SUGGESTED GUIDELINES

FOR RESOLUTION OF

FABRICATION ERRORS

Using

FIXS - Fabrication error Indexed eXamples and Solutions

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Task Group 5: Repair Database

FIXS Regional Pooled Fund Study

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ABSTRACT

Errors occurring during the fabrication of steel bridges need to be recognized and corrected according to the situation. To arrive at the best solutions, engineers need not only knowledge of material and fabrication specifications but also experience and good understanding of the practical limitations faced by fabricators. This expertise is scattered and varies among individuals.

The "Fabrication error Indexed eXamples and Solutions" (FIXS) research project at the University of Kansas gathers and reconciles this expertise. The project's objective is to make this information readily available at the time and place needed. Both print and web media are being used to meet the project goal of information dissemination. This project has assembled a database of actual cases of errors and their resolutions to provide guidance and improve confidence in solutions to unusual but non-unique problems. A web site < http://www.ceae.ku.edu/fixs.html > is under development to provide access to this case-base, including mechanisms for adding and discussing cases.

This report provides suggested guidelines, in a form intended to assist engineers, inspectors, and fabricators, introducing the issues necessary for good resolutions of fabrication error situations. This document is being submitted to AASHTO/NSBA Steel Bridge Collaboration < http://www.steelbridge.org > for balloting and eventual publication as an SBC document.

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LIST OF ABBREVIATIONS

AASHTO..................... **A**merican **A**ssociation of **S**tate **H**ighway and **T**ransportation **O**fficials
AISC...**A**merican **I**nstitute of **S**teel **C**onstruction
ASNT..**A**merican **S**ociety for **N**ondestructive **T**esting
ASTM.. **A**merican **S**ociety for **T**esting and **M**aterials
AWS .... **A**merican **W**elding **S**ociety
CJP .... **C**omplete **J**oint **P**enetration
CVN...**C**harpy **V**-**N**otch
DOT...**D**epartment **O**f **T**ransportation
FHWA .... **F**ederal **H**igh**W**ay **A**dministration
FIXS ....**F**abrication error **I**ndexed e**X**amples and **S**olutions
HAZ..**H**eat **A**ffected **Z**one
LFD ....**L**oad **F**actor **D**esign
LRFD...**L**oad and **R**esistance **F**actor **D**esign
MT ....**M**agnetic **P**article **T**esting
MTR .... **M**ill **T**est **R**eport
NDE .... **N**on-**D**estructive **E**xamination
NSBA...**N**ational **S**teel **B**ridge **A**lliance
PT .... **L**iquid **P**enetrant **T**esting
QA ....**Q**uality **A**ssurance
QAI...**Q**uality **A**ssurance **I**nspector
QC ....**Q**uality **C**ontrol
QCI...**Q**uality **C**ontrol **I**nspector
RCSC.. **R**esearch **C**ouncil on **S**tructural **C**onnections
RT...**R**adiographic **T**esting
SAW ....**S**ubmerged **A**rc **W**elding
SMAW.. **S**hielded **M**etal **A**rc **W**elding
URL ....**U**niform **R**esource **L**ocator (an Internet address)
UT...**U**ltrasonic **T**esting
VT...**V**isual **T**esting
WPS... **W**elding **P**rocedure **S**pecification

1 - INTRODUCTION

1.1 Suggested Guidelines

Fabrication errors in the bridge industry are seldom identical, but are often similar. These guidelines are intended to assist engineers, inspectors, and fabricators in categorizing situations and determining the best possible solutions, so fabricators may propose established methods and owners may have adequate background information to permit acceptance.

This document covers common error situations. Coverage of each topic begins with a statement of the problem issue, followed by a description of the recommended resolution, and concludes with commentary on the issue and recommendation. If several recommendations are made on a single issue, they are presented in the order of preference.

The "Errant Holes" section covers evaluation of holes drilled in areas that do not comply with the original design, as well as missized or misshaped holes. The "Filling Holes" section covers structural and cosmetic repair of holes and addresses correction of improperly weld-restored holes. The "Stiffeners and Connection Plates" section covers common problems with intermediate stiffeners, bearing stiffeners, and connections plates. The "Miscut Members" section deals with plates, girders, webs, and flanges cut incorrectly or by mistake. The section on "Web or Flange Replacement or Repair" covers web or flange damage, overgrinding, sole plate, and rolling direction corrections. The "Material Defects, Nicks, and Gouges" section covers surface quality, base metal defects, erroneous cuts, gouges, arc strikes, bending and restraint-induced fractures, material substitution, and raw material. Finally, the "Heat Application" section is limited to the application of heat to repair fabrication errors.

1.2 Errors

Errors may include, but are not limited to, material or weld discontinuities, weld flaws or geometry problems, dimensional errors, substandard material, and coating problems. Any deficiency serious enough to cause rejection is a "defect." According to Annex V in D1.5, a defect is "A discontinuity or discontinuities that by nature or accumulated effect (for example, total crack length) render a part or product unable to meet minimum applicable acceptance standards or specifications." If a fabrication error occurs in a minor member, such as a splice plate, unattached connection plate, or cross-frame piece, it is usually most economical to just replace the member. For major members, errors may be economically corrected using the methods described in this document.

Deficiencies may be discovered as the material is being handled, during various stages of the fabrication process, or during loading, shipping, and field erection operations. Some items can only be found after the material is blasted. The surfaces and edges of members and holes should be thoroughly inspected for defects during each step.

The Contractor's QC and the Owner's QA personnel are responsible to identify and document significant deficiencies. S4.1 provides an excellent description of QC/QA duties and activities, in particular Sections 2.4, 9.1, 9.6, and their relevant commentary.

To correct fabrication errors listed throughout this document, the QAI must be involved and notified of all work, so that all appropriate parties are notified. All correspondence should go through the QAI. Written error repair procedures including applicable details, such as welding procedure specifications (WPSs) should be submitted by the Fabricator.

1.3 Nondestructive Examination

Based on the requirements of A 6 and D1.5, repairs to base metal and welds may require a formal repair plan approved by the Engineer or they may be permitted by the mill or shop using standard operating procedures. In any event, the repair process will entail some nondestructive examination (NDE), which in addition to visual testing and inspection (VT) may include magnetic particle (MT), liquid penetrant (PT), ultrasonic (UT), radiographic (RT), or other testing methods to evaluate the defect, its removal, and the subsequent repair. For an overview of nondestructive examination in steel bridge fabrication, refer to Section 6 in D1.5, Volume 1 in the AWS Welding Handbook, and publications by the American Society for Nondestructive Testing (ASNT) <http://www.asnt.org/home.htm>.

1.4 Definitions and Responsibilities

Throughout this document, the terms "Contractor," "Engineer," "Fabricator," and "Owner" are used frequently. The following definitions are in accordance with the AASHTO/NSBA Steel Bridge Collaboration standards:

- **Contractor:** The Contractor is responsible for proper completion of all tasks required by the Contract. Subcontractors, including fabricators, erectors, and field painters, may be used by the Contractor, but the Contractor retains responsibility for all material, operations, and the final product. The Contractor should permit direct subcontractor interaction with the Owner to expedite the project, but subcontractors must inform the Contractor of any proposed modifications to Contract requirements accepted by the Owner.

- **Engineer:** In this document, the Engineer is the Owner's authorized representative, responsible for monitoring of the Fabricator's work, and other duties and responsibilities detailed by the Owner. The Engineer has the authority to allow exceptions to Contract document requirements.
- **Fabricator:** In this document, "Fabricator" refers to the facility or facilities performing such shop activities as cutting, welding, drilling, punching, cleaning, and painting of structural steel. "Fabricator" also includes any agents of the Fabricator, such as those who prepare shop detail drawings. In some cases the Fabricator may also be the Contractor, but usually the Fabricator is a subcontractor.
- **Owner:** In this document, "Owner" refers to the entity paying the Contractor to fulfill the terms of the Contract. The Owner also encompasses the following: those preparing the Contract documents, including those responsible for the structure's adequate design; and those authorized to represent the Owner during construction, commonly called the "Engineer" and the "Inspector". The Engineer and Inspector may be employees either of the Owner or of professional firms contracted for the work.

1.5 Standard Units of Measurement

This document makes use of both U.S. Customary Units and the International System of Units (SI). The default system for this document is U.S. units with SI units shown within brackets [ ]. The measurements may not be exact equivalents, so each system shall be used independently of the other.

2 - ERRANT HOLES

Holes are sometimes drilled too close to the ends or edges of flanges, splice plates, or adjacent holes. These errors can occur due to shifted templates or layout dimension errors. Errant holes will normally need to be filled or covered (see Section 3, “Filling Holes”).

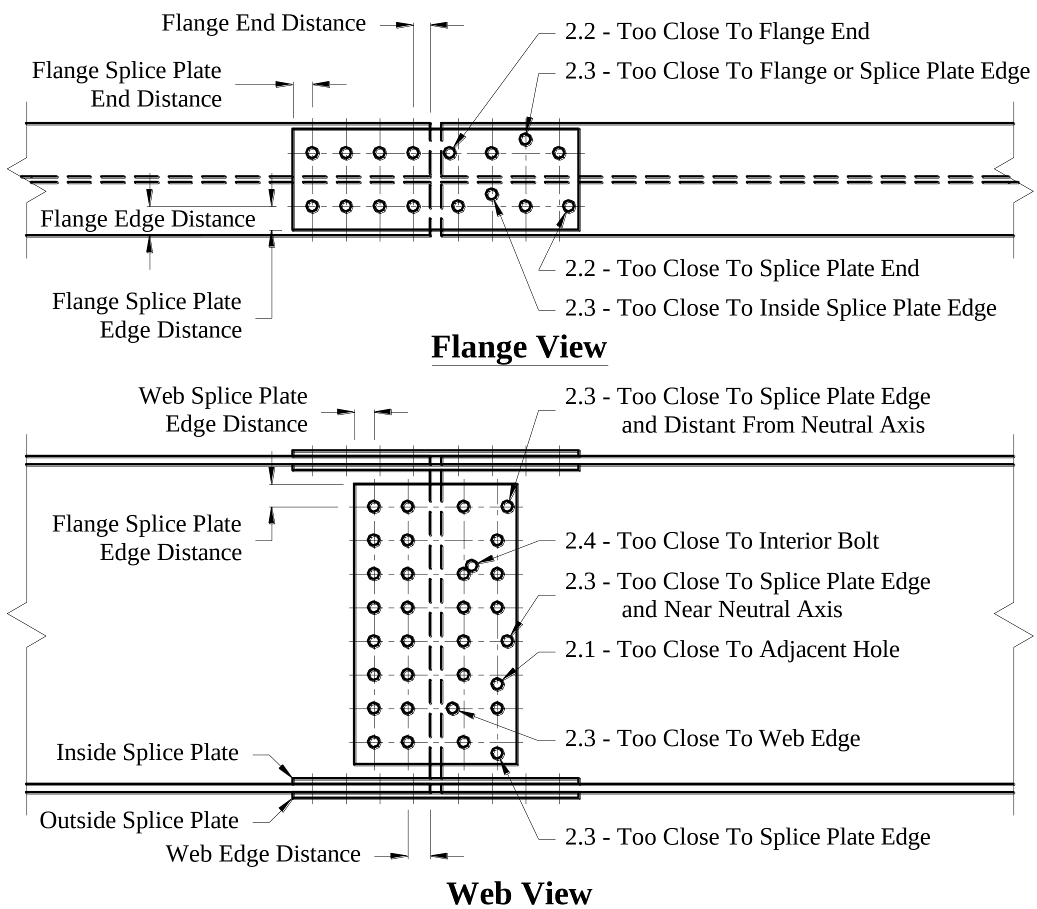

Figure 1: Errant Holes At A Splice

Depending on the design criteria used for the structure, refer to Section 6.13.2.6 in the AASHTO LRFD Bridge Design Specifications for fastener spacing, edge distance, and/or end distance requirements or Section 10.24.5 - .7 in Division I of the AASHTO Standard

Specifications for Highway Bridges (for LFD). The LRFD code closely parallels AISC limitations and distinguishes between end and edge distances but does not permit reducing clearances for planed edges. Conversely, the LFD code allows reduced edge or end distances for milling thermal cut faces, but it has more conservative minimums. When designing with the LRFD code, edge and end distances specified should be about 1/4" [6 mm] greater than the minimum clearances prescribed, allowing some fabrication tolerance.

