# Guidelines for Field Repairs and Retrofits of Steel Bridges

## G14.2—2023


American Association of State Highway and Transportation Officials
AASHTO

National Steel Bridge Alliance NSBA Founded 1995

**AASHTO/NSBA STEEL BRIDGE COLLABORATION**

American Association of State Highway and Transportation Officials

National Steel Bridge Alliance

## 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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AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS
555 12th Street, N.W., Suite 1000
Washington, D.C. 20004

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# DEFINITIONS AND ABBREVIATIONS

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| 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 | *(AASHTO LRFD) Bridge Design Specifications* |
| 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 | *Manual for Bridge Evaluation* |
| 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 $20^{th}$-century Carnegie (left) and Bethlehem (right) rolled shape catalogs.

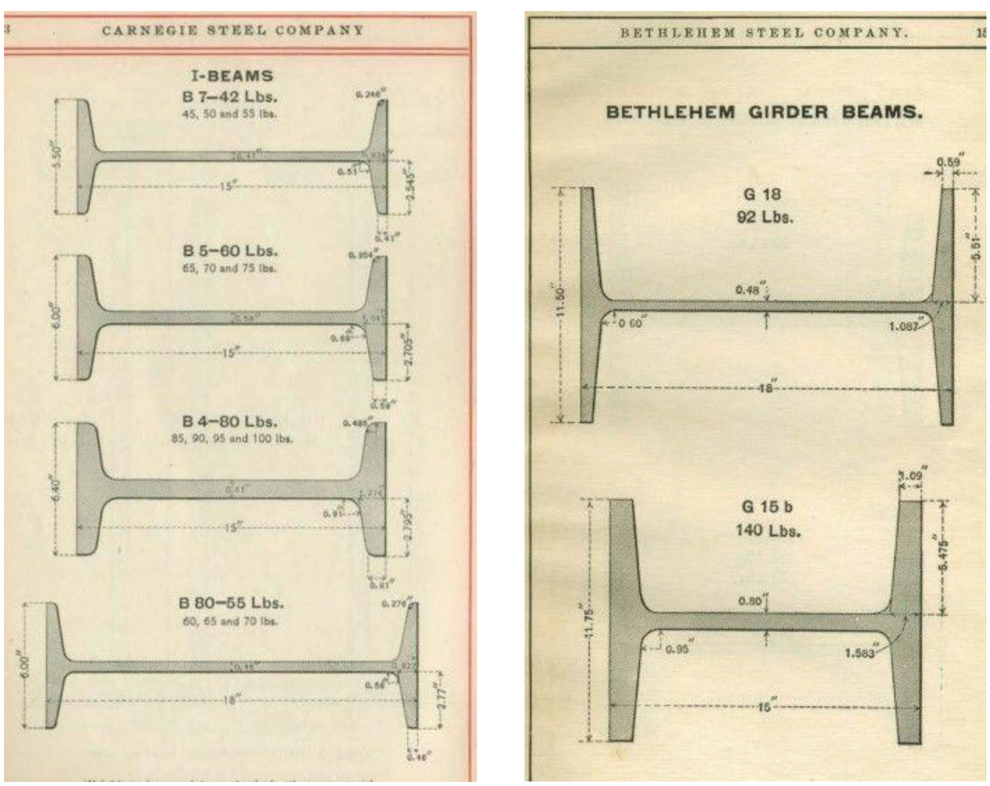


**Figure 1-1—Examples taken from Carnegie and Bethlehem historical steel shape reference catalogs**

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**

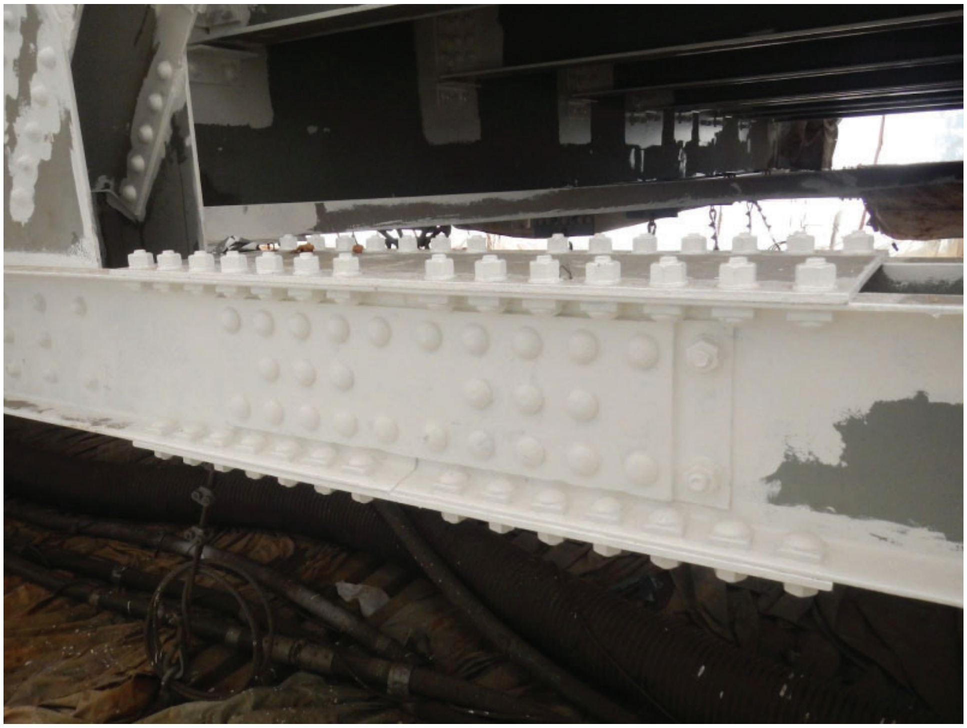

**Figure 2-2—Truss bottom chord field splice (outside view)**

**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 ½ 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.

### 3.2.1—Plating Repairs

Plating repairs to address section loss are usually designed and detailed based on inspection reports and existing plans in conjunction with supplemental field visits to take detailed measurements. The existing coating system should be removed prior to installing any plating repairs. In isolated locations, the coating system can be removed only at the contact surfaces between the existing members and the new plating. For bridges where widespread repairs are needed, it may be beneficial to perform the repairs in conjunction with a full-scale cleaning and painting contract. The limits and magnitude of section loss measured during design are often underestimated and substantial additional section loss can be uncovered when the Contractor removes the existing coating system or the build-up of pack rust. In some cases, the observed section loss after coating removal may warrant supplemental evaluation by the Engineer to determine the appropriate limits of repair and whether a plating repair is still preferable. However, the need for and mechanism to trigger the additional evaluation can vary significantly depending on the Owner, Contract type, Contractor, and construction inspection staff. Therefore, convey tolerable limits to the Contractor and be flexible with the limits of repair details.

General plating repairs can take a variety of forms depending on the member type being plated and the region of section loss. Plating repairs can be made with flat plate, bent plates, rolled members, or built-up sections depending on location and preference. The use of flat plates offers the most flexibility in the field by simplifying procurement and fabrication and should be the first choice wherever practical. Flat plates are viable options wherever repairs or retrofits can be limited to an isolated region on the member. For instance, regions where plating can be limited to the flange or web of a member are ideally suited to the use of flat plates. If the repair or retrofit needs to include the flange-to-web interface, then a bent plate, angle, built-up angle, channel, or built-up channel member may be considered. Articles 3.2.1.1 through 3.2.1.3 discuss specific design and detailing considerations associated with particular types of repairs, as well as those that are general in nature.

#### 3.2.1.1—Bent Plate

A versatile method of plating across irregular geometries is bent plate. Flat plate can be cold-bent to match the geometry of existing circular shapes, plate girders, box girders, or rolled shapes. The Fabricator cannot bend a plate to a prescribed minute and second accuracy. Variability in the process will limit the precision to approximately ±2 degrees. The Designer should consider these fabrication tolerances for both the new and existing elements and determine whether the stiffness of the plating will enable it to deform as the bolts are tightened to conform to the shape of the in-situ member.

When detailing plating repairs using bent plate, it is important to understand the practical geometric limitations of the bending process. The AASHTO BDS and AASHTO *Steel Bridge Fabrication Specifications* (SBFS) call for a minimum inside bending radius (i.e., measured to the concave face of the plate) of five times the plate thickness (5*t*) in most situations to prevent the degradation of the toughness of the steel in the region of the bend. Prior to 2012, smaller minimum radii were required based on preventing cracking during bending. There are several factors that contribute to the minimum bending radius, such as the tensile strength, rolling direction, and thickness of the material. However, the minimum 5*t* bending radius may not fit the geometric requirements of many plating situations. The Designer must assess the fatigue and ductility demand of the situation. Where degradation of these particular properties is less of a concern, particularly in common plating scenarios in which the bent plating does not make up the whole cross-section, the older requirements may be referenced and a minimum bending radius of twice the plate thickness, or 2*t*, can often be used as a rule of thumb. The older AASHTO bending radii can be found in an appendix to ASTM A6.

When planning a plating repair, the bend radius should be considered when determining bolt layout and the outside radius should be compared to the existing geometry to identify potential fit-up issues. In Figure 3-1, where the web and inside face flange of a W-shape member are repaired using a bent plate, the inside radius of the bent plate may restrict how closely to the web the bolts can be placed and may limit the number of rows of bolts that can be specified, especially for thicker plates. The outside radius of the bent plate may not match the flange-to-web fillet of welded plate sections or the k-section profile of the existing rolled member, and a filler material may be needed to adequately seal the repair. Figure 3-1 illustrates how the bending radius associated with the thicker plate may not yield the desired results. In this scenario, multiple thinner bent plates can be nested to reduce the minimum required bending radius. This configuration will introduce a second gap between the nested repair plates as illustrated in Figure 3-1. A rolled or built-up angle can also be considered.

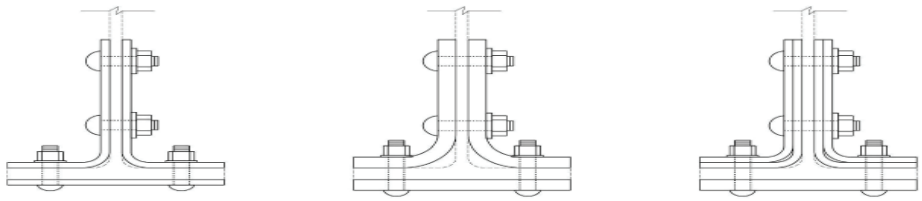

**Figure 3-1—Plating an existing web and flange with bent plate for thin plates vs. thick plates**

### 3.2.1.2—Rolled and Built-up Members

Rolled or built-up angle and channel shapes are also routinely used for plating flange-to-web interfaces. Channel shapes are commonly used for web plating against one flange of an existing member. However, channels have square outside corners which must be ground to clear the beam's flange-to-web radius or flange-to-web fillet weld. Channels may also prove troublesome when the goal is to plate the inside face of both flanges continuously with the web. The ability for field staff to install a channel shape between existing flanges is dependent on the channel being no deeper than the clear distance between flanges. Both built-up and rolled members have fabrication or mill tolerances. Even minor variations in clear distance due to fabrication or rolling tolerance can prevent installation of the member. For this reason, it is preferable to instead use a pair of angles and a flat plate to give the Contractor more flexibility to match the existing geometry. An example can be seen in Figure 3-2.

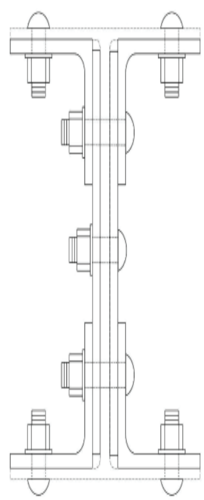

**Figure 3-2—Plating an existing W-shape with angles and flat plate**

Similarly, if it is not critical that the plating be continuous across the web, then a pair of angles can be used, as long as a gap is provided between the ends of the angle legs along the web to accommodate any needed tolerance. An example of this scenario can be seen in Figures 3-3 and 3-4.

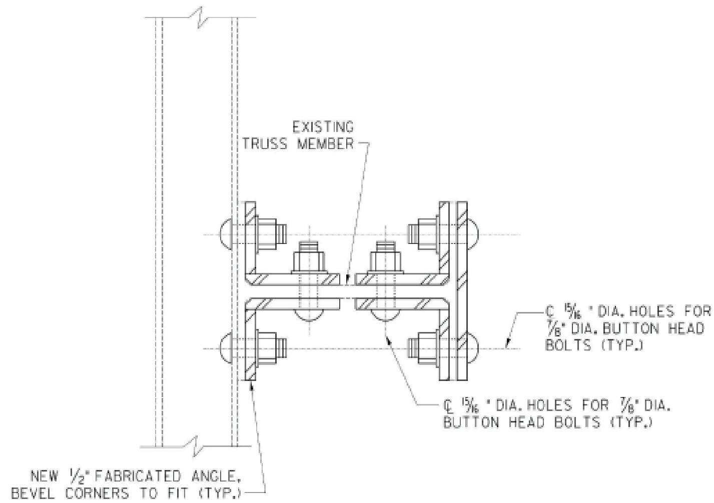


**Figure 3-3—Plating an existing W-shape with built-up angles**

**Figure 3-4—Photo of plating an existing W-shape with built-up angles**

Another detail to consider when using angle or channel sections is how to provide clearance around the flange-to-web weld or fillet. Often this is handled by grinding off the heel of the angle or channel to fit. Sample details using this approach are illustrated in Figures 3-5 and 3-6.

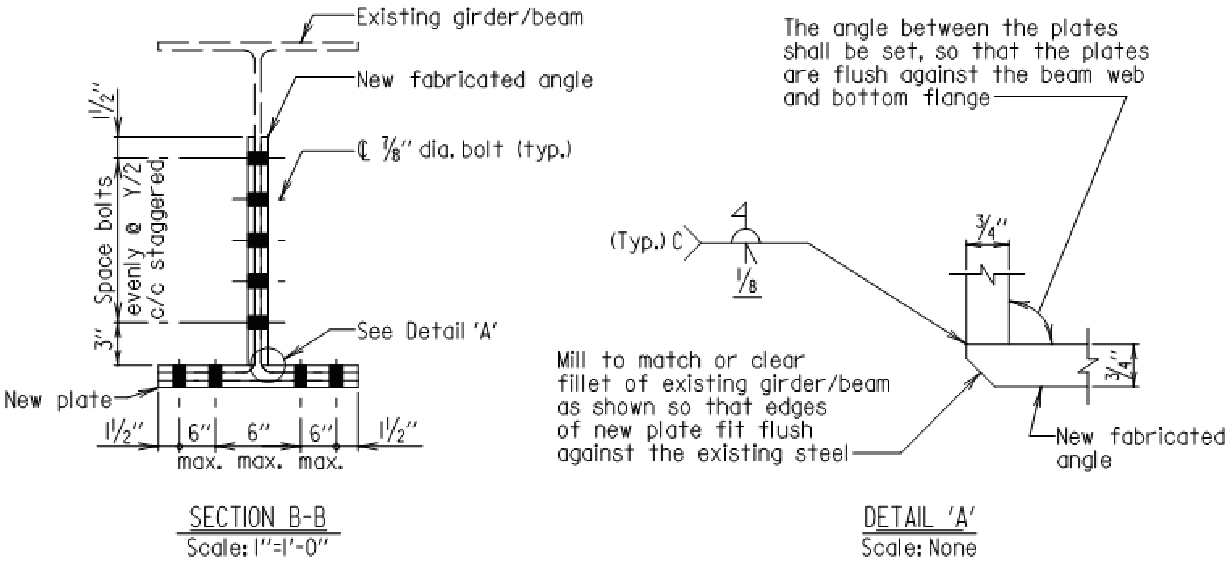


**Figure 3-5—Grinding heel of built-up angle (Maryland Department of Transportation [DOT] State Highway Administration Detail No. SRST(SR)105)**

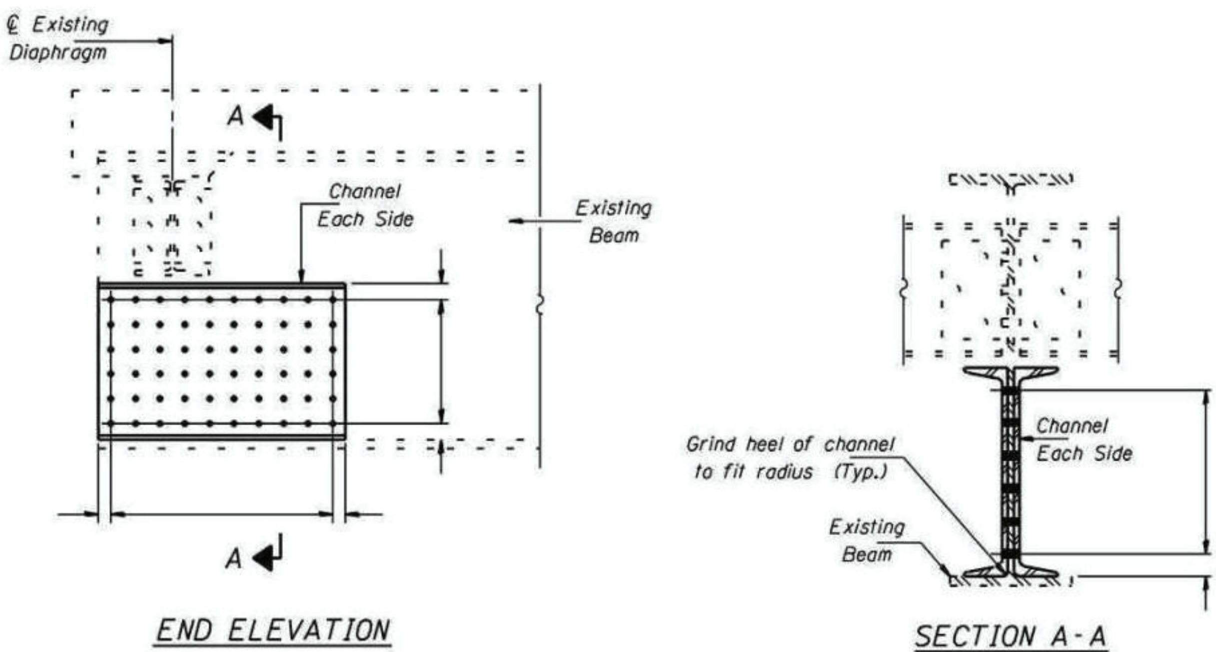


**Figure 3-6—Grinding heel of rolled channel (retired Illinois DOT *Structural Services Manual* Figure 2.15-4) Note: Illinois DOT has revised this standard detail to use a bent plate in a pending update.**

As an alternative to grinding, when using built-up angles, the plates could be offset in order to avoid the flange-to-web weld or fillet. An example of this detail is shown in Figure 3-7. In this case, a minimum plate thickness of ¾ in. was selected to provide a ¼-in. overlap between the plates along with adequate edge distance beyond the fillet weld. Some fabrication advantages of this offset include the use of fillet welds instead of groove welds and eliminating the need to mill the heel.

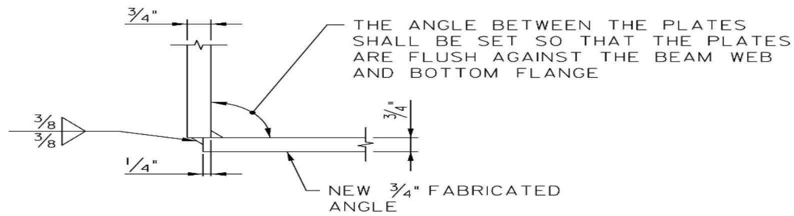


**Figure 3-7—Built-up angle with offset plates**

### 3.2.1.3—Design and Analysis Considerations

Design and analysis considerations associated with plating repairs can vary significantly depending on the application. The first determining factor is whether the plating will be required to add capacity to the section, to reestablish the original capacity of the section, or simply to cover and seal distressed areas. This is an important distinction when determining how far to extend the plating.

When adding capacity to a section or reestablishing the original capacity of the section, the plating must be developed beyond the point where the additional capacity is no longer required. This is analogous to development of reinforcing steel beyond the point where the bar is required to carry tension. Consider the tension flange of a beam, for instance: the plating will not experience any load at the end of the plate until the first row of bolts transfers some load from the flange into the plating. Therefore, the plating would not be considered to be fully developed until the point where enough bolts have been provided to transfer the required amount of force into the plating. An illustration can be seen in Figure 3-8.

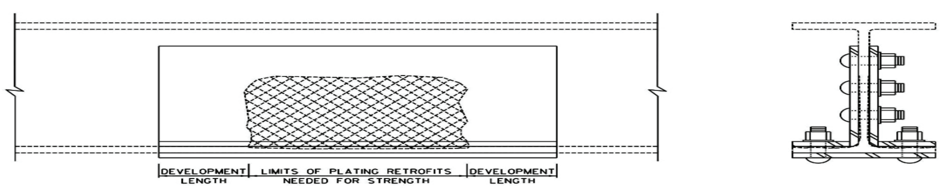


**Figure 3-8—Plating detail illustrating development limits**

The Engineer should determine how much force needs to be carried by the plating. It may be practical to develop the entire capacity of the additional plating but geometric constraints or the number of bolts required to develop the capacity may only accommodate developing a reduced supplemental capacity before a critical location on the member. Additional capacity will continue to develop at subsequent rows of bolts on longer plating repairs.

When plating tension members, compare the gross section capacity to the net section tensile capacity of the existing member, particularly when there is already section loss present. The layout of the bolts in the development zone of the plating should be evaluated to guard against reducing the overall capacity of the member at the first row of bolts attaching the new plating. At this location, the plating has not developed any capacity, so the introduction of bolt holes may reduce the capacity of the member if governed by net section tensile capacity. This effect may be tolerable but should be evaluated on a case-by-case basis.

When plating the flanges of flexural members, net section tensile capacity concerns may require that the limits of plating initiate in a lower stress region. For pure tension members (such as in trusses), the end connections are often bolted or riveted and may already govern the net section capacity. For many member types, limiting the number of bolt holes to match the end connections may avoid capacity reduction concerns. This may not hold true at locations with significant section loss near the gusset plates where the plating may need to be anchored to the end connection. Members where plating is added to regions not already containing bolt holes at the end connections may also warrant evaluation of the net section tensile capacity. An example might be an I-beam or W-shape section connected to gusset plates along the outside face of each flange. If plating is proposed for both flanges and the web, then net section tensile capacity may need to be evaluated and the holes for the web plating may need to be staggered with respect to the holes for the flange plating.

It is important to remember that repairs and retrofits are usually performed while the existing member is in a loaded state. The dead load is carried by the existing steel and the transient loads are carried by a combination of the existing and new material. In general, this is not a concern because the ductility of the steel allows for load sharing between the existing and new steel elements. Special consideration may be necessary when welding to members under load, reinforcing compression members for a stability limit state, or partially disassembling a member in order to implement a repair (AISC, 2018).

As previously noted, removal of the existing coating system often shows that the section loss measured by routine safety inspections is initially underestimated. Figure 3-9 provides an illustration of a poorly executed repair where substantial additional section loss was observed upon removal of the existing coating system.

**Figure 3-9—Plating repair with severe section loss and inadequately prepared irregular surface**

There are materials that can be used as filler to address minor variations in existing material thickness and to promote flush plating repairs free of voids which could allow water infiltration and adversely affect the durability of the repair. Steel-reinforced epoxy putty is one such common material since it adheres well to vertical surfaces, has a rapid functional cure time, and can be machined after it is fully cured. However, it is important that the Engineer and Contractor understand the limitations of these materials and that they do not contribute to the structural capacity of the section. Since these materials are meant to address minor variations in thickness, they can be difficult to apply in situations where section loss is more severe. It is also important that the Contractor follow the manufacturer's recommended installation procedures. If the material sets up prior to the plating and bolts being installed, then gaps are likely to form due to the stiffness of the filler materials unless they are properly machined. An example of this (as well as poor caulking to seal residual gaps) can be seen in Figure 3-10.

**Figure 3-10—Plating repair where epoxy filler hardened prior to plate installation**

There are some details that were common on older bridges that are rarely used in modern steel construction. One such detail pertains to bolted field splices on riveted bridges where filler plates were commonly extended beyond the gusset or splice material and anchored with rivets. In some cases, these filler plates may be thin and flexible and, if the existing rivets are replaced by high-strength bolts, could be prone to warping when the bolts are tightened. Consideration may need to be given to adding a thicker (stiffer) cover plate to protect the existing filler plate. An example of this effect can be seen in Figure 3-11, where replacing the existing rivets with high-strength bolts caused tearing of the thin filler plate. Thin plates are also more susceptible to warping from pack rust, which is also evident in Figure 3-11.

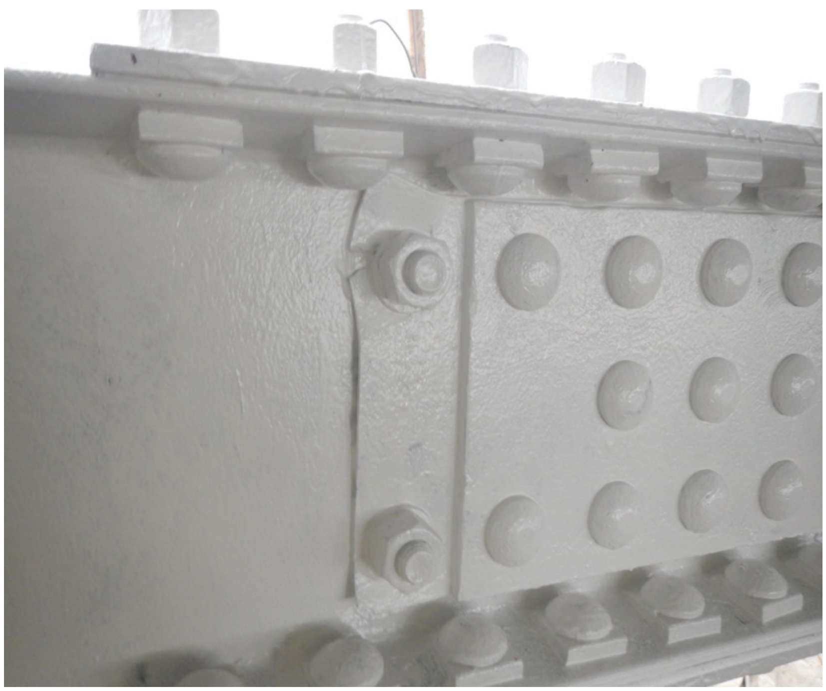

**Figure 3-11—Damaged filler plate due to bolt tensioning**

Additional considerations associated with older structures (especially truss bridges) relate to changes in standard rolled steel shapes over time. As discussed in Article 1.5, the AISC website maintains a database of historical shape references that should be referenced in cases where a new AISC shape is proposed to replace a historical shape. Even minor differences in width or thickness can create significant fit-up issues in the field. Take, for example, a case where a W-shape truss vertical member with a riveted connection to gusset plates along each flange is replaced. If the new AISC shape is even ⅛ in. thicker than the existing historical member, then the flanges would require grinding to match the original member depth. Gusset plates that are connected to multiple members can demonstrate significant stiffness, and the Contractor may have trouble forcing the new member into the existing void between gusset plates. Doing so would also cause a deformation of the gusset plate that induces additional stress into the adjacent bolts or rivets and out-of-plane bending of the gusset plate itself.

### 3.2.2—FRP Repairs

Owing to their light weight, durability, and excellent fatigue and corrosion resistance characteristics, the use of FRPs or carbon-fiber-reinforced polymers (CFRPs) for the repair of steel structures can offer significant potential as a viable repair method. In general, research efforts on retrofitting steel elements have examined the following four areas as noted in Shaat et al. (2004):

1. repair of naturally deteriorated steel girders,
2. repair of a girder or steel plates artificially notched to simulate fatigue cracks,
3. strengthening an intact section to increase the girder stiffness and flexural capacity, and
4. increasing the composite action between the steel girder and concrete deck in bridge application.

Figure 3-12 shows one of the few examples found on the use of FRP for strengthening a steel bridge girder. Various experimental tests and analytical studies investigating the effect of FRP bars and CFRP strips on strengthening steel elements have been carried out, many of which are summarized in Mahmoud and Riveros (2013). Recent studies have focused on the repair of precracked steel panels (10 percent section loss) in wet and corrosive environments (Hudak, 2019). The remaining fatigue life was used as the primary metric for assessing the efficacy of the retrofit method. Results indicated that the use of both CFRP and basalt-fiber-reinforced polymers (BFRPs) is effective at extending fatigue life of the precracked steel panels subjected to 8 ksi stress range. One of the benefits of using BFRPs is that basalt is not a conductor and, as such, galvanic corrosion does not develop and the use of fiberglass layers is not needed. The repair resulted in a substantial increase in fatigue life, which was a function of the size of the patches and whether a single layer or double layers were used.

**Figure 3-12—Application of FRP bar for bridge repair (Wipf et al., 2003)**

#### 3.2.2.1—Application to Steel Bridges

Despite the number of experimental and analytical studies on the use of CFRP patches and plates for the fatigue enhancement of steel elements, limited studies were found in the literature to demonstrate the use of CFRP for repairing steel bridges in the U.S. Examples of such applications can be found in Phares et al. (2003), where a steel girder bridge in Guthrie County, Iowa, was strengthened with post-tensioning CFRP rods; and in Mertz et al. (2002), where the 1-704 Bridge over Christina Creek in Delaware was strengthened with CFRP. Despite the limited applications to U.S. bridges, CFRP has been successfully used to repair numerous Army Corps of Engineers steel hydraulic structures. In Europe, the use of CFRP for the repair of steel bridges is common, as summarized in Hollaway et al. (2002).

