1—Tensile and Shear Capacity

Determining the tensile and shear strength of rivets for load rating or other engineering calculations can be challenging simply because the material properties may not be known. Typically, historical steel bridge design plans will not name the rivet specification used. However, if the year the bridge was built is known, then, generally, a reasonable material specification conclusion can be made based on what ASTM specification was the most common at that time. AISC's *Design Guide 15* (2018) summarizes some of the structural rivet specifications used during periods of time. For example, the publication states that the 1969 AISC specification included only Grades 1 and 2 of ASTM A502, a rivet specification originally published in 1964 that combined discontinued rivet steel specifications A141 and A195. The publication further states that the 1978 AISC Specification and subsequent editions have included A502 Grades 1, 2, and 3. Thus, from this information, an Engineer could deduce that a rivet would likely not be A502 Grade 3 (with higher hardness and higher strength) on a structure built prior to 1978.

When the structural rivet steel properties are unknown and testing is not desired or not feasible, AASHTO MBE provides conservative estimates for factored shear strength. Rivets of unknown type and origin are assigned a factored shear strength of 27 ksi, ASTM A141 or ASTM A502 Grade 1 rivets are listed as 32 ksi, and ASTM A502 Grade 2 rivets are listed as 43 ksi. These strengths include a resistance factor of 0.80 and a shear/tension ratio assumed to be 0.67. There are also additional shear strength penalties based on rivet connection length that may apply. Finally, in-situ field testing of rivet strengths via portable hardness testers can be used, as discussed in Article 2.2.2.

### 2.2.2—In-Situ Testing of Rivets for Material Properties

Estimating rivet shear strength by the year the bridge was built will most often result in a conservative appraisal of member or connection capacity. In cases where performance ratios (i.e., demand–capacity ratio) are greater than 1, for example, it may be desirable to obtain field-based measurements of rivet shear

strength to improve the accuracy of the engineering calculations. Stevens et al. (2021) propose a relatively simple nondestructive testing method that correlates field-measured hardness data to the shear strength of rivets. Their proposed method uses field-measured surface hardness from the rivet heads and correlates it directly to rivet shear strength using a multiplier. The field-driven head is the head of the rivet formed while driving the rivet in the field during construction of the bridge. The field-driven head may show signs of slight misalignment with the hole, misshapen head, or other minor imperfection indicating it was driven by a hammer in the field.

Stevens et al. (2021) report that the ASTM A370 tensile strength approximation was found to consistently underestimate the rivet ultimate strength and that the AASHTO MBE factor to approximate shear strength consistently underestimated the actual shear strength. Additionally, they found that the estimated rivet shear strength calculated using the 0.85 factor from Ocel et al. (2013) was found to be within 6 percent of actual strength when using the shop-formed head hardness, but for the field-driven head using the 0.85 factor overestimated shear strength by as much as 27 percent. Thus, to simplify the process, Stevens et al. (2021) propose a direct hardness-to-shear strength multiplier of 0.7 ksi/HRB (Rockwell B hardness). Field-measured hardness of the shop-formed head, determined using a portable hardness tester, was shown to correlate to within a few percent of the actual shear strength of the tested rivets. In situations where it is not possible to distinguish between shop and field-formed heads, the authors recommend testing both heads and using the lower of the two hardness measurements. This is a conservative method since hardness has a linear correlation to strength. The hardness testing involved lightly grinding the rivet head to produce a flattened, exposed bare metal surface at the crown of the rivet head, on which three hardness tests were performed using a small, portable tester.

The proposed method was applied to three in-service bridges (Stevens et al., 2021). The authors note that there is too much variation in a small subset of data to apply an average or lower-bound value to the entire bridge. Instead, they propose testing a subset of rivets on a particular element, such as a gusset plate connection of interest, and estimating the shear strength of rivets on that element, based on the tested rivet subset. This approach would account for members or connections built from the same heat of rivets using similar driving techniques and ambient conditions. Whether the average or lower-bound hardness (and therefore shear strength) is used would be at the discretion of the Engineer.

## 2.3—FIELD BOLTING CONSIDERATIONS

Repair and retrofit projects often present constructability challenges due to access restrictions. Existing structures have concrete elements (e.g., decks and abutment backwalls) that were not constructed until after the steel superstructures were erected and bolted. These elements limit space and may restrict the access necessary for construction personnel and equipment to perform the scheduled repairs and retrofits. Many older bridges, particularly trusses, include built-up riveted members that have restricted access. Considering the access needed for field bolting during the design process can help avoid delays and undesirable results during construction.