If the errant hole location is close to another bolt, installation and tightening might be difficult or impossible. The clearance requirements for high-strength bolts are given in Tables 7-3a and 7-3b in AISC's LRFD Manual of Steel Construction, 3rd Edition.

2.1 Too Close to Adjacent Hole

Issue:

An incorrectly drilled hole is too close to an adjacent hole.

Recommendation:

Use Section 5.3 in the RCSC Specification to calculate the reduced bolt bearing strength. If the calculated strength is adequate, and the bolt can be correctly installed to perform as slip-critical, then the inadequate hole spacing may be accepted "as is." The hole must either be covered (i.e., by design material or a plate washer) or filled to exclude water. When a fill is not structurally required, a cosmetic plug may be installed per Section 3.2, "Cosmetic Repair of Errant Holes Using Steel Pins," or Section 3.3, "Cosmetic Repair of Errant Holes Using Molten Zinc."

If it is determined that the "as is" condition is not acceptable, then consider variations of the other fixes proposed within this subsection.

Commentary:

Connections in steel bridges are normally specified as slip-critical. In this type of connection, the prevention of slip in the service load range is the limit state. Since the load transfer mechanism is friction on the faying surfaces, it is a design assumption that the slip resistance at each fastener is equal and additive with that at the other fasteners. Therefore, all locations must develop the slip force before a total joint slip can occur at that plane. However, although a slip-critical connection is designed to not slip into bearing under service loads, the connection must also meet the bearing requirements. This results in a final connection that does not slip under service loads, but also performs in bearing under extreme loads.

It is only for the bearing load transfer mechanism that the hole spacing is treated as a direct design parameter. The bearing strength is a function of the hole spacing. Inadequate hole spacing reduces the bearing strength.

For the friction load transfer mechanism, the clamped area of the plates in contact around the bolt must provide for friction load transfer. There must be enough room to correctly install the bolt.

See Section 2.4, "Interior Bolt Within A Splice," covering similar errors.

2.2 Too Close to End

Issue:

A hole is drilled closer to the end of a load-carrying connection than permitted by the applicable design specifications. This applies to the end of a flange or flange splice plate.

Recommendation:

If bolt placement is based on LFD criteria (see the second introductory paragraph in Section 2, "Errant Holes"), for errors up to 1/8" [3 mm] grind the end of the plate to allow a smaller end distance than for a flame-cut edge.

Errors reducing clearance below LRFD-specified minimums cannot be corrected with grinding, so then, although a bolt must be placed in the errant hole, neglect it in calculating the splice's total capacity. Inserting a bolt in the errant hole will maintain the sealing pitch, exclude debris, and avoid confusion on future inspections.

If neglecting the bolt makes the splice inadequate: for a flange end error, provide a sufficiently longer splice plate to add one row of bolts (see Figure 2); for a flange splice error, provide a sufficiently longer splice plate for all designed holes to be used.

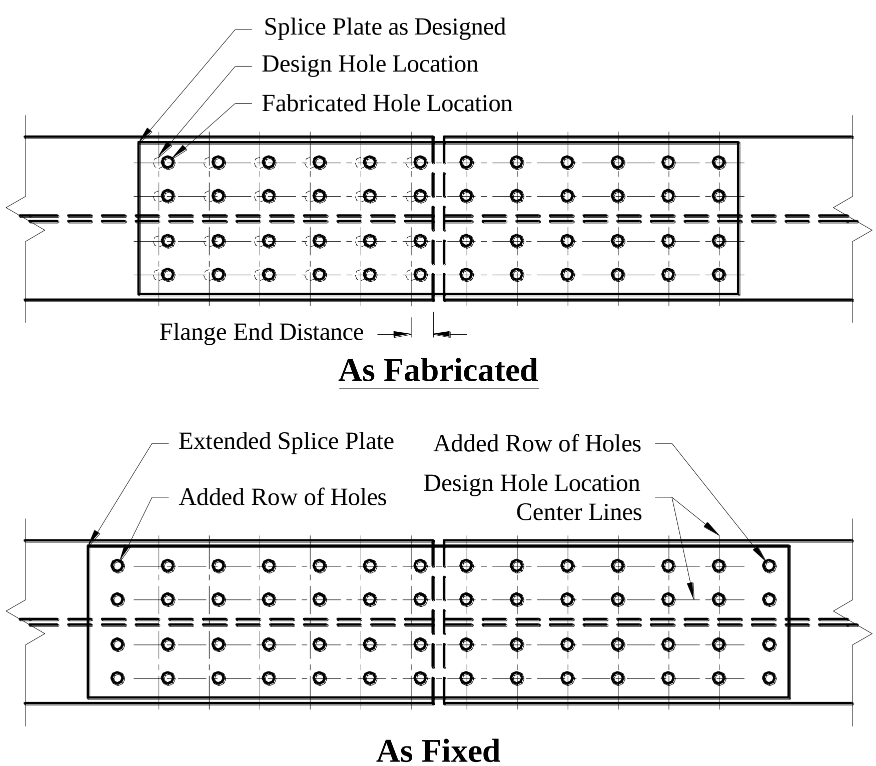

Figure 2: Too Close to End of Flange

Commentary:

An "end" is perpendicular to the direction of primary force in a bolt. Theoretically, load transfers through friction along the contact length between the flange and the splice plate, so there is minimal stress remaining in the flange or the splice plate at their respective ends. See the commentary in Section 2.3, "Too Close to Edge," for why webs do not have any "ends" at a splice.

2.3 Too Close to Edge

Issue:

A hole is drilled closer to the edge of a load-carrying connection than permitted by the applicable design specifications. This applies to the longitudinal edges of flanges and flange splice plates, the vertical edge of webs, the perimeter of web splice plates, and bracing connections (e.g., cross-frames, diaphragms, lateral bracing, etc.).

Recommendation:

Refer to the second introductory paragraph in Section 2, "Errant Holes," concerning LRFD and LFD criteria. For LRFD, if edge clearances do not satisfy Section 6.3.2.6 of the AASHTO LRFD Bridge Design Specifications, the connection must be enlarged to provide adequate clearances. For LFD, the provisions of Section 2.2, "Too Close to End," also apply to edges. This either permits inclusion of all designed bolts, or allows sufficient bolts to be added. For inadequate clearance at web edges, adding a row of bolts to only one side of a splice increases eccentricity of the bolt group, requiring redesign. Adding a row of bolts to both sides of the splice avoids this increase in eccentricity. For flanges, if there is less than one bolt diameter between the edge of the hole and the edge of the flange, then insert bolts to fill the holes (see Section 3.1, “Bolted Repair of Errant Holes”), but lengthen the splice in order to add the required number of extra bolts or, if the situation permits, add a row of bolts to each side of the unmodified splice (see Figure 3).

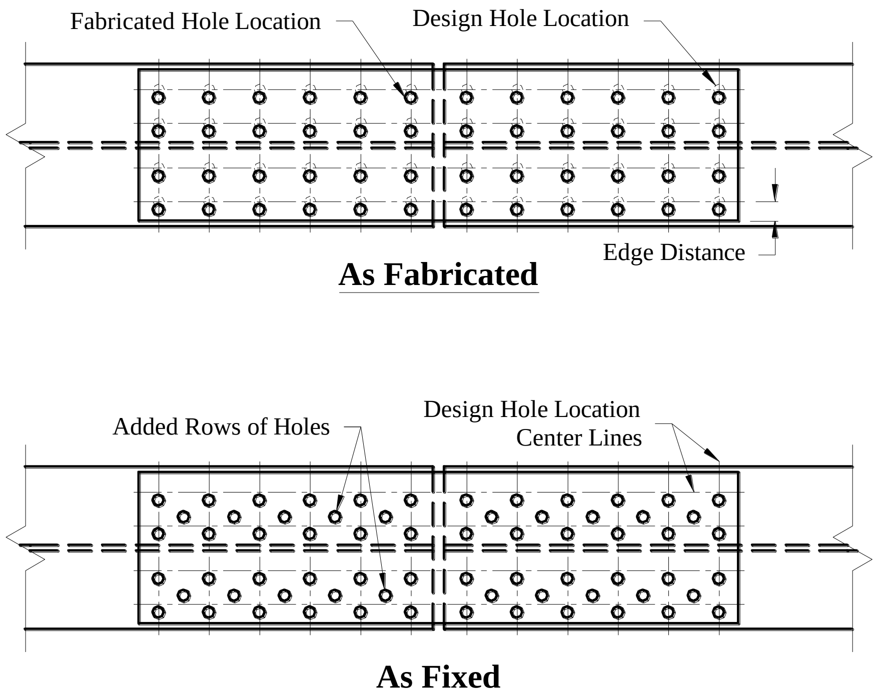

Figure 3: Too Close to Edge of Flange

A hole breaking the edge of the flange or splice plate creates a severe stress concentration. The splice plate must be replaced unless the defect is at the end bolt row on the splice and the bolt can be ignored, in which case the Engineer may determine acceptability. If the flange edge is broken, further evaluation will be necessary, based on the defect’s depth and location. Similarly, if the edge of a flange or splice plate is under the contact area of the washer, documentation shall be submitted to the Engineer to determine if it is necessary to replace the splice plate, add bolts to fully develop the splice, repair the base metal, or trim the washer.

Commentary:

An “edge” is parallel to the direction of primary force in a bolt. In a web splice, most bolts primarily carry shear, but the bolts furthest from the neutral axis are considered to primarily carry longitudinal force. Therefore, a web splice plate’s perimeter consists of all “edges” and no “ends”.

For holes too close to the edge at a web splice, the position of the errant hole, relative to the neutral axis, is significant. Bolts near the neutral axis have the least amount of the splice capacity and in some cases may be acceptable to leave “as is.” For errant holes distant from the neutral axis, the affected bolt contributes significantly to the moment of inertia of the bolt group. Therefore, the need to add bolts within the splice is more likely. Cases with bolt holes too close to the edge of web splice plates need to be evaluated by the Engineer on a case-by-case basis. Factors include, but are not limited to, composite or non-composite neutral axis and location with respect to the applicable neutral axis.

2.4 Interior Bolt Within A Splice

Issue:

A hole is drilled incorrectly within a splice, but does not violate edge or end distance requirements.

Recommendation:

Check if the spacing still satisfies applicable code requirements for minimum and maximum spacing, and if so, leave “as is.”

If it is too close to an adjacent bolt, but there is sufficient room to install the bolt, consider the amount by which the code minimum spacing is violated. If the violation is less than or equal to 3/8" [10 mm], then fill the hole with a bolt (see Section 3.1, “Bolted Repair of Errant Holes”) and neglect the error. If the violation is more than 3/8" [10 mm], then install a bolt, but evaluate the splice without the bolt to see if an additional bolt is needed and if sufficient room (i.e., additional hole will not cause any new spacing violations) exists for an additional bolt. If an additional bolt is installed within one bolt space of the errant hole, the connection bolts do not need to be re-evaluated.

If the hole is too close to another fastener to install a bolt, consider leaving the hole unfilled and adding a bolt as close as possible to the correct location. If the errant and correct holes slightly overlap (e.g., look like a snowman), but the splice plates will cover the extra hole space like a hardened washer, then the hole may be ignored. If two holes overlap more than 50% they constitute an oversize or slotted hole and must be evaluated based on their design load. If the calculated load exceeds the allowable, another bolt must be added.