#### 3.2.2.2—Application to Steel Structures in Harsh Environments

Deterioration in the form of excessive corrosion, corrosion fatigue, and stress corrosion cracking can be present due to rain, salt air near coasts, or the use of deicing salt on bridge decks (seeping through bridge joints) or on roadways below bridges (uplifted and transported to bridge girders due to traffic underneath the bridge). There are various examples of CFRP applications for ship and submarine structures. Allan et al. (1988) used CFRP patches for the repair of aluminum ships and added a moisture barrier composed of foil sheets and strand glass laminate to cover and seal the CFRP patch. CFRP patches were also installed on a Royal Australian Navy frigate to prevent the recurrence of superstructure fatigue cracking, as noted in Grabovac and Whittaker (2009). The repaired ship has been in service for 15 years following the repair, and the patches were effective in eliminating crack growth except for one crack that reappeared due to debonding failure. Similarly, Det Norske Veritas, a Norwegian foundation, used CFRP patches to conduct field repair of cracked bulkheads in the Norne floating production storage and offloading (FPSO) units and corrosion pits on the repair of the floating storage and offloading (FSO) vessel ABU in the Abu Cluster (DNV.GL 2015). Various field repairs have been performed to repair fatigue cracks observed in underwater steel hydraulic structures. For example, significant deterioration was observed in the Pickwick Lock and Dam located on the Tennessee River. Specifically, the strut arms of a vertically framed Tainter valve were repaired using CFRP patches, as shown in Figure 3-13. The strut arms are subjected to large axial cyclic stresses. The durability of CFRP repairs has been successfully demonstrated through their application to various U.S. Army Corps locks for the period of five years. Long-term (i.e., 20 years or more) durability studies should be conducted to evaluate the use of this repair method over a longer time span.

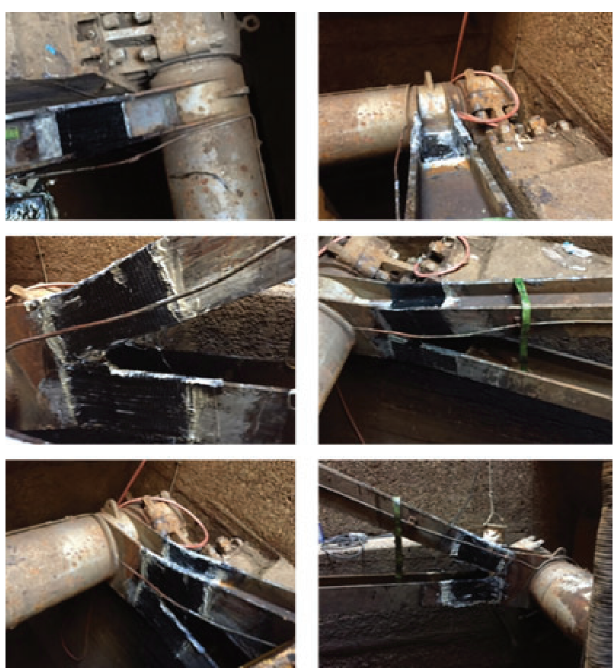

**Figure 3-13—Applied CFRP on the problematic detail (Riveros et al., 2018)**

#### 3.2.2.3—Recommended Installation Process and Sequence

The steps required for installing the repairs are as follows:

1. The surface on which the patches will be installed should be roughened using a grinder or sandblaster.
2. The roughened surfaces should be thoroughly cleaned with acetone to remove particles from grinding or sandblasting.
3. The required fiber patches should be cut to size and wiped with a cloth dampened with acetone to remove any dust.
4. Glass fibers should be cut approximately ½ in. larger in both dimensions than the carbon fibers to provide an adequate insulating layer should there be any slight misalignment during application.
5. Adhesive should be spread on the roughened steel surface in a uniform layer using a spatula.
6. A layer of fibers should then be placed on top of the adhesive and gently pressed into the adhesive and smoothed to remove any air bubbles and align fibers.
7. It is recommended that carbon fibers and basalt fibers be saturated with the adhesive (the glass fiber fabric layers are not to be saturated) before they are placed.
8. The fiber patches should be placed such that the unidirectional fibers are aligned perpendicular to the crack plane.
9. After all the layers are applied, spew fillets (i.e., rounded surface of a fillet of adhesive) around the patches should be formed by hand from additional adhesive to create a smooth transition between the patch and the steel.
10. The epoxies should be allowed to harden before steel is handled (new steel) or exposed to live load (existing steel) per the manufacturer's specified cure time.

## 3.3—CONSIDERATIONS FOR SPECIALIZED REPAIR REGIONS

### 3.3.1—Gusset Plate Repairs

Gusset plates are routinely used in truss bridges to efficiently transfer forces between members at the panel points. There was a heightened awareness of the importance of gusset plates after the 2007 collapse of the I-35W highway bridge over the Mississippi River in Minneapolis, Minnesota. Following the collapse, the FHWA issued Technical Advisory T5140.29 (2008), which recommends bridge Owners include the gusset plates in a bridge's load rating. These recommendations included revisiting previous load rating analyses for existing bridges in order to evaluate the capacity of the gusset plates with consideration of the existing condition (deterioration, distress, etc.). The FHWA Technical Advisory resulted in the identification of numerous gusset plate connections throughout the United States that required repairs to improve a deficient load rating factor. Gusset plate repairs are often more costly than other strengthening repairs due to the limited room available to implement the repair, the unique nature of the repair design to accommodate individual gusset plate geometry, and the need for partial disassembly, supplemental load support, or maintaining alignment of the members and gusset plates during the repair. Therefore, it is important to accurately analyze, design, and properly detail gusset plate repairs because the cost of a thorough upfront analysis is usually insignificant compared to the cost of the repair itself, as well as the cost and risk of an ineffective repair. This Article focuses on primary gusset plate connections for truss chords, diagonals, and verticals that are under stress during the repair. Repair of secondary gusset plates, such as lateral bracing connections, would be similar but may not require detailed consideration of the repair sequence as long as the bracing of the truss chord at the panel point is maintained throughout the duration of the work.

#### 3.3.1.1—Gusset Plate Inspection and Documentation

Documentation of the as-built gusset plate geometry and current condition is the first step in determining the capacity for a load rating analysis. A hands-on inspection is commonly performed to obtain, or confirm, information typically shown on the shop drawings, including measurements of the overall gusset plate geometry, thickness, and bolt or rivet size and spacing.

A gusset plate inspection will also include measurements of remaining thickness in areas of corrosion-related deterioration, pitting, and section loss. Due to the uneven surfaces of corrosion-related deterioration, UT methods are the most appropriate for measuring the remaining thickness of corroded gusset plates. However, the results for UT thickness measurements must be verified through some form of calibration because uneven surfaces due to pitting can lead to erroneous results. In cases where uneven surfaces are present on both faces of the gusset plate, pit-depth gauges, a straightedge, or other mechanical devices are sometimes used if the measurement tool can be placed on the adjacent, undeteriorated plate surface. Gusset plate deterioration is often most severe along edges of the connecting members where salt-laden water can pond on the surface and these locations are often the critical section for gusset plate shear, such as the example shown in Figure 3-14. Therefore, the gusset plate deterioration can have a significant influence on the connection capacity and calculated load rating factors.

**Figure 3-14—Gusset plate connection with severe pitting and through-thickness loss along the horizontal plane located just above the truss bottom chord**

It is essential to develop detailed documentation—such as that shown in Figure 3-15—of gusset plate pitting, including both magnitude and location of section loss, in order to document the remaining plate area at each of the critical sections. For example, reporting only the minimum remaining thickness (0.58 in.) of the gusset plate in Figure 3-15 corresponds to thickness loss of approximately 23 percent; however, detailed documentation of the remaining thickness along the horizontal shear section, just above the bottom chord, indicates only approximately a 6 percent section loss. Overly simplified or incomplete field notes can often lead to a conservative load rating analysis and installation of unnecessary repairs.

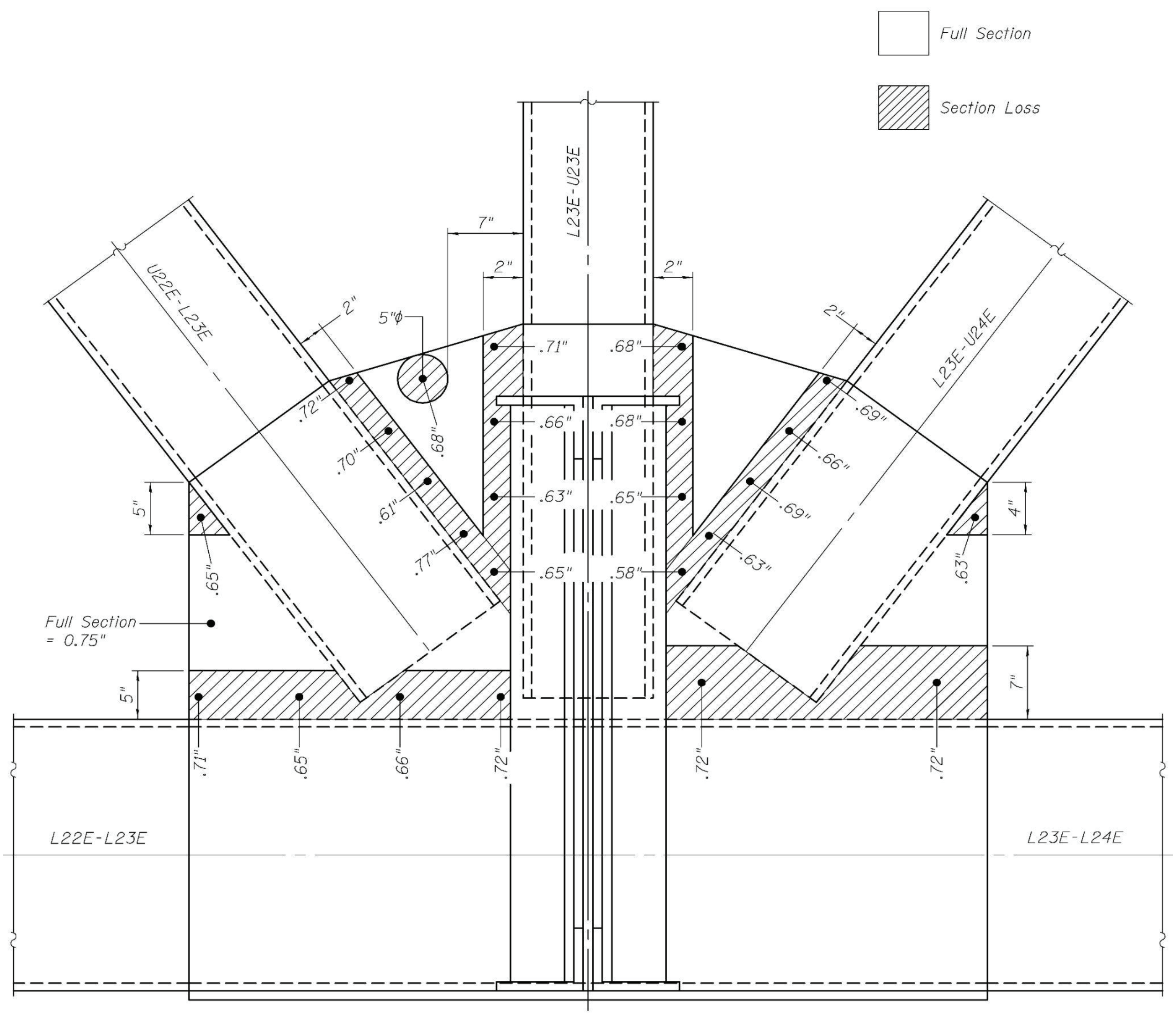


**Figure 3-15—Example of detailed documentation of gusset plate section loss**

Warping, distortion, buckling, or other distress must also be documented during the field inspection. Depending on the severity of the observed conditions, the distress may need to be immediately reported to the Owner. Repair of gusset plate distress is determined based on the likely cause. Photographs of each gusset plate and the existing conditions are recommended to supplement the field notes.

### 3.3.1.2—Gusset Plate Strength Evaluation

Gusset plates throughout a truss structure often have unique dimensions due to the geometry of the members framing into them, calculated design forces, and the existing deterioration. As a result, each deteriorated gusset plate in a truss bridge should be evaluated individually. Given the unique geometry at each panel point, the wide range of potential limit states, and the limited room available to install the repair, gusset plate repairs are rarely repetitive and are, therefore, costly to implement. Conservative analysis assumptions can result in a significant underestimation of gusset plate capacity, which will likely lead to these costly repairs. Using more accurate analysis methods to calculate gusset plate capacity for each controlling limit state will potentially result in a reduced number of repairs and substantial cost savings.

A gusset plate load rating typically includes the calculation of each potential controlling limit state with consideration of the documented section loss. Typical strength limit states include fastener shear, block shear, compression (e.g., Whitmore), tension, vertical shear, horizontal shear, and partial shear. An example of the horizontal shear limit state is shown in Figure 3-16. Publication FHWA-IF-09-014 (FHWA, 2009) was issued after the collapse of the I-35W highway bridge in order to provide a relatively simple and straightforward methodology for evaluating the various gusset plate limit states. The provisions in Publication FHWA-IF-09-014 have been updated and incorporated into AASHTO MBE.

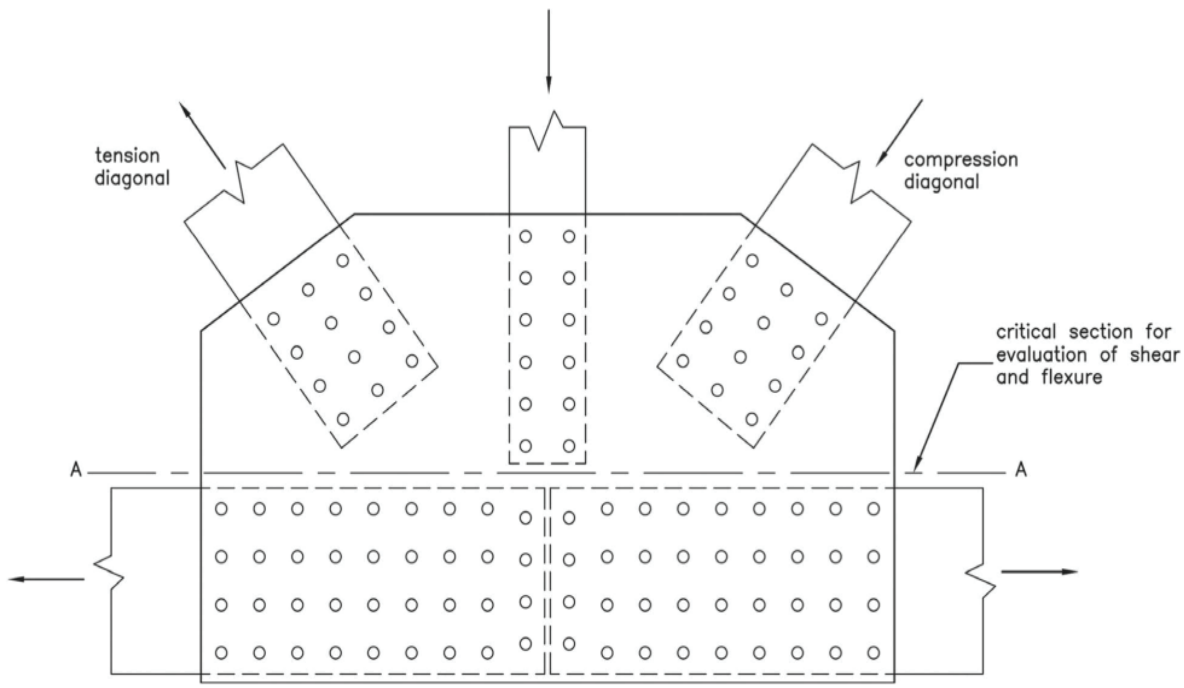


**Figure 3-16—Example gusset plate showing the horizontal shear limit state (Section A-A)**

The gusset plate provisions in AASHTO MBE are applicable to a wide range of gusset plate configurations, and so the provisions may yield conservative results in certain cases. As a result, gusset plates that are found to have a load rating factor less than 1.0 using AASHTO MBE should not automatically be considered deficient. A more rigorous approach should be used to further improve the calculated gusset plate capacity, which may include consideration of concurrent gusset plate forces instead of envelope forces, material sampling (from a non-critical area) to determine the actual yield strength, or by using refined analysis methods such as those presented in Ocel et al. (2013), Hill (2014), or IDOT (2014).

Gusset plate deterioration and section loss further complicate the gusset plate analysis because several cross-sections may need to be considered on a case-by-case basis for a single limit state. For example, the critical sections may need to be selected based on the original area without section loss, a reduced area that considers section loss, a net area that considers fastener holes, or some combination thereof. The example shown in Figure 3-17 shows a tension diagonal (red arrow) framing into one side of a gusset plate. When performing a partial shear check, or refined "corner check" analysis, the controlling limit state may be the longer red line with a reduction for the section loss along the horizontal plane or the shorter yellow line that represents a net section with a reduction in area for the rivet holes, but without section loss. Whether using standard or refined analyses methods, analyses can typically be completed using hand- or spreadsheet-based calculations; three-dimensional finite element modeling is typically not necessary, except in rare instances.

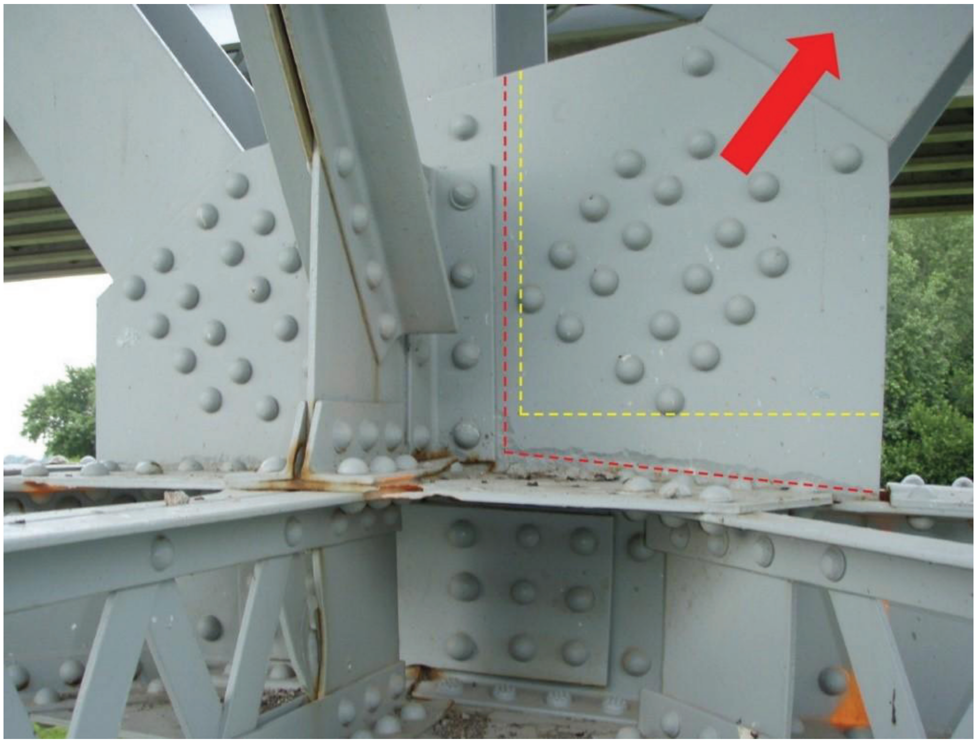

**Figure 3-17—Example of a deteriorated gusset plate showing the partial shear (or corner check) limit state along the deteriorated section (red) and the minimum gross section adjacent to the rivet holes (yellow)**

### 3.3.1.3—Gusset Plate Repair Options

Based on the results from the more rigorous load rating analysis, repairs may be required where the load rating factors are deficient. Efficient repair design will recognize the current capacity of the existing gusset plate. Even severely deteriorated gusset plates have adequate capacity to carry the dead load forces, possibly with a reduced factor of safety; otherwise, the gusset plate connection would have failed. Because of the ductile nature of structural steel, gusset plate repairs will carry their share of the applied loads due to yielding and plastic deformation under full factored design loads. Gusset plate repairs typically consist of localized strengthening of an element or critical section, partial gusset plate repairs to address a larger area, supplemental gusset plates to "sister" another gusset plate on the original plate, or gusset plate replacement. Once a gusset plate is designated for strengthening, typical steps of the repair process include:

1. Measure and document required repair element dimensions, bolt size, bolt spacing, and potential interferences (see Figure 3-18).
2. Fabricate steel repair elements (see Figure 3-19).
3. Remove existing fasteners and drill holes for new fasteners (see Figures 3-20 and 3-21).
4. Prepare areas of through-thickness section loss (create radiused corners and smooth cut edges) (see Figure 3-22).
5. Clean and paint exposed steel surfaces that will be covered by the repair elements (see Figure 3-23).
6. Install steel elements and tighten bolts (see Figure 3-24).
7. Seal contact surfaces (to prevent future water entry and pack rust) and apply topcoat (see Figure 3-25).

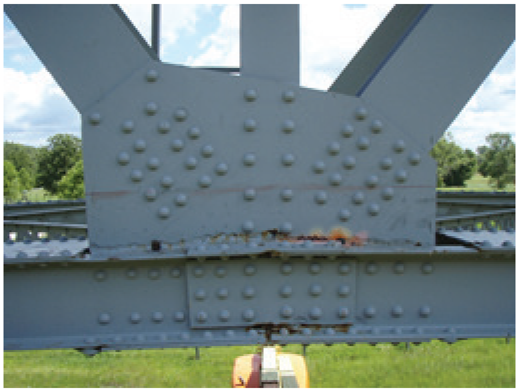

**Figure 3-18—Field-verifying repair dimensions**

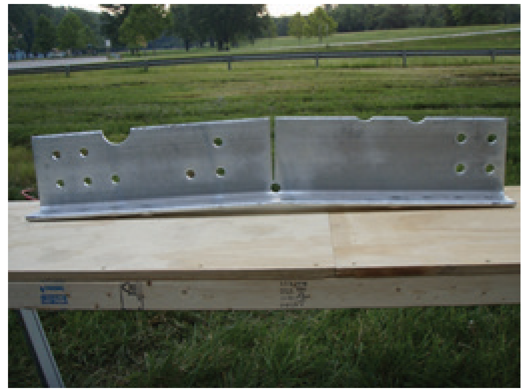

**Figure 3-19—Fabricating steel repair elements**

**Figure 3-20—Removing existing fasteners and drilling holes**

**Figure 3-21—Holes drilled on existing gusset plate**

**(a)** **(b)**

**Figure 3-22—Preparing through-thickness section loss by smoothing and rounding edges: (a) original condition and (b) after preparation**

**Figure 3-23—Coating exposed steel surfaces**

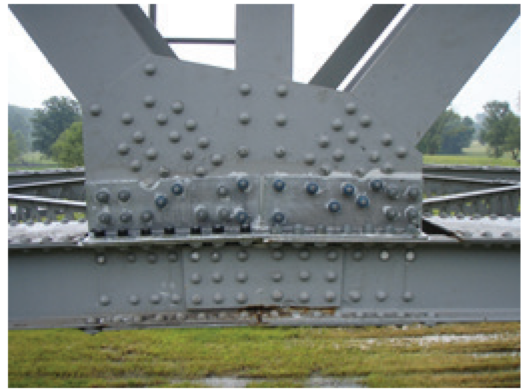

**Figure 3-24—Installing steel repair and tightening bolts**


**Figure 3-25—Final repair installation after sealing contact surfaces and application of topcoat**

*3.3.1.3.1—Localized Strengthening Repairs*

Localized gusset plate strengthening consists of the installation of flat plate, bent plate, or angle elements to reinforce members with areas of localized section loss. Examples of localized strengthening repairs to address section loss on a horizontal shear plane are shown in Figure 3-26, Figure 3-27, and Figure 3-28. Strengthening elements are typically installed on the face of the gusset plate with the fewest interferences. For example, Figure 3-28 has an angle element (left) and flat plate element (right) that are each installed on the opposite side of the gusset plate from the diagonal members in order to avoid interference. Localized strengthening repairs can also be installed to address other areas of deterioration and critical sections, including vertical shear, partial shear, block shear, compression buckling, and tension yielding. Figure 3-29 shows a triangular plate that was added below a compression diagonal to help prevent buckling and to provide a direct load path, in bearing, between the diagonal member and the gusset's doubler plate.

**Figure 3-26—Localized angle repair to address section loss along the horizontal shear plane**

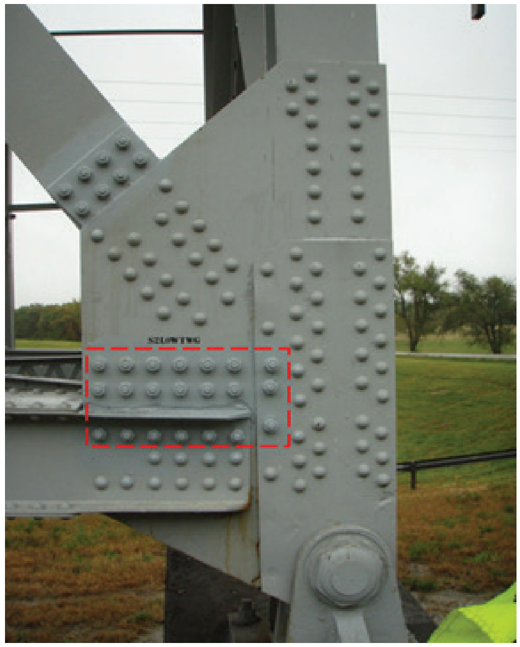

**Figure 3-27—Plate installation (red outline) on the inside of the gusset plate to address section loss on the horizontal shear plane**

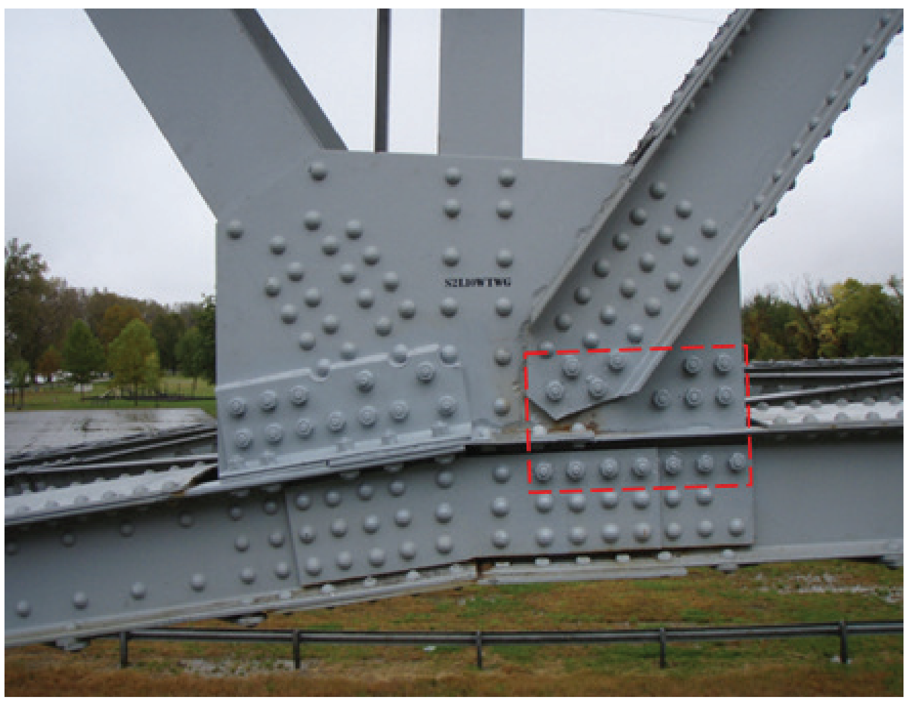

**Figure 3-28—Installation of an angle element (left side of gusset) and a plate element (dashed red line on right side of gusset represents plate installed on inside face of gusset) to address section loss on a horizontal shear plane**

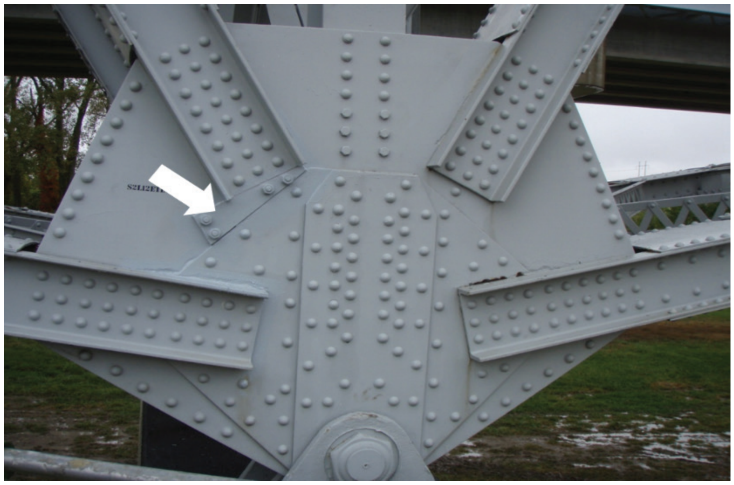

**Figure 3-29—Plate repair (arrow) to address severe section loss below a compression diagonal**

*3.3.1.3.2—Partial Gusset Plate Strengthening*

In areas with more widespread deterioration, or when multiple gusset plate limit states need to be addressed, local strengthening repairs may not be adequate. In these cases, the gusset plate may be strengthened with a larger plate that extends over multiple areas of section loss. Because of the size of these repairs, the gusset plate and connecting members often require a significant amount of disassembly to install the plate repairs. Partial disassembly of the gusset plate potentially changes the capacity of the gusset plate by modifying the load path, reducing the number of fasteners, or increasing the unbraced length of the gusset plate for the compression resistance. Therefore, if partial disassembly is done as part of the repair, each limit state must be checked in the disassembled condition to confirm the repair work does not decrease the capacity and create an unsafe condition. Various measures may be taken during the repair work to address reductions in the gusset plate capacity, including limiting or omitting live load from the members during the repair work, providing temporary bracing or other modifications to the load path, or keeping various fasteners in place to maintain an acceptable unbraced length for compression.

An example of a partial gusset plate repair is shown in Figure 3-30, in which a plate was installed to address the vertical shear, horizontal shear, and compression limit states. This repair required the removal of several rivets to install the plate, which significantly increased the unbraced length for compression buckling. Checks of the reduced compression capacity were found to be inadequate for the forces due to dead load and a reduced live load. To address this condition, four existing rivets were replaced with pretensioned bolts prior to removing the additional rivets. These four bolts are shown with arrows in Figure 3-30. By keeping these fasteners present during the repair work, the compression buckling capacity was maintained during the gusset plate disassembly. In order to install the gusset plate strengthening repair, the repair plate had to be fabricated with enlarged holes to accommodate the bolts that remained in place. These enlarged holes are shown with red circles in Figure 3-31.