For some older riveted structures, historic preservation agencies require that “button-head” bolts be substituted for hex-head bolts to mimic the appearance of rivets, as illustrated in Figures 2-1 through 2-3. Button-head bolts cannot be gripped on the bolt head and come as a fastener assembly that includes a bolt with a spline end. The installation tool for these assemblies grips the spline while turning the nut to tighten the fastener to the required pretension of the fastener. Therefore, tightening of button-head bolts requires enough working space for the installation tool to access the bolt location from the nut end of the bolt assembly. Since the intent of using this type of bolt is to mimic the appearance of a rivet, the head is expected to be on the exposed face of the member, which means installation will need to be done from the more congested internal face. Many riveted built-up members are composed of some combination of rolled members (often angles or channels), cover plates, and either batten plates or lacing bars. Depending on the size of the member, there may not be enough space to access the nut end of the bolt with the installation tool.

Along with access limitations, it is also important to consider the duration of the activities associated with field repairs. In some cases, specific repairs are specified to be performed under partial or full closure of the bridge due to access (e.g., through-truss elements over traffic lanes) or loading concerns. Field repairs

may not be able to be expedited by increasing the number of crews or crew size because most of the work must be performed in situ at the member being repaired.

Field drilling of bolt holes is commonplace in steel repair and retrofit operations and should be considered when scheduling repairs. Bolted repair or retrofit connections generally require field drilling bolt holes in one or more of the connected plies to facilitate fit-up. Figure 2-1 shows a plating repair on the bottom chord of a riveted, built-up through-truss member. In this case, the outside repair plate (visible in the photo) had the holes pre-drilled and was used as a template to field drill holes in the existing channel member and inside repair plate (not visible). Figures 2-2 and 2-3 show a field splice along the same member where corroded splice plates were replaced. This time, the holes in the existing channel member were used as a template to drill holes through the outside and inside splice plates. Field drilling bolt holes cannot be avoided by shop drilling using detailed field measurements, or even lidar scans, of the existing bolt or rivet locations because the center of the bolt or rivet heads do not necessarily represent the center of the hole.



**Figure 2-1—Truss bottom chord plating repair showing button-head bolts and beveled plate washers**

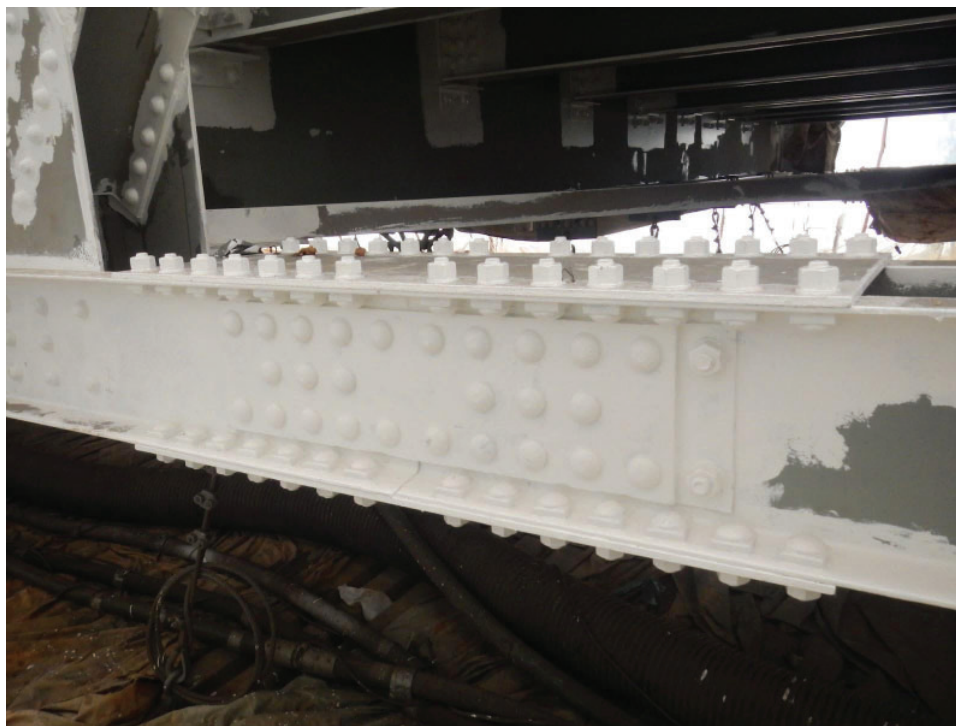


**Figure 2-2—Truss bottom chord field splice (outside view)**



**Figure 2-3—Truss bottom chord field splice (inside view)**

As specified in the Research Council on Structural Connections (RCSC) *Specification for Structural Joints Using High-Strength Bolts*, beveled plate washers should be specified when installing plating to any surface sloped more than 1:20 normal to the bolt axis (e.g., the inside face of a rolled channel flange).

Figure 2-1 shows beveled plate washers used on a built-up member composed of two channels with batten plates. Plating has been installed on the web with button-head bolts and the batten plates across channel flanges have been replaced. The original batten plates were riveted, and the replacement web plates were attached with button-head bolts.