Commentary:

Shops usually drill a few holes in the web and flanges and corresponding ones in the splice plates to position splice plates for drilling the remaining holes. Locating these holes in the splice plates is straightforward, based on the pattern, but positioning holes in the web or flange are sometimes incorrectly located. Rather than intentionally making a matching errant hole in the adjacent splice plate, it may be better to leave the errant hole unused, especially if splice plates enclose the hole. This is common in webs, but flanges may or may not have inside splice plates. For an errant hole in a flange with only outside splice plates, if it can be left unused, the Owner and/or Contractor may want the hole filled for cosmetic reasons (see Sections 3.2, “Cosmetic Repair of Errant Holes Using Steel Pins,” and 3.3, “Cosmetic Repair of Errant Holes Using Molten Zinc”) or to prevent confusion in the field.

See Section 2.1, “Too Close to Adjacent Hole,” covering similar errors.

2.5 In Web Near Abutment

Issue:

Missized or mislocated holes occur at the integral or semi-integral abutment end of a member's web. The holes are to be used to allow the passage of the abutment's reinforcing steel through the member.

Recommendation:

If the holes are undersized to allow passage of reinforcing bars, enlarge the holes.

If the holes are mislocated one hole diameter or less, extend the holes towards their intended locations, making them slotted (see Figure 4).

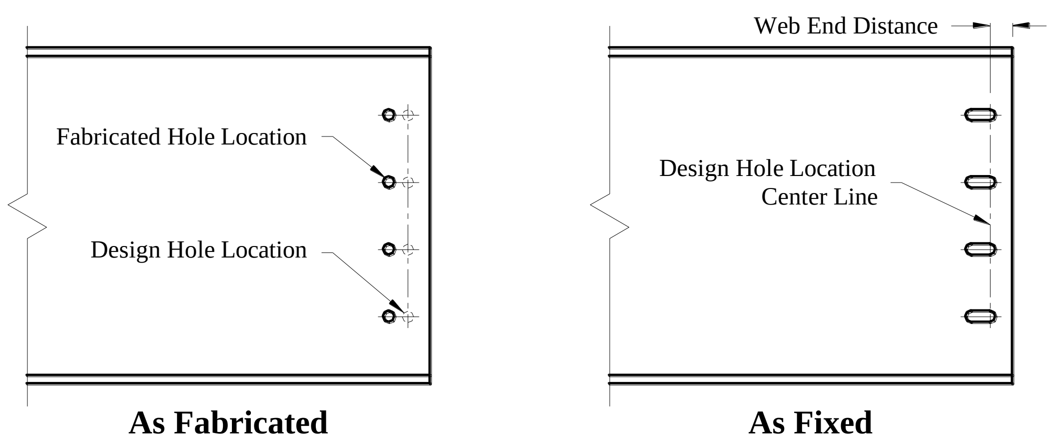

Figure 4: Slotted Holes In Web Near Abutment

If the holes are mislocated more than one diameter, then leave the holes "as is", fill with tightened bolts, and provide new holes at the proper locations.

Another instance of errant holes near an abutment could result from splice plate holes being drilled at the wrong end or mislocated diaphragm connection holes. For such cases, the holes may be filled with fully-tightened bolts to restore the full vertical shear capacity of the section. If the additional holes/bolts would cause an aesthetic problem at a highly visible location, the holes could be filled non-structurally (see Sections 3.2, “Cosmetic Repair of Errant Holes Using Steel Pins,” and 3.3, “Cosmetic Repair of Errant Holes Using Molten Zinc”). End bracing connections must be considered with such retrofits.

Commentary:

A common problem is a hole being too small, due to overlooking the bar diameter including deformations, the effects of abutment skew with holes drilled perpendicular, and physical limitations of threading heavy, long bars through holes in successive beams.

2.6 Elongated and Oversize Holes

Issue:

A hole detailed as standard size (see Table 6.13.2.4.2-1 in LRFD or Table 10.24.2 in LFD) is incorrectly drilled, resulting in an oversize, misshaped, or elongated hole.

Recommendation:

For most cases where the oversize dimension is not severe, it should be acceptable to leave the hole “as is,” but a hardened washer must cover any exposed non-standard hole, as required by the RCSC Specification.

If there are a number of non-conforming holes at a single splice location, consider changing all bolts at that location to one larger size if edge and end distances permit. If there is poor hole quality and inadequate edge/end distance, then that splice may need to be re-designed (e.g., lengthen the flange splice and/or add rows of bolts to a web splice, considering the effect of oversize holes on capacity) or a portion of the member may have to be replaced. Replacement may entail relocating the splice or removing and replacing part of the member, but these are last-choice options due to the potential for weld defects. Costs of additional material, increased erection labor, and engineering to design and verify alternates are the Contractor's responsibility.

Commentary:

Avoid changing to specialty bolts and washers only for certain holes. Reaming or re-drilling the hole as necessary and increasing the bolt one size (e.g., 3/4" to 7/8") at those locations, informing the erector, and marking the locations of the larger bolt(s) clearly for field personnel may seem like a reasonable approach. Additional costs for field installation of the larger bolts (e.g., different air and torque wrenches, additional bolt lots to test and calibrate, etc.) are the Fabricator's responsibility. This approach has a high potential for field installation errors and, therefore, is NOT RECOMMENDED.

2.7 Partially Drilled Holes

Issue:

A hole is partially drilled in the wrong location.

Recommendation:

Sometimes a hole is initiated into a member in a wrong location. If the hole penetrates less than 1/8 the material's thickness, it can be fixed by feathering out at a 10:1 slope (see Figure 5).

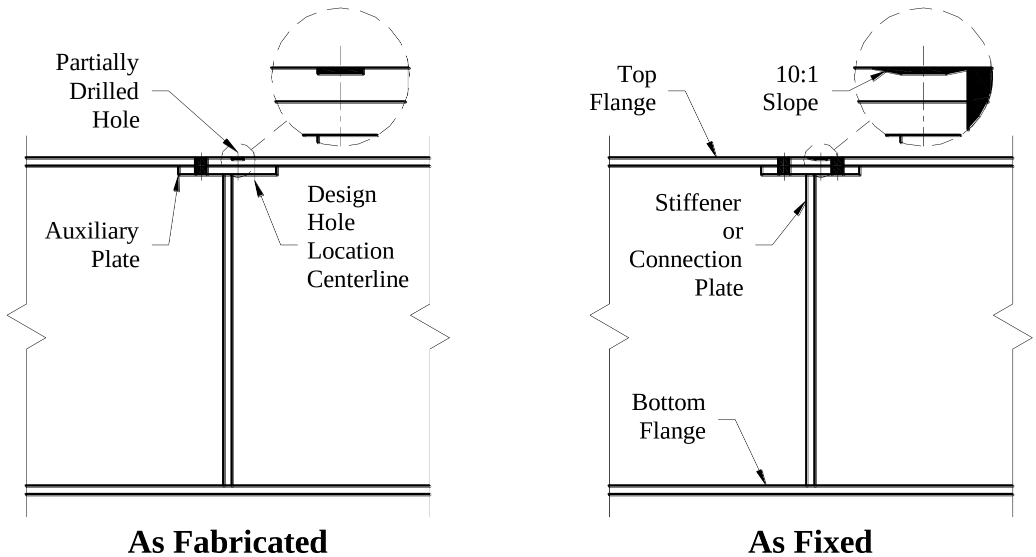

Figure 5: Partially Drilled Hole

Commentary:

This repair approach is only valid for isolated, shallow, partial holes. If there are several neighboring errors, the feathering approach cannot be used since it would interfere with the contact areas of adjacent bolts. If the hole is deeper and the situation permits, it may be preferable to drill the hole completely and fill it with a bolt (see Section 3.1, "Bolted Repair of Errant Holes"). However, this may affect the design and must be pre-approved by the Engineer.

3 - FILLING HOLES

Leaving unfilled holes is confusing to erection personnel and disconcerting to the public. If holes will be covered (e.g., by a diaphragm connection angle), there is no need to fill the hole. Even if the hole will be exposed after erection, if it will not be readily visible to the public or confuse the erector, it may remain unfilled, but for a painted structure, the inside (perimeter) of the hole must then be painted. However, in high moisture climates the hole may need to be sealed to prevent main member deterioration due to moisture between adjacent members.

3.1 Bolted Repair of Errant Holes

Issue:

Bolt holes were drilled out of alignment or by mistake.

Recommendation:

For primary members, fill the hole with a high-strength, torqued bolt when adequate clearance exists. For secondary members (i.e., not designed for carrying load), A307 bolts may be used unless weathering properties are needed. For unpainted situations (e.g., AASHTO M270 / ASTM A709 Gr. 50W [345] or Gr. HPS70W [485W]), AASHTO M164 / ASTM A325 Type 3 bolts must be used for all locations. For painted or metalized structures, bolt surface treatment should be similar to that specified for permanent bolts, but substituting hot-dip galvanizing for mechanical galvanizing should be permitted.

For round holes, the bolt may be either 1/16" [2 mm] smaller than the hole without washers or 3/16" [5 mm] smaller than the hole with appropriate hardened washers under the head and nut. (For slotted holes, see the RCSC Specification requirements.)

Commentary:

Filling holes with high-strength, torqued bolts is cosmetic, but also provides the structural benefit of pre-compressing the edge of the extraneous hole so that fatigue cracking is discouraged.

See Sections 3.4, "Welded Repair of Errant Holes," and 3.5, "Correcting Weld-Restored Holes."

3.2 Cosmetic Repair of Errant Holes Using Steel Pins

Issue:

An errant hole will be objectionably visible in the final structure, or may cause confusion during erection, but stresses do not require solid metal and utilization or proximity of other connections precludes filling it with a bolt.

Recommendation:

Fill the errant hole with a steel pin secured by brazing, epoxy, or other non-stress-concentrating methods. Then grind flush.

Commentary:

Welding the steel pin in place is NOT RECOMMENDED. Welding a plug into the hole is unacceptable from a fatigue standpoint. Installing a pin with very small welds may result in high residual stresses and defects that could initiate cracking in otherwise lightly loaded members. Web and bracing members are seldom more than 1/2" [12 mm] thick, so the D1.5 1/4" [6 mm] minimum weld size from both sides would result in a full welded repair.

See Section 3.3 for an alternate cosmetic approach.

3.3 Cosmetic Repair of Errant Holes Using Molten Zinc

Issue:

An errant hole will be objectionably visible in the final structure, or may cause confusion during erection, but stresses do not require solid metal and utilization or proximity of other connections precludes filling it with a bolt.

Recommendation:

If the Owner or the Fabricator prefer, a plug of molten zinc may be used instead of the steel pin approach in Section 3.2. This may be preferred since there will be no question about the type or setting time for epoxy, qualifications required for brazing, or possible welding problems. The zinc will also provide corrosion protection.

The first step is to make 45º chamfers, 1/8" [3 mm] deep into the plate on each side (see Figure 6). Then clean and coat the inside of the hole with a zinc-rich primer or a flux that will permit adhesion. Next, preheat the member and backing to 200º F [90º C] and fill the hole with molten zinc. Finally, remove the backing plate, grind flush, and clean and coat the area.

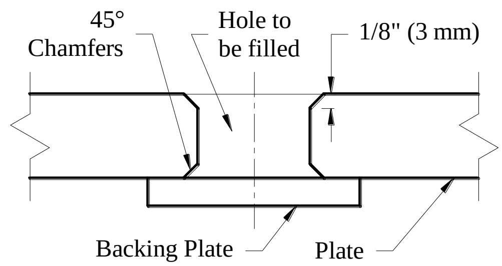

Figure 6: Molten Zinc Repair

Commentary:

This repair approach is not valid for errant holes in weathering steel.