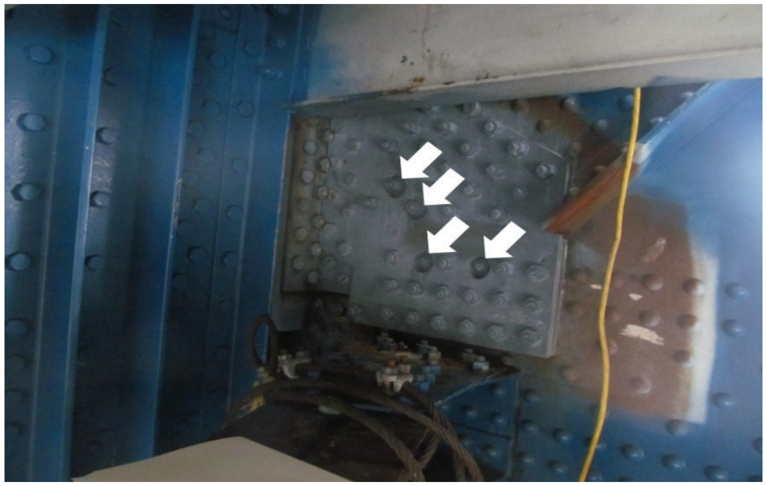

**Figure 3-30—Partial gusset plate repair that kept four fasteners in place (arrows indicate four original rivets replaced with bolts prior to disassembly) to improve the compression buckling capacity during the disassembly, see Figure 3-31**

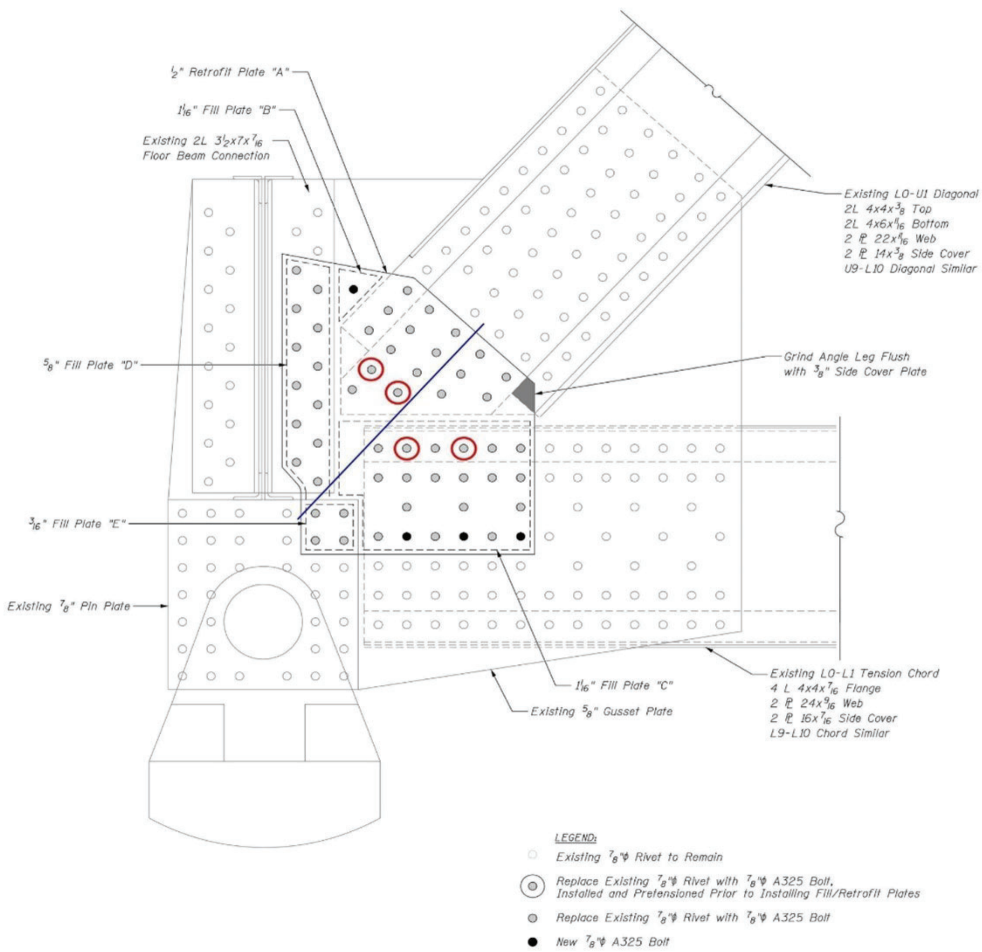


**Figure 3-31—Oversized holes (red circles) in the partial gusset plate repair to accommodate the four bolts that remained (refer to Figure 3-30) during the disassembly work, blue line indicates unbraced length of gusset plate if these four bolts had been removed**

*3.3.1.3.3—Supplemental Gusset Plates*

In some cases, when geometry permits, the gusset plate is reinforced with a supplemental gusset plate. The supplemental gusset plate is nearly the full size of the original, existing gusset plate. In order to install a supplemental gusset plate without a significant amount of disassembly, each fastener is replaced, one by one, with a fully threaded elongated bolt or high-strength threaded rod. Next, a spacer plate, or "cheese" plate, is installed with oversized holes to accommodate the nuts and washers of each fastener assembly. The thickness of the spacer plate must be slightly larger than the thickness of a nut and washer. Finally, the supplemental gusset plate, with standard holes, is installed over the spacer plate and secured with nuts and washers installed on the threaded extensions of the bolts or rods. An example of a supplemental gusset plate repair is shown in Figure 3-32. Benefits of the spacer plate and supplemental gusset plate repair include a reduced level of analysis, so they may be installed as emergency repairs. Challenges include developing procedures for tightening two different nuts on the extended bolts and determining the level of load sharing between the existing and supplemental plates due to the presence of the relatively thick spacer plate between the two gusset plates. In addition, the presence of truss members on the face of the gusset plate may require additional shim plates for this repair to be implemented.

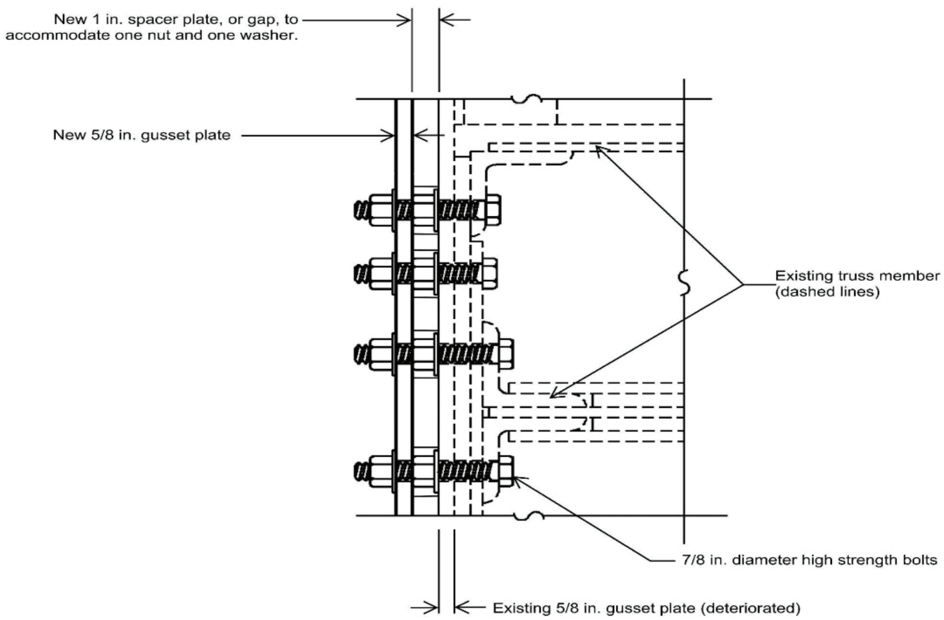


**Figure 3-32—Example of a supplemental gusset plate repair**

*3.3.1.3.4—Gusset Plate Replacement*

In rare instances of severe deterioration, replacement of the gusset plate may be warranted. This repair option should be considered a last resort due to the substantial amount of temporary shoring and bracing work that is required to bypass the gusset plate prior to disassembly, particularly if the bridge will remain open during the work. Less work is likely necessary if the bridge is undergoing a rehabilitation project that includes removal of the deck. Examples of temporary supplemental members to remove the load from the existing truss chord are shown in Figures 3-33 and 3-34. Alternately, shoring may be used to remove the load from the existing gusset plates prior to disassembly, as shown in Figure 3-35. In order to prevent differential movement of the connecting members, the gusset plates should be replaced one at a time. The connection geometry is maintained by keeping one gusset plate connected at all times.

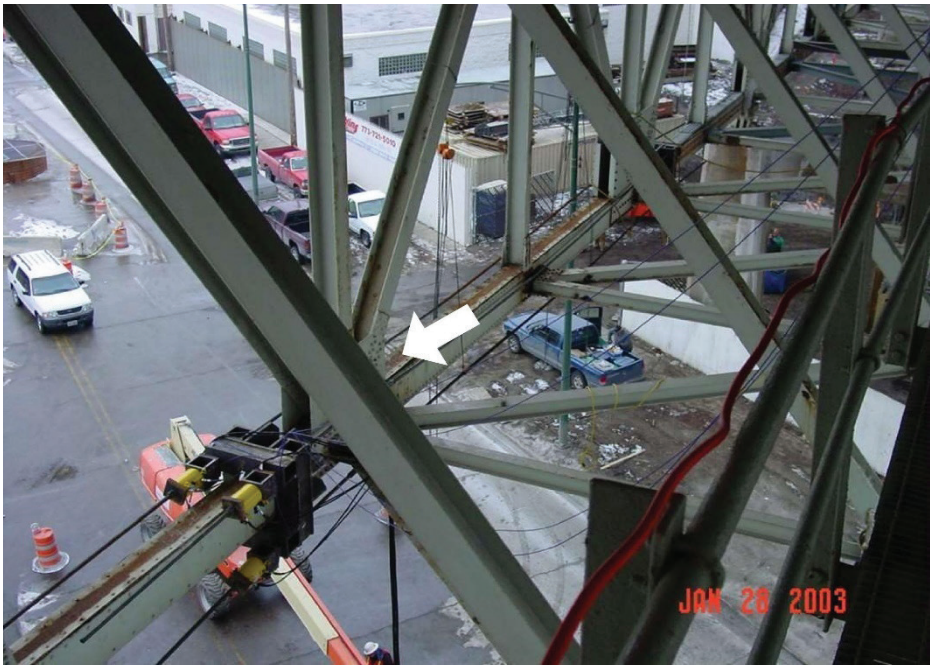


**Figure 3-33—Supplemental tie rods to remove the existing load in the truss tension chord (arrow indicates gusset plate being repaired)**

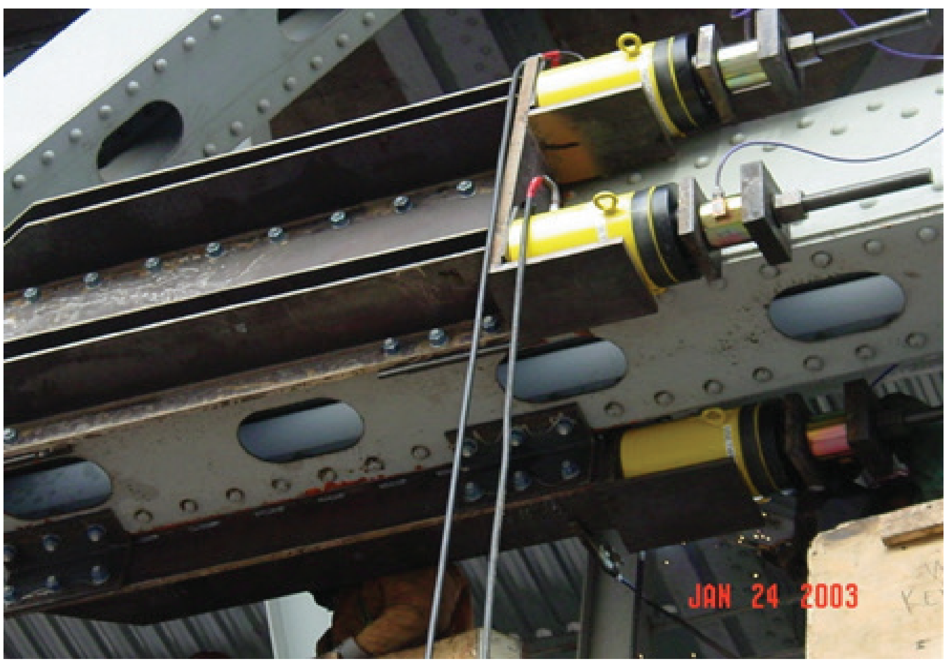


**Figure 3-34—Close-up of the load transfer bracket for the supplemental tie rods shown in Figure 3-33**

**Figure 3-35—Example of supplemental truss shoring to perform a gusset plate replacement; circles indicate bolted connections for temporary truss support on the shoring tower header beam**

### 3.3.2—Repair of Girder End Damage

Section loss at the girder ends or at supports directly below expansion joints is a relatively common form of corrosion damage. Expansion joints are prone to tears from roadway objects and debris and typically have a service life between five and fifteen years. Failure of the expansion joints exposes girders to higher time of wetness in combination with deicing salts carried by runoff from the roadway surfaces above. This form of damage can result in section loss at the web and flanges, bearing stiffener section loss, sharpened plate edges, and irregularly shaped holes, and can lead to decreased bearing and shear capacity of the girder ends.

Articles 3.3.2.1 through 3.3.2.3 discuss several options for repair of girder end section loss. However, any leaking expansion joints must also be addressed with each of these or the potential for corrosion damage will continue, reducing the repair service life. The extent of the repair will vary from case to case, requiring a somewhat customized application for each repair, considering how much of the damaged girder needs to be repaired.

#### 3.3.2.1—Welded Partial Replacement

The welded partial section replacement repair requires field welding. These repairs have been successfully performed many times, and several Owners use welded partial section replacement as a standard repair approach. However, some Owners prohibit field welding, in which case this alternative will not be possible.

The advantage to the welded partial section replacement is the complete removal and replacement of the damaged section, restoring the girder ends to their original capacity without changing the original appearance. Before this repair is carried out, the Owner should confirm that the existing girder is made from a steel that is favorable to welding. In some cases, if the type of steel is not known for historical structures, minor material sampling and chemical analysis may be required before welding in order to establish an acceptable WPS (see Article 2.4.2.8.1 for further detail). In this scenario, it is important that the material sampling for analysis is representative of the intended repair section.

Items required for this repair include hydraulic lifting equipment, replacement materials, and welding equipment. The general procedure is as follows:

1. Close all or a portion of the bridge to traffic to allow for hydraulic jacking.
2. Remove diaphragms, cross-frames, connection plates, or bearing stiffeners, as required, to access and remove the damaged area. Depending on the condition, these elements may be reinstalled following repair.
3. Relieve the load at the bearing through hydraulic jacking from below (jacks on columns) or above (transfer beam above deck with cables). Take care to limit differential jacking between adjacent girders to avoid damage to the deck and any remaining diaphragms or cross-frames. Use blocking to distribute jacking loads, if necessary. If the extent of the corrosion damage is such that the jacks cannot be placed under sound portions of the girders and adequately support the corroded area, overhead support or temporary shoring of the corroded area may be required.
4. Mark and cut out the corrosion-damaged area using rounded corners of 3- to 4-in. diameter or greater at direction changes, as illustrated by the example detail in Figure 3-36.

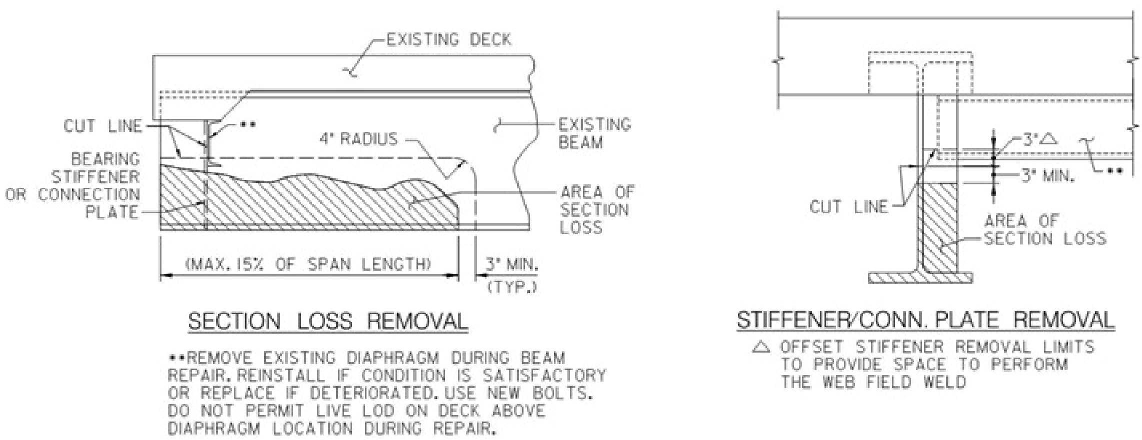


**Figure 3-36—Illustration of section loss removal**

5. Mechanically clean rust, scale, or existing paint at least 3 in. beyond the repair area.
6. Prepare the cut edge of the remaining web or flange plates for the CJP weld.
7. Install the cut-to-fit replacement section. (It is recommended that the replacement section have beveled edges prepared for field welding.) Fully weld along the top and sides of the web plate using CJP groove welds. If the new section is not cut from a rolled beam, then use fillet welds to connect the new web plate to the new flange plate prior to installing the new section, as shown in the example detail illustrated in Figure 3-37. All welding procedures should be in accordance with governing Owner standards and specifications. Weld sizes may vary based on applicable design.

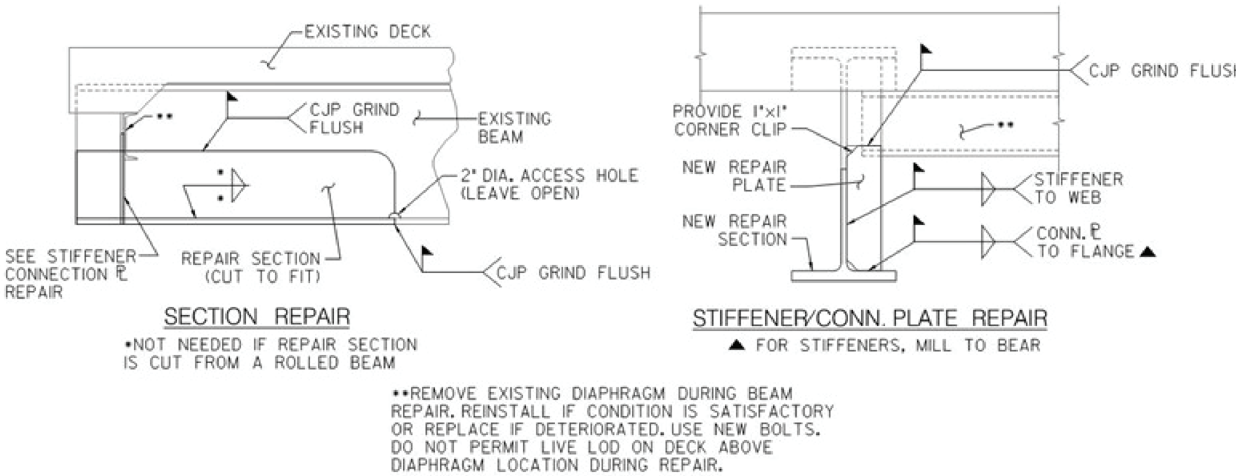


**Figure 3-37—Example illustration of section repair**

8. Once the welding is completed, perform appropriate welding inspection and nondestructive testing, as required.
9. Grind the weld flush, taking care to not reduce web or flange plate thickness. Grind smooth any gouges or impact indentations caused during the repair. Refer to AASHTO/NSBA G14.1 for guidance on grinding repairs of gouges and nicks.
10. Replace removed bearing stiffeners and connection plates with new material, ensuring finish-to-bear on flanges, and weld to bottom flange. If bearing stiffeners or connection plates were partially removed, join replacement material with PJP welds from each side. Lower member onto bearing ensuring full contact, adjusting bearing if necessary. Reinstall all removed diaphragms or cross-frames.
11. Clean the repair area, removing debris and oils. Paint to match the adjacent existing girder section, as desired.

#### 3.3.2.2—Bolted Cover Plates or Angles

Web plates, flange plates, or angles can be bolted over section loss to restore section properties without requiring removal of the corrosion-damaged section and without requiring hydraulic jacking of the bridge. These are significant advantages in themselves, but also make it possible to leave the bridge open to traffic throughout the repair operation. Items required for this repair include magnetic-base drilling equipment with annular cutter, new angle or plate materials, and bolt tightening equipment (e.g., appropriate fastener assembly wrench).

Web and bottom flange repairs are straightforward unless clearance prevents cover plates on the underside of the bottom flange, necessitating angles above the flange be bolted to the flange. For the top flange, angles and blind bolts installed through flange with one-sided access through holes in flange and into deck, angles bolted through the web, or plates welded to the flange may be considered depending on the loading and Owner restrictions.

An example of this type of repair using four rolled angles bolted onto a bearing stiffener and web plate is provided by the Maryland DOT State Highway Administration in Figure 3-38. An example of an implemented repair is shown in Figure 3-39. This same repair concept could also be accomplished using only two thicker angles, one on each side of the web plate, thereby reducing by half the number of components to be fabricated and installed. The general procedure is as follows:

1. Design and fabricate the new stiffening angles or plates.
2. Use the angles or plates as a template for the holes in the existing web and stiffener plates. Clamp the

angle or plate in place and use a transfer punch to mark the locations of the holes to be drilled, or, alternatively, the angle or plate can be left in place for use as a drilling template.

3. If using transfer punching, remove the new stiffening components and clamps. Using a magnetic drill with annular cutter and pilot pin, drill the bolt holes in the web and bearing stiffeners $^{1}/_{16}$ in. larger than the nominal diameter of the bolts, using the transfer punch indentations or the holes in the new components as guides. Secure the magnetic drill to the girder using sturdy clamps and chain or rope so that if power is lost, the drill will not fall.
4. If the new components were used as templates during drilling, remove them prior to this cleaning step. Clean and degrease all surfaces within the area of the repair where drilling was performed. Remove dirt, cutting oils, drilling shards, and other debris from the area, plus an additional 6 in. or more outside the repair area.
5. Fill the areas of section loss and pitting with approved metal-reinforced epoxy filler just prior to installing the new angles or plates, such that the filler is not allowed to harden before the new angles or plates are in place. See Article 3.2.1.3 for further discussion on use of epoxy filler.
6. Install the new angles or plates and tighten the bolts. Starting at the middle of the new angles or plates, methodically move outward through the bolts, snug-tightening and then fully tensioning the bolts according to current RCSC specifications for pretensioned connections. RCSC does not endorse the use of metal-reinforced epoxy fillers in slip-critical connections; recommended use of the RCSC specification in this case is for bolt tensioning procedures only.
7. Complete the repair by cleaning and painting according to Owner specifications.

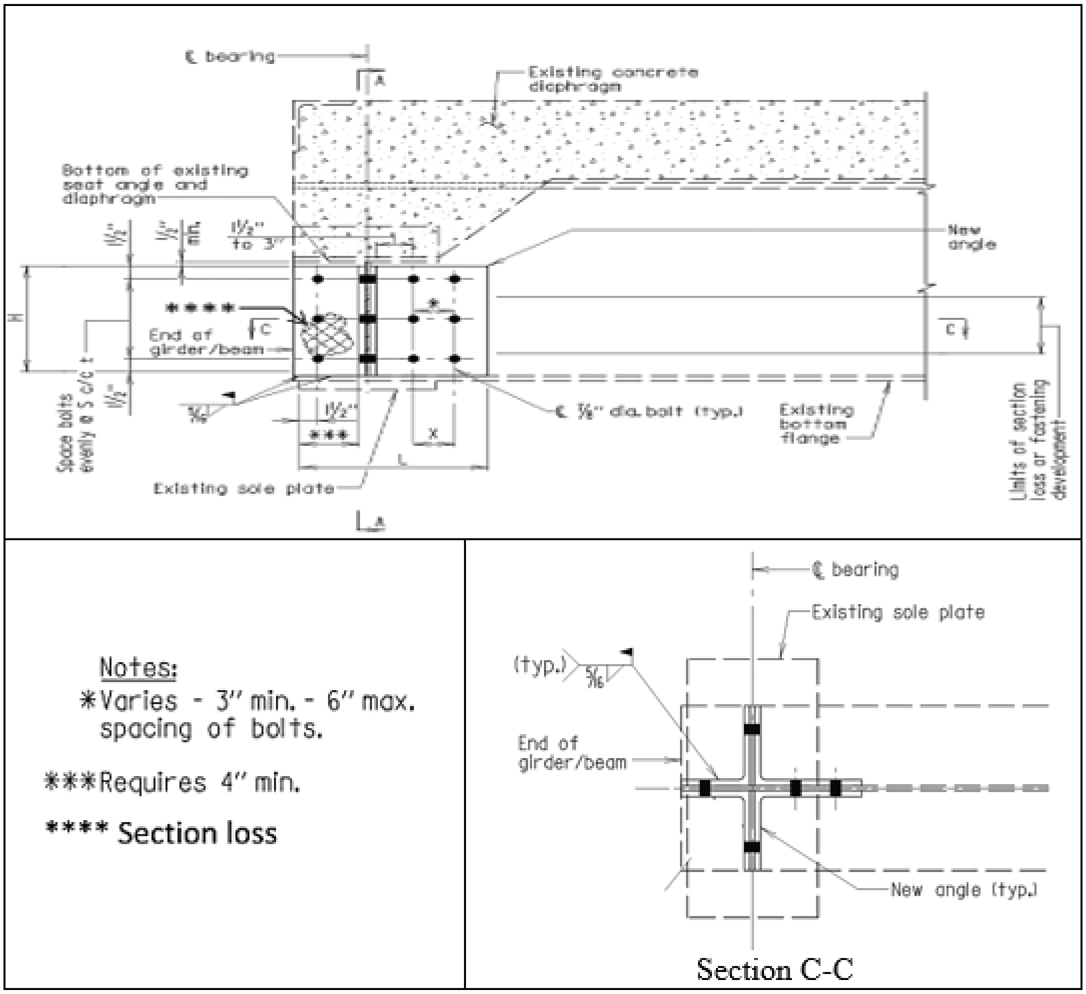


**Figure 3-38—Illustration of bearing area repair (Maryland DOT State Highway Administration Detail No. SR-ST(BSR)-102)**

**(a) Beam end with through-thickness section loss** **(b) Implemented beam end repair**

**Figure 3-39—Example of a deteriorated beam end (a) repaired with web cover plate and stiffener elements (b)**

#### 3.3.2.3—Ultra High-Performance Concrete Encasement

The UHPC encasement repair detail was developed through research at the University of Connecticut for corroded steel bridge girder ends (Zaghi et al., 2017). It offers an alternative to welding or bolted cover plates and angles. There are several advantages to this method, including simplicity, minimized disruptions to traffic, and no need for girder jacking, but one drawback is that it must be completed using UHPC rather than normal-weight reinforced concrete. UHPC has higher strength and impermeability that will protect against corrosion of the girder ends so long as the encasement remains resistant to moisture intrusion. Drawbacks to the use of UHPC could include the expense of the material, inexperience of local Contractors, or the potential need for the supplier representative to be onsite to oversee the mixing operation. These factors should be considered before selecting this alternative.

Materials required for this repair include wood forms, shear studs, UHPC materials, and a deck coring drill. The general procedure is as follows:

1. Design the repair, including the required number of shear studs.
2. Grind to bright metal all locations on the girder web plate where shear studs will be welded. Studs should be welded to sound steel. For cases where welding studs is not feasible due to lack of access or a lack of sound base metal, bolting rods (threaded rods through the web with hand-tightened nuts on each side) of comparable diameter can be considered. Zaghi et al. (2017) reported that the pattern and placement of shear studs does not have a significant effect on performance of this repair. The studs should not be placed closer to each other than three stud diameters to facilitate installation, but otherwise the studs can be welded in place in any pattern or configuration that meets the minimum number of studs required by the design.
3. Weld the shear studs on (or install the bolting rods). See Figures 3-40 and 3-41 showing the shear studs welded to both sides of the web.