## 2.4—FIELD WELDING CONSIDERATIONS

Welding may be a viable repair alternative to bolted connections and can be done successfully in the field subject to existing material weldability, adequate access, and use of appropriate procedures and controls.

### 2.4.1—Advantages and Challenges of Field Welding

Advantages of welding over bolted repair methods include:

- The repair may require less material.
- The area of repair may be reduced compared to the additional material (e.g., tab plates or double plates) required for a bolted connection.
- The repair area may not be able to accommodate the number of bolts required (insufficient room for a bolted connection).
- A welded repair may require less time than drilling bolt holes or preparing faying surfaces.
- Welded connections are less susceptible to corrosion issues (e.g., pack rust).

Challenges of field welding over bolted repairs or shop fabrication include:

- Environmental conditions (wind, precipitation, and low temperatures) must be mitigated. For example, AASHTO/AWS D1.5M/D1.5 prohibits welding at ambient temperatures below 0°F.
- Some historical materials may require special processes, consumables, preheat, or postheat, while others may not accommodate welding at all. See Article 1.3 for more information related to historical material properties.
- Bolted connections may provide more redundancy and can be less fatigue-sensitive.
- Experienced field welders capable of making groove welds that pass volumetric inspection, or out-of-position, primary load-carrying welds may not be readily available on short notice.
- Access to the repair location may be limited. For example, accessing both sides for two-sided complete joint penetration (CJP) groove welds may not be possible.

### 2.4.2—Owner (Designer) Considerations

There are many stakeholders and variables to consider when developing a field-welded repair.

#### 2.4.2.1—General EOR Considerations

##### 2.4.2.1.1—Owner Experience

Owner experience with field welding may influence the decision to select this repair type. Where experience is lacking, an Owner may contract an individual or organization to evaluate possible actions and develop a repair specification.

#### 2.4.2.1.2—Weld Design Principles

Incorporate the following practices:

- Design economical welds.
- Specify the most efficient weld type and size (e.g., fillet welds instead of groove welds, welded splice plates instead of CJP repairs to existing elements).
- Avoid special treatments (e.g., post-weld stress relief).
- Use AASHTO/AWS D1.5M/D1.5 standard weld joint details.
- Incorporate simple, direct load paths.
- Consider the stiffness of connecting elements.
- Minimize stress concentrations, and avoid details with poor fatigue resistance.
- Design inspectable welds.
- Establish reasonable tolerances for situations not addressed by the code.
- Consider welding access needed.

For further recommendations:

- Chapter 9 of FHWA's *Bridge Welding Reference Manual* (FHWA, 2019) discusses details for welded bridges.
- *Design Guide 21: Welded Connections—A Primer for Engineers* (AISC, 2017) provides a complete overview of topics related to structural welding, including selection of weld types, weld design, metallurgy, weld repair, weld procedure specifications, quality, inspection, economy, and safety.
- The article “Field Welding to Existing Steel Structures” (Ricker, 1988) provides insight on various aspects of welding on in-service structures.

#### 2.4.2.2—Governing Code and Specification Requirements

By scope, AASHTO/AWS D1.5M/D1.5 is applicable to field welding because it is suitable for all bridge welding, whether in the field or in the shop. However, AASHTO/AWS D1.5M/D1.5 may not provide the particulars needed for special situations like repairs and retrofits of older steels or other materials. For example, AASHTO/AWS D1.5M/D1.5 prescribes welding consumables for ASTM A709 base metals, so if the base metal is different, additional instructions are needed. Also, within AASHTO/AWS D1.5M/D1.5 the EOR has flexibility for certain aspects of welding, such as alternative NDE processes or acceptance criteria and use of non-standard joints. Other codes, such as AWS D1.1/D1.1M and AWS D1.7/D1.7M, can be used in conjunction with AASHTO/AWS D1.5M/D1.5 to help fill gaps. If so, the EOR should use Contract language that avoids conflicts with other applicable codes.

#### 2.4.2.3—Nonredundant Member Considerations

Welding nonredundant steel tension members (NSTMs) will require adherence to the fracture control plan (FCP) primarily provided in Clause 12 of AASHTO/AWS D1.5M/D1.5. The FCP includes weld metal toughness limits, use and handling of low-hydrogen electrodes, base metal repairs, welding procedure qualification, heat input requirements, welder and NDE technician qualifications (unless stricter requirements are needed), and acceptance criteria (unless alternate criteria are specified). Some criteria may not apply where existing base metal does not meet FCP standards such as toughness and soundness. Weldability and chemical composition may be concerns with some steels. In these cases, alternative acceptance criteria and heat input can be established to account for the existing steel properties. Locations where welding is required to be performed to the FCP should

be clearly identified on the drawings. The Designer has the responsibility for establishing the testing acceptance criteria required. While the Designer may choose to define the existing component as NSTM and invoke the FCP, the existing material may not have been originally fabricated to FCP criteria. In such cases, trying to impose the FCP on existing bridge members may not be possible, and adding modern FCP-compliant material to a structure built before FCP provisions were first published in 1978 may not improve performance.