Filling a hole with molten zinc is a viable fix for some owners. This type of repair is especially effective when a plate partially covers the errant hole, as it will seal the joint against water.

After painting, it is difficult to tell where the misdrilled hole was. Also, grinding and blast-cleaning are unlikely to dislodge the plug.

See Section 3.2 for an alternate cosmetic approach.

3.4 Welded Repair of Errant Holes

Issue:

High design stresses, coupled with adjacent holes within one bolt diameter, welds within one inch [25 mm], insufficient clearance to install a high-strength bolt (see the third introductory paragraph of Section 2, “Errant Holes”), or other considerations may dictate that an errant hole must be repaired by restoring the solid metal.

Recommendation:

Insert a steel fill pin halfway into the errant hole. Prep the transition into/out of the hole by grinding or gouging, and weld with stringer passes. Go to the second side of the hole and grind or gouge out the fill pin to sound weld metal and weld with stringer passes (see Figure 7). Grind both sides flush and UT and/or RT. (See Section C3.7.7 in D1.5.)

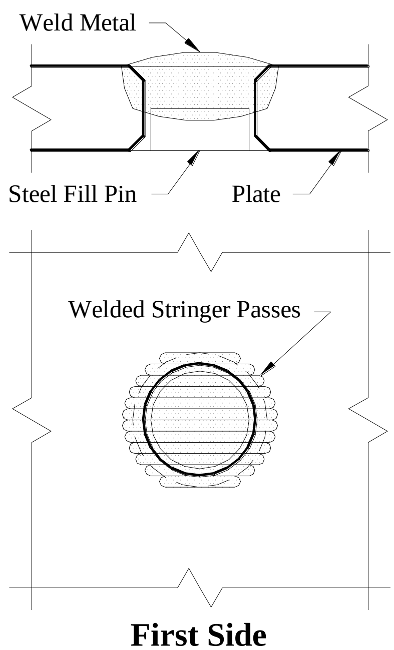

Figure 7: Welding Stringer Passes Repair

The recommended weld repair procedure uses parallel stringer passes to avoid trapping slag or high residual stresses. Although removed during the second side weld, the steel fill pin should be similar to the material being repaired for equivalent weldability and chemistry. It can be a steel rod or even a punch-out, but its diameter should be within 1/8" [3 mm] (1/16" [2 mm] is preferred) of the hole size for weld continuity. The fill may be tacked inside the hole on the first side, since that tack will be subsequently consumed or removed. Tack welds should not be allowed on the surface of the second side. Gouging and grinding should provide a smooth transition into and out of the original hole so the weld can be continuous. Slopes should be about 1:6 for shallow excavations (i.e., up to approximately 1/4" [6 mm]), 1:4 from 1/4" [6 mm] to 1/2" [12 mm] material, and 1:2 for over 1/2" [12 mm]. Welds should continue onto the surface before terminating.

Commentary:

Although sometimes mistakenly referred to as "plug welding," properly executed welded repairs of holes can restore the full section of the member. Plug welding is discouraged by D1.5, and not permitted in tension or stress reversal areas. Plug welds usually start around the perimeter of the hole and spiral to the center, with either backing or another member behind the hole. The weld is made quickly so the adjacent weld more easily melts slag, but slag inclusions are common. The greatest problem is the weld shrinkage during solidification and cooling, generating extremely high residual stresses at the center of the plug, which is the last to solidify. This results in micro-cracks in the initial weld, coupled with near yield point residual stresses that may initiate cracking due to applied stresses on the structure that are far less than the predicted fatigue limit.

For high-stress situations or on thick material (i.e., over 2" [50 mm]), thermal stress relief may be necessary, and the Engineer and Fabricator should agree on this before initiating the repair, so that stress relief can immediately follow welding.

UT is the preferred method to assess full-thickness weld repairs, but MT may be used with the Engineer's concurrence on thin material (i.e., up to 1/4" [6 mm]) due to limitations on those NDE systems.

3.5 Correcting Weld-Restored Holes

Issue:

An errant hole was repaired improperly by welding. Either the repair weld is defective or a welded repair was made in an inappropriate location.

Recommendation:

Determine whether non-defective portions of the weld can remain, since removal may cause additional problems. This will depend on the weld procedure employed, the location, and anticipated service load conditions.

If the entire weld is considered deficient, it must be removed. For an improper welded hole restoration, drill out the entire weld and inspect the area. The drill bit must be slightly (i.e., 1/16" [2 mm]) larger than and centered on the original hole's location to remove most of the heat affected zone (HAZ) and potential micro-cracks. The remaining area should be inspected with MT. If possible, install a high-strength bolt per Section 3.1, "Bolted Repair of Errant Holes." If installation of a bolt is not possible, a welded restoration is required per Section 3.4, "Welded Repair of Errant Holes."

If a welded restoration has already been attempted (per Section 3.4) and is unacceptable due to weld quality, method, or location, full removal of the weld would result in a hole much larger than the original. If there are weld defects but the weld procedure is acceptable, either weld-repair just the defect(s), or drill out the defect(s) and install a bolt. If the location or weld method (e.g., wrong consumables, non-qualified procedure or welder, etc.) dictate complete removal, the Fabricator should submit a comprehensive proposal for the Engineer's review. This may entail gouging out the entire weld and rewelding with a custom procedure and/or specialized NDE.

Other options, usually less desirable to both the Engineer and the Fabricator, may be considered, but their effect on the service life of the structure (including inspectability and maintenance) must be considered:

1. When stresses do not require solid metal restoration of the hole, drill out and inspect the welded area as described above. Cosmetically fill the hole using methods described earlier in Sections 3.2 and 3.3.
2. Leave the weld repair in place and cover it with a bolted connection capable of carrying the full load on the member if the weld caused fracture. For example, sandwich the weld-repaired plate between new plates and bolt similar to a splice to provide an adequate load path across the unacceptable weld repair.
3. Stress relief (e.g., thermal, vibratory, ultrasonic, or other methods).
4. Removal and replacement of the portion of the member (see Section 6, "Web or Flange Replacement or Repair").

Commentary:

The entire weld would be considered deficient in cases including, but not limited to, improper welded hole restoration or improper weld consumables. Improper welded hole restoration may result in many weld defects, such as lack of fusion, if a fill is not completely removed before the weld on the second side. This problem may arise for repairs in thick plates where fills are large and, therefore, excavations deep. Using SMAW with low heat input during repair welding may also cause lack of fusion due to insufficient heat penetration.

Plates may not be welded on for Option 2, since the fracture could then propagate through the welds.

Application of Option 3, stress relief in lieu of a repair, is limited, as it is difficult to quantitatively verify results, and unwanted distortion is possible.

Option 4 above, involving replacement of material, is not advisable for rolled beams and should be avoided for all members if at all possible. However, this may be the only feasible approach when multiple unacceptable weld restorations occur in one area.

4 - STIFFENERS AND CONNECTION PLATES

Strictly speaking, stiffeners and connection plates are different in terms of their structural purposes, but the same plate can fulfill both functions. Recent changes in design requirements create additional possibilities for errors with stiffeners and connection plates.

AASHTO requires cross-frame and diaphragm connection plates to be positively connected to both flanges to prevent distortion-induced fatigue. Section 6.10.8.1.1 in the AASHTO LRFD Bridge Design Specifications specifies: "Stiffeners used as connecting plates for diaphragms or cross-frames shall be connected by welding or bolting to both flanges." This is also specified in Sections 10.34.4.6 and 10.34.4.9 of the AASHTO Standard Specifications for Highway Bridges: "…transverse stiffeners which connect diaphragms or cross-frames to the beam or girder shall be rigidly connected to both the top and bottom flanges." State details vary on welding or bolting the connection plate to the tension flange, so contract documents govern.

Connection plate attachment to the compression flange is generally by welding, unless there are special considerations for erection stresses or other unusual conditions.

Transverse intermediate stiffeners not used as connection plates are tight fit or connected to the compression flange, but need not be in contact or connected to the tension flange.

The most common types of errors related to stiffeners or connection plates are improper flange attachment, mislocation on the member, and incorrect hole placement. Repairs vary, depending on the error and/or whether a stiffener or a connection plate is involved. The stage of fabrication completion may also affect decisions about repairs.

4.1 Erroneously Welding To A Tension Area

Issue:

A connection plate or stiffener is welded to a tension flange where welding was not specified.

Recommendation:

If a connection plate or stiffener is welded to the tension flange instead of the compression flange, consider leaving the tension flange welds and, if a connection plate, welding to the compression flange as well. MT the tension flange welds.

Depending on the structural situation and extent of the welding done, the QAI may require the Fabricator to remove the weld and, if called for on the shop drawings, install a bolted connection to the tension flange.

If removing the weld is not possible due to other factors, then the Fabricator must propose an alternative solution to the Engineer.

Commentary:

If the welds are of good quality and the fatigue range is below the limit set for Category C, the Owner should consider allowing the errant weld(s) to remain. This avoids potential problems caused by gouging and grinding out a weld, a process that still leaves some weld effects (e.g., fusion and HAZs) just below the flange surface based on weld penetration.

Welds to the tension flange may be prohibited by owner policy, regardless of service stresses, or may be prohibited due to fatigue stresses exceeding those permitted by AASHTO for Category C (flange-to-web = Category B, stiffener or connection plate to flange and web = Category C). If welds are prohibited due to stress range, especially on a fracture-critical flange, remove in accordance with Section 4.10, "Plate Removal." Removal must go slightly below the flange surface and the area should subsequently be examined by MT. Hardness testing may also be needed if the HAZ must be removed.

4.2 Mislocated Plate Tacked In Place

Issue:

A stiffener or connection plate has been tack-welded at the wrong location.

Recommendation:

If an error is noticed before the plate has been completely welded, remove the plate and place it in the correct location. Completely remove the tack welds. Grind them out approximately 1/16" [2 mm] below the base metal surface, and MT each removal area. If steel with a yield strength of 100 ksi [690 MPa] is present, perform hardness testing to insure sufficient removal, so the remaining weld area HAZ has approximately the same hardness as the adjacent base metal surface. Brinell, Rockwell, or Vickers hardness testing may be used.

If the stiffener has been fully welded, follow the procedure in Section 4.10, “Plate Removal.”

Commentary:

Tack welds must be completely removed because they typically do not meet quality standards for permanent welds. The HAZ is more pronounced in tack welds due to external, rapid cooling and solidification. This rapid solidification can lead to crater cracking at the center of the weld, which solidifies last. Other defects are common with tack welds, including, but not limited to, arc strikes (see Section 7.5, “Arc Strikes”), inclusions, lack of fusion, and roll-over.

When removing tack welds, grinding is preferable to gouging. Grinding avoids both excessive removal of material and additional heat input. Small die grinders are used for final finishing and severing of the weld at the root. The majority of the weld must be severed by grinding. Do not try to break tacks by prying plates apart since cracks may initiate in the base metal.

4.3 Mislocated Intermediate Stiffeners

Issue:

An intermediate stiffener is fitted at the wrong location.

Recommendation:

If the stiffener at the present location does not interfere with any other members or attachments, leaving it in place is preferable to removal. If the present location is more than 12 inches [300 mm] from the specified location, add an additional stiffener at the specified location. If the present location is less than 12 inches [300 mm] from the specified location, an additional stiffener may not be required. If the location of the stiffener exceeds the allowable stiffener spacing and calculated stresses, an additional stiffener may be added to reduce the spacing and meet design requirements.

If the stiffener at the present location interferes with other members or attachments, minimize the amount of stiffener removal required to provide necessary clearance.

Commentary:

Intermediate stiffeners not exactly located as shown on the shop drawings may not present serious problems and may usually be left in place. Leaving the stiffener in place is better than removal, which is a time-consuming task that risks damaging the girder. Partial removal must be done carefully to avoid damaging the web. If weld removal extends into portions that will remain, the QCI and the QAI must determine if and/or how the weld shall be restored.