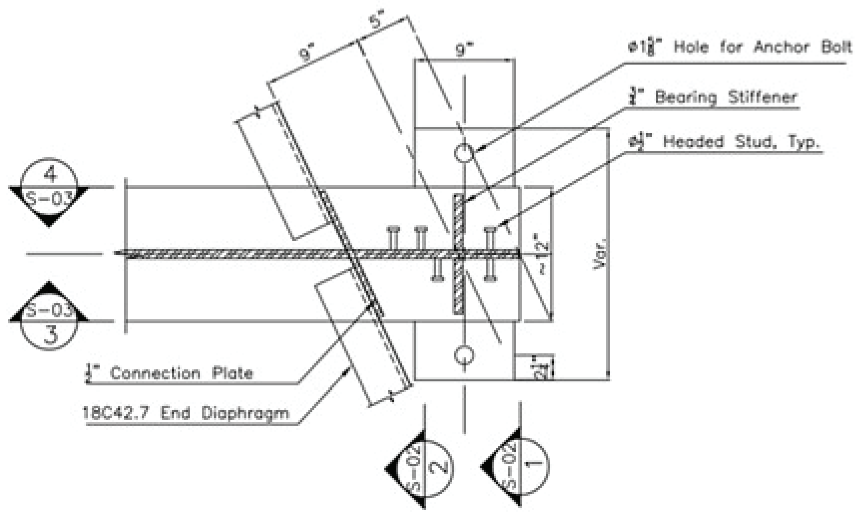


**Figure 3-40—Sample plan view of stud layout for UHPC repair (Zaghi et al., 2017)**

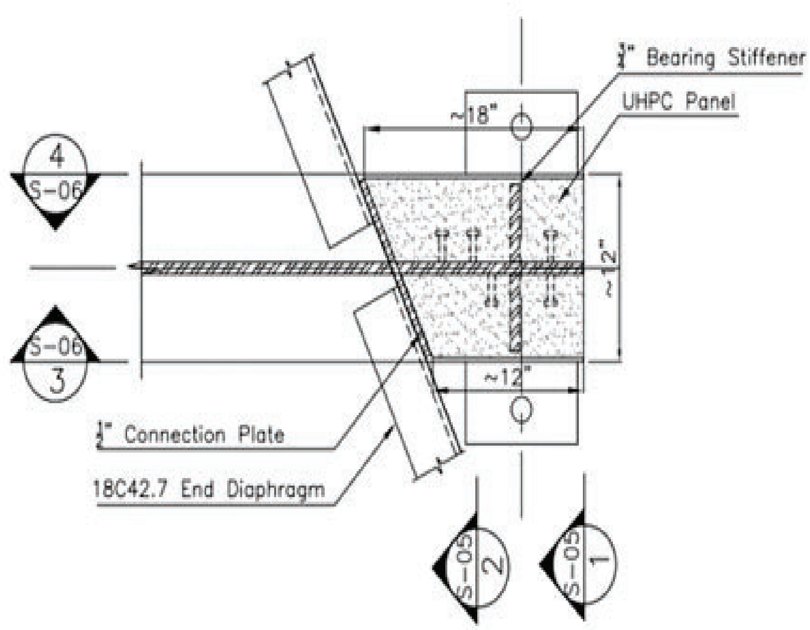


**Figure 3-41—Sample plan view of UHPC repair with concrete placed (Zaghi et al., 2017)**

### 3.3.3—Pin and Hanger Connections

Pin and hanger connections (or assemblies), or hinged joints, are often used in steel girder bridges, particularly those constructed prior to the 1970s. Pin and hanger connections simplify girder design and construction by providing a moment release and allowing rotation while providing a location to accommodate thermal movements in continuous girders. These connections are most often located below expansion joints in the bridge deck and are susceptible to corrosion-related deterioration from leaking joint seals because they are designed to accommodate longitudinal thermal movements of the bridge girders. Examples of pin and hanger connections (also known as pin and hanger assemblies) are shown in Figure 3-42. Following the failure of a pin and hanger connection that caused the collapse of the Mianus River Bridge in Greenwich, Connecticut on June 28, 1983, there was an increased interest in the inspection and evaluation of these connections. Pin and hanger details are no longer recommended for use in modern steel bridge design practices.

#### 3.3.3.1—Typical Construction

The primary components of a pin and hanger connection typically consist of two vertically oriented steel plates, or hangers, with pins at the top and bottom of the plate, similar to the example shown in Figure 3-43. Other components of a pin and hanger connection may include spacer washers, pin nuts, pin caps, through bolts, and cotter pins. To accommodate the large bearing forces imposed by the pins, plate girder webs often have increased thickness and rolled beam webs are thickened using bolted or welded doubler plates. Newer pin and hanger connections may also include a bushing between the pin and hanger to reduce the friction between the components and prevent seizure due to corrosion products. Grease tubes or grooves may also be present on the pin to allow for the components to be lubricated during service.

(a) Pin and hanger with exposed pins

(b) Pin and hanger with pin caps to cover pins

**Figure 3-42—Typical examples of pin and hanger connections**

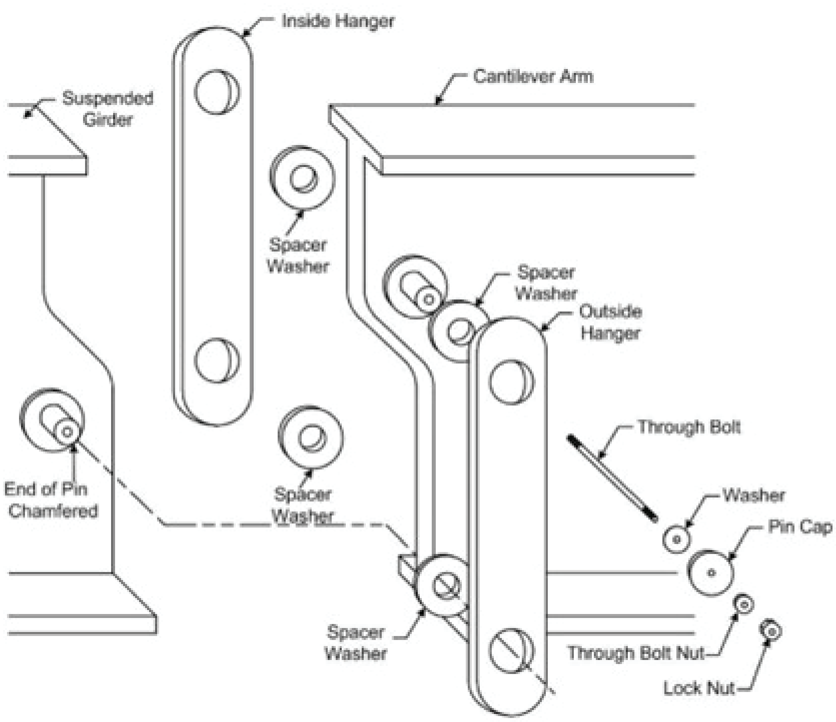


**Figure 3-43—Typical construction of a pin and hanger connection (Ryan et al., 2012)**

Pin and hanger connections, such as those shown in Figures 3-42 and 3-43, create an expansion joint that allows longitudinal expansion movement and rotation while still transferring girder shear across the joint.

Pin and hanger connections may also be used at bearings that are subjected to uplift (see Figure 3-44a). If accommodating thermal movements is not required at a connection, and only joint rotation is needed, pin-connected members can be used in truss members (see Figure 3-44b) and bridge girders (see Figure 3-44d). Pin and eyebar members are also used in truss tension members, such as the example shown in Figure 3-44c.

**(a) Tie-down bearing**

**(b) Pin-connected truss members**

**(c) Pin and eyebar truss chords**

(d) Hinge at a multi-girder superstructure

**Figure 3-44—Other examples of pin-connected members in bridges**

### 3.3.3.2—Inspection

Pin and hanger connections are rarely included in new bridge construction; however, these connections can be quite common on older bridges that remain in service. In addition, these connections may be found on NSTMs or other members with unknown redundancy. Therefore, careful inspection of these assemblies is important for identifying conditions that may affect the performance of the pinned connection. In general, inspection of pinned connections can be quite difficult and often requires special equipment or rope access to get within arm's length of the components. Also, while corrosion may be assessed visually, many pin defects, such as wear grooves and fatigue cracks, are not visible because they are located at the interface between the pin and hanger. As a result, UT is a common method for performing detailed inspection of in-service pins. Additional inspection methods and information related to the inspection of pin and hanger assemblies can be found in the *Bridge Inspector's Reference Manual* (Ryan et al., 2012) and *Guidelines for Ultrasonic Inspection of Hanger Pins* (Moore, 2004).

Visual inspection of pin and hanger assemblies can reveal conditions such as coating failures, surface corrosion, and pack rust. Failed expansion joints often lead to salt-laden water dripping on the pin and hanger connections. Corrosion of these assemblies, such as that shown in Figure 3-45, can lead to section loss on the hangers, girder webs, and pins. Corrosion can also lead to a lack of lubrication and increased friction on the outer surface of the pin. Even moderate pin corrosion can cause the pin to "freeze" and restrict the ability of the pin to rotate relative to the hanger plates. Frozen pins can generate significant torsional stresses in the pin and hanger components. Similarly, pack rust between the hanger plates (see Figure 3-46) can cause prying of the plates and tension stress in the pin. Generally, the hanger plates are designed for tension and the pins are designed for shear. Unintended torsion and tension in the pin can lead to distress and possibly failure. Fixed or partially fixed pin connections may also result in performance issues in other areas of the bridge. Other conditions that may be identified from a visual inspection include broken spacer washers (see Figure 3-47), excessive expansion or contraction movement, and thermal movement that does not match the ambient temperature, such as expansion movement in cold weather.

**Figure 3-45—Severe corrosion of a pin and hanger connection that may create unintended torsional stress on the pin**

**Figure 3-46—Pack rust between the pin plates (arrows) that creates unintended tensile stress in the pin**

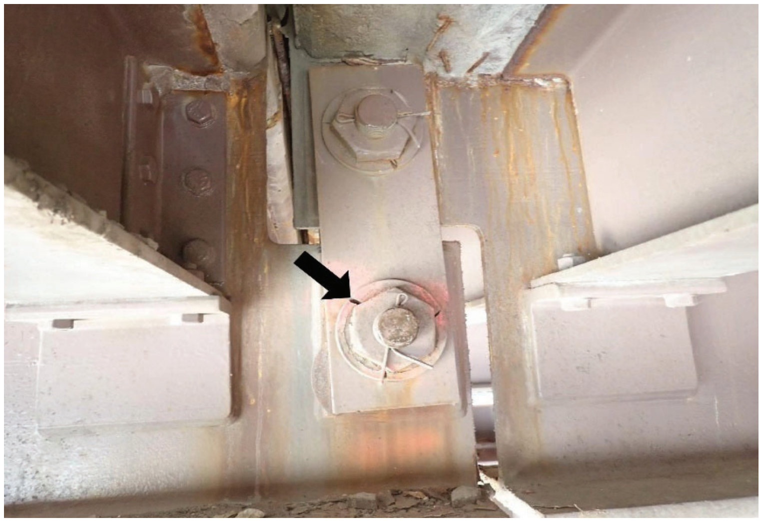

**Figure 3-47—Broken spacer washers (arrow) in a pin and hanger connection**

#### 3.3.3.3—Repair and Retrofit of Pin and Hanger Connections

Pin and hanger deterioration must be addressed to maintain performance of the assembly and safety of the structure. Most minor deterioration can be addressed by routine maintenance such as cleaning the deck drains and joints, replacement of leaking or failed expansion joints, sealing cracks and repairing deck deterioration above the pin and hanger connections, and cleaning and painting of the assemblies.

Pack rust between pin plates (see Figure 3-46) can cause tensile stress in the pin. It may be possible to eliminate the tensile stress by mechanically removing the pack rust, loosening the nuts on the pin, or a combination of these methods. Pack rust at a girder hinge connection, such as the one shown in Figure 3-44d, may also cause prying that can crack the connecting welds (see Figure 3-48). In this case the crack tip should be ground out to prevent crack propagation. If the crack did not propagate into base metal, the weld may be re-welded to restore the strength of the connection. If the fracture extends into base metal, the connection must be evaluated to determine the extent of repairs needed. Bolts may be installed in the pin plate to transfer the loads imposed by the pin. The supplemental bolts have the added benefit of clamping the pin plates together in order to prevent future pack rust between the plates. An example of the bolted retrofit for a girder hinge plate is shown in Figure 3-49. In either case, the underlying cause of pack rust build-up (usually a leaking joint) should be evaluated and addressed.


**(a) Overall view; arrow shows the view of the close-up photo**

**(b) Close-up view of the cracked fillet weld (arrow)**

**Figure 3-48—Cracked fillet weld at a girder hinge connection due to pack rust between the pin plates**

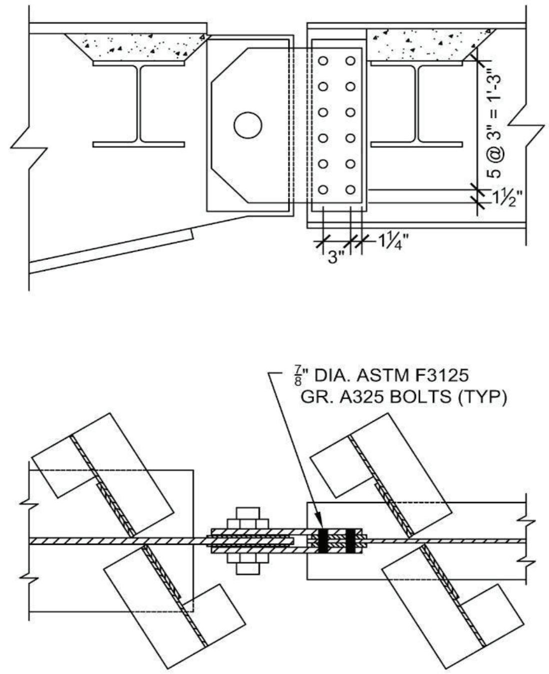


**Figure 3-49—Bolted retrofit to address pack rust between pin plates of a girder hinge connection, similar to the condition shown in Figure 3-48**

More advanced corrosion-related deterioration on the girder web may require plate strengthening repairs, such as those shown in Figure 3-50. Because the girder web adjacent to a pin and hanger connection is often congested with doubler plates, stiffeners, and diaphragms, these plate strengthening repairs may not always be feasible. In addition, multiple layers of plates may be necessary to properly develop the strengthening plate into the undeteriorated girder web. Corrosion-related deterioration on the hanger plates and pins is not easily repaired and may require one of the retrofit options described in the paragraphs following, depending on the severity of the section loss.

**Figure 3-50—Plate strengthening repairs to address section loss of a girder web and flange adjacent to a pin and hanger connection**

Advanced corrosion of the assembly, or section loss on the hanger plates or pin, may require more complex retrofit options, such as replacement of the pin and hanger components. Some Owners encourage replacing the pin and hanger assemblies with a different structural system, if possible, in order to improve the inspectability, maintenance, and long-term serviceability of the connection. One option is to install an underslung beam, or "catcher" beam, which would act as a support and provide redundancy in the event that the pin and hanger connection fails. Examples of a catcher beam system are shown in Figures 3-51 and 3-52. The catcher beam will significantly reduce the clearance below the structure, so it may not be possible when the bridge is located over other roadways or navigable waterways. The catcher beam retrofit may be installed as a permanent load path or in situations where an emergency repair is required to address a critical condition. Because the load path may change, checking the design of the existing girders, floor beams, and stringers is necessary to evaluate the adequacy of these members.

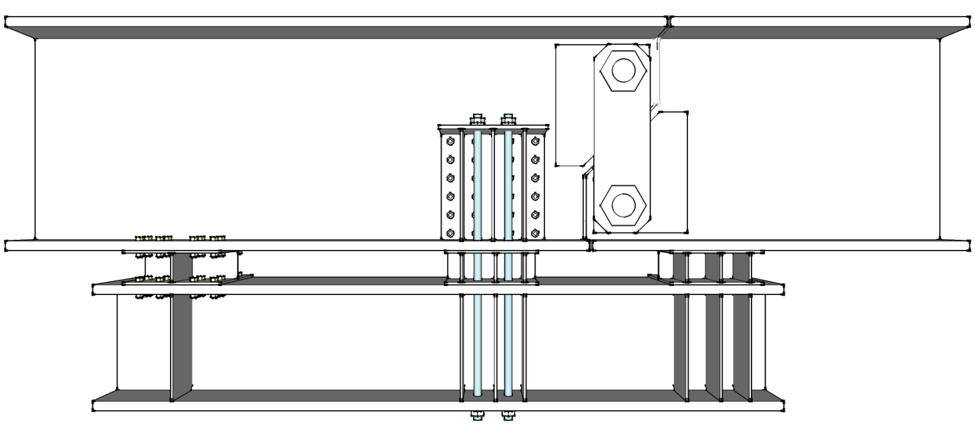

**Figure 3-51—Example of catcher system details at a pin and hanger connection**

(a) Catcher beam for a bridge girder

(b) Tie rod catcher system for a truss

**Figure 3-52—Example catcher systems installed at various pin and hanger connections**

Another option is to remove the pin and hanger connection and replace it with a segment of girder spliced to each side, or with a new splice connection at the location of an existing pin and hanger connection (see Figure 3-53). This option, often done in conjunction with a deck replacement, requires a detailed analysis to confirm that the continuous girder superstructure, bearings, and substructure can accommodate thermal and volume change movements without damaging other portions of the structure. In addition, splicing in a girder segment creates a continuous girder with different moment and shear forces than considered in the original design, so the design of the existing girders on each side of the joint must be checked for these forces. If girder continuity cannot be accommodated, replacing the pin and hanger connection with a seated beam connection is another option (see Figure 3-54). The seated beam connection does not affect the design forces for the girders and it is easier to identify deterioration and distress during an inspection; however, this retrofit approach involves rebuilding the girders at the joint and can be more costly relative to other retrofit options. Expansion bearings may be used in seated beam retrofits to permit thermal movement.

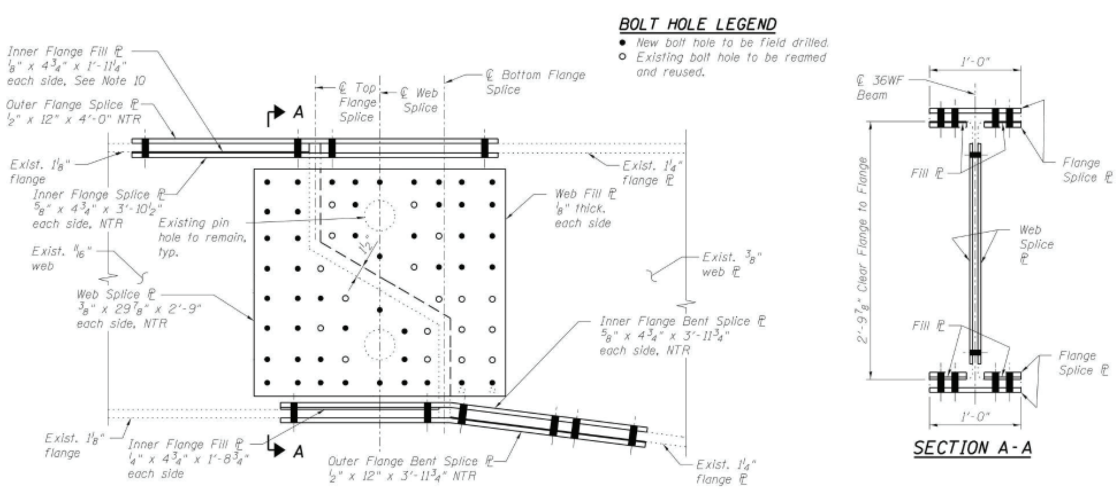


**Figure 3-53—Example of a pin and hanger assembly made continuous with girder splice plates**

**Figure 3-54—Example of a pin and hanger assembly replaced with seated beam connection (Linzell et al., 2017)**

Replacement of the pin and hanger assemblies may be the most appropriate repair option given cost, access, and other constraints imposed by the bridge design and geometry. In this case, replacement of the components with different materials to improve long-term performance should be considered. These materials may include link bars and stainless steel pins with high fracture toughness at low temperatures, polytetrafluoroethylene (PTFE) bushings, or other more durable components that will be less susceptible to corrosion or fracture over time. Wear grooves in the pin or misalignment of the connection will increase the difficulty of pin and hanger replacement. Line boring is a method used to field drill holes in the girder webs and pin plates after the pin has been removed (see Figure 3-55). Line boring is used to remove edge wear on the plates and make misaligned holes round and concentric; however, line boring increases the diameter of the existing holes in the connecting plates and the net section capacity of the connecting elements will need to be checked. An example of pin replacement using standard components and PTFE bushings is shown in Figure 3-56. To facilitate future UT, double pin nuts are used instead of single nuts with cotter pins and the full pin diameter is threaded instead of stepped-down diameter pins with recessed pin nuts.

Retrofits or replacement of pin and hanger assemblies can be quite complex. Any disassembly of the connection for repair or retrofit will require a temporary support system to provide an alternate load path during repair. The stability of the disassembled structure must also be considered. Due to the wide range of details associated with pin and hanger assemblies, repair guidance is available in more detail in other sources. Additional information related to repair, retrofit, catcher systems, and replacement of pin and hanger connections can be found in Connor et al. (2005), Gregg et al. (2015), IDOT (2017), Linzell et al. (2017), and South et al. (1992).

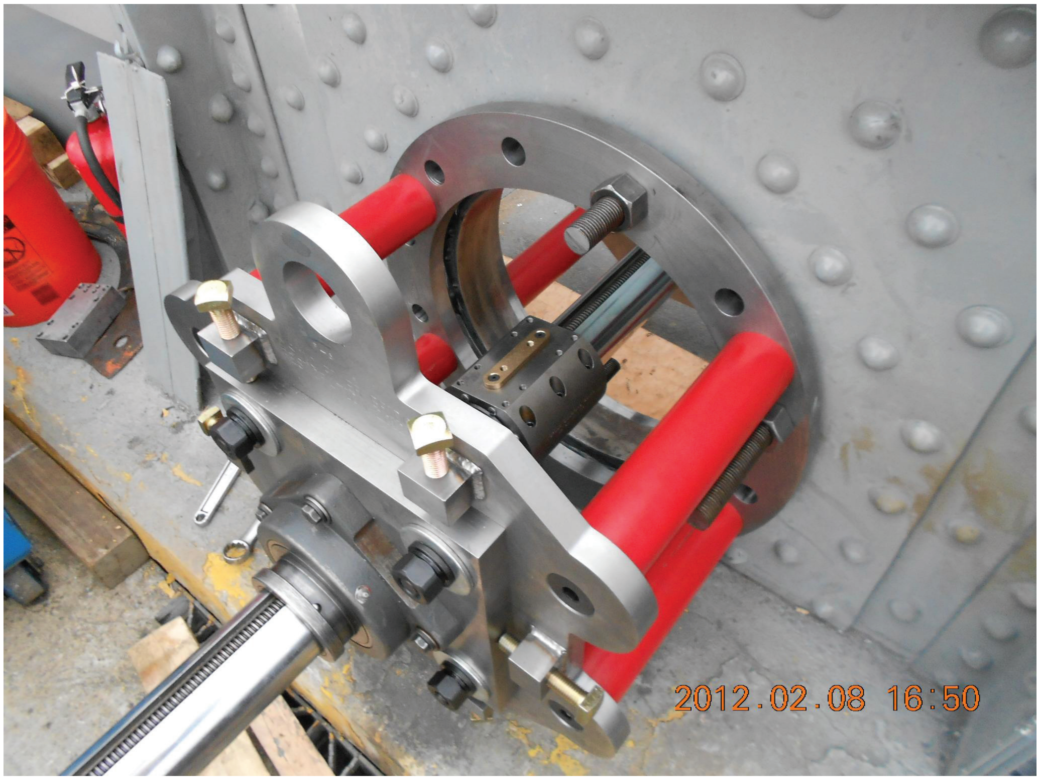


**Figure 3-55—Line boring to facilitate pin replacement**

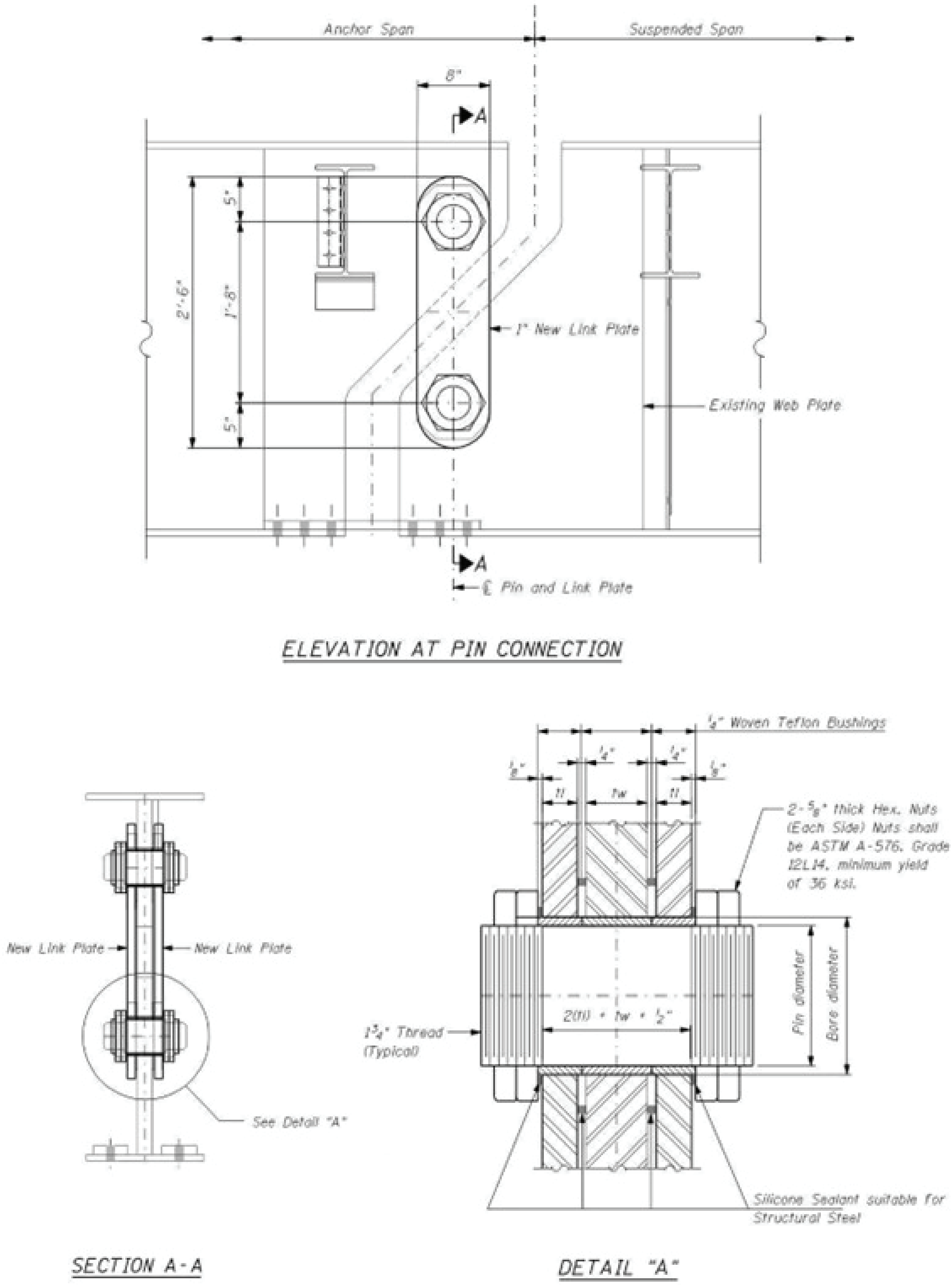


**Figure 3-56—Example of pin and hanger replacement details**

# SECTION 4: STRENGTHENING

Strengthening is a generalized term used to describe an intervention to increase or restore the load capacity of an element. This Section will focus on the detailing, installation, and access requirements to address strengthening needs related to changes in loads, rather than changes in capacity due to section loss, which are addressed in Section 3.

## 4.1—OVERVIEW

Strengthening may be necessary as a result of increases in:

- Dead loads (new barriers, additional overlay thickness, etc.)
- Live loads (new permit vehicles, additional traffic added to bridge, etc.)

Prior to planning strengthening work, it is important that the Owner establish the goal of the work for the entire structure. An understanding of the scope of the project will be useful in determining the design criteria for any strengthening needs. To illustrate this, consider the following brief summaries of two example project scopes and objectives:

1. A bridge is scheduled for a preservation project to extend the useful service life of the deck. The expected service life extension of this work is 15 to 20 years. The scope of the work does not result in any increase in dead load. If the bridge was designed using Allowable Stress Design (ASD) and the routine safety inspection indicates no signs of distress, the Owner may elect to not perform an updated load rating on the bridge. As such, no strengthening would be required for this project.
2. A bridge is scheduled for a rehabilitation project to replace the deck. The expected service life extension of this work is 30 to 40 years. The bridge, originally designed using ASD, was subject to an updated Load and Resistance Factor Rating (LRFR). The rating indicated that permit loads would be restricted without strengthening. As such, strengthening will be included to allow for travel by permit vehicles upon completion of the project.

With any strengthening endeavor, the cost of the work should be considered compared to the total service life goal of the bridge. An Owner may elect to manage a bridge through the use of permit restrictions in lieu of expensive strengthening options if the bridge is nearing the end of useful service life. Alternatively, the cost of strengthening an individual bridge may be minor compared to any required geometric or roadway improvements to facilitate new bridge construction.

## 4.2—OPTIONS TO ADDRESS STRENGTHENING

There are a variety of techniques available to address strengthening. Common repairs to beams which are addressed in this Section include:

- Addition of stiffener angles to webs for shear capacity
- Addition of diaphragms to increase lateral-torsional buckling (LTB) capacity
- Addition of plating to flanges for moment capacity
- Addition of shear connectors for composite action
- Bolt or rivet replacement

When developing alternatives for strengthening, it is important to evaluate alternatives against known site or project constraints. These may include:

- Distance above ground or water
- Weight of repair elements
- Lifting equipment and access requirements
- Installation equipment and access requirements
- Confined space restrictions
- Traffic on and under structure
- Surface preparation requirements
- Material procurement and availability
- Deterioration in adjacent elements
- Project type (preservation, rehabilitation, emergency, etc.)
- Sequence of load application (during and after work)
- Regional availability of skilled labor

## 4.3—DETAILING AND SURFACE PREPARATION

When planning any repairs, a thorough review of existing documents is necessary to produce a constructible solution. These documents may include:

- Existing design plans and provisions
- Construction as-built plans
- Shop drawings
- Requests for information or design change requests from previous projects
- Construction inspection notes or photos

Ideally, the Designer will be able to visit the bridge to gain a better understanding of access requirements and be able to translate plan details to as-constructed observations.

For any attachment, whether bolted or welded, the following items should be considered:

- To determine the correct length, field verification is the most accurate method. In absence of this, the as-built girder drawings should be referenced in order to account for any changes during fabrication beyond tolerances.
- For attachments that require finish-to-bear fit between existing elements, specify permissible field modifications in the special provisions or plan sheets to achieve this condition. Unless the procedure is detailed within the Contract, a submittal by the Contractor should be required for review by the Owner. Permissible methods may include:
    - Specifying as finished-to-bear at load transfer points and tight-fit at opposite ends to allow minor length variations in the field.
    - The use of jacks to slightly push the flanges apart to allow insertion of a stiffener may be permitted. If permitted, limits related to distortion or jacking forces should be specified.
    - Detail lengths slightly longer than required to allow for field modifications. The additional length can be based on code tolerances for fabrication.

- Any necessary modifications to existing elements that are in conflict should be detailed in the plans. For example, a longitudinal web stiffener may be in conflict with a proposed transverse stiffener.
- Elements that will be in contact with both the web and flanges must be clipped to account for the rolled fillet (*k* region, i.e., the area encompassed by the *k* dimension in the AISC section properties tables) or the fillet weld between the web and the flange. Refer to Figure 4-1 for a typical detail. Refer to AASHTO BDS Article 6.10.11 for how to determine the appropriate clip size.
- Attachments may use bolted connections. The bolt hole size should be determined by the installation procedure.
    - Pre-drilled oversized holes should be used in the new attachment when holes are already formed in the existing element. Special consideration should be given to hole size if slip-critical connections are required for load transfer. Location of existing holes should be field-verified.
    - Pre-drilled standard holes should be used when the new attachment will be used as a template for field drilling in existing elements.
    - The existing holes can be used as a template to form holes in the new attachment. In this case, the holes will match the size measured in the existing element. This might require several drilling sequences in multiple plates.