System redundant members (SRMs) are defined by FHWA as members “in a bridge system without load path redundancy, such that fracture of the cross-section at one location of a primary member will not cause a portion of or the entire bridge to collapse” (2022). A nationally recognized method to identify SRMs can be found in the *AASHTO Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members*. SRMs are redundant members by definition, but are required to be fabricated and repaired in accordance with the FCP.

#### 2.4.2.4—Welding Processes

Any bridge welding process found in AASHTO/AWS D1.5M/D1.5 is suitable for field welding if proper conditions are provided, although some processes are more suited to field use than others. A discussion of each process is found in the FHWA *Bridge Welding Reference Manual*, Chapter 3. Each process has some advantages or disadvantages, some of which are compounded under field conditions. For example, if gas-shielded processes are used, the Contractor must use adequate wind protection shelter. Generally, this requires a complete enclosure.

The better design practice is not to specify a particular welding process but rather to allow the Contractor to choose the process best suited to the situation, considering access, welding position constraints, and the Contractor’s skills and equipment. Equipment portability and access are important field considerations for the Contractor. For example, the Contractor may choose shielded metal arc welding (SMAW) for small overhead welds to be performed from a bucket truck or submerged arc welding (SAW) for long, straight deck joint welds that can be performed in the flat position.

#### 2.4.2.5—Inspection Processes

Specify the type, extent, and frequency of NDE for all welded repairs. Typical methods include magnetic particle testing (MT), liquid penetrant testing (PT), radiographic testing (RT), ultrasonic testing (UT), and visual testing (VT). MT or PT can be used to identify surface-breaking cracks. RT may not be appropriate for all field repair situations due to the exposure restrictions involved and because it is cumbersome to handle in the field. UT may be used in lieu of RT at the direction of the EOR under AASHTO/AWS D1.5M/D1.5 requirements and, given its portability, is better suited to field inspection than RT. VT should be conducted on all welds. Phased-array ultrasonic testing (PAUT) is an advanced UT method (AWS D1.1/D1.1M:2020, Annex H; AASHTO/AWS D1.5M/D1.5:2020, Annex J) that may be considered instead of RT or conventional UT. Coatings of the repair area will need to be removed in advance of performing any MT or UT, unless special procedures are qualified to address the presence of coatings.

The frequency of applying each method should be defined by the Contract. For smaller repair work 100 percent NDE, in addition to VT, is reasonable. For larger repairs, the frequency of NDE for each specified method at each repair or joint type should reflect the risk associated with the project. For more information about the suitability of NDE methods for bridge welds, consult the FHWA *Bridge Welding Reference Manual*, Chapter 6.

The Contractor QC activities should include the following:

- Conduct NDE prior to, during, and after completion of the weld.
- Conduct MT on weld repair excavations to verify the defect has been removed.
- Prior to welding, verify that fit-up, alignment, joint prep, and joint cleanliness requirements are met.
- During welding, verify the welding is completed per the WPS.
- Test welds with the required NDE methods after the weld has cooled.

To increase the likelihood that any delayed cold cracking has occurred by the time of inspection, waiting periods before final acceptance testing are required for members subject to the FCP, and may also be recommended for highly restrained joints and materials with high carbon equivalent, which may be found in some historical steels.

NDE performed as a part of Owner inspection may be considered based on risk and confidence in QC capabilities. However, Owner inspection NDE other than VT is not common.

#### **2.4.2.6—Inspection Procedures and Acceptance Criteria**

AASHTO/AWS D1.5M/D1.5 provides acceptance criteria for weld NDE. If AASHTO/AWS D1.5M/D1.5 is specified, these criteria will apply. However, AASHTO/AWS D1.5M/D1.5 acceptance criteria include workmanship provisions that may be overly conservative for certain field repair situations. More lenient levels of acceptance may be suitable in some situations. There are other procedures and criteria that may be considered, including the following:

- PAUT procedures and acceptance criteria are addressed in AASHTO/AWS D1.5M/D1.5, Annex J and AWS D1.1/D1.1M, Annex H.
- AWS D1.1/D1.1M, Annex O gives alternative UT acceptance procedures and criteria derived from the American Petroleum Institute (API).
- Visual acceptance criteria (e.g., undercut, weld profiles) can be modified by the EOR.