4.4 Misaligned and Misproportioned Bearing Stiffeners

Issue:

A bearing stiffener is welded at a different alignment than shown on the shop drawings or not directly over the final bearing centerline. This may include out of plumb, not normal to the flange, or not perpendicular to the web.

Recommendation:

Check the alignment of the bearing stiffener to determine the location of the bottom of the stiffener and the misalignment, as a percentage of its height.

If the bottom location of the bearing stiffener is within the middle 50% of the sole plate or bearing contact area (see Figure 8) and if the misalignment is less than 5%, then consider leaving the bearing stiffener "as is." If other members connect to the stiffener, such as diaphragms or cross-frames, its vertical and lateral (i.e., non-perpendicularity to the web) misalignment must permit a positive connection by the use of shims or other means.

If the bottom location of the bearing stiffener is within the middle 50% of the sole plate or bearing contact area and its misalignment is greater than 5%, but necessary connections can still be completed, then leave the stiffener "as is" and add stiffeners at balanced locations about the specified bearing point. Space the additional stiffeners between 8 and 12 inches [200 and 300 mm] on center (see Figure 8). It is usually very conservative to make them the same thickness as the bearing stiffener. Typically such additional stiffeners may use the same thickness as interior connection plates, but the Contractor's engineer must provide the Owner with adequate design documentation that this will be adequate to prevent local buckling.

If the bottom location of the bearing stiffener is outside the middle 50% of the sole plate or bearing contact area, see Section 4.5, "Mislocated Bearing Stiffeners." If the bearing stiffener's location precludes the required attachment of other elements, the stiffener must be removed and replaced unless the Fabricator can propose an alternate scheme acceptable to the Owner. If possible, avoid removal of a bearing stiffener already welded into place.

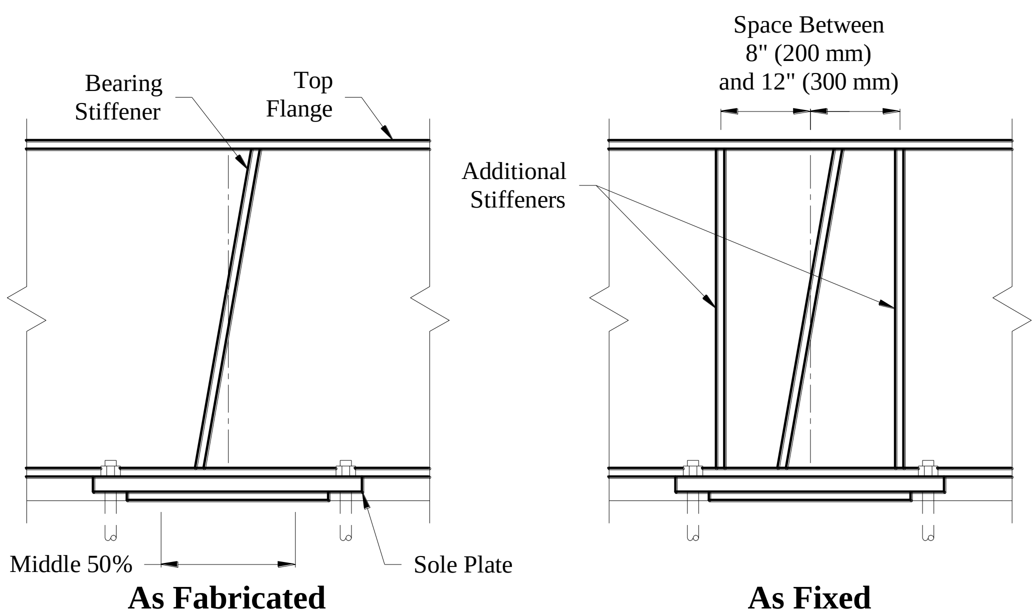

Figure 8: Misaligned Bearing Stiffener

Commentary:

The primary role of the bearing stiffener is to act with a portion of the web and form a column to transfer high gravity-induced forces through the sole plate. Therefore, the location of the bottom of the bearing stiffener with respect to the sole plate is more important than the location of the top of the bearing stiffener.

Exact vertical alignment is not critical to the capacity of the web-stiffener column, so bearing stiffeners typically may be either plumb (i.e., truly vertical) or normal to the bottom flange, even for structures on profile grades of 10%. Misalignments of 5% of member depth should not significantly affect total vertical load carrying capacity.

Air carbon arc gouging and grinding to remove welds are detrimental to remaining material and should be avoided if possible, especially if stiffeners have CJP welds to the bearing flange.

4.5 Mislocated Bearing Stiffeners

Issue:

A bearing stiffener is placed at the wrong location, outside the middle 50% of the sole plate or bearing contact area.

Recommendation:

If the location varies from the design plans or approved shop drawings by more than 8 inches [200 mm], then it may be possible to leave the stiffener in place, fill any holes with bolts (see Section 3.1, "Bolted Repair of Errant Holes"), and add a new stiffener at the specified location.

Bearing stiffeners mislocated by only a small amount (i.e., less than 8" [200 mm]), if left in place, may prevent adding a stiffener at the proper location due to the clearance needed for welding both sides of the stiffener. In this case, determine if an extra stiffener can be added to the other side of the bearing centerline, but still be above the sole plate or bearing contact area. The mislocated stiffener plus the added stiffener may provide adequate vertical load capacity, but calculations by the Contractor’s engineer must be submitted for the Owner’s acceptance.

If neither of the above approaches is acceptable, the stiffener must be removed in such a way that the stiffener is sacrificed, but the girder is not damaged. Follow the guidelines for removal per Section 4.10, “Plate Removal.”

Commentary:

Leaving the stiffener in place poses less risk to the member than removal, which may damage the web or flanges and require re-welding over previous weld locations.

If a diaphragm or cross-frame is to be attached to a mislocated or added bearing stiffener, a viable connection scheme must be prepared by the Fabricator for the Owner’s and the Contractor’s acceptance.

4.6 Mislocated Connection Plates

Issue:

A connection plate is fitted at the wrong location.

Recommendation:

When connection plates are fitted in a location other than shown on the design plans or approved shop drawings by more than 6 inches [150 mm], the preferred solution is to leave the plate in place, fill any holes with bolts (see Section 3.1, “Bolted Repair of Errant Holes”), and add a new connection plate at the proper location. This solution depends on the lack of any interference and access to the location for subsequent fabrication.

When connection plates are mislocated by a small amount (i.e., less than 6" [150 mm]), the “as fabricated” condition may be acceptable. However, this may result in the diaphragm or cross-frame to be attached not being perpendicular to the girder and possibly requiring modification. Care should be taken not to create a situation where buckling in the diaphragm or cross-frame may occur due to force fitting. The gusset plate should be in full contact with the connection plate before bolts are fully tightened.

A connection plate incorrectly located even by a small amount may be unacceptable in some situations such as a curved structure, where the diaphragms or cross-frames align across the structure to resist the horizontal forces due to curvature. In these situations, the connection plate must be removed in such a way that it is sacrificed, but the girder is not damaged. Follow the guidelines for removal per Section 4.10, "Plate Removal."

Commentary:

When unique diaphragms/cross-frames are made for misaligned stiffeners, adequate piece marking is essential to avoid erection difficulties. If a stiffener is skewed incorrectly, a bent plate should be used to adapt the fit.

4.7 Mislocated Hole in a Connection Plate

Issue:

A hole is at an incorrect location in a connection plate.

Recommendation:

When holes are mislocated in a connection plate, a bearing stiffener, or an intermediate stiffener where a diaphragm or cross-frame is to be attached, several options exist:

1. If the stiffener or connection plate has not been fit to the girder, the piece can be re-fabricated.

2. If the plate is already attached to the member, fill the hole with a bolt (see Section 3.1, “Bolted Repair of Errant Holes”), or leave the hole open if the bolt would be in conflict with other elements.
3. If the distance the mislocated holes are off from the specified location is small, check the hole spacing and loading conditions to see if slotting of holes is acceptable to the Owner. If so, slot holes for attachment and provide plate washers and appropriate length bolts, with appropriate notification to the erector.
4. If the plate has been attached to the girder, additional holes can be drilled or new gusset plates can be fabricated to match holes already drilled in the plate.
5. A new diaphragm or cross-frame can be custom-fabricated to match the plates.
6. If none of these options are acceptable and it is determined that the member must be removed, follow the procedure in Section 4.10, “Plate Removal.”

Commentary:

Other possibilities besides the hole being drilled incorrectly include, but are not limited to, installing a member with the wrong piece mark or installing a member upside down.

Holes may be slotted vertically in the connection plate/bearing stiffener and/or in the bracing itself. Slots may be placed in one connecting member with the other member having standard holes or in both elements. Slots under the head or nut must be completely covered by either individual plate washers or common bars with multiple standard size holes. The RCSC Specification defines washer requirements. Slots may not be acceptable if cross-frames or diaphragms carry design loads, such as in a curved structure, so the Owner must always be consulted before adding slots or oversize holes not shown on the contract plans.

If a custom diaphragm or cross-frame is fabricated to fit in this location, the shop drawings should be revised and submitted reflecting the "as fabricated" condition to the Owner and the Contractor. The erector must be aware of the existence of a special diaphragm or cross-frame, where it is to be located, and the field bolt and washer requirements.

4.8 Incorrectly Sized Plates

Issue:

An installed stiffener or connection plate does not provide the specified fit to the flange.

Recommendation:

The recommended repair varies depending on whether an intermediate stiffener, a bearing stiffener, or a connection plate is involved.

For interior stiffeners or connection plates with gaps up to 3/16" [5 mm] at the compression flange, increase the vertical leg size of the fillet weld (see Figure 9). For larger gaps, either deposit weld on the flange (see Section 3.3.1 in D1.5), or insert a tab plate so elements can be connected with separate fillet welds (see Figure 9). Tab plates must be thick enough to permit the minimum fillet weld size per Table 2.1 and Figure 2.2 in D1.5, so the stiffener or connection plate may need to be trimmed slightly. For painted structures, the tab plate will only be welded along three edges, so sealing or other measures may be needed along the web edge, and the contact faces of the flange and tab plate should be free of rust and scale before welding. Priming of the contact faces before welding is not recommended, since it may cause weld defects. Use of a small fill between the flange and stiffener or connection plate is restricted by Section 3.3.1.2 in D1.5.

The treatment is the same for connection plates and intermediate stiffeners specifying welding to the tension flange. If a bolted connection using tab plates is required at the tension flange, the tab thickness may be increased. If an angle connection is shown, the plate-flange gap between the angles may be increased, but some permanent non-corroding fill material (e.g., metal filled epoxy, zinc, or stainless steel bar) may be needed in the gap to reduce corrosion.

For bearing stiffeners, a direct metal-to-metal or solid metal load transfer condition is required at the vertical load transfer end (i.e., usually the bottom flange). If a "finish to bear" ("grind to bear", "mill to bear") fit is specified, but the criteria of Section 3.5.1.9 in D1.5 are not met, consider either a CJP weld or drive-fitting a machined fill (see Figure 9) that will subsequently be covered by the specified fillet weld. If the machined fill is the width of the stiffener, then its thickness is limited to 3/16" [5 mm] (see Section 3.3.1 in D1.5). If the gap between the stiffener and the flange is greater, consider either trimming and preparing the stiffener so a tab plate may be installed (see above) or utilizing a CJP weld. However, providing a high-quality CJP weld is very difficult for the following reasons:

- Weld terminations occur at the bearing stiffener clip near the flange-to-web weld and close to the flange edge.
- For a gap exceeding the root opening dimensions of Figure 2.4 in D1.5, joint qualification testing may be needed unless steel backing is allowed.
- For a stiffener already attached to the web, preparing the joint for welding may damage the flange.