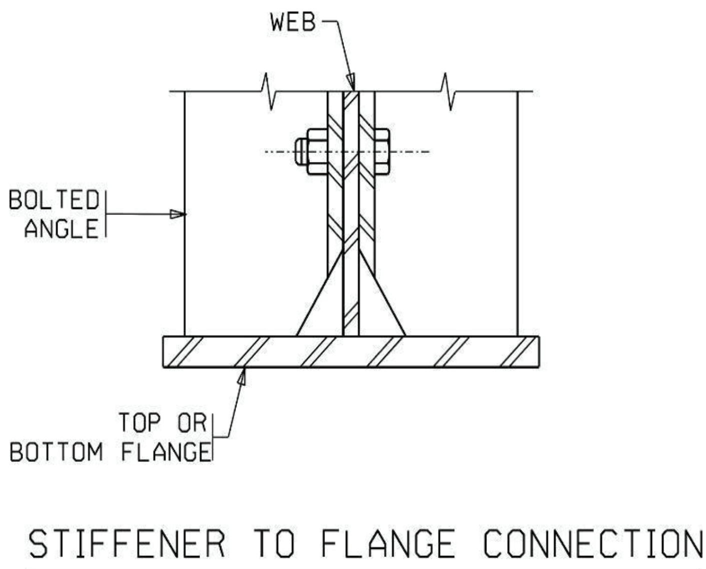


**Figure 4-1—Clipped end detail to accommodate transition fillet or weld (Minnesota DOT)**

Surface preparation is important to provide a durable solution. The following items should be specified in the Contract provisions:

- Remove loose scale and rust from existing girders at contact surfaces.
- Remove existing paint systems when:
    - In poor condition.
    - Attachment is welded to existing element.
    - Bolted attachment requires a slip-critical connection.
- Consider allowing existing paint systems to remain when:
    - Quick response/emergency work is required.
    - The intended service life of the repair is limited.

  - The connection does not need to be slip-critical.
- When painting is required:
  - Clean contact surfaces to the required surface preparation standard. Typically, this would be the level that is required by the coating manufacturer for the required performance. For example, if SSPC-SP 10 is generally required before shop priming new steel, it may not be attainable on existing steel, while SSPC-SP 6 may suffice per the Manufacturer's product data sheet.
  - Apply a primer to this surface as soon as possible to avoid contamination or significant corrosion. The primer should meet the manufacturer's curing recommendations on the faying surfaces prior to making any connections.
  - The new attachments should be shop-primed with a system that is compatible with that which will be used in the field. At a minimum, the faying surfaces on the existing element and new attachment must be primed prior to installation.
- Existing element surfaces may be irregular, resulting in non-uniform contact with new attachments. Consider specifying:
  - Filler paste for designs that require full contact between elements. (With any filler paste, the impact on the slip resistance of the faying surface must be considered if the design requires a slip-critical connection.)
  - Caulk around bolted attachments to seal against moisture intrusion.

The lists provided in this Section are not intended to be all-inclusive. Designers must consider the project specifics to identify other possible constraints. Further, it is important to consider the level of detail presented in the plan and prescription of means and methods. Generally, the Contractor takes on less risk with more detail included, which may lead to lower costs. However, the Owner is responsible for the cost of deviations encountered in the field. The Owner should give consideration to what elements should be field-verified by the Contractor or field-verified by the Owner and included in the Contract plans.

## 4.4—INCREASING SHEAR CAPACITY OF PLATE GIRDERS

Shear capacity issues can be remedied by adding stiffening elements. Dimensioning of these pieces is primarily based on the controlling resistance (thickness and projecting width). For bolted attachments, the available contact surface and bolt installation access must be considered as well. The Designer must review any as-built and shop drawings to determine if the additional stiffeners will fit between existing elements and that there is sufficient space for installation. Locations for additional stiffeners are determined in accordance with AASHTO BDS.

It is important to properly characterize controlling capacity factors and identify other structural elements in close proximity when detailing the retrofit. While low shear capacity can occur anywhere within a span, it is most commonly an issue near supports. The calculated resistance may be influenced by:

- Material strength
- Girder section properties (web thickness, web height, etc.)
- Stiffener element sizes (spacing, thickness, projecting width, etc.)

### 4.4.1—Bolted Angles

Angles are readily available and are produced in a range of sizes. Design can be completed in accordance with AASHTO BDS; Designers should be aware of the differences between bolted and welded attachments for this application.

The Designer must consider all design code requirements, including:

- Bolt spacing to satisfy sealing requirements
- Fatigue
- Net section changes (particularly if drilling into the flanges)

Fit and attachment of the angles with respect to the flanges must be specified in the plans. In general, for stiffeners not used as bracing connection plates, the following is required:

- For straight girders:
  - Compression flange: tight-fit or attached to flange
  - Tension flange: need not be in contact with flange
- For curved girders or adjacent to bearing stiffeners:
  - Single-sided stiffeners must be rigidly attached to both flanges.
  - Pairs of stiffeners must be rigidly attached to both flanges or tight-fit against both flanges.

Figure 4-2 shows an example where rigid attachment is not required at the flanges. Rigid attachment of the projecting angle leg to the flanges may be detailed as a bolted WT shape or welded plates between the end of the projecting angle leg and the flange, as shown in Figure 4-3.

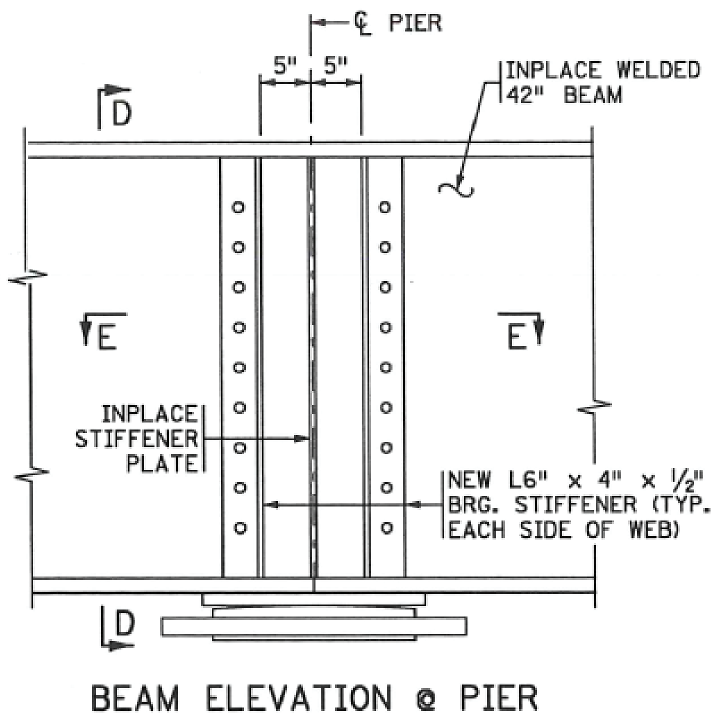


**Figure 4-2—Elevation view of supplemental stiffeners added near interior bearing (Minnesota DOT)**

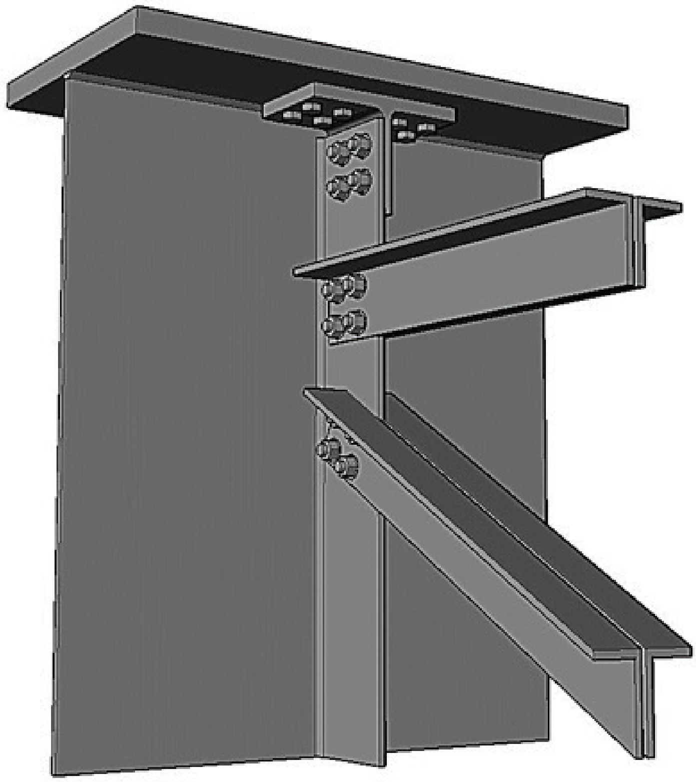

**Figure 4-3—Completed bolted splice retrofit showing WT stiffener attachment element (AASHTO/NSBA G14.1)**

#### 4.4.2—Welded Stiffener Plates

As an alternative to bolting, stiffeners can be welded at specified locations using fillet welds in most cases where the existing materials permit (see Article 1.3). This process is similar to shop fabrication. Contract documents may include the following:

- Location of new stiffener with fit requirements (see similar requirements in Article 4.4.1)
- Weld sizes and lengths
- Control of flange distortion
- Welder qualifications specific to material characteristics and welding position
- NDE requirements and qualifications for performing this work
- Contractor access

Depending on the complexity of the repair, it may be prudent to consider requiring a mockup, as discussed in Article 2.1.8.

#### 4.4.3—Other Options

Shear capacity can also be increased using channel sections or web plating. These details can be developed as bolted or welded. As with any repair, attention to conflicts with existing bridge elements and fit-up must be given in order to complete a successful repair. Additionally, these elements tend to be heavier than the previously discussed options. Consider field access, installation, and handling during design.

## 4.5—ADDITION OF BRACING TO INCREASE LATERAL-TORSIONAL BUCKLING CAPACITY

LTB was not accounted for in bridges designed using ASD. When bridges are load-rated using the modern provisions, existing diaphragm spacing often results in reduced flexural resistance. Additional diaphragms or cross-frames may be used to reduce the unbraced length of the compression flange.

The design of the bracing components must consider both strength and overall stiffness. Without sufficient stiffness, the additional bracing may not resist the twist of the girder. Design of bracing components is thoroughly addressed in the NSBA *Steel Bridge Design Handbook*, Chapter 13.

However, if the flexural resistance is determined to be insufficient in the load rating, a more robust analysis may determine that the current system has sufficient capacity. Analysis may give consideration to the construction sequence and the stability offered by the hardened deck. The critical stage for LTB typically occurs during deck placement, when the entire construction load is supported by the steel girders. During this sequence of loading, both flanges can translate laterally and twist at points away from bracing. However, once the deck is installed and hardened, the top flange may be restrained from movement. The amount of restraint will be dependent on the connection between the deck and the top flange (e.g., shear connectors, encasement). For composite sections, the deck is to be considered as a continuous support for positive flexure when the shear connectors are designed in accordance with AASHTO BDS. However, the contribution of the deck is conservatively neglected for negative flexure and translation of both flanges is assumed. Realistically, resistance for this composite section in the negative flexure region is influenced by the rigidity of the bottom flange and flexibility of the web. Additionally, AASHTO MBE Article 6.9.3 further addresses the level of restraint available based on the condition of the flange.

Because of the contribution of the deck, Owners may waive the requirements to address LTB under certain types of projects. An example of this would be where the deck will not be removed and there are no indications of overstress.

For narrow structures, such as two- or three-girder systems, global stability must also be considered to prevent all girders from buckling together regardless of the cross-frame spacing. This is particularly critical prior to placement and curing of a composite concrete deck.

### 4.5.1—Connection Plates

The connection between the new bracing element and existing beam must be sufficiently stiff. Ideally, a new bracing element can be connected to an existing transverse stiffener or flange. However, typically it is necessary to provide additional connection plates at the locations identified during analysis.

For connection plates that extend the full height of the web, considerations are similar to those discussed in Article 4.4. The AASHTO BDS discusses when transverse connection plates must be attached to both flanges. Detailing of the connection plate and attachment is important to avoid distortion-induced fatigue issues. Attachment to the flanges can be made using welds or bolted elements. The Designer needs to consider the presence of the concrete deck. In cases where the top flange is encased in concrete, preventing routine bolted attachments to the flanges, the Designer may consider welded connections, use of "blind bolts" (from below, into partial-depth holes in the deck), or a softened connection detail. Softening is a technique that allows for movement of an element to be distributed over a greater distance, reducing the risk of damage due to out-of-plane displacement. In Figure 4-4, the large gap between the top of the connection plate and the top flange represents the "softened" portion of the connection. In comparison, Figure 4-5 shows the connection plate in contact with both flanges of the girder. Refer to AASHTO/NSBA G14.1, Chapter 5 for additional examples and guidance on how to detail for a softened connection.

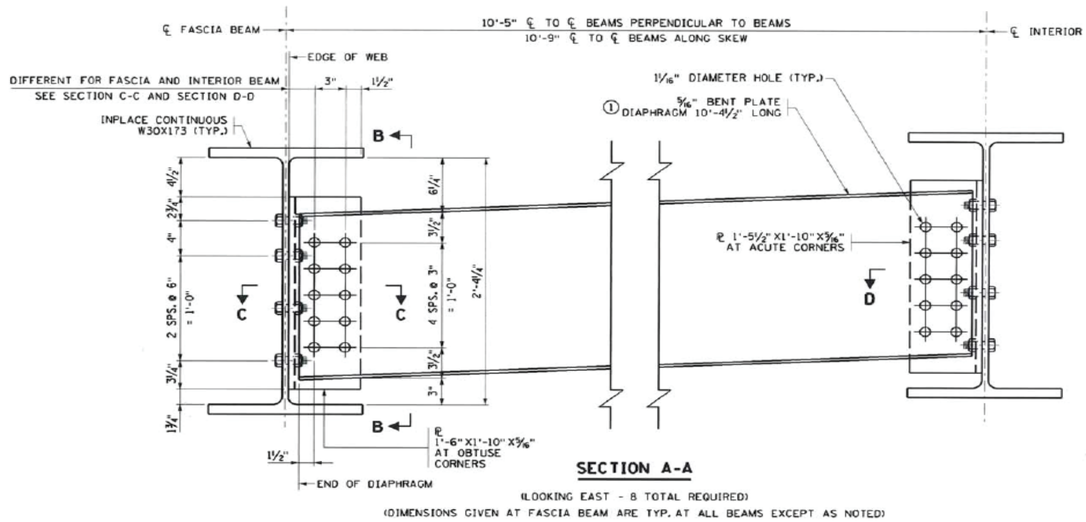


**Figure 4-4—Elevation view of new bent plate bracing element (Minnesota DOT)**

For flange bracing, detailing can be similar to that used for attachment of lateral bracing against wind. Bracing may consist of bolted double angles, WT shapes, or built-up plate shapes. It is important to consider the thickness of these elements. Refer to the section on web gap stiffening in AASHTO/NSBA G14.1.

Field installation of these elements can be completed using welding, bolting, or a combination of both. General considerations for these options are similar to those discussed in Article 4.4. Figures 4-5 and 4-6 provide an example of a bolted connection plate detail. The specific location as identified in design must be evaluated for access and installation constraints. For example, if the existing deck is to remain while the new bracing connection is installed, a welded connection to the top flange is suitable. Alternatively, localized deck removal may be required to provide a bolted connection to the top flange if a bolted connection is preferred. The type of connection detail should be considered against the goal of the repair, the anticipated remaining service life, and impacts that access creates to the initial schedule, traffic, and cost.

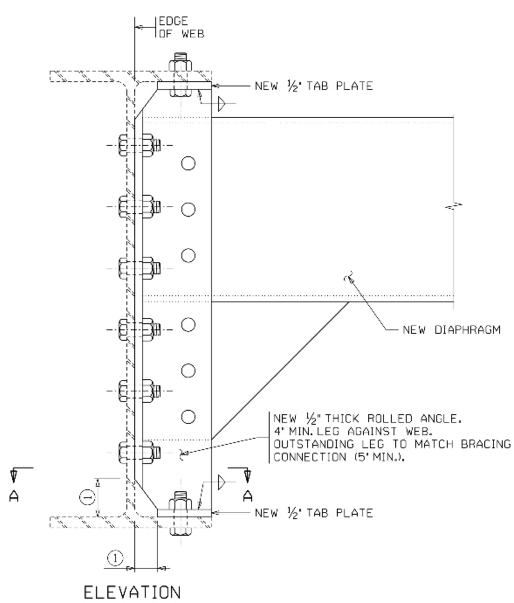


**Figure 4-5—Elevation view of new bracing connection (Minnesota DOT)**

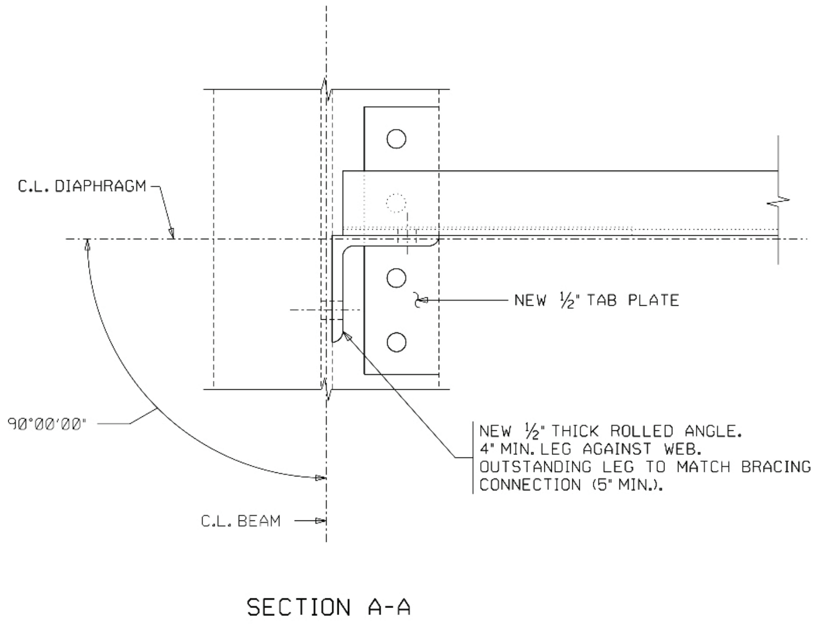


**Figure 4-6—Plan view of new bracing connection (Minnesota DOT)**

### 4.5.2—Bracing

The type of bracing selected must be compatible with the existing system. When possible, use a bracing system detail similar to what exists on the bridge. The detail must be designed to accommodate element-specific forces and be integrated into the analysis of the entire bridge. The stiffness of the brace may result in redistribution of forces along the beam, particularly near piers of bridges with skewed substructures.

Several brace types can be used in this application. Both the strength and stiffness of each bracing type must be considered to ensure they are viable to act as a brace.

1. Bent plate or rolled shape diaphragm: this system uses a beam element spanning between beams. Diaphragms can be bent plates (see Figure 4-7) or rolled sections such as channels or W-shapes.
2. Built-up cross-frames: this system consists of multiple elements to form a discrete brace point along a beam. Cross-frames use rolled shapes such as angles or WT shapes. Refer to Figure 4-8.
3. Lean-on bracing: this system uses a discrete brace, such as a cross-frame, in between one set of beams. All other adjacent beams are connected by struts, which lean into the brace. Refer to Figure 4-9.
4. Continuous bracing: this system utilizes the composite deck to provide continuous resistance.

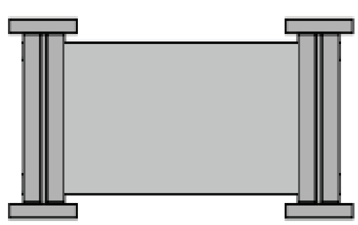

**Figure 4-7—Bent plate diaphragm (FHWA Bracing Systems)**

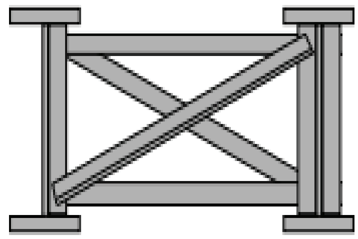

**Figure 4-8—Cross-frame bracing (FHWA Bracing Systems)**

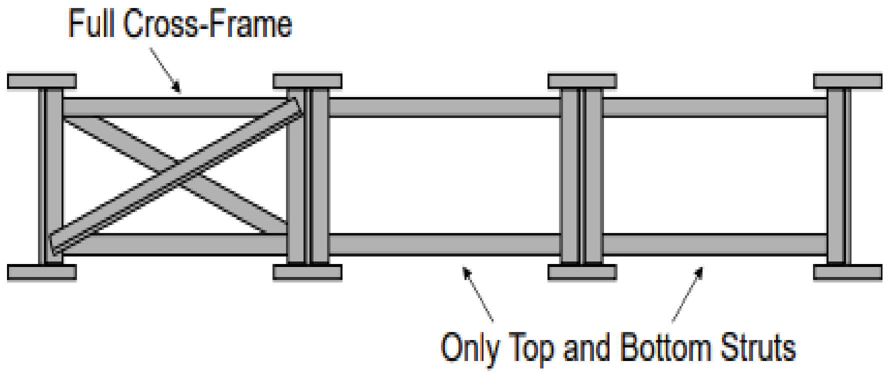


**Figure 4-9—Lean-on bracing with cross-frame (FHWA Bracing Systems)**

Selection of type needs to consider installation and access constraints. If the deck will be removed at the time of installation, it is likely the brace can be lifted into place using a crane. The size and weight of a brace is limited only by the crane capacity for the lift radius. However, if the deck will be in place during installation, handling the brace becomes an important consideration. The Contractor may have to use forklifts or come-alongs attached to the deck or other existing members or may need to assemble the brace piecewise.

Conflicts with existing elements must be considered. The following list illustrates several examples:

1. A new brace is specified in between beams that are connected with lateral bracing attached to the bottom flange. The brace must be detailed to accommodate the lateral bracing. Additionally, consider including provisions to remove and reinstall the lateral bracing if necessary for access. Refer to Figure 4-10.
2. An existing beam has wide flanges and closely spaced stiffeners. The new brace may have to be rotated to drop in between the beams. To accommodate this, consider locating the connection plates at slightly different locations along the length of each beam to allow the brace to be rotated into place.
3. An existing beam has previous impact damage that resulted in variable spacing of the beams. Heat straightening has restored the spacing to within 2 in. of the original spacing. To allow for field fit-up if bolted construction is used, the brace should be detailed with oversized or slotted holes, or field-welded to connection plates. Additional length in elements may be provided, but the Designer must consider how the brace may be cut in the field. Alternatively, the beam spaces could be measured and included in the plan, or the Contractor could be required to measure the in-situ condition and detail custom-length braces. If the Contractor is required to take measures, consider the time required for measurement, procurement of material, and fabrication.

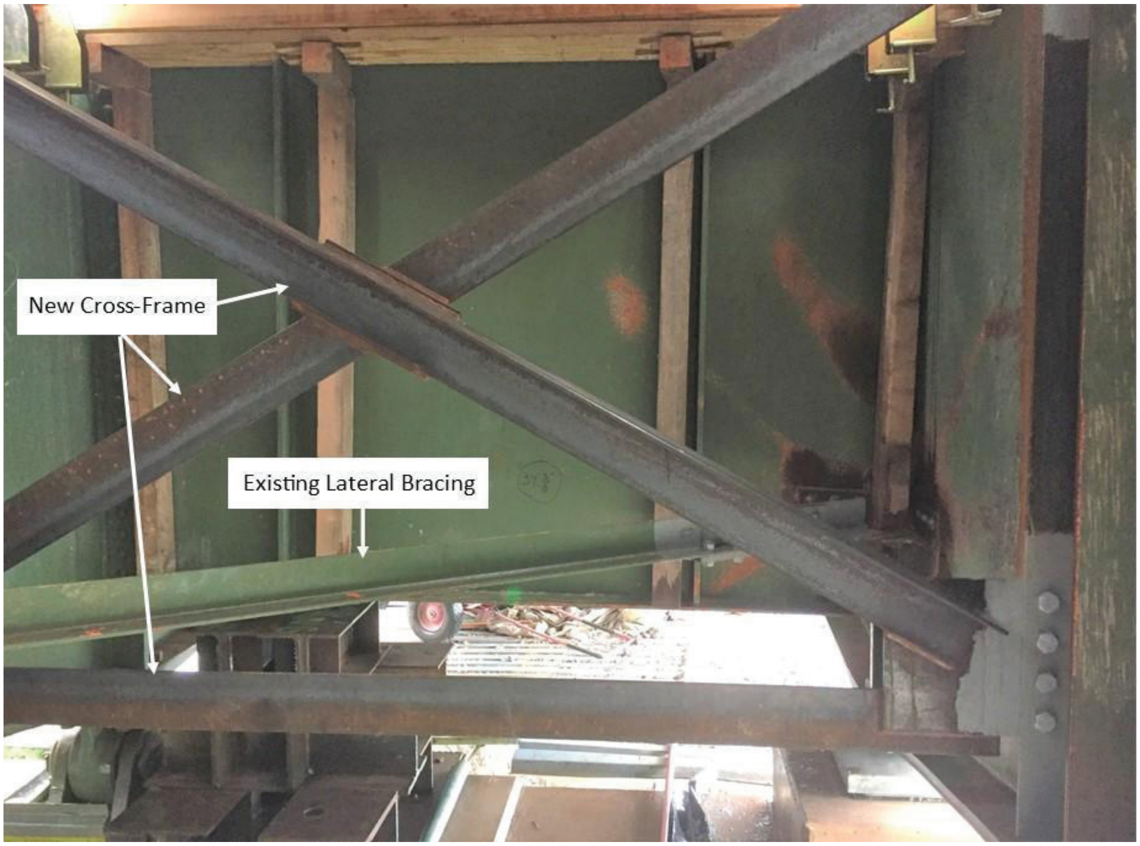


**Figure 4-10—Existing lateral bracing extending through new cross-frame (Minnesota DOT)**

When appropriate, the Designer may require an installation plan to be submitted prior to the work. The installation plan should address how the Contractor intends to address access, fit-up, and attachment.

## 4.6—ADDITION OF PLATING TO FLANGES FOR MOMENT CAPACITY

In rehabilitation, moment capacity can be increased through additional plates attached by welding or bolting. The load present on the bridge at the time the strengthening plates are attached must be considered in the design. For example, if the new strengthening plate is attached with the deck in place, the new plate will only be subjected to service level live load stresses. An example of a strengthening plate is shown in Figure 4-11.

When adding plating for moment capacity, consider the following:

- Plating repair design for the fatigue limit state.
- Member ductility to accommodate local yielding of the built-up components and load sharing between the new and existing elements in order to achieve the full factored design loading.
- Attachment design for possible interference with existing elements, such as bolts, cover plates, stiffeners, or shear connectors, and maintaining clearance over traffic below.
- Transitions in flange thicknesses. Fill plates may be required in order to satisfy fit-up requirements and allow the connection to be fully developed.
- Net section capacity for bolted attachments.
- Bearing interference for plating to the bottom flange in the negative moment region. Additional elements may be necessary to facilitate jack placement in order to keep the work area clear for installation.

Refer to Section 3 for additional guidance related to plating repairs.

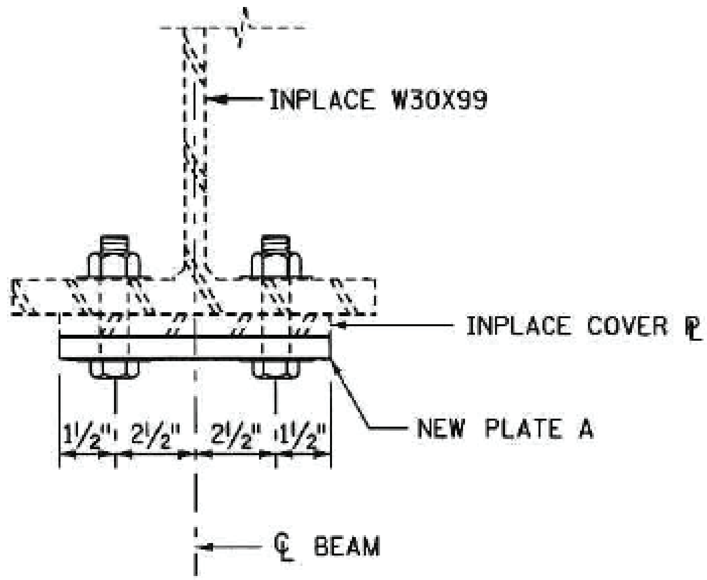


**Figure 4-11—Cross-section of plating added to existing beam (Minnesota DOT)**

## 4.7—ADDITION OF SHEAR CONNECTORS FOR COMPOSITE ACTION

Shear connectors (or shear studs) are necessary to create a composite section between the steel girders and the deck. Bridges may have been originally detailed to omit shear connectors in the negative moment regions. The addition of shear connectors during a redecking project can increase the load capacity; however, Designers should be cognizant of the redistribution of forces that results from the change in beam stiffness. Additional locations may require strengthening as a result of this change. Further, fatigue needs to be evaluated at the location where the shear connectors are added. AASHTO/AWS D1.5M/D1.5 provides greater detail on requirements for welding shear connectors.

Existing shear connectors may be shear studs or channel lugs as shown in Figure 4-12. The Contract plans should identify the existing shear connector type to assist the Contractor in planning the deck removal. Any new shear connectors added to regions where shear connectors are already present should be detailed to accommodate the existing spacing and a reasonable placement tolerance should be provided to accommodate as-constructed conditions.