Fitness for service (FFS) is a method for developing alternative acceptance criteria based on fracture mechanics. Accurate discontinuity sizing is essential when applying the FFS method. NDE technicians and sizing procedures should be specially qualified to increase the reliability of the analysis.

#### **2.4.2.7—Personnel Qualifications**

##### *2.4.2.7.1—Welders*

Welders should be qualified in accordance with applicable AWS welding codes. For complex repairs, in addition to code-required qualification, further welder qualification may be required using a mockup that is representative of the repair. Under AWS D1.1/D1.1M and AASHTO/AWS D1.5M/D1.5, Contractors and Fabricators qualify their own personnel to the requirements of these standards.

##### *2.4.2.7.2—NDE Technicians*

NDE technicians may be qualified based upon American Society for Nondestructive Testing (ASNT) certifications or similar. AASHTO/AWS D1.5M/D1.5 requires certification to ASNT requirements. Special qualification of technicians using mockups should be considered for some projects. Example specifications that contain qualification standards include AWS D1.8/D1.8M and publications produced by API and the California Department of Transportation (Caltrans).

Further discussion of NDE technician certifications is found in the FHWA *Bridge Welding Reference Manual*, Section 6.11.2.

#### **2.4.2.8—Material Considerations**

##### *2.4.2.8.1—Existing Base Metal*

Weldability must be part of the decision process when considering welded repairs. Establish existing base metal properties to facilitate development of a WPS. Properties can be derived from as-built drawings, shop drawings, design specifications, date-built tables, or testing. Establish material properties through testing (see below) where existing information does not exist. Material properties must be established so that a proper WPS can be developed.

Typically, the Owner identifies the base material of the existing structure. If unknown, conduct material testing to determine material properties. Use test results to confirm performance requirements are met and to develop a WPS.

Test material should be obtained from a representative location where effects of removal are minimal or can be easily repaired. Chemical composition can be obtained from a small material sample or by using a portable positive material identification analyzer on in-place materials. A tensile test typically requires a specimen at least 2 in. by 8 in., and a set of CVN tests can be obtained from a 3-in. core sample. When possible, the tensile sample should be oriented longitudinally, which can almost always be assumed to be the direction of rolling. ASTM A673/A673M should be consulted for the orientation of CVN samples, and the longitudinal direction should be marked on the sample. Minimum material thickness needed for a full-size CVN test is about 0.5 in.

For bolted or riveted structures, carbon equivalent should be determined from the chemical composition test to determine weldability. Welded structures were likely constructed of weldable steels, but the chemical composition should still be determined.

#### *2.4.2.8.2—Selection of Repair Base Metal*

New base metal properties need not exceed the requirements of the existing base metal except where performance requirements dictate. ASTM A709 grades should be sufficient in most cases. Weathering steel (e.g., W grades of A709) is generally suitable for unpainted applications. Corrosion-resistant steel (e.g., A709 Grade 50CR) may be desirable where very high corrosion potential exists. Additional consideration may be needed when using steels that are not listed in A709. Repair plates to replace tension flanges or flange splice plates should have rolling direction parallel to design tensile stress whenever possible.

#### *2.4.2.8.3—Consumable Selection*

AASHTO/AWS D1.5M/D1.5 defines which consumable classifications may be used with the materials found in the code, and the Contractor should be permitted to select the consumables accordingly. If the material is not found in AASHTO/AWS D1.5M/D1.5 but is found in a different AWS D1 codes, such as AWS D1.1/D1.1M, then specify use of that different code and add CVN requirements if needed.

If welding outside of the AWS D1 codes, specify toughness and hydrogen control requirements as needed. Historical steels may be weldable with special heating. Consider retaining an expert who has experience welding such materials to help develop the best practices and define them in the Contract. Consider undermatching weld strength for higher-strength base metals to minimize hydrogen-induced cracking potential.

Combining different welding consumables in a single joint (e.g., for repair of an existing weld or for tack welds) is a common practice in fabrication and allowed by welding codes, including AASHTO/AWS D1.5M/D1.5. However, the codes prohibit welding over FCAW-S with other processes unless special qualification testing is performed; the FHWA *Bridge Welding Reference Manual* addresses this concern in Section 3.6.6.

#### **2.4.2.9—Weld Joint Capacity and Geometry**

The weld joint types (fillet, partial joint penetration [PJP], and CJP) are specified by the Engineer. If AASHTO/AWS D1.5M/D1.5 or AWS D1.1/D1.1M is specified, the Contractor has the latitude to use the standard joints in the specified code. If the standard joints are not suitable or the Contractor would prefer to use a detail other than a standard joint, the Contractor can propose the alternative for the EOR's consideration. There are provisions in AASHTO/AWS D1.5M/D1.5 for qualifying alternative joint geometries. The development of a weldable joint must take into account the following key points:

- Minimum and maximum weld root gap
- Weld joint detail tolerances (as detailed and as fit-up)
- Use of tack welds, size considerations

- WPS
- Thickness and width transitions
- Misaligned members, offsets
- Effects of preheat weld heat input, shrinkage, and restraint on material properties and final geometry
- Use of backing to avoid overhead welding or when far-side access is restricted

Seal welds may be desirable to exclude moisture from the joint and prevent corrosion. Seal welds are typically fillet welds and should be the minimum size required for the work and not produce fatigue-sensitive details in high-tension areas (e.g., a longitudinal seal weld terminating on the tension flange). Seal welds should be indicated on the Contract drawings.