Bearing stiffeners at the non-load transfer end (i.e., usually the top flange) are treated as described above for connection plates and intermediate stiffeners.

For connection plates, positive attachment to both flanges is required. If a transverse intermediate stiffener stopping short of the tension flange was mistakenly installed at a connection plate location, consider extending it to the tension flange by installing a rolled or welded T-section in the gap (see Figure 9). This can be welded or bolted to the flange, per contract document connection plate details, and either welded directly to the stiffener edge or lap-spliced, based on the conditions.

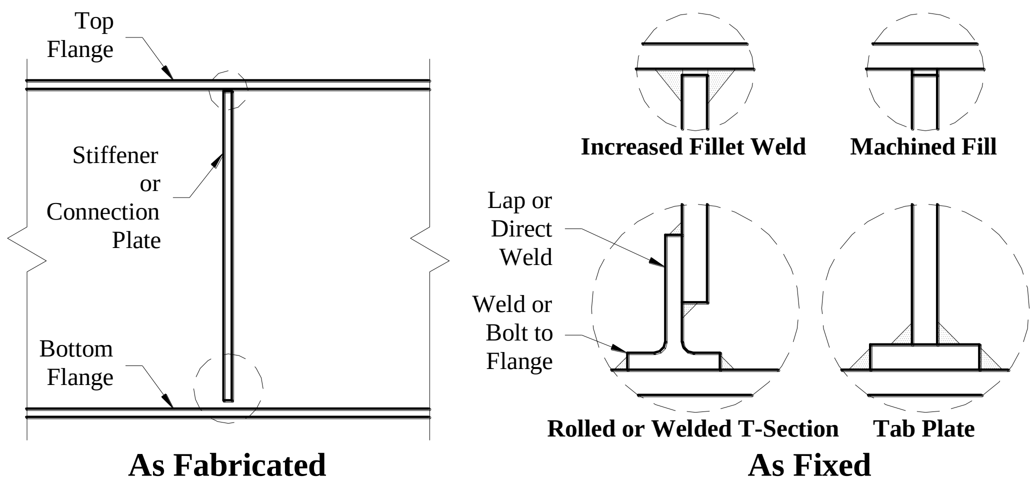

Figure 9: Stiffener or Connection Plate Fit Repairs

Commentary:

For partial-depth, transverse intermediate stiffeners, stopping short of the compression flange is of concern primarily due to the lack of attachment. A partial depth stiffener is not intended to reach both flanges, so buckling resistance is typically not affected by the stiffener being short.

However, if the stiffener is welded to the tension flange instead of the compression flange, the Engineer and the Contractor's engineer should determine if the stiffener can be left "as is," based on the fatigue conditions. The web will be braced against buckling and removing the welds on the flange and web may cause more problems than leaving them. This problem is most common near the point of dead load contraflexure, where "tension" and "compression" flange identities change.

Force fitting of stiffeners or connection plates must be avoided, as this can result in bowing of the installed plate, high residual stresses, and even flange tilt or web distortion. Correctly preparing the plate during fitting avoids this problem.

4.9 Exterior Face

Issue:

A transverse stiffener or connection plate is erroneously installed on the outside face of an exterior girder.

Recommendation:

The Owner has the option to require removal and reinstallation in the required location for aesthetics. Follow the removal procedure in Section 4.10, "Plate Removal." However, for low visibility areas (i.e., over streams, railroads, industrial areas, etc.) where aesthetics are not a significant concern, leaving the element in place avoids potential damage from removal operations, as long as fatigue conditions are satisfied.

Commentary:

If a stiffener is located on the wrong face of a girder, it will still function as intended and does not need to be relocated unless this is necessary for aesthetic reasons. A connection plate on the outside face of a girder may be allowed to remain if aesthetics permit, but another connection plate on the correct side must be added.

4.10 Plate Removal

Issue:

It has been determined that a stiffener or connection plate has been installed and must be removed.

Recommendation:

1. Remove the plate by flame cutting to 1/8" [3 mm] above the fillet welds.
2. Using air carbon arc gouging, being extremely careful not to damage the web/flange base metal, remove the weld and remaining plate to within 1/8" [3 mm] of the base metal. Protect the adjacent base metal from spatter, such as with sheet metal shields clamped in place.
3. Grind the remaining fillet weld/plate smooth and flush with the surrounding base metal. Final grinding should be parallel to the direction of applied stress (i.e., typically longitudinal on webs and flanges).
4. 100% MT the weld removal areas.

An alternate procedure would be to skip the flame cutting and remove the welds to within 1/8" [3 mm] of the base metal using air carbon arc as described in Step 2, then grind welds to separate the errant plate, grind flush, and MT per Steps 3 and 4.

Commentary:

Removal of plates may be required for a variety of reasons. However, leaving the plate in place is a more attractive solution to avoid the risk of damaging the girder. See the commentary in Section 4.2, "Mislocated Plate Tacked In Place," regarding removal of welds. Do not try to cut plates by arc gouging, as it is difficult to control, removes greater width, and more heat is applied into surrounding material than thermal cutting methods (e.g., oxy-gas or plasma).

5 - MISCUT MEMBERS

5.1 Plates

Issue:

A plate is cut incorrectly.

Recommendation:

The first option that should be investigated is whether the plate can be used in a different location. Refer to the shop drawings and material cutting sheets to study this alternative.

Another possibility, if the mistake is caught early enough, is to cut an adjacent plate to compensate (similar to Section 5.2, "Entire Girder Cut Short").

Yet another alternative, assuming material is available, is splicing on material by butt-welding. The weld must be reflected in the shop drawings and meet D1.5 quality criteria.

If none of these are feasible, the piece may have to be replaced.

Commentary:

NDE requirements for a butt-welded splice per D1.5 would apply. Note that CJP butt-welds have the same fatigue category (B) as flange-to-web fillets, so the permitted stress range for plate girders will not change. Butt-welds should be located at least 6 inches [150 mm] from other transverse welds (e.g., web or flange butts, fillets on stiffeners or connection plates, etc.).

5.2 Entire Girder Cut Short

Issue:

During burn-off for length, the girder was cut shorter than the specified length.

Recommendation:

First, determine if other girders in the same line have sufficient additional length to compensate for the miscut piece. Relocation of a field splice up to a few feet [1 m] is not structurally significant for most girders, but bearing stiffeners and connection plates may require relocation and final splice locations must clear bracing connections. If other segments cannot satisfactorily compensate, then either the miscut section or another segment must be lengthened or replaced.

If any segment has not been "shafted" (i.e., flange-to-web welds completed), consider lengthening that element by butt-welding extensions to individual web and flange components.

If the miscut member must be lengthened, the webs and flanges may be either completely or partially disconnected, or an additional segment may be bolt-spliced. For a relatively short girder, completely removing flange-to-web welds simplifies butt-welding extensions and permits the flange and web extensions to be at opposite ends of the piece. For longer girders, partial removal of the flange-to-web welds permits welding flange and web extensions without requiring full reassembly of the web and flanges. Reattaching the miscut segment using bolted splices at each end might be employed when a substantial portion of the girder was erroneously removed. This could be shop-bolted and avoids removing finished welds, but the splice must be designed by the Contractor's engineer and submitted for the Owner's approval, so delays and engineering costs may outweigh any benefits.

For partial removal of flange-to-web welds to add material, arc-gouge the welds. The flanges must be pulled away from the end of the web to permit joint preparation and welding without kinking or permanently deforming the flange. The Fabricator should determine the distance based on flange thickness, but it will usually be about 6 feet [2 m] (see Figure 10). Either the flanges or web must be cut to provide at least 6 inches [150 mm] of separation between adjacent butt-welds. The Fabricator should be allowed to determine these locations, but typically the web is extended further to allow clearance for run-off tabs. Flanges usually have their inside face welded first so backgouging is uninterrupted on the outside face, but this is also up to the Fabricator. The added sections of the newly fabricated web and flange sections should be a minimum of 3 feet [1 m] in length in order to control distortion during welding. Butt-welds must be ground per Section 3.6.3 in D1.5 and then NDE of each is performed per governing specifications prior to rejoining the web and flanges. Remove temporary braces and bring the flanges and web into the required alignment. After aligning and tacking components, but prior to welding the flanges to the web, remove an additional 2 inches [50 mm] of flange-to-web weld at the "Point of Attachment" labeled in Figure 10 to insure any cracks caused by the rework are eliminated. Weld the flanges to the web and 100% MT. Trim the girder to the correct length.

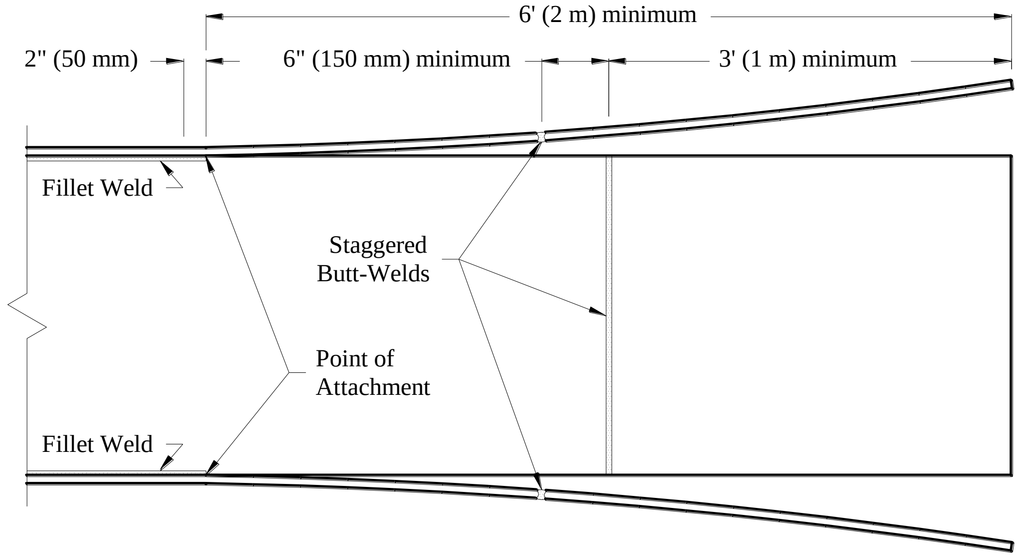

Figure 10: Staggered Butt-Weld Repair

If a rolled beam was cut short and must be weld-spliced (i.e., a bolted splice is not a viable alternative) or if a plate girder near (i.e., within 10% of the span) a simple support and primarily carrying shear with very low moment-induced stresses is miscut, then consider adding a complete section to the cut end (see Figure 11). Make weld joint preparations on flanges and webs and cut weld access holes per Figure 5.2 details in D1.1 (also see Figure 7.1 in S2.1). Note that AASHTO has not adopted a fatigue category for weld access holes, but Table 2.4 in D1.1 does provide this criterion. The Contractor's engineer must evaluate the situation at the proposed full-depth joint and provide calculations and a proposal including welding and NDE procedures for the Owner's review and approval. Access holes are not to be welded to reduce their size, but may be covered for aesthetic reasons if approved by the Owner. After all butt-welds are completed, NDE each per governing specifications. For plate girders, make the flange-to-web fillet welds, including wrapping the welds at the access hole locations, after the butt-welds are accepted. 100% MT and trim to the correct length.