(a) Shear studs

(b) Channel lugs

**Figure 4-12—Examples of shear connectors after deck removal (Minnesota DOT)**

A common issue with existing shear connectors is how to address damage that occurs during deck removal. Damage may be the result of impact due to jack hammering or saw cutting. The design should include provisions to address common findings. Below is a list of suggested mitigation efforts:

1. If the existing channel lug or shear stud has been sawed through:
   a. If the saw cut has not penetrated into the beam flange and is oriented parallel to the longitudinal axis of the beam, no correction is necessary.
   b. If the saw cut penetrates into the beam flange, see Article 7.1.
2. If the existing channel lug or shear stud was bent or deformed during deck removal, it is recommended that the connector be removed and replaced with a new connector. All of the existing weld must be ground smooth to avoid the introduction of stress-risers. However, the Owner may consider allowing damaged shear connectors to remain. The following items could be used to determine the necessity to replace damaged shear connectors:
   a. Carefully examine the weld which attaches the connector to the flange. If no damage is visible and the connector is in a positive moment region, it is likely that no correction is necessary.
   b. If no damage to the weld is visible and the connector is in a negative moment region, the weld can further be inspected using nondestructive testing if the connector is a channel lug. Alternatively, if the connector is a shear stud, it can be tested with a few blows using a 10-to-15-pound hammer to determine if this level of force results in damage. Generally, a shear stud produces a distinct "pinging" sound if it is still rigidly attached.
   c. If the connector is bent over greater than 45 degrees to the vertical, removal and replacement should be considered based on an evaluation of:
      i. The ability for concrete to be consolidated around the connector.
      ii. The ability for the connector to engage in composite action due to the lack of projection into the deck.
   d. If all shear studs in a row are damaged, it may be advisable to remove and replace them.

## 4.8—BOLT OR RIVET REPLACEMENT

When bolt or rivet removal is required, it is important that the design specify the maximum number of elements that can be removed prior to new installation so that the Contractor can appropriately plan the sequence of work. Additionally, the Designer must specify any load restrictions on the bridge to limit loads such that the connecting element has adequate strength while some of the connectors are removed.

Rivet removal is discussed in Article 2.2 and AASHTO/NSBA G14.1. Bolt removal can occur using shearing or drilling methods. Prior to installing new bolts, the existing hole must be prepared to remove any defects that could result in future cracking.

Equipment required to form new holes or enlarge existing holes must be considered. For example, it is difficult to use a magnetic drill within the center of a large, bolted plate, as the area available for the drill to be seated can be limited.

# SECTION 5: IMPACT REPAIRS

Impact damage can occur any time an object in motion collides with a portion of a bridge. While impact damage can include floating debris during high-water events and errant vessels, the most common cause is overheight vehicles. Damage resulting from these collisions can vary from cosmetic to causing complete collapse of the structure.

## 5.1—OVERVIEW OF IMPACT DAMAGE

Overheight vehicle strikes commonly impact the primary steel members of multi-beam (or girder) bridges and the secondary members of through-truss bridges. In each case, the damage may not be isolated to the members that were directly impacted. The force of the impact to an exterior beam or girder can also damage nearby cross-frames or diaphragms, as well as bearings or adjacent beams. Similarly, impacts to overhead secondary truss elements can cause damage to the primary truss members at the connections. Some of the repair concepts in Articles 5.2 through 5.6 can be applied to a variety of member and/or structure types, while others are more specific to a particular geometric configuration.

When significant impact damage is discovered, the bridge Owner should be notified immediately. Many Owners have documented procedures that should be followed to evaluate structures that have been damaged by impact. For more information, refer to publication FHWA-HIF-20-087, *Response to Bridge Impacts—An Overview of State Practices* (FHWA, 2020), which is a case study that outlines the standard practices for a cross-section of state DOTs.

When responding to impact damage, bridge Owners' primary focus is to ensure public safety, followed by restoring mobility as quickly as possible. Whereas most bridge repair initiatives are aimed at improving durability to limit future maintenance efforts, impact repairs may prioritize restoring functionality with the understanding that follow-up repairs can usually be implemented after mobility is restored.

When assessing impact damage, the first step is to determine whether the bridge is safe to remain in service. This assessment should also include an evaluation as to the potential for damage to vehicles or assets that may be beneath the bridge upon which debris or components may fall. When impact damage is severe, the affected shoulder or lanes of a redundant, multi-beam bridge may need to be closed to traffic. When severe damage is encountered on a bridge without load path redundancy (such as a truss bridge), the entire structure may need to be closed.

Once public safety is addressed, stabilization and repair options can be considered to restore functionality and mobility. *NCHRP Report 271* (Shanafelt and Horn, 1984) provides guidelines for damage inspection, assessment, and selection of the appropriate repair methodology to address impact damage. Damage can be addressed using methodologies associated with gouge repairs, straightening, or partial or full replacement, in ascending order of magnitude. The following Articles discuss these repair options in greater detail.

## 5.2—GOUGE REPAIRS RESULTING FROM IMPACT

Even in less severe instances of vehicle impact, where overall member distortion does not occur, damage may include scratches, nicks, gouges, local distortion, or cracks in girder flanges or projecting elements of truss members. An example of local distortion of a truss vertical is shown in Figure 5-1.

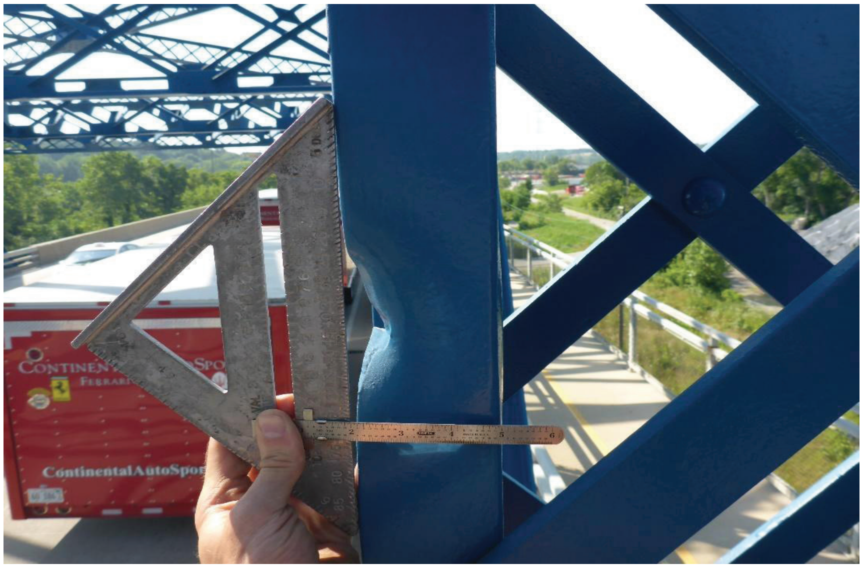

**Figure 5-1—Local distortion of the projecting flange of a channel section used in a truss vertical member**

Repair of nicks, gouges, and shallow cracks is commonly done by grinding to fair, or blend, the notches to the adjacent undamaged steel. A minimum fairing slope of 2.5:1 is recommended for elements in compression. A fairing slope of 10:1 (AASHTO/AWS D1.5M/D1.5) in the longitudinal direction is preferred for tension elements to improve the fatigue performance of the repair. When a fairing slope of 10:1 cannot be achieved due to interferences with the grinding area, a minimum fairing slope of 5:1 in the longitudinal direction is often acceptable for tension elements (AASHTO/NSBA G14.1). Nondestructive evaluation, such as MT, is recommended to confirm all cracks have been removed from the damaged area. The section remaining after grinding must have adequate capacity to resist the anticipated forces and fatigue stresses in the element. A more detailed discussion related to assessing the damage, calculating the remaining section and capacity, and potential options for repair is presented in Article 7.1 for gouges and saw cuts in girder flanges. The approaches in Article 7.1 may also be used to address localized damage resulting from vehicle impact.

An example of gouge damage from vehicle impact is shown in Figure 5-2. The damage shown in this figure was repaired by fairing the gouges at a slope of 1 to 10 in the longitudinal direction to create a smooth transition. The ground surfaces were buffed to a surface roughness average, $R_a$, of 500 µin. MT was performed after the grinding was completed to confirm that no cracks in the steel remained. Finally, the area was painted with a coating system approved by the Owner.

(a) (b)

**Figure 5-2—Gouging in a girder bottom flange (a) as discovered and (b) after grinding repairs**

## 5.3—MECHANICAL STRAIGHTENING

*NCHRP Report 271* (Shanafelt and Horn, 1984) provides guidelines for choosing the appropriate repair methodology to address impact damage based on strain evaluation. Mechanical straightening is one option that may be considered in cases where straightening, or straightening and strengthening, is determined to be appropriate. The appropriate method should be selected based on thorough evaluation of the damage and the effects of the repair technique on the material properties. In addition to *NCHRP Report 271* (Shanafelt and Horn, 1984), the FHWA *Manual for Heat Straightening, Heat Curving and Cold Bending of Bridge Components*, publication FHWA-HIF-2023-003, also discusses mechanical straightening repairs.

Mechanical straightening involves cold bending the deformed member using a jack or winch device in conjunction with blocking systems. The following schematic details have been successfully used for mechanical straightening of mildly deformed multi-beam bridges. Figures 5-3 and 5-4 illustrate schematic details for horizontal mechanical straightening using a winch or jack system, respectively, while Figure 5-5 illustrates a schematic detail for vertical mechanical straightening using a jack system.

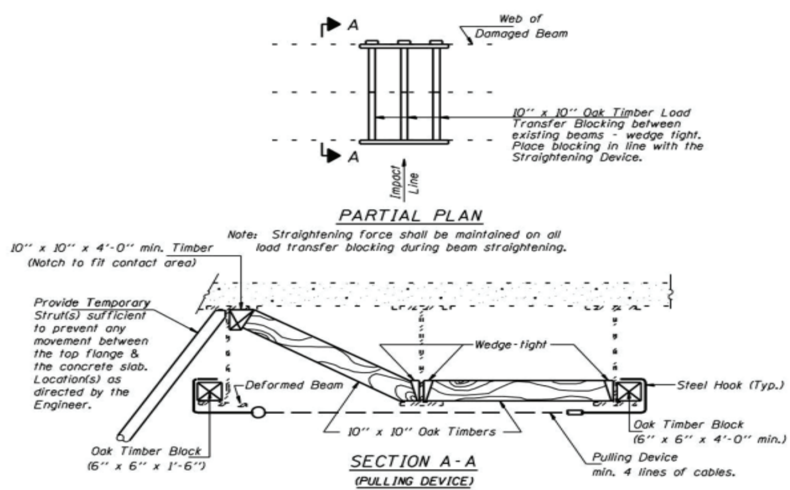


**Figure 5-3—Schematic detail for horizontal mechanical straightening using a winch system (Illinois DOT *Structural Services Manual*, Figure 2.13-4)**

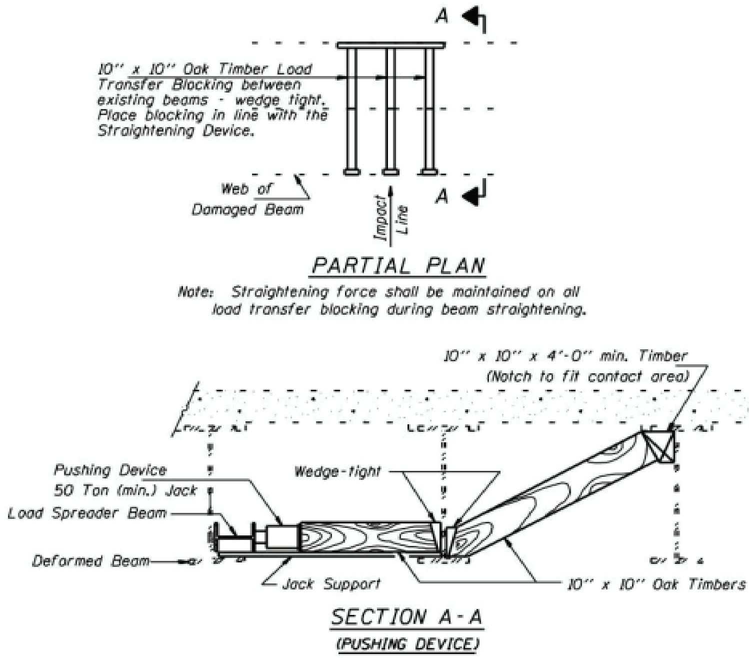


**Figure 5-4—Schematic detail for horizontal mechanical straightening using a jacking system (Illinois DO*T Structural Services Manual*, Figure 2.13-3)**

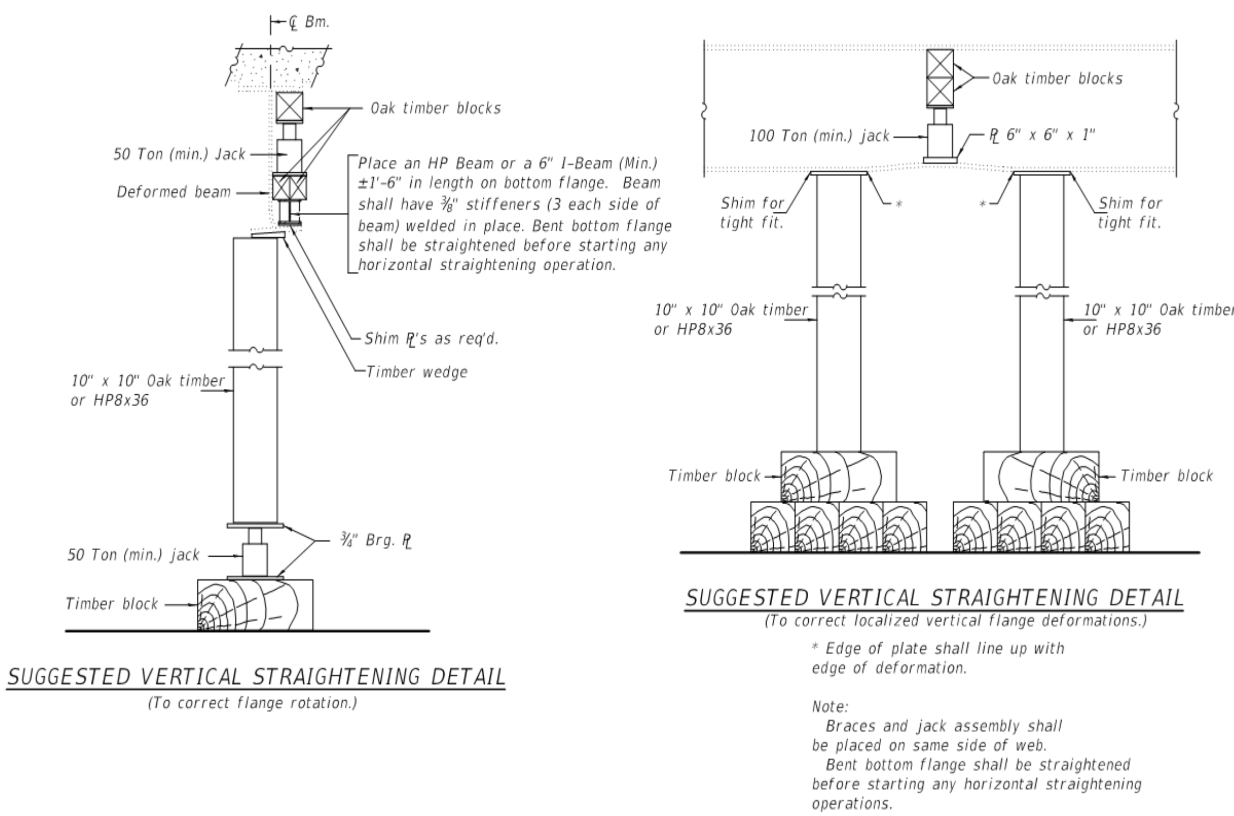


**Figure 5-5—Schematic detail for vertical mechanical straightening using a jacking system (Illinois DOT Standard Detail REP-1)**

## 5.4—HEAT STRAIGHTENING

Heat straightening, with or without mechanical methods, has been used extensively and has proven to be a reliable and cost-effective repair technique when proper procedures are used. The FHWA *Manual for Heat Straightening, Heat Curving and Cold Bending of Bridge Components* (2023), publication number FHWA-HIF-23-003, thoroughly addresses heat straightening repairs. Accordingly, no further technical discussion is provided herein.

## 5.5—PARTIAL MEMBER REPLACEMENT

There are situations where impact damage to a beam or girder within its span is too extensive for mechanical or heat straightening to be effective. In those instances, a partial-depth member replacement is necessary. Partial member replacement refers to the removal of the damaged or distorted portion of the beam or girder and replacing it with a similar section that will restore the section properties and capacity of the damaged member. Typically, this repair method involves replacement of a portion of the bottom flange along with a portion of the member web. Replacement sections can be rolled members or built-up sections, depending on the section properties required.

### 5.5.1—Recommended Sequence of Work

The general sequence of work includes the following:

1. Conduct a safety inspection of the damaged structure.
2. If necessary, shift traffic away from damaged member.
3. Identify limits of damage and conduct detailed field measurements.
4. Determine appropriate means of removing loads from damaged member.
5. Perform necessary engineering for new partial member and any temporary support systems.
6. Fabricate or procure materials.
7. Install temporary support system.
8. Remove damaged portion of existing member.
9. Install new member section and complete all connections.
10. Remove temporary support system.
11. Open bridge to normal operations.

The above steps provide a basic outline of the work to be performed. The Engineer should prepare a detailed sequence of construction that is appropriate for the repair being proposed along with any site-specific restrictions.

#### 5.5.1.1—Defining the Removal Limits

Before starting any analysis or design, accurate field measurements are necessary to define the limits of removal on the damaged member. Recommendations on what measurements should be obtained can be found in *NCHRP Report 271* (Shanafelt and Horn, 1984) or obtained from the Owner. In many cases, Owners already have procedures and requirements in place for taking measurements on damaged elements and these should be followed as appropriate.

Knowing the limits of removal will determine how the damaged section will be replaced. For example, Figure 5-6 shows an exterior girder with the damaged section already removed. The vertical limits of damage extended to approximately mid-depth of the girder web. At this location, a simple bolted splice connection or a field-welded connection with a rolled WT or fabricated inverted T-section could be used effectively.

**Figure 5-6—Partial-Depth Removal (Maryland Transportation Authority)**

A different situation is presented in Figure 5-7, where the damage on the girder extends almost to the top flange. In this situation, it is likely that only a welded connection to the top flange would work due to the tight constraints and limited depth to work with on the existing member. Another option is to replace the entire section, including a portion of the deck slab, either from existing field splice to existing field splice, or at newly created field splices.


**Figure 5-7—Full-Depth Removal (Maryland Transportation Authority)**

#### 5.5.1.2—Determining Loads on Existing Member

If the original design calculations for the bridge are available, they can be used to help assess the loads being carried by the damaged girder. However, in many cases, calculations are not available, and the Engineer will need to make some assumptions regarding load distributions and perform load calculations to determine the approximate loads being carried by the damaged girder. Simple-span members or bridges with simple geometry may only require a line girder analysis approach to determine load effects on the damaged member. In those cases, using simple tributary areas for dead load distribution should be sufficient. AASHTO live load distribution factors will be appropriate for live loads. For complex structures, such as those with variable girder spacings or curvature, it may be necessary to perform some refined analysis—such as grid models or 3D finite element analysis—to better estimate the loads being carried by the damaged member and the effects of connected bracing members.

#### 5.5.1.3—Additional Dead Load Considerations

In addition to normal dead loads, the Engineer will need to account for some amplification of those loads to address the influence from the overall structural system. Two major contributors are the deck slab and traffic barriers mounted on the bridge.

While assuming a tributary area for load calculations is a typical approach, it does not account for the continuity of the deck slab across the damaged member and adjacent members. This continuity has an impact on how any temporary support will effectively remove loads from the damaged member. Often referred to as stiffness or deck stiffness, this amplification usually takes the form of a multiplier applied to the deck slab dead load. The intent is to account for some extra resistance (or stiffness) coming from the continuous deck slab that connects to the damaged member and the rest of the bridge system.

Perhaps less well-known, but also important to consider, are the effects from continuous traffic barriers mounted on the bridge. Continuous barriers provide stiffness to the overall system and can have an effect on a temporary support system. Similar to the effect from the deck slab, a continuous barrier will impart additional stiffness that must be overcome by the temporary support. Overcoming the additional stiffness is achieved, in some cases, by multiplying the barrier dead load that must be supported.

#### 5.5.1.4—Removing Load from Members Prior to Partial Removal

Relieving the loads from the damaged girder prior to any repair operations is critical. Both dead loads and live loads need to be addressed. For live loads, a common approach when the bridge cannot be closed is to shift traffic away from the damaged member. This usually involves setting temporary concrete barriers on the bridge to protect the damaged area as well as the future work zone. Any temporary barrier set on the bridge must be accounted for in the load analysis during the design of the repair, particularly if the temporary supports are carrying the dead load of the barrier.

Depending on the cross-section of the bridge, there are two common approaches to removing the dead load from a damaged girder. In one approach, one or more temporary "sister" beams are installed adjacent to the damaged beam to carry the loads (see Figure 5-8).

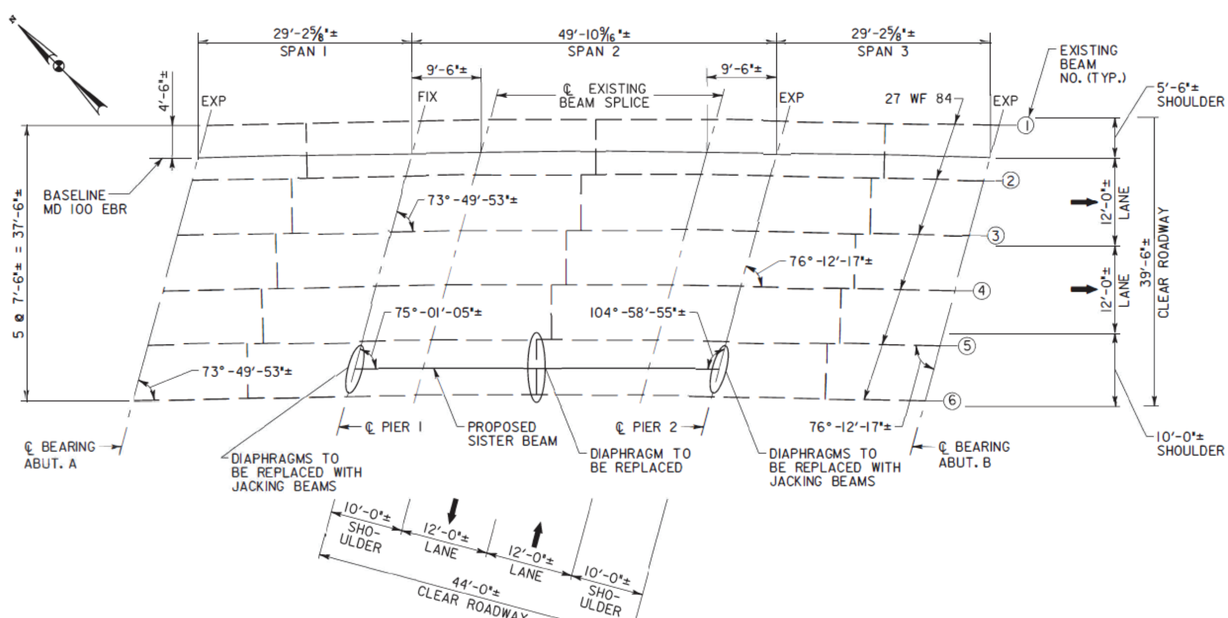


**Figure 5-8—Example sister beam system (Maryland DOT State Highway Administration)**

In the second approach, as shown in Figure 5-9, temporary "carrier" beams can be erected directly over the damaged beam to lift the load off the damaged section to be repaired. These methods will be discussed further in Article 5.5.1.5.

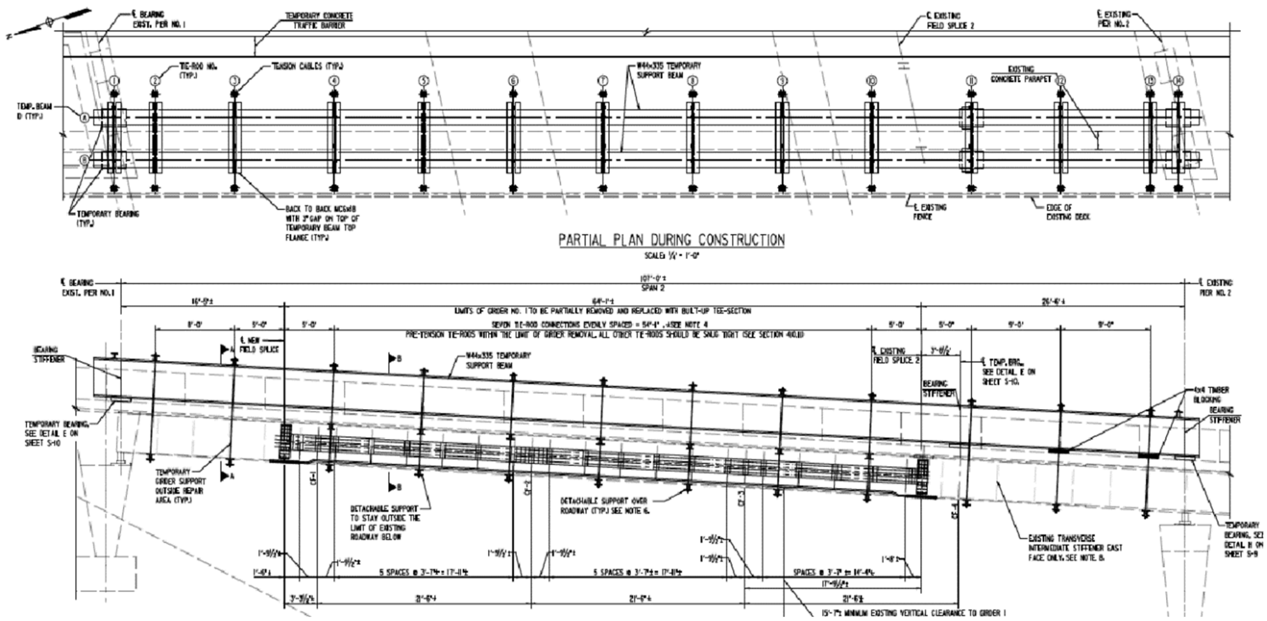


**Figure 5-9—Example carrier beam system (Maryland Transportation Authority)**

#### 5.5.1.5—Temporary Works to Facilitate Repairs

The analysis and design of a partial-depth member replacement involves more than just sizing the section required to repair the damaged member. The process also involves designing a temporary system to remove as much load as possible from the damaged member, if not all of the load, prior to starting any removal operations. As mentioned in Article 5.5.1.4, there are two common approaches: sister beams adjacent to the beam or girder to be repaired, or carrier beams erected over the damaged member.

Using sister beams typically requires the Contractor to carefully fit the temporary beams up under the existing bridge deck without damaging surrounding existing structural elements such as other beams or supporting substructure. The sister beams need to be placed close enough to the damaged beam to remove the dead loads from it but with enough space between them to allow repair work to be performed. Depending on the location of the damaged member within the bridge superstructure (i.e., exterior or interior beam/girder), one or two sister beams can be provided. For a damaged exterior girder, the typical setup uses a single sister beam located between the exterior member and the first interior girder. When the damaged member is located at an interior girder line, the setup can use two sister beams, one located on either side of the damaged beam to remove loads. Both scenarios are illustrated in Figure 5-10.

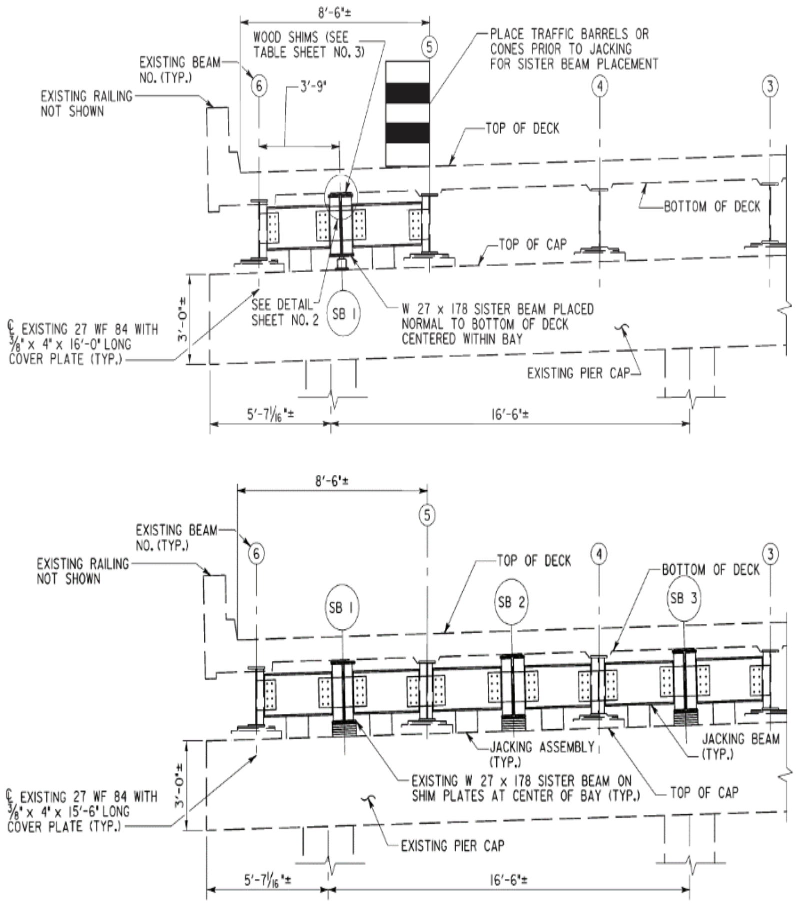


**Figure 5-10—Example sister beam systems (Maryland DOT State Highway Administration)**

Support for the sister beams is usually achieved using the existing substructure units. However, in some circumstances, when the existing substructure cannot support the sister beams and the area under the bridge is available for use, temporary shoring towers can be used to support the sister beams while the damaged girder is repaired. In this case, the placement of the towers should be analyzed, since shoring towers modify the load path of the structure and can influence effective span lengths and beam continuity.

Carrier beams are another method of removing loads from damaged members. This type of temporary support uses beams erected over the damaged beam or girder. Typically, they are set over the existing bridge deck with temporary bearings or blocking located directly over the existing bearing lines. Blocking located in this manner provides a direct load path to the substructure below. Depending on the conditions, one beam aligned over the existing beam may be enough. However, there may be situations where two carrier beams set side by side may be required to provide more stiffness or more load-carrying capacity. Both configurations are shown in Figure 5-11. The Engineer will need to assess which scenario will work best for the problem at hand.