Three types of permanent welds may be used: fillet, PJP, and CJP. Plug welds are not allowed; however, bolt holes can be restored using an excavation that facilitates welding with stringer passes, as described in AASHTO/AWS D1.5M/D1.5 commentary on mislocated holes. For cyclically loaded areas, the required fatigue resistance per AASHTO BDS or AWS D1.1/D1.1M may limit which weld types are acceptable.

Fillet and PJP welds are preferred for shear loads parallel to the weld throat such as flange-to-web welds or compression perpendicular to the weld throat such as at bearing stiffeners. The allowed fatigue range for those weld types limits their use where tensile stress is transverse to the weld throat such as transversely loaded attachments (Category C or C') and the ends of cover plates and welded splice plates (Category E or E').

The termination of fillet and PJP welds carrying shear within a member is a fatigue concern (Category E or E'), so flange-to-web welds should use run-on and run-off tabs. Intermittent welds have potential start and stop defects at each end and should not be used in bridge members or structural members carrying cyclic tensile loads.

When a design requires fillet welds larger than  $\frac{1}{2}$  in., PJP welds with equal weld throat are more efficient and reduce the weld deposit. PJP welds also avoid the need to extend stacked plate edges and may avoid interference with adjacent elements.

For connections subject to bending about the weld axis, fillet and PJP welds should have rotation restrained by a far-side weld or returns at weld ends to avoid tension at the weld root.

PJP and CJP welds may be welded from one or two sides. PJP weld sizes should be based on design and code requirements and do not require through-thickness continuity. If a two-sided PJP weld is detailed with total groove depths equal to the material thickness, fusion between sides is not required and volumetric NDE (UT or RT) is not mandatory by the code. The joint strength will be similar to a CJP weld without requiring back-gouging and volumetric NDE, but its fatigue resistance will be reduced.

CJP welds made from one side using standard joints must use fused (steel) backing on the far side. If unfused backing (ceramic, water-cooled copper) or a non-standard joint geometry is desired, the configuration must be qualified per code requirements. Fused backing must not be tack welded outside the weld joint. Fused backing may be removed after welding, but that requires extensive work and risks damaging the base metal. However, fused backing left in place can result in stress concentrations that promote fatigue-initiated fracture and complicate UT. Removal of backing should be considered where it is transverse to tension or where the backing compromises UT. For recommendations on retrofit for locations with backing bars left in place, see AASHTO/NSBA G14.1.

CJP welds made from two sides require back-gouging the root pass of the first-side weld to assure the second-side weld root pass can fully fuse to the base metal and first-side weld. Back-gouging may be done by air carbon-arc, oxy-fuel, or machining (grinding, milling, etc.), forming a groove shape similar to a standard joint without excessive removal of base metal. Thermal back-gouging is followed by grinding to bright metal to remove carbon, copper, or other residue.

#### 2.4.2.10—Tolerances

Fabrication and fit-up tolerances are addressed in AWS D1.1/D1.1M and AASHTO/AWS D1.5M/D1.5 while raw material tolerances are defined in ASTM A6 for rolled material. The EOR may specify different or additional tolerances when the standard tolerances are not appropriate.

#### 2.4.2.11—Restraint During Welding

Members restrained along one axis during welding are able to move or stretch in the other two axes due to Poisson's effect and thus behave in a ductile manner. Even when restrained on two axes, a member can deform and maintain some ductility. When fully restrained on all three axes, little or no deformation is possible, and the member and its connections constitute what is known as triaxial constraint. Members with triaxial constraint can fail in a brittle fracture mode known as constraint-induced fracture. The failures are sudden, and the extent of the failure is a function of the level of constraint, amount of energy stored in the members, and locally brittle nature of the material under constraint.

Intersecting welds should not be confused with conditions of triaxial constraint. Intersecting welds often occur in the fabrication and repair of steel members, such as at welded splice details or at the intersection of stiffeners and longitudinal flange-to-web welds, and do not constitute triaxial constraints. The FHWA has clarified this concern in *Evaluation of Steel Bridge Details for Susceptibility to Constraint-Induced Fracture* (2021).