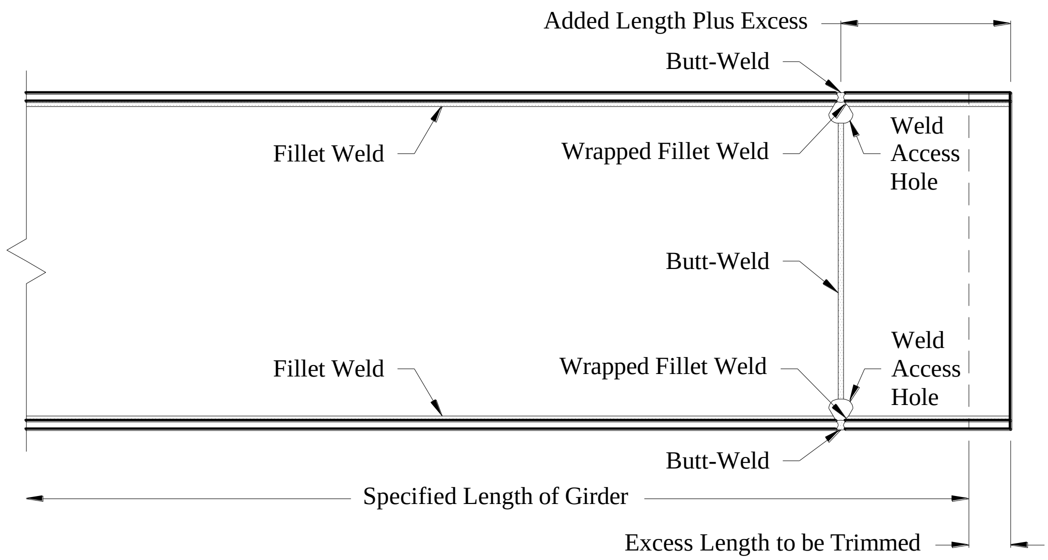

Figure 11: Non-Staggered Butt-Weld Repair

Commentary:

If the error is caught early enough, it may be possible to keep the short girder and lengthen an adjacent girder to accommodate the insufficient length. However, this may throw off many other considerations, such as stiffener spacing and lateral support. Careful observation and planning is needed if this option is pursued.

The retrofit depends on the member type and structural behavior required at that location. Usually, a butt-weld retrofit will be suitable. However, an additional bolted splice (i.e., an extra splice within a member specified as a single member) may be preferable, based on the situation.

If heavier (i.e., thicker or wider) material is added based on the miscut member properties, splices may need to be redesigned by the Contractor and submitted for the Owner's acceptance. AASHTO requires bolted-splices to be designed based on both the properties of the sections joined and also the maximum stresses at the splice location. If heavier material is butt-spliced to lighter (i.e., thinner or narrower) material to extend a flange, and/or if the bolted-splice location is significantly changed, the minimum splice requirements may change.

If a girder carries significant moment as well as shear, and butt-splices are used for the repair, they are made with the web and flanges separated to allow full-width butt-welds and continuous flange-to-web fillets without access holes. The welds are also staggered to reduce the possibility of crack propagation along common HAZ boundaries. This butt-splice without access holes retrofit is usually applied for large errors in length.

For a rolled beam or for a girder primarily loaded in shear, access holes are permitted without separating the web and flanges, and staggering the butt-welds is not necessary. This butt-splice with access holes retrofit is often more suitable near end supports and when the length to be replaced is relatively small, especially if it will be within a bolted splice. If weld access holes need to be filled for cosmetic or moisture reasons, use bolted covers, a metal-filled epoxy suitable for long-term exterior exposure, or other non-structural methods, but do not fill them with weld.

If the repair is located within a bolted splice area, the holes should not coincide with the butt-weld locations since the weld metal is harder and may deflect the drill, so adjustment may be necessary. If the web splice plates must be elongated to adjust for the butt-weld locations, more bolts may be needed, requiring changes in hole spacing. Any such changes in structural items must be designed by the Contractor's engineer and the calculations and proposed revisions then submitted to the Owner for review and acceptance.

5.3 Web Not Cut Correctly

Issue:

If the web is cut short at one or both flanges, a repair is needed.

Recommendation:

The entire member can be cut to "match" the error, with an adjacent connecting member lengthened to compensate for the error.

If the error is extreme at a bolted-splice location, the web may need to be cut back and a butt-welded section added to correct the error (see Section 5.2, "Entire Girder Cut Short").

For errors at a simple support such as an abutment:

- If it is an integral design, or if the web is cut correctly at the bottom flange but short above that, consider leaving it "as is."
- If the web is cut short at the bottom flange, but full contact with the bearing plate and proper positioning of the bearing stiffeners is possible, it should be considered for acceptance based on the required stiffener/web column capacity.
- If the web is cut so short at the bottom flange that the bearing will not have full contact or the stiffener/web column will be deficient, consider either shifting the bearing position on the seat or moving the entire girder line slightly toward that support.

Commentary:

When trimming girders to length with the web horizontal, the web is typically cut before the flanges due to the weight of the flanges and low transverse strength of the web.

The cut may be normal to the flanges or skewed slightly if the girder slope changes at the splice.

Shifting bearing position or moving the girder line requires evaluating effects on other bearing locations, alignment of bracing members, effects on modular or finger-type joints, and other ramifications of such changes.

If the error is extreme at a bolted-splice location, a special extended bolted-splice may be designed and proposed by the Contractor for the Owner's acceptance. Depending on the situation (e.g., shipping time, vertical shear, other design loads, and potential problems with weld-splicing on plates to extend the girder), it may be more efficient to design a longer bolted splice that includes an added segment placed between the two girders. The added segment may be as short as 4 inches [100 mm] or as long as many feet [~3 m]. This approach could be very un-aesthetic, but avoids potentially destructive repairs to otherwise sound girders.

5.4 Flange Transitions

Issue:

A flange transition thickness or width is incorrectly fabricated.

Recommendation:

First, the Contractor's engineer must determine if the piece can be used "as fabricated," and if so, propose this to the Owner. If leaving the transition is structurally unacceptable, it must be removed and an evaluation made for using the element with that length removed. If the resulting girder would be structurally deficient, the transition must be removed and an additional section butt-welded to restore the planned section (see Sections 5.1, “Plates,” and 5.2, “Entire Girder Cut Short”).

If a misfabricated flange is welded to the web before the erroneous transition is discovered and it must be corrected, the flange-to-web fillet weld must be removed to the point where the transition splice weld can be removed. The flange section to remain in-place is prepared for welding and a plate of correct thickness or width added (see Section 5.2, “Entire Girder Cut Short”).

If repairs are significant, it may be more desirable for the entire flange to be replaced.

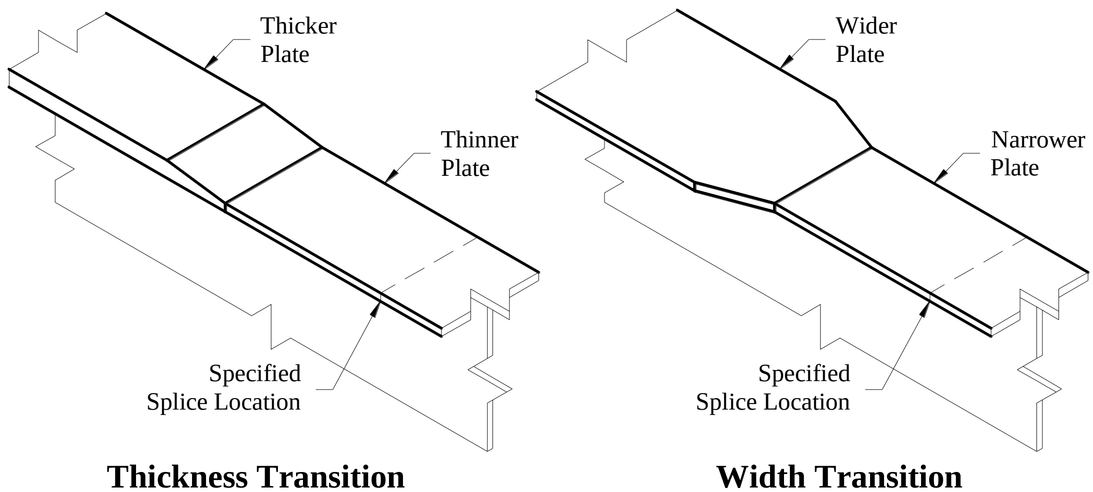

Figure 12: Flange Transitions

If a thicker plate exceeds the length specified and does not cause interference or significantly affect structural behavior, leave “as is.” If it interferes with a bolted connection to the thinner plate, the thicker plate may be ground to match the thinner plate or a modified connection detail provided (see Figure 13).

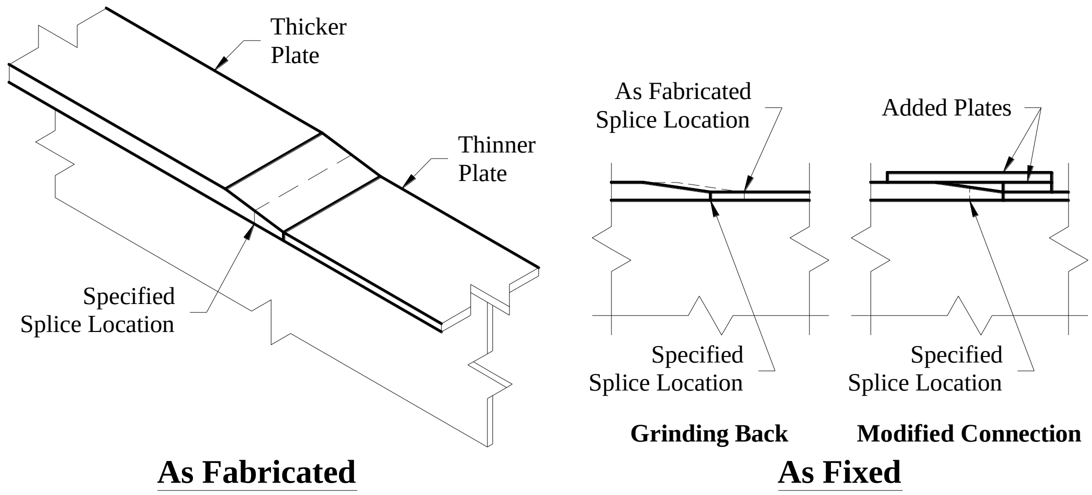

Figure 13: Connection Interference Repair

If a thicker or wider plate extends significantly beyond its intended termination, the girder's structural properties and behavior change. Redistribution of loads and changes in deflection may not be acceptable. If heavier material is to extend more than a few percent of the span length beyond plan dimensions, the Contractor's engineer must define effects on the overall structure and submit calculations and a proposal to the Owner.

Commentary:

This type of error may be caused by misread dimensions or other shop drawing details or by workers confusing concurrent jobs.

Before the QCI requests the QAI to permit significant changes in flange size or transition locations, the Contractor's engineer or the Fabricator's engineer should check the structural effects of the error and submit documentation and a proposal to the Owner to see if the flange can be left "as is." The QCI and QAI may be able to discuss small errors of 1% to 2% of the span length, and then obtain the Owner's concurrence without formal calculations. "As-built" shop drawings must accurately reflect the final conditions for future reference.

The Contractor's engineer should document the actual yield stress and ultimate strength of the flange material, based on MTRs. Yield stresses are commonly 10% to 15% higher than the minimums required, especially for 36 ksi [250 MPa] material, so this may provide more latitude for allowing a premature transition to a smaller plate to remain.

Any additional welded joints require the same NDE as specified by the contract for designed joints.

Sometimes a flange transition taper is cut in both top and bottom flanges when only one flange is specified. Transition tapers at a bolted splice, where no transition was required, may normally be left "as is" if the transition does not extend more than 1/2" [12 mm] under the splice plate and does not compromise the minimum edge distance for bolts.

If the contract plans detail either a straight taper (i.e., 2-1/2:1 slope) or a radiused width transition, but the other is provided during fabrication, this should not require rework unless quenched and tempered Gr. 100W [690W] or Gr. HPS 70W [485W] is involved. See Section 2.17.5.3 and Figure 2.8 in D1.5.