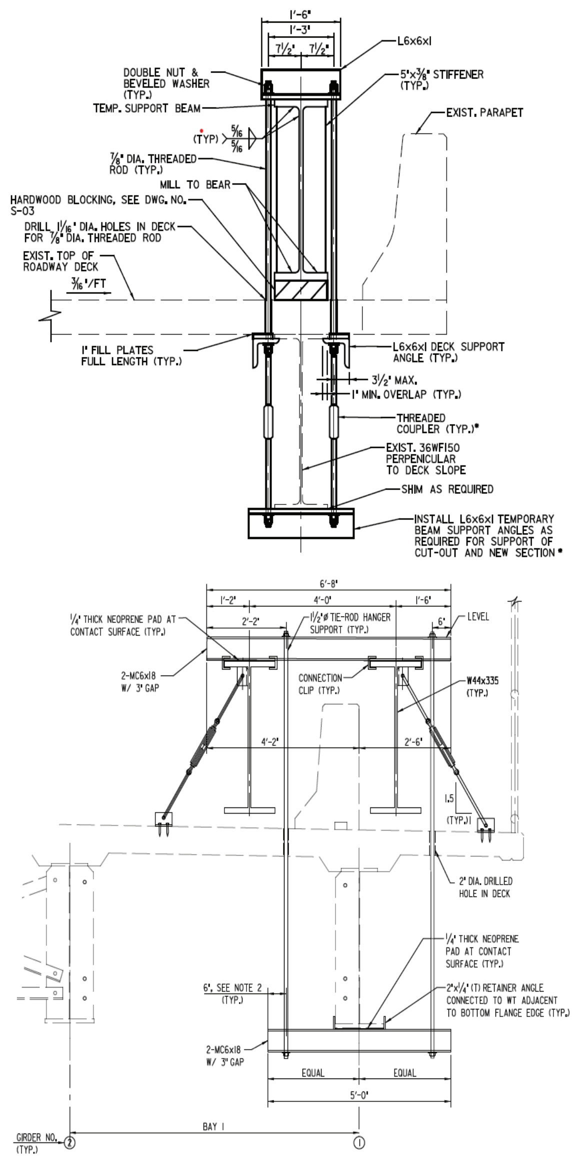


**Figure 5-11—Temporary systems using one and two carrier beams (Maryland Transportation Authority)**

To transfer the load off the damaged beam or girder, carrier beams usually have a system of hangers spaced along their length. These hangers use high-strength rods that pass through the deck slab and either engage the bottom of the slab or extend to a lifting beam just below the damaged member. Hangers and lifting beams are designed to carry a tributary length of the beam and connected deck. Check with the bridge Owner for any standard details related to the hangers. Some Owners have standardized hanger details that should be followed. For example, Illinois DOT has a standardized detail which provides the maximum spacing allowed between hangers based on the rod design.

### 5.5.2—New Partial Member Considerations

Article 5.5.1.1 noted that defining the required depth of replacement in many cases determined the type of new member (e.g., fabricated inverted T-section or a rolled WT). The main goal is to return the damaged member to its full cross-section, load-carrying capacity, or both. Other factors influencing the type of replacement member include the existing member that is damaged, material availability, and the schedule for completing the necessary repairs.

If a rolled beam is damaged, it is likely that a rolled WT or a section cut from a similar rolled beam section could be used to return the existing member to full capacity, or close to full capacity. For a plate girder section, a rolled section could also provide compatible section properties as long as the splice connections between the existing and proposed sections can be achieved and the proposed section does not greatly alter the required section properties of the restored member. Sections should be selected to minimize or avoid changes in thickness requiring filler plates, which can complicate the installation. Replacement sections could also be fabricated by welding plates together. Welded plate sections offer more flexibility to the Engineer, given the various plate sizes available from suppliers. Plates can be trimmed to provide a more tailored fit-up to the existing section.

There are many instances where material availability and schedule are more critical to the bridge Owner and these factors need to be considered early in the process. A rolled section may appear as a cheaper alternative due to reduced fabrication effort. However, rolling schedules and material availability could delay getting the needed section for repair. In that situation, a fabricated section, while more labor intensive, could provide the Owner with a faster repair schedule. The Engineer needs to consult with the bridge Owner to determine the preferred path forward.

#### 5.5.2.1—Bolted Connections

When evaluating the limits of removal on a damaged beam, consider the use of existing bolted connections, when possible, to attach the new sections to existing steel. On simple-span members, this may not be possible since there may not be any existing field splices, and new web and flange splices will need to be introduced. For continuous spans with existing field splices, the Engineer should evaluate using existing splices as connections for the repair, even if it means extending the limits to reach the splice locations. This approach will simplify the design and construction.

For vertical splice locations, the web and flange connections are required to be designed in accordance with the AASHTO BDS. Horizontal splices between existing and new steel with a partial-depth removal are designed for horizontal shear, similar to a flange-to-web weld, to determine an appropriate bolt size and spacing. Figure 5-12 shows portions of a sample detail that includes both horizontal and vertical web splices. Figure 5-13 shows photos of the primary stages of a partial beam replacement of a damaged beam.

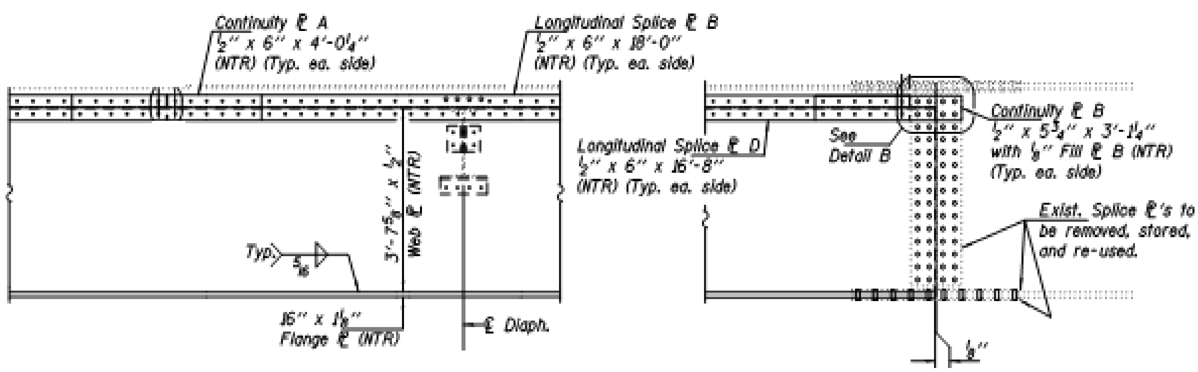


**Figure 5-12—Sample bolted horizontal and vertical web splice detail (Illinois DOT)**


**Figure 5-13—Photos illustrating primary stages of partial beam replacement (Illinois DOT)**

*5.5.2.1.1—Welded Connections*

When welded connections are preferred over bolted ones, follow the Owner's requirements for field welding. In this case, the repair is analogous to a beam end repair as discussed in Article 3.3.2.1. Fillet welds should be used wherever possible to simplify construction, although it may be necessary to use CJP welds (see Article 2.4.2.9). An example of a welded repair to address impact damage is shown in Figures 5-14 and 5-15.

**Figure 5-14—Impact damage to an interior steel girder of a multi-girder bridge**

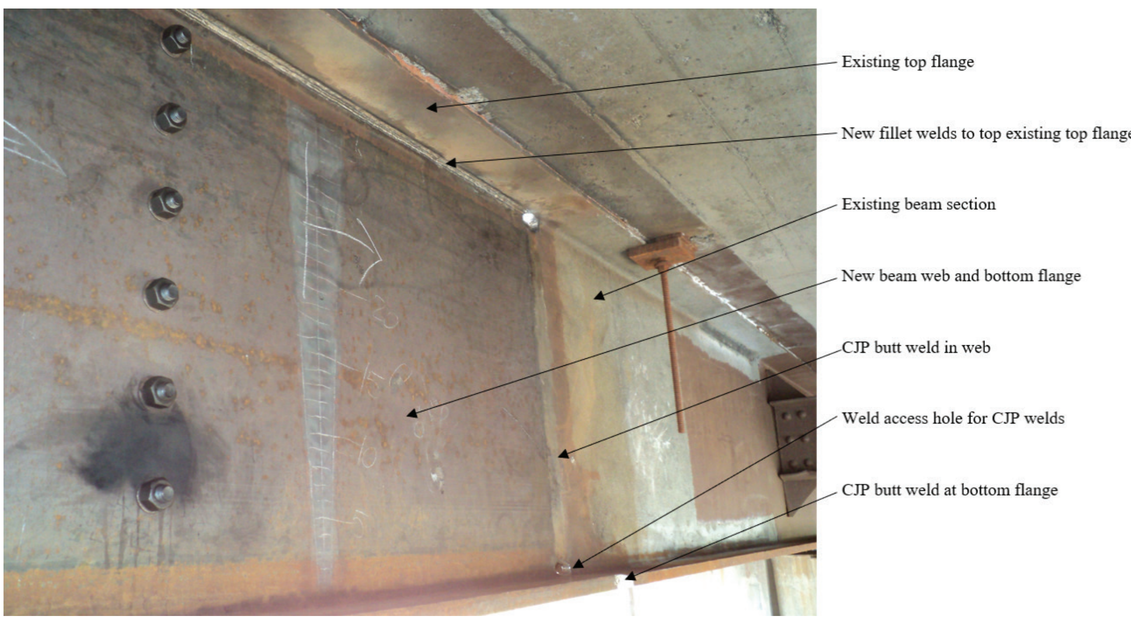


**Figure 5-15—Web and bottom flange replacement to repair the impact damage shown in Figure 5-14**

*5.5.2.1.2—Heat Straightening at Limits of Repair*

Removing the distortion caused by an impact as part of a partial replacement repair is desirable, but some minor distortion may remain in the existing member where bolted or welded splices are installed. Localized mechanical or heat straightening can be used to reduce or eliminate the distortion for a better fit between new and existing elements. Bolted splices may pull slightly distorted webs into alignment. Refer to Article 5.4 for additional information on heat straightening.

## 5.6—CLEARANCE MODIFICATIONS

It may be feasible to prevent future impacts by improving vertical clearance. Adequate vertical clearance can be achieved by raising the bridge, lowering the roadway below the bridge, or a combination of both. It is prudent to measure the thickness of the paving under the bridge via coring before determining how much to raise the bridge and determine if there are potential gains in removing paving. It is also prudent to consider the build-up of future paving overlays when determining how much to raise the bridge. The magnitude of the necessary profile change and cost of associated approach roadway modifications should be considered when evaluating this option.

Bridges are raised primarily by raising the bearings seats at the bents. It is common to replace or repair the bearings during the bridge raising since both procedures are closely related. The actual bridge raising is typically performed via jacking and is discussed in Article 6.3, which includes discussion of live loads, stability during jacking, and requirements of hydraulic jack systems.

The beam seats can be raised in a number of ways, depending on the bearing type, distance to be raised, presence of live load during installation, and duration of traffic closure, if any. The range of applicable heights for the solutions offered here is usually dictated by Owner preference, but the Designer must still confirm that a solution is stable when subjected to the anticipated combinations of vertical and horizontal load. The hydraulic jacks used to lift bridges often have relatively small strokes, and the means of raising the beam seats typically must allow for the jacks to be stroked more than once. This is often done by adding steel plates alternately at the bearing locations and under the jacks as the system works upward. The simplest means of raising the bearing seat is to leave the steel shim plates in place under the bearing. Plates are typically limited to ½-in. thickness to allow for handling. The plates are joined together by small intermittent welds. Grout pads are a common alternative to plates when the beam seat is being raised a short distance.

A common means of raising the bearing seats a larger distance is to construct or raise the concrete bearing pedestals. This is typically done by chipping out concrete and then doweling in new reinforcement for each pedestal. This method requires a longer closure time, as the concrete must achieve adequate strength prior to applying live load.

Another common method to raise beam seats larger distances is to install steel pedestals or bolsters made up of plate, steel castings, or W or HP steel sections with base plates welded to both ends. The use of pedestals places the bearing at the top of the pedestal directly under the beam. A simple statics check can be used to evaluate the stability of the pedestals under horizontal loading. Ideally, the existing anchor rods can be used to connect the pedestals to the bent to resist the overturning loads if needed. An alternate method is to connect steel bolsters to the bottoms of the beams. This method places the bearing at the bottom of the bolsters, and horizontal reactions at the bearing will induce moments in the girders. The use of steel pedestals or bolsters requires a shorter closure time than installing or raising the concrete bearing pedestals. Sometimes steel pedestals are used as a temporary means of allowing live load on the bridge while concrete pedestals are built around them. Figures 5-16 and 5-17 show example details of steel pedestals and bolsters used on prior projects. Connections to the beam or girder can be field-welded or bolted depending on the Owner's preferences.

Approach slabs and bridge-mounted utilities must be considered when raising a bridge. For bridges with a backwall at the end bent, an overlay on the approach slab is adequate to raise the top-of-roadway elevation to match the new bridge elevation. The approach slabs often must be cut for bridges with an end wall or similar detail where the approach slab is supported on the superstructure. Sometimes a new approach slab can be placed on top of the existing one. Sometimes a new extension to the existing approach can be used. In cases where the bridge backwall or end wall is raised with the superstructure, bridge-mounted utilities must be "freed" where they pass through the concrete. This can be done by coring horizontally through the backwall or end wall to create a series of overlapping holes around the utility. The utility hangers may also have to be adjusted to accommodate the superstructure movement since the superstructure is being raised relative to the utility.

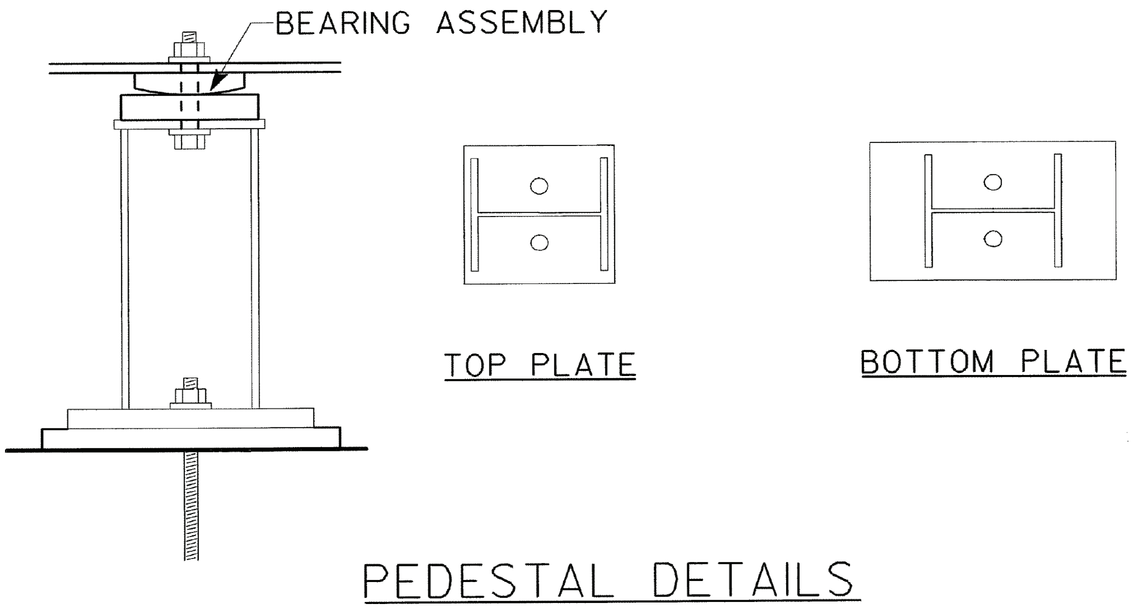


**Figure 5-16—Examples of steel pedestals under bearings**

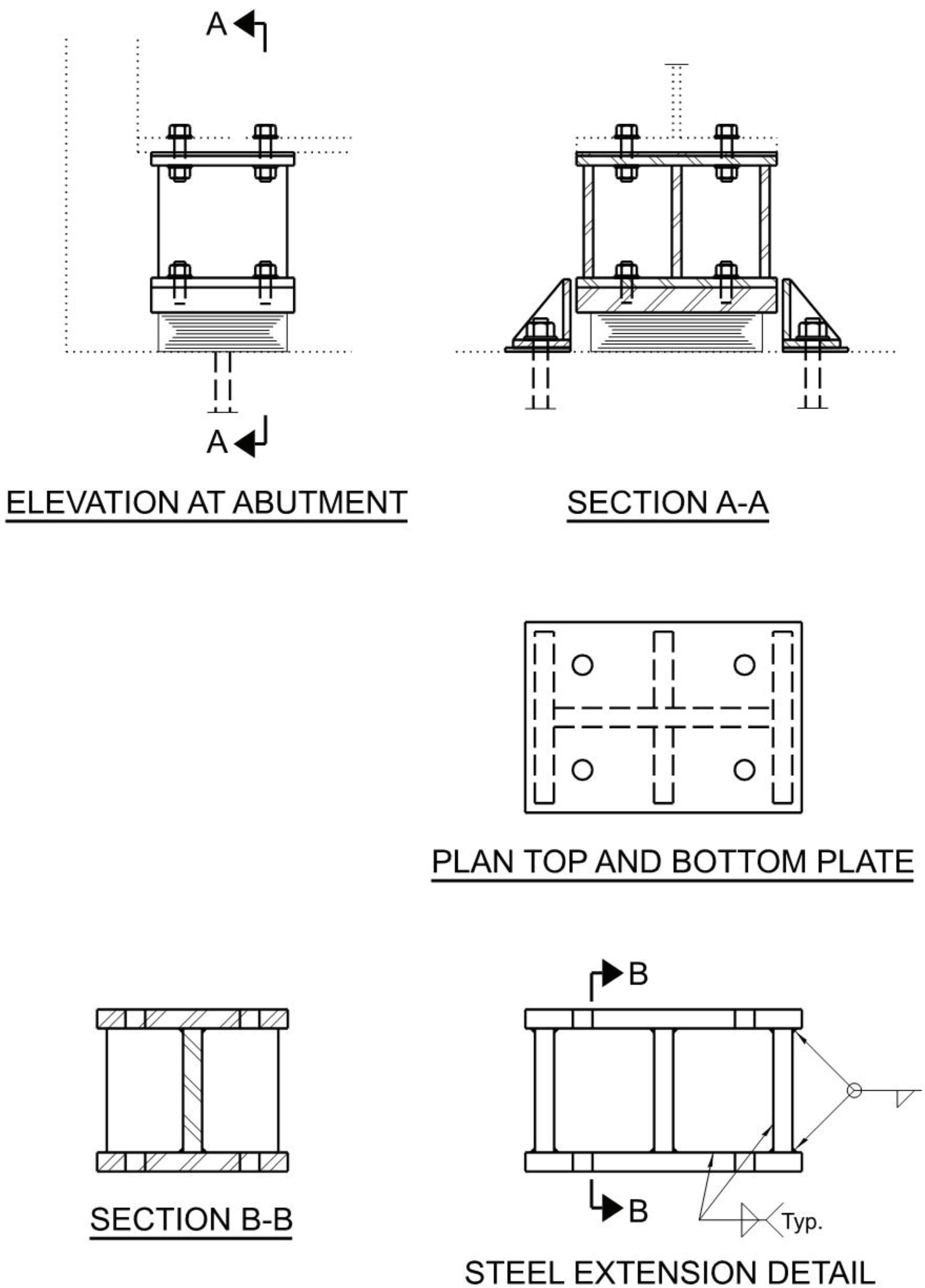


**Figure 5-17—Examples of steel bolsters over bearings (Illinois DOT)**

# SECTION 6:
# BEARINGS AND ANCHOR RODS

Bearings and anchor rods may require repair or replacement due to corrosion loss or damage from applied loads. Leaking deck joints allow water carrying corrosive material to drain onto the bearings. The corrosive material, together with dirt and animal waste, causes build-up on and around the bearings, contributes to bearing corrosion, and obstructs intended rotation or translation. The build-up changes how horizontal loads are transferred within the structural system and can result in bearing and anchor rod failure. Inadequate design or installation can also cause unanticipated system behavior and lead to failure. Finally, bearings and anchor rods must convey the dynamic loads applied to the bridge, especially those caused by live load and temperature changes.

Bearings in particular are considered to be replaceable items rather than permanent parts of the bridge. It is common to replace bearings with improved designs when work is performed to increase a bridge's vertical clearance since the procedures are closely related. Bearing replacement can be avoided when preventative maintenance is routinely performed.

Bearings can be maintained through planned preventative maintenance activities. This includes keeping these elements free of debris and corrosion, as well as lubricating where necessary. Depending on the level of deterioration, bearings that are inoperative or misaligned may be able to be cleaned and reset. This process involves jacking of the superstructure, cleaning surfaces or pins, lubricating moving elements, resetting elements to the correct position, and setting the superstructure back onto the bearings. Much of Article 6.3 may apply to this work. Once the plan has been developed, that same plan can be reused for future preventative maintenance work on the bearings.

## 6.1—BEARING REPAIRS AND REPLACEMENTS

Bearings are typically replaced with improved designs rather than repaired, as older designs are difficult to repair and retain undesirable features for performance and maintenance. Replacement involves raising the bridge, usually with hydraulic jacks, to remove the existing bearing and install the new bearing. This may be performed while the bridge is in limited service if the bearings are easy to replace and the existing structure permits the vertical displacement. Potential negative effects on existing finger plates, modular expansion joints, embedded conduits, metal railings, diaphragms, and bridge decks must be considered. Steel bearings, rocker bearings, and similar designs should be replaced with elastomeric bearings when possible. High-load multi-rotational (HLMR) disc bearings are preferred where elastomeric bearings will not work. See AASHTO/NSBA G9.1, *Steel Bridge Bearing Design and Detailing Guidelines*, for additional information.

Some primary objectives for bearing replacement are achieving the correct bearing height, resolving problems with the anchor rods, and addressing concerns with deterioration and movement of the supported existing structure. When changing bearing height, it may be necessary to adjust the bearing seat elevation to accommodate the new bearing geometry. See Article 5.6 for discussion of bearing seat adjustments. Anchor rods are discussed in Article 6.2. Modern bearings are designed to be replaceable with only small jacking heights, but older bearings—such as articulated rocker bearings and early pot bearings—may require more clearance. Cutting the accessible portion of the bearing may be possible, allowing the embedded portion to remain. In other cases, partially demolishing the bearing seats to remove embeds and concrete may be necessary.

## 6.2—ANCHOR RODS

It is prudent to inspect existing anchor rods to determine whether they can be reused or whether they will require repair or replacement to permit bearing replacement or during the process of bearing replacement. Broken, bent, loose, or marred anchor rods are common, especially on highly skewed bridges. Corrosion and fracture often occur where the rod passes through the bottom plate of the bearing, since water and debris collect in the anchor rod holes. This distress may not be evident during inspection. Anchor rods often cannot be readily repaired without removing the bearing. Repairs may require removing concrete around the existing anchor rods to expose enough of each rod to splice new anchor rods via welding or threaded bar couplers. Confirm that the existing anchor rod material can be welded prior to using a weld repair. Occasional broken anchor rods at expansion bearings are often monitored rather than repaired. Where there is redundancy and horizontal fixity is not critical, the superstructure redistributes the horizontal loads to other anchor rods. Replacement bearings may be designed and detailed so that anchor rods are not required.

Anchor rods are sometimes replaced in the same location by core drilling over and around the anchor rod until it is released. Extending the core through the tail of bent anchor rods may be necessary. An alternative for removal of long anchor rods is to core horizontally from the face of the bent cap through the rod to sever it. Care must be taken to guard against primary reinforcement being damaged or cut by the core barrel. This coring method is more predictable where anchor rods were grouted into formed wells since their location is known and primary reinforcement can be avoided. Coring can be difficult where cross-frames, edge beams, or the beam flanges prevent placement of the core rig. Removing a cross-frame may facilitate coring. The Designer should determine what is allowable and communicate it in the bearing replacement or anchor rod replacement notes or plans. Once the old anchor rod is removed, the new anchor rod is installed with two-component epoxy capsules for smaller rods (up to about 1½-in. diameter and 12-in. embedment, depending on manufacturer recommendations) or grouted into place for larger rods.

Consider designing the bearing replacement to use new anchor rods in new locations. New anchor rod locations may avoid existing reinforcement in the cap or accommodate space requirements for the core rig. New locations may also be selected to increase the overturning capacity of a steel bearing pedestal if one is being used. Require the Contractor to field-verify the locations of the new anchor rods and existing primary reinforcement near anchor rod locations prior to fabricating the new bearing components. The new anchor rods are typically installed by rotating them into a two-component epoxy capsule in the cored holes.

Reducing the size and number of anchor rods or eliminating them entirely by introducing other means of resisting horizontal loads may be possible. One way to eliminate horizontal load in bearings and anchor rods is to form shear blocks or guide blocks on the bent caps. When required, only two blocks are needed at each bent if properly positioned. The blocks should be configured such that transverse expansion and contraction will not cause distress. A slightly radiused stainless steel plate or hot-dip galvanized assembly can be cast into the blocks to facilitate longitudinal sliding while resisting lateral loads. Stainless steel sliding plates are also attached to the mating surfaces of flanges. Guide blocks have been found to allow for very simple bearing and anchor rod arrangements. They also simplify design since the Designer can easily determine the horizontal load path. Figure 6-1 shows example details of a guide block.

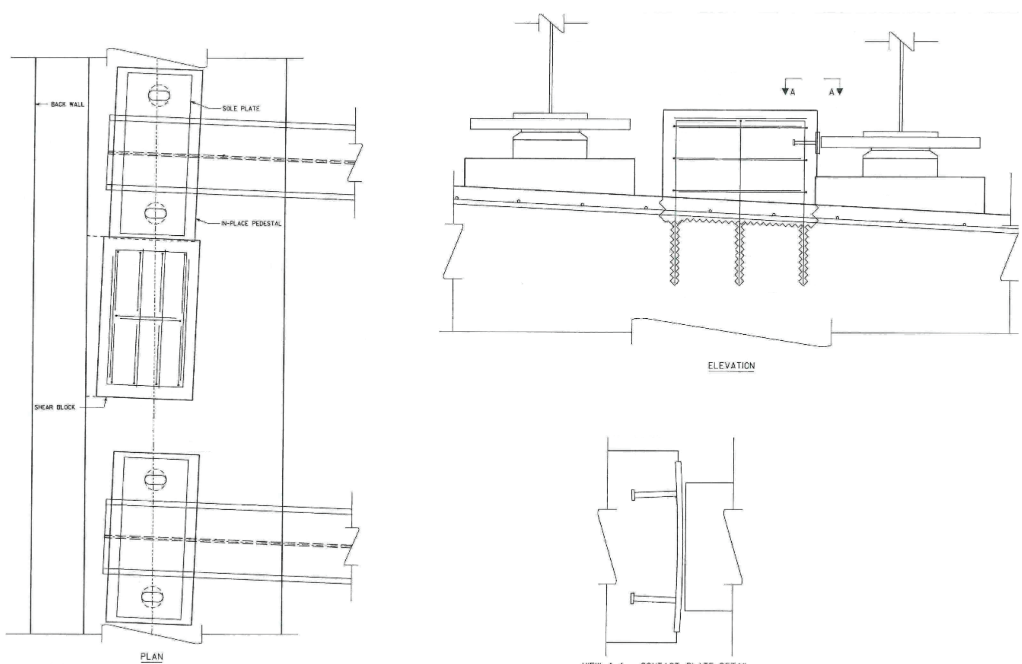


**Figure 6-1—Examples of guide blocks used in lieu of anchor rods**

## 6.3—DESIGN, ANALYSIS, AND DETAILING CONSIDERATIONS

Bearing replacements, anchor rod repairs, and vertical clearance adjustments typically do not require explicit analysis themselves. They require that portions of the bridge support and restraint systems be temporarily removed, sometimes while under traffic. Analysis and design of this temporary system include determining where the structure can be lifted based on geometric and capacity constraints, what strengthening or modifications are needed to allow for the lifting, and how the lifting will be performed.

The Designer should evaluate the magnitude of the loads anticipated at the hydraulic jacks and specify all requirements associated with carrying live load during bearing replacements. In general, raising all girders at one bent relative to another bent does not usually cause a significant stress effect. However, differential raising of the girders at an individual bent line can have a significant effect on the girders, cross-frames, jacks, and supporting structures. The Designer should evaluate these effects and consider the potential for additional break-away load associated with soil friction at the endwalls or other constraints. The design must consider how the jacks will remain plumb while the structure is elevated. Placing the jacks on a common manifold promotes equal load distribution. However, the bearings do not all carry the same load, especially if some live load is on the structure (traffic or construction vehicles), so they will have different displacements. The bridge cross-section might twist before the bearing that supports the most load lifts off. Alternately, the jacks can all be individually controlled to maintain constant displacement. In practice, this may not be possible. An alternative way to raise a bridge is to raise the jacks sequentially and iteratively, one at a time, by a defined amount, until the desired geometry is achieved. The Designer should determine how much displacement at a single location is acceptable and communicate this displacement, along with anticipated loads, in the bearing replacement notes or plans.

The Designer should also evaluate the magnitude and paths of the horizontal loads that must be resolved during jacking. Hydraulic jacks cannot tolerate horizontal (lateral or longitudinal) loading and will cease to operate if side-loading occurs. They must be protected from horizontal loading while actively engaged. Some jacks with locking collars can tolerate some horizontal load, but only while the collars are locked. It may be valid to assume that horizontal loads will be resisted if the anchor rods at some bearings remain in place, but this assumption often requires displacements that would disable the jacks. Performing short strokes at one jack at a time while all other jacks are on their lock collars may be the best way to protect the jacks. Timber blocking is typically placed in all open deck joints during jacking to help maintain the relative superstructure geometry. The Designer should determine how much horizontal load is anticipated and communicate it, along with some means of resisting the horizontal load, in the bearing replacement notes or plans.