#### 2.4.2.12—Temperature Control—Preheat and Interpass Temperature

Preheat and interpass temperatures are stated in the WPS. AASHTO/AWS D1.5M/D1.5 lists preheat temperatures based on thickness, steel specification and grade, and certain other parameters for welds that are subject to the FCP. Maximum preheat and interpass temperatures are also listed for certain steel types. Alternative methods for determining preheat and interpass temperatures are presented in AASHTO/AWS D1.5M/D1.5, Annex F, "Guidelines on Alternative Methods for Determining Preheat." Annex F should be consulted where higher restraint or elevated hydrogen levels may be present or for steels with high carbon content beyond the bounds of what is represented in AASHTO/AWS D1.5M/D1.5. If a minimum preheat or interpass temperature other than that specified in the welding code preheat tables is required, the EOR should specify that preheat.

If the base metal is manufactured using heat treatment processes, such as quenching and tempering, and is not listed in AASHTO/AWS D1.5M/D1.5, the steel producer or a metallurgist should be contacted regarding appropriate preheat and interpass temperature.

#### 2.4.2.13—Laminations and Lamellar Tearing

Lamellar tearing is separation of the base metal caused by welding shrinkage stresses perpendicular to planes of weakness such as laminations in the steel. Laminations are caused by inclusions such as silicates and voids in the base metal, flattened into thin discontinuities during rolling and are roughly parallel to the steel surface. Because individual inclusions may be in different planes, a fracture may result in a stair-stepped pattern. Lamellar tearing may or may not extend to the surface or edges of the steel. Groove-welded corner and T-joints are susceptible to lamellar tearing because the weld shrinkage strains are perpendicular to laminations parallel to the surface. Steel with low through-thickness ductility is more susceptible to lamellar tearing. See AISC *Design Guide 21* (2017) and FHWA *Bridge Welding Reference Manual* for weld design to avoid or minimize lamellar tearing.

If laminations are suspected, straight-beam UT should be performed on the base metal prior to welding to detect laminations, especially before large welds are made on the surface of thick material. For laminations or delamination at base metal cut edges, follow the provisions in AASHTO/AWS D1.5M/D1.5 for base metal cut edges.

### 2.4.3—Contractor Considerations

#### 2.4.3.1—Welding Position

Welding position will be dictated by the orientation of the repair. Welding codes allow welding certain standard joints in all positions, but welders must be qualified for the position of the repair weld.

### 2.4.3.2—Distortion and Residual Stresses

Distortion and residual stresses can be controlled, but addressing one problem can exacerbate others. Excessive distortion can be controlled by balancing heat input and by providing appropriate restraint conditions. Greater emphasis on heat and restraint controls is required for asymmetric members with components joined by large welds.

Distortion is caused when the member is acted upon by external or internal stresses that exceed the member's resistance to displacement. Distortion can be due to heat application, weld shrinkage, external loads, or restraint. Excessive distortion results in misalignment, improper fit-up, exceeding tolerances, and unintended loading due to member eccentricities.

Distortion can be managed through the control of heat (and thus shrinkage) or by providing restraint (preventing or controlling shrinkage). When welding or heating complex assemblies, the Contractor should develop a distortion control plan covering routine issues and add provisions for other job-specific conditions. Common distortion control measures include (see also AASHTO/AWS D1.5M/D1.5, Clause 5):

- Positioning material to anticipate weld shrinkage.
- Weld sequencing (mapping) can be defined to reduce non-uniform and overall shrinkage or allow uniform shrinkage. Methods may include:
  - Balancing welds.
    - Place weld passes on alternating sides for a two-sided weld (e.g., a flange-to-web groove weld) or alternating about the neutral axis (e.g., a U- or C-shape), with successive passes counteracting the distortion caused by the previous weld.
    - Place welds near the neutral axis to avoid eccentric loading (shrinkage).
  - Welding joints with greater expected shrinkage first so they are made with less restraint.
  - Use of subassemblies to allow minimally restrained shrinkage prior to incorporation into the final assembly.
  - In rare cases where continuous welding of thin material will create significant distortion, employ backstep welding. Place short segments of weld starting at the end with the greatest restraint and weld towards that point. Place successive welds away from the end with the greatest restraint.
- Restraint Controls. Restraint is added to prevent movement due to shrinkage through use of jigs, clamps, strongbacks, fixtures, or tack or temporary welds. Excessive external restraint can cause fracture in a manner similar to internal restraint (see Article 2.4.2.11).
- Heat control. Distortion can be reduced by distributing heat through weld sequence, preheat and interpass temperatures, and slowing cooling rates.
- Peening and thermal stress relieving. Peening can be conducted on a weld bead as it cools to relieve shrinkage stresses and thus shrinkage. Thermal stress relieving tends to permit additional distortion unless parts are restrained. These should only be performed if required by contract or permitted by the EOR.

Residual stresses occur through a variety of mechanisms including solidification and differential cooling in the production of steel (base metals have residual stresses), thermal gradients from welding and cutting, inelastic (plastic) deformations from cold forming or impact damage, and variations in the coefficient of thermal expansion for different adjoining materials.

Residual stresses are created in areas that are restrained from shrinkage. Residual stresses can reach the yield strength of the material, causing stress to redistribute to other locations. Welded connections subjected to residual stresses near or at yield stress have performed satisfactorily but may be susceptible to distortion, shrinkage cracking, fatigue cracking, and lamellar tearing.

The Contractor should make note of any of these deficiencies when observed during the welding process and related inspection. Many of the guidelines to control distortion can also reduce residual stresses. These

include weld sequencing, heat control, and peening. They do not include restraint. Residual stress has the potential to cause cracking; crack avoidance should take precedence over distortion control. Some common controls on residual stress and related cracking include:

- Use undermatching or more ductile weld metal when permitted by design (the maximum yield stress and thus residual stress is reduced).
- Weld joints with greater expected shrinkage first so they are made with less restraint.
- Use higher heat input and slower cooling rates and increase the preheated area to uniformly distribute and lower the magnitude of localized residual stresses.
- Limit restraint and restrictions to movement during heating, welding, and cooling while continuing to provide stability.
- Peen welds in between passes in large groove welds (typically not needed except for thick weldments, i.e., over 2 in.). Do not peen the surface of the final weld.

Where highly restrained conditions exist, particularly when combined with thick and large members (e.g., flanges over 3 in.), alternative preheat and interpass temperatures should be considered following the guidelines of AASHTO/AWS D1.5M/D1.5, Annex F. Hydrogen diffusion postheat should also be considered to prolong and slow the cooling (see Clause 12 of AASHTO/AWS D1.5M/D1.5 for recommended parameters). Heat soaking using electric resistance blankets should also be considered to maintain uniform heat throughout a larger area.

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## SECTION 3: SECTION LOSS

This Section addresses the meaning of section loss, its effect on the load-carrying capacity of the structure, and remediation considerations depending on its location.

### 3.1—OVERVIEW OF SECTION LOSS

Section loss is a generalized term used to describe a reduction in thickness of a structural steel element. Most commonly, section loss is attributed to corrosion of the steel due to exposure to moisture and chlorides. Bridges frequently exhibit distress (e.g., corrosion or spalling) in isolated areas where bridge elements are exposed to water and deicing salts from the roadway. The exposure often stems from poor detailing (e.g., those which trap water), salt spray from a roadway below, or deterioration of the overlying bridge elements intended to provide a barrier between the bridge deck runoff and the underlying bridge components. For steel bridge elements, deterioration is exacerbated by insufficient bridge cleaning and maintenance of protective coatings. Some locations where section loss is most commonly observed are:

- below deteriorated portions of leaking bridge deck expansion joints;
- beneath bridge deck drains or scuppers which are not functioning properly;
- along exposed portions of through-truss, cable-stay, or suspension bridges near the top of the bridge parapets;
- lower portions of stringers and deck-truss elements exposed to moisture from below due to under-passing traffic or streams; and
- at the water line or ground line for steel substructures and piles.

As section loss advances, it can cause reduced load-carrying capacity or even contribute to partial or complete failure of the bridge.

### 3.2—OPTIONS TO ADDRESS SECTION LOSS

The transportation industry has employed a number of techniques to address section loss in bridges. Before planning any work, it is important to identify the cause of the section loss. The Owner should first determine whether the source of the issue should or can be corrected. This will be useful in determining the proper technique and design criteria for addressing the section loss. The most frequently used techniques are plating, partial-depth member replacement, or full-depth member replacement.

Most commonly, section loss is addressed by plating the member. Plating is a term used to describe a repair where supplemental steel plates, or other members, are connected to the existing member to create a built-up member. This repair strategy is used for strengthening and to improve the load rating. It may also result in an improvement to the National Bridge Inspection Standards condition rating of the element.

In more extreme cases, or in locations where geometry makes plating less practical, partial-depth or full-depth member replacement can be performed. This approach is often more practical at locations near connections and bearings where several layers of cover plates and fill plates would otherwise be required to address the distressed region while still transmitting the desired capacity to the connection. Gusset plates and beam ends (e.g., regions near bearings or where transverse and longitudinal members frame into one another) are among the most common of these locations and are discussed in detail in Article 3.3.

There is ongoing research into alternatives to plating or replacement of distressed members. Technologies being evaluated include synthetic coatings, such as fiber-reinforced polymer (FRP), encasement with ultra high-performance concrete (UHPC), and thermally applied metals to restore the original section. In addition, there is ongoing research examining the actual effects of section loss and pack-out corrosion on the strength and service limit states of steel members.