Jacks are often placed either under the girders or under the cross-frames at the bearing centerline. Each must be evaluated for the anticipated jacking load, and the loads may not be distributed uniformly in a system with redundant jacks. When lifting under the girders, the existing girder web over the jack is often not adequate to handle the loads due to jacking. The web can be strengthened in a number of ways, including attaching tube or angle stiffener members to the web, adding cover plates to the web, infilling the web with concrete made composite with the web via shear studs, or by adding jacking stiffeners. In most cases, for economy and ease of installation, jacking stiffeners need not be the full height of the web. Fill all open holes in the permanent members with fully tensioned bolts after the temporary members are removed. Bolted angle stiffeners, web cover plates, and concrete infills can be made permanent. If bolting is not feasible, consider field-welding stiffening components to the members. Evaluate all modifications to avoid introducing detrimental fatigue conditions. Figure 6-2 shows example details of a jacking scheme with jacks under each girder.

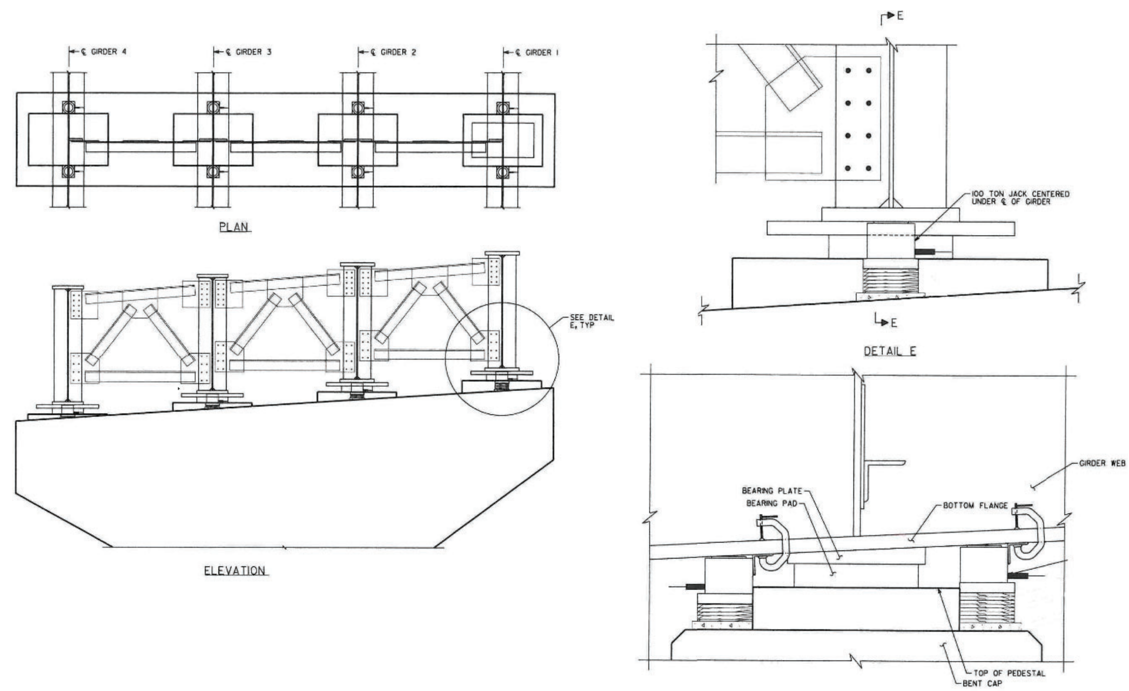


**Figure 6-2—Example of jacking scheme with jacks under each girder**

Space limitations often make it easier to lift at the cross-frames than at the girders. The cross-frames may require modification or even replacement to accommodate the jacking forces. Figure 6-3 illustrates common modifications to cross-frames for jacking. If cross-frames require modification, it may be more efficient to use only two jacks at each bent. Using fewer jacks will reduce the number of frames to modify and make the lifts and associated jack loads easier to anticipate and control.

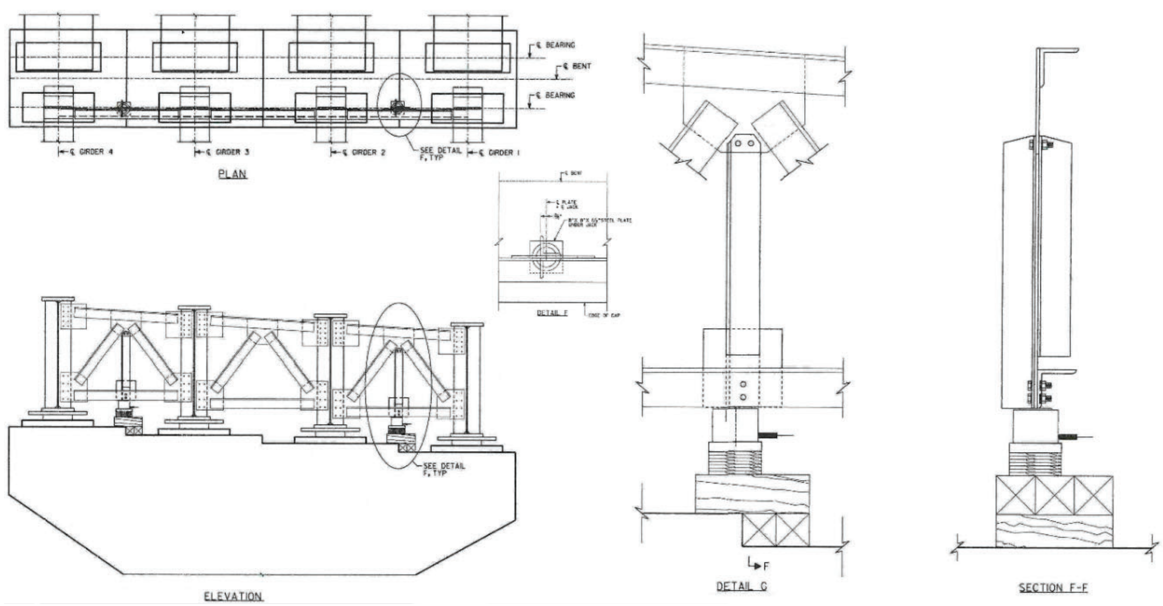


**Figure 6-3—Example of jacking scheme with jacks under modified K-frame**

Jacks are sometimes used at other locations to lift a structure. If present, floor beams can provide robust lift points for a structure. Where access below the bridge is difficult, strongbacks (or carrier beams) can be mounted to the superstructure ends from above and the jacks can lift from the adjacent span or end bent.

The supporting substructures must also be considered. Jacks are often supported by the existing bents. The edges of bent caps and bearing pedestals are often lightly reinforced and the reinforcement may have large bend radii. This makes the areas of concrete near the edges, which are not encapsulated by the reinforcement, prone to spalling under load. Figure 6-4 illustrates a geometric check used to locate the jacks in order to minimize spalling. Where space and headroom allow, this can be overcome by using load plates under the jacks. Bent caps are not usually finished to a flatness criterion, so the use of thin elastomeric pads, grouts, or other means of distributing load may be needed. Where jacks cannot be supported on the bent cap, they may be supported by members mounted to the bent cap. Steel beams running parallel to the bent cap can be hung from the cap using saddle beams. In this configuration, the loads from both the back and ahead spans must be carefully controlled to balance the saddle system. Figure 6-5 shows example details for a cradle beam jacking system. Steel beams can also be mounted to the bent cap or columns using standard column-mounted screw jacks. Individual steel bolsters can be mounted to the bent cap face under each girder to support the jacks. The deflection of the support system must be included in assessments of the system displacements, especially if restroking the jacks will not be possible.

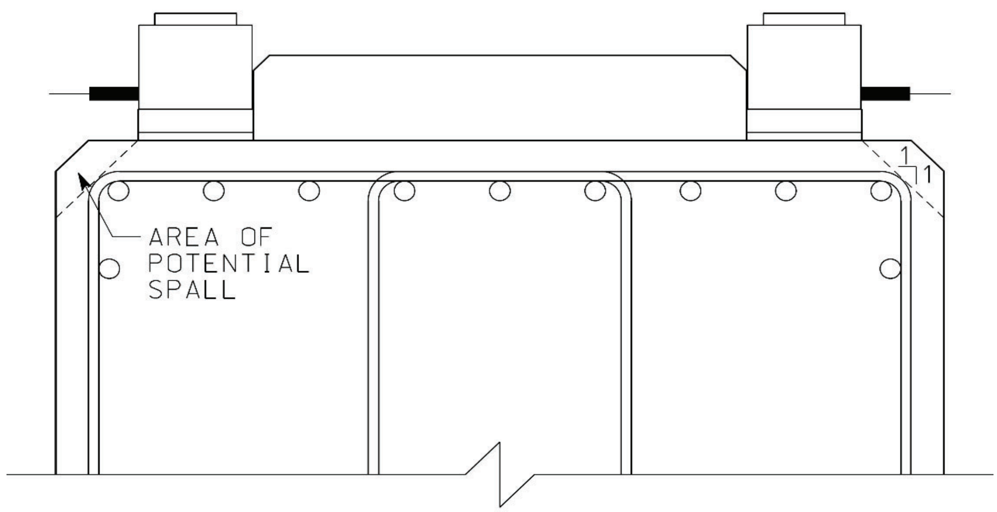


**Figure 6-4—Location of jack to minimize spalling at edge of bent cap**

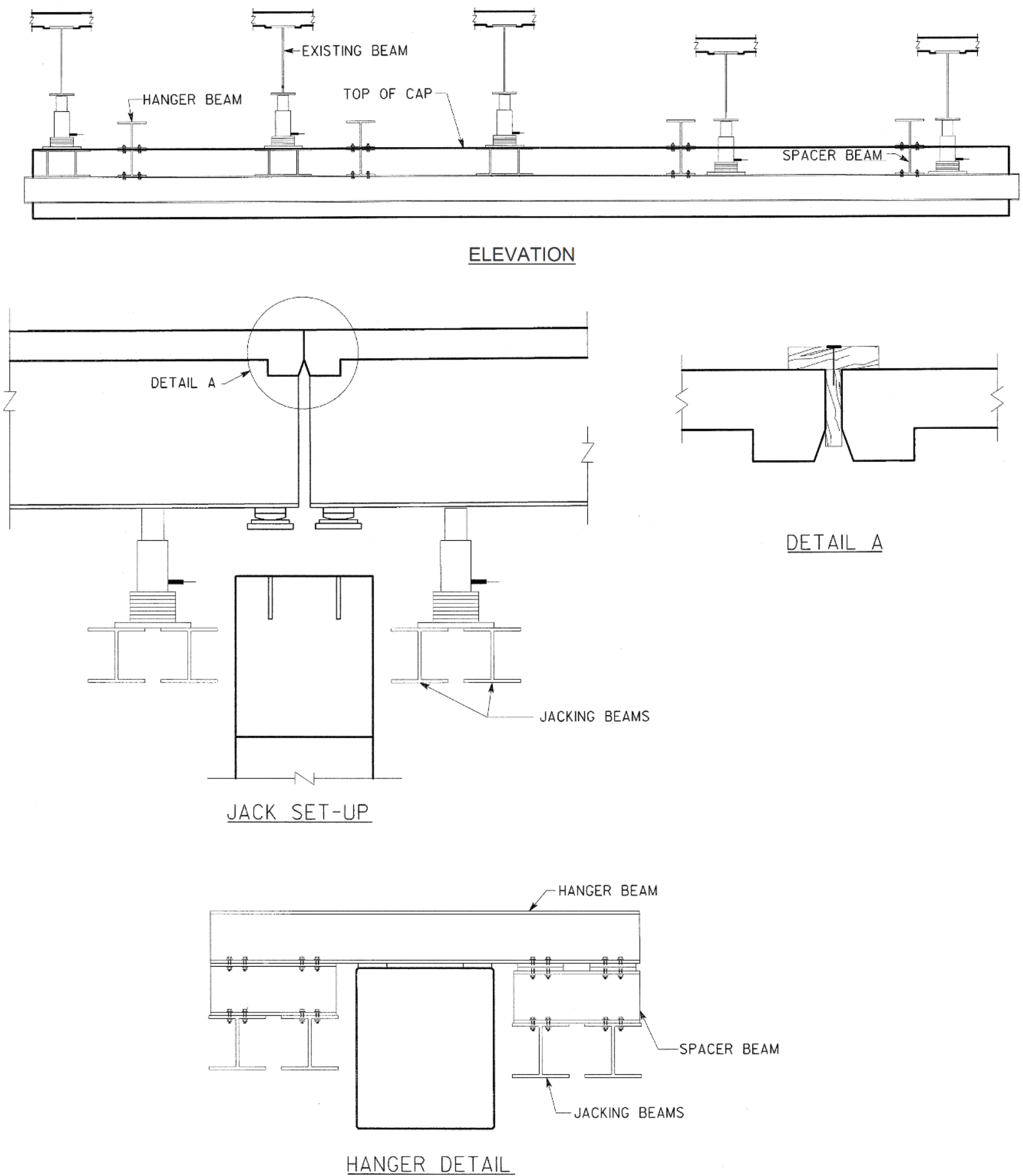


**Figure 6-5—Examples cradle beam system to support jacks**

# SECTION 7: DAMAGE OCCURRING DURING CONSTRUCTION

Field repair may be required for damage occurring during new construction or bridge rehabilitation. Frequently, damage occurs during concrete removal operations associated with deck replacement projects or localized concrete repairs.

## 7.1—SAW-CUT DAMAGE TO STEEL GIRDER FLANGES

Saw-cut damage to the top flange of steel girders may occur during bridge rehabilitation projects involving demolition and replacement of the concrete deck. This type of damage is often a result of setting the depth of the saw blade too deep, variable depth deck fillet heights, or not adjusting the depth of the blade in negative moment regions where the flanges are typically thicker or include cover plates. The damage may range from a single shallow saw cut across the flange width to multiple, closely spaced saw cuts over a long length. The cuts may also be oriented parallel or perpendicular to the length of the beam. In order to properly assess the damage, each saw cut must be fully documented, including length, depth, orientation, and location within the span. Each location of damage can then be evaluated for the amount of section loss, available strength (remaining load-carrying capacity), fatigue, and serviceability. The analysis and repair concepts presented in this Section also apply to gouges in flange edges from deck-removal equipment, accidental gouge damage in girders and truss members during construction, or vehicle-impact notches and gouges in member edges without member distortion.

### 7.1.1—Strength Evaluation

An analysis to determine the severity of the saw cuts is required, as each saw cut reduces the cross-sectional area of the top flange. The reduced flange area must be checked for the dead load and design live load stresses acting on the remaining flange area. If the girder will be composite when the deck is recast, the analysis must consider loads acting on both the non-composite and composite sections. Flanges with isolated shallow cuts behave like a flange net section, similar to a flange with a row of bolt holes, and can be evaluated using net section limit states. Flanges with multiple cuts may no longer behave as a net section when grinding is used to provide a smooth transition for cuts and the areas of grinding overlap. In these cases, the capacity should be evaluated based on the reduced flange area and the corresponding reduction in the section modulus.

Each saw cut location should be evaluated individually because the moment demands and maximum stresses vary from location to location. Saw cuts found to require strengthening repairs at the location of maximum moment may not need to be repaired at the locations closer to the inflection point.

#### 7.1.1.1—Compression Flanges

AASHTO BDS allows the use of the plastic moment for the capacity of beams and girders that have compact elements and are sufficiently braced. In order to be compact, a flange must be proportioned to preclude local buckling. In positive moment regions, the girder top flange is in compression. Saw-cut damage is often repaired by grinding smooth transitions, as discussed in Article 7.1.3.

Grinding out saw cuts will reduce the overall thickness of the compression flange, possibly resulting in a non-compact flange and a reduction in the capacity; however, the grinding must take place over a significant length in order to affect the local buckling capacity of the flange. In addition, the top flange of composite girders is often near the neutral axis, so a reduction in flange area will not typically have a noticeable reduction on the capacity of a composite girder. If the remaining section thickness is too thin to accommodate remediation by grinding, then cuts can be repaired by welding (see Article 7.1.3.3) or plating (see Article 7.1.3.2).

#### 7.1.1.2—Tension Flanges

While the concrete deck often provides continuous support for the compression flange in positive moment regions, the bottom (compression) flange in negative moment regions is typically only discretely braced by cross-frames or diaphragms. As a result, the girder capacity may be limited to the yield moment or less, depending on the spacing of the bracing elements.

Top flanges in negative moment regions are in tension and the effects of the saw-cut damage are checked by determining if the reduced cross-section can accommodate the dead load and design live load moment demands, with consideration of composite behavior if necessary. As discussed above, single or widely spaced saw cuts may be evaluated for strength as a localized net section, similar to a row of bolt holes.

Saw-cut damage is often repaired by grinding smooth transitions, as discussed in Article 7.1.3. If the remaining section thickness is too thin to accommodate remediation by grinding, then cuts can be repaired by welding (see Article 7.1.3.3) or plating (see Article 7.1.3.2).

### 7.1.2—Fatigue and Serviceability

Although a saw-cut kerf is not a crack, it is a severe notch in the top flange. Notches in the tension regions of the flange will reduce the fatigue performance due to stress concentrations at the bottom of the notch. Therefore, removal of the notches by grinding or other methods is necessary to provide smooth transitions for the stress flow. Even after the sharp notches are eliminated, the live load stresses on the reduced flange area should be evaluated to confirm they are below the published fatigue stress limits.

For reference, AASHTO BDS specifies that tapered flange thickness transitions at welded joints that are ground smooth are a Category B fatigue detail. Straight longitudinal cuts are parallel to the direction of stress and are, therefore, less sensitive to fatigue. However, notch-like conditions still exist at the ends of longitudinal cuts, or if the cuts meander. Other serviceability checks, such as a deflection analysis, are often not necessary because localized reductions in girder stiffness will not have a significant effect on the maximum deflections.

### 7.1.3—Repair of Saw-Cut Flanges

#### 7.1.3.1—Saw Cut Removal

Saw-cut damage to the heads of bolts or rivets is most easily addressed by removing these connectors and replacing them with high-strength bolts. Similarly, after removing shear studs or connectors with saw-cut damage, the damaged shear connectors may be reinstalled, or the girder may be analyzed with the reduced number of shear connectors. Saw cuts in splice plates are also a common occurrence because these plates project above the girder's top flange. Replacement of a damaged splice plate is often more cost-effective than grinding or plating repairs, which are discussed in Article 7.1.3.2.

Shallow transverse saw cuts in tension flanges found to be adequate for strength and fatigue limit states will require removal of the saw cut to provide a smooth transition and reduce the stress concentrations. The grinding should be performed to provide a taper from the deepest part of the saw cut to the top surface of the flange. In addition, a radiused root (¾ in. or larger) is preferred to transition the slopes on each side of the notch. A minimum 2.5:1 fairing slope, as required in AASHTO/AWS D1.5M/D1.5 for flange thickness transitions, is the steepest slope that should be used to eliminate the notch; however, shallower transition slopes are often required depending on controlling project standards or specifications, or to further reduce stress concentrations. In tension flanges or other fatigue-sensitive cases, a 10:1 fairing slope is often preferred. When a fairing slope of 10:1 cannot be achieved due to interferences with the grinding area, such as shear studs, rivets, bolts, splice plates, or other components, a minimum fairing slope of 5:1 in the longitudinal direction is often acceptable for tension elements (AASHTO/NSBA G14.1). Saw cuts that do not extend the full width of the flange should be faired in the longitudinal and transverse directions. A typical saw cut repair detail is shown in Figure 7-1.

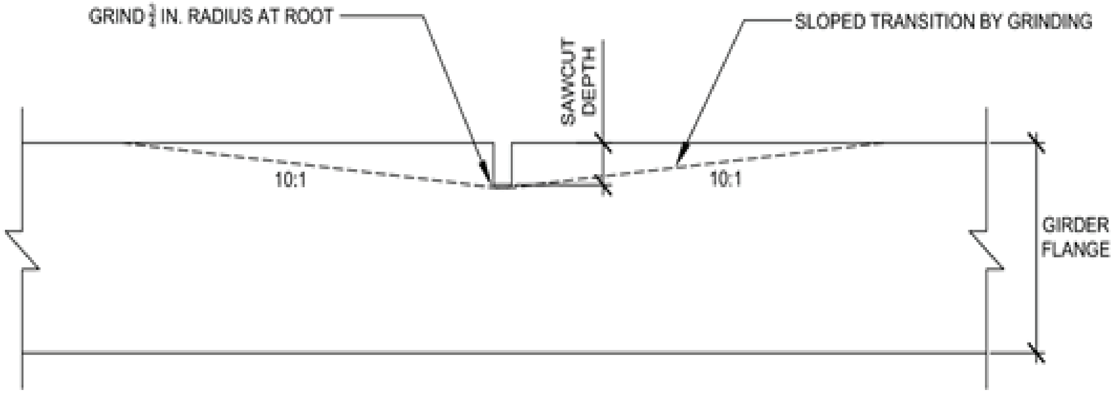


**Figure 7-1—Girder flange elevation showing a typical grinding profile to repair a transverse saw cut**

It is recommended that the saw cuts in tension flanges be ground so that grinding marks from the cutter or abrasive disk are parallel to the direction of primary stress. A surface roughness value, $R_a$, of about 500 µin. on the final surface is recommended. This surface roughness value is achievable using sanding disks (80 to 100 grit) for the final passes. MT is recommended to confirm any notches or crack-like indications are removed. A typical grinding repair for a shallow transverse saw cut is shown in Figure 7-2.

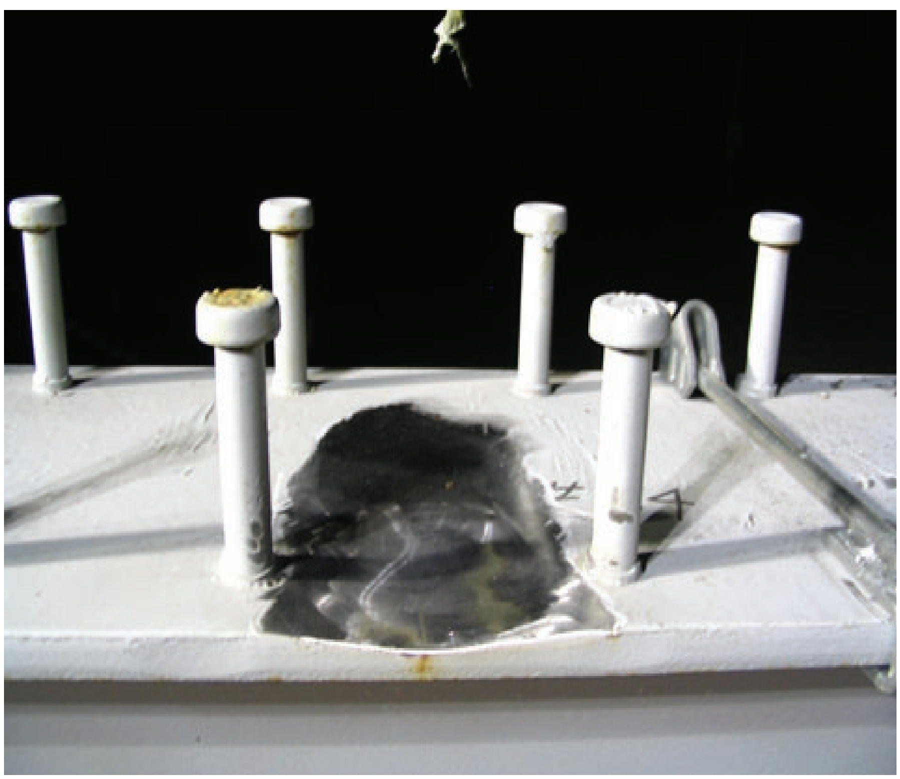

**Figure 7-2—Example of a grinding repair performed on a transverse saw cut**

Transverse saw cuts in compression flanges without stress reversal and away from welded splices are generally not susceptible to fatigue damage, so there is no technical reason to remove the notches. However, they do not represent good workmanship and may be removed on this basis if desired. This removal may be accomplished by grinding in the vicinity of the saw cut to fair, or blend, the notch to the adjacent undamaged steel. A minimum slope of 2.5:1 is recommended for compression flanges, but the slope may need to be reduced in local areas in order to avoid existing shear studs, if present.

The ends of longitudinal cuts in tension flanges are often faired to remove the notch created by the saw cut termination. The need to grind the full length of a longitudinal cut should be evaluated. In curved girders or areas sensitive to fatigue, grinding the full length of a longitudinal saw cut may be necessary. Because a longitudinal saw cut is typically parallel to the direction of stress in straight girders, grinding the full length of the cut to provide a transverse taper is often not necessary. When grinding is required, the fairing slope may vary between 5:1 and 10:1, as discussed earlier in this Section. An example of a longitudinal saw cut is shown in Figure 7-3.

**Figure 7-3—Example of a longitudinal saw cut in a built-up girder flange; arrow indicates a transverse saw cut**

#### 7.1.3.2—Strengthening Plate Repairs

Flanges determined not to have sufficient remaining area for strength or fatigue after removal of gouges or defects will require the flange area to be increased using cover plates or similar methods. In these cases, grinding is still recommended to remove the saw-cut notch. Cover plates add load-carrying capacity by replacing the lost cross-sectional area and must be adequately developed past the location where they are no longer required. A sufficient number of bolts is necessary to develop the required force in the new cover plate beyond where the plate is no longer needed based on the analysis. Bolt spacing may need to be reduced to seal the cover plate to the existing flange in order to help prevent the formation of pack rust in the future. Shim plates may also be needed to make up differences in thickness where grinding is performed. Alternatively, strengthening plates may be installed on the underside of the top flange if there is no interference with transverse stiffeners. Depending on the location of the repair, shear studs may need to be installed on the top flange cover plates to restore composite action, or the cover plate bolts may be used to attach channels or other shear connectors. In addition, new cover plates may project into the bottom surface of the new bridge deck, which may require an adjustment to the fillet heights, changes to the proposed grade elevation, or installing the cover plates on the underside of the girder top flange.

In some instances, the depth of the saw cut may be too deep to practically strengthen the flange with cover plates. For example, if the depth of a single saw cut is near the full thickness of the top flange, it may be desirable to sever the top flange completely and design splice plates to transfer the required force in the now-severed flange. Severing the top flange moves the bottom of the notch into the top of the web, so a large core hole (such as 3-in.- or 4-in.-diameter) would be needed to eliminate the notch in the top of the web and provide a smoother transition of stresses. Severed cover plates in built-up members, such as the example shown in Figure 7-4, can be repaired by replacing the cover plates over a given length and properly splicing the new plates with the existing flange elements.

**Figure 7-4—Saw-cut damage in a built-up top flange causing severing of the top plate and near-full section loss of the second plate from the top**

In many cases, damage to the steel girders during deck demolition is noticed before it gets too severe. In rare cases, repair of multiple deep saw cuts using a flange replacement retrofit over the damaged length could be considered. This would involve removal of the damaged flange and installation of a new top flange using a bolted splice at each end unless field welding is permitted. The new flange plate will also require a bolted or welded connection to the existing girder web plate in order to transfer the horizontal shear between the flange and web. In extreme cases of damage, member replacement may be warranted.

#### 7.1.3.3—Welded Repairs

If a welded repair is pursued, as for any welding, the provisions of AASHTO/AWS D1.5M/D1.5 should be followed. It is recommended that the area to be welded be prepared in a single-U-groove joint configuration similar to those listed in AASHTO/AWS D1.5M/D1.5. For tension areas, the surface of the weld should be ground flush with the base metal surface.

### 7.1.4—Saw Cut Avoidance

While Articles 7.1.1 through 7.1.3 discuss analysis and repair options for saw-cut flanges, it is always best to avoid any saw-cut damage to the flanges during construction. Common measures to prevent saw-cut damage include:

- Review of design drawings to identify and mark the locations of girder splices and changes in flange thickness.
- Use of ground-penetrating radar or through-thickness drilling of the concrete bridge deck to confirm the as-built thickness along the length of the bridge.
- Stopping the transverse saw cuts short of the girder flange.
- Painting the top surface of the bridge deck to indicate removal restrictions, splice locations, flange thickness transitions, and potential areas of reduced deck thickness prior to demolition.

Some Owners provide financial incentives, such as a lump sum payment, to Contractors who take precautionary measures and avoid all damage to the existing structural steel. Other Owners incorporate provisions in the Contract documents to deter Contractors from performing demolition without any precautionary measures, which may include liquidated damages or holding the Contractor responsible for any repair costs and delays.

## 7.2—GOUGES OR DISTORTION RESULTING FROM CONSTRUCTION

Construction of steel superstructures may result in localized scratches, nicks, gouges, and impact distortion in projecting elements of girders or truss members. More detailed discussions related to assessing the damage, calculating the remaining section and capacity, and potential options for repair is presented in Section 5 for impact gouges and distortion and Article 7.1 for saw cuts in girder flanges. The approaches described therein may also be used to address other localized damage from construction. AASHTO/AWS D1.5M/D1.5 includes requirements for how gouges are to be addressed. Because this damage is often caused by impact, MT is recommended to confirm all cracks have been removed from the damaged area. Minor local damage does not usually require a strengthening repair, but an analysis should be performed to confirm the remaining section has adequate strength and serviceability. An example of straightening operations to address flange distortion caused during a rehabilitation project is shown in Figure 7-5.

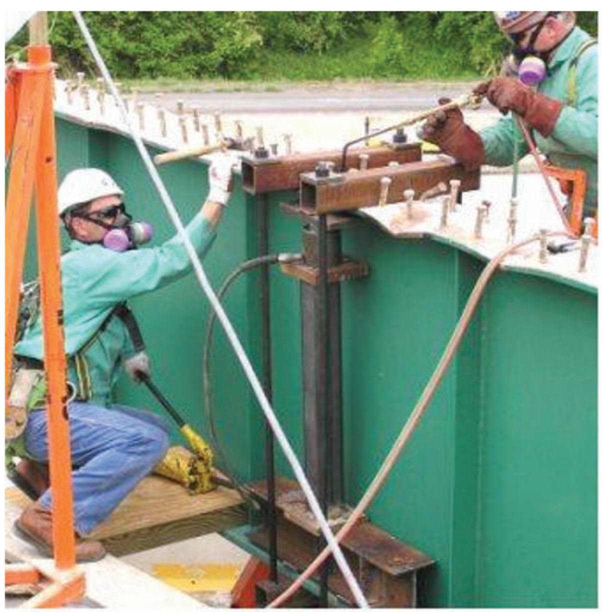

**Figure 7-5—Straightening to Address Flange Distortion**

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