If a radiused or straight taper flange width transition is mistakenly cut at an end to be butt-welded, remove the tapered portion and relocate the butt-weld. This will shorten the flange (i.e., usually about 5" [125 mm] to 10" [250 mm]), so determine if the two flange elements have enough excess to compensate. If they do not, a short piece should be added, preferably near one end. Butt-welding on material to replace the mistakenly cut transition tapers is not recommended, since these welds would intersect the transverse butt joint and defects at the flange edge are likely.

5.5 Flange Corners

Issue:

The flange corners were cut incorrectly or where they were not specified.

Recommendation:

If part of the flange is coped to the web on the incorrect side (see Figure 14), determine whether the item can be used "as is" by altering installation. If the cope must be on the opposite side, determine if the piece may be salvaged by repairing the erroneously cut side and then cutting the correct side. Attempting to reattach the miscut pieces using CJP welds is not recommended due to the cost and location of the welds. Coping the flange on both sides of the web would leave the web unstiffened, so the repair must provide adequate lateral stiffness. "Repair" could entail fillet welding a plate to the miscut area of the flange or attaching a channel to that side of the girder (see Figure 15). Other potential repairs could be proposed by the Fabricator for the Owner's acceptance. Note that bolts or welds used to attach repairs must not interfere with intended use.

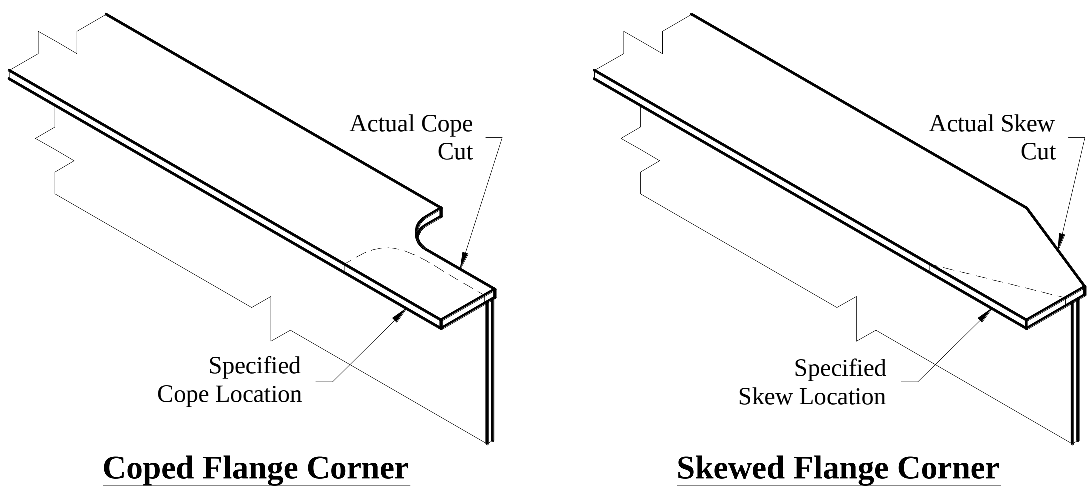

Figure 14: Wrong Flange Corner Cut

If the skew is cut on the wrong flange corner (see Figure 15), consider cutting the specified skew on the correct side or flange corner and leaving the other side “as is.” If the miscut portion is required to support an expansion joint or for bearing area, a new piece of plate could be CJP welded along the cut-line and then cut to match the required configuration. NDE would be based on anticipated loading and contract requirements.

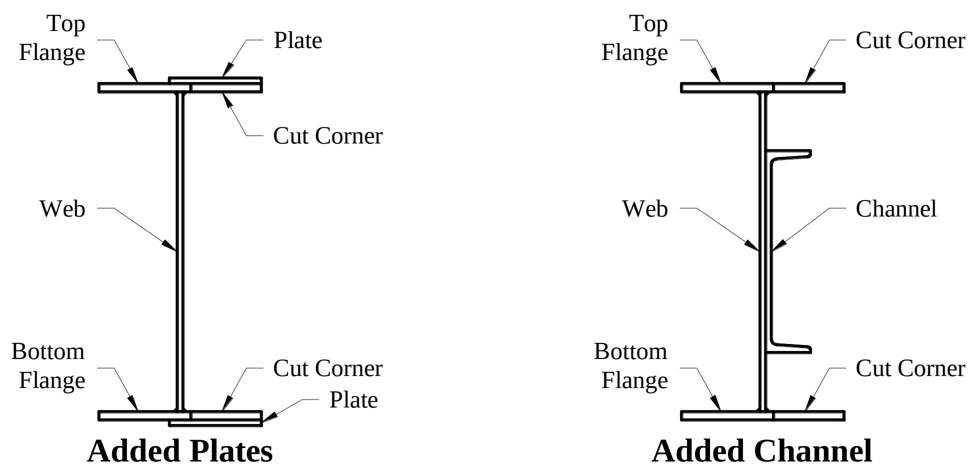

Figure 15: Potential Repairs of Unstiffened Webs

Commentary:

A flange corner may be clipped or coped for a variety of reasons. A skewed clip is typically used at simple support ends to clear expansion joints, abutment backwalls, or other obstructions. There is usually very little stress in the flange at simple supports unless modular or finger-plate expansion joints are supported.

Flanges of floor beams, diaphragms, or cross-frame members may be coped to allow a direct connection to the web. For these situations, the primary load is shear in the web so the flange may not be needed. If the member is detailed with gusset plates or to bolt directly to connection plates on the main girders, it may be possible to connect to opposite sides of the member at its ends, that is to say the member is to be installed on different faces of the two connections plates, but this must be accepted by both the Owner and the Contractor’s erector.

6 - WEB OR FLANGE REPLACEMENT OR REPAIR

6.1 Web or Flange Damage

Issue:

A web or flange is damaged during or after fabrication.

Recommendation:

If the web or flange plate is damaged prior to "shafting" (i.e., fitting and welding the flanges to the web), repair or replace the damaged section. For replacing part of the plate, cut it full-width and add a new piece with a butt-weld. Do not attempt to cut out a partial-width area and weld a "patch" by welding around its edge (i.e., either two or three sides or completely around its perimeter) (See Section 5.1, "Plates.")

Repairs may be made if the QCI and the QAI (and if appropriate, the Owner) agree the damage may be corrected by methods of Section 3.2 in D1.5, A 6, or other applicable criteria. Repairs may entail welded restoration, heat-assisted or cold straightening, reinforcing by bolted or welded plates, additional stiffeners, or other owner-approved methods. Any welded repairs must utilize owner-approved weld procedures.

If the damaged portion is attached to the member and cannot be repaired, it will require removal. If near the end of a member, removal and replacement is similar to the repair methods in Section 5, "Miscut Members." If the damaged portion is not near the end, the Fabricator may consider suggesting one or more additional bolted splices (i.e., extra splices within a member specified as a single member). Full or partial (e.g., flange portion only) splices could be placed over the defect. Alternately, the splices could be used to insert a new segment within the existing girder. Additional splices, positioned to miss bracing connections, would be shop bolted and the repaired member would be the original specified length.

Commentary:

Common causes of such damage include: inadvertent bending or kinking due to improperly handling long material without spreaders; impact damage during fabrication, transport or erection, and cutting or welding errors.

For material defects, see Section 7, "Material Defects, Nicks, and Gouges."

6.2 Overgrinding

Issue:

Material thickness is reduced due to overgrinding of butt-welds and fillet welds or while removing surface defects.

Recommendation:

There are three approaches to correcting this problem:

1. Add weld metal to restore the required section, and then finish grind, followed by the appropriate NDE.
2. Request the use of the piece "as is." In this case the Contractor's engineer would have to determine that the reduction in thickness would not affect the required design capacity of the piece and provide documentation accompanying a proposal to the Owner.
3. Remove and replace the piece.

Commentary:

Overground butt-welds must be considered on a case-by-case basis. Section 3.2 in D1.5 and A 6 provide criteria for acceptable material removal limits without requiring repair. Section 3.6 in D1.5 defines weld profile tolerances.

Defect repairs are covered in Section 7, "Material Defects, Nicks, and Gouges."

6.3 Sole Plate in the Wrong Location

Issue:

A sole plate is welded to the bottom flange in the wrong location.

Recommendation:

If discovered in the shop, remove the sole plate (see Figure 16) and re-attach it in the correct location. Large welds may be partially air-carbon arc gouged to remove the bulk of the deposit, but the final portion requires careful grinding. The weld within 1/8" [3 mm] of the flange and sole plate should be removed with a hand grinder to minimize damage, especially to the girder flange. After removal of the sole plate, the girder flange should be examined visually and with MT to ensure no cracks resulted from the removal process. The edges of the sole plate should be ground to remove small gouges and carbon deposits, providing a good weld face, but minor gouges or material loss can be restored during rewelding. Reposition the sole plate in the correct location, weld it to the flange, and perform NDE as required by the contract.

If the discrepancy is discovered in the field, determine if the masonry plate can be relocated, especially for small discrepancies. If the anchor bolts have not been installed and can be repositioned without conflicting with reinforcement in the substructure, moving the bearing a few inches [~100 mm] is not usually significant. If the anchor bolt position cannot be changed, consider shifting the entire girder line. This will affect many more locations, but may prevent significant delays to remove and reposition the sole plate. If no other options are possible, the sole plate may be relocated, but the Contractor must have welders qualified for overhead welding, approved procedures, and address all damage at the bearing, especially with a painted structure.

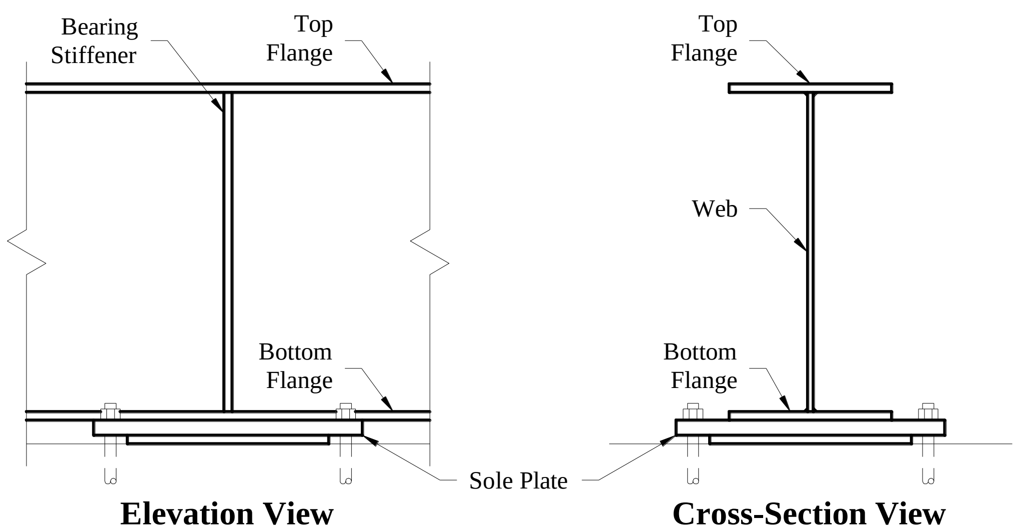

Figure 16:   Sole Plate

Commentary:

Minimize damage to the bottom flange of the girder, even though it is typically in compression at continuous piers and has low stress at simple supports. Damage to the sole plate can usually be addressed during rewelding to the flange, since it does not carry tensile stresses. (See Section 6.1, "Web or Flange Damage.")

6.4 Rolling Direction Not Parallel to Primary Stress

Issue:

The primary rolling direction of a splice plate is not parallel to the design tensile stress.

Recommendation:

The rolling direction of webs and flanges will necessarily be correct, except possibly in very short members, such as a transverse bearing stiffener/floorbeam. However, splice plates might be incorrectly oriented with their prime rolling direction transverse to the highest stress. Flange splices are the most critical, since they see the highest tensile stresses, and essentially take axial loading. Web splice plates carry shear (i.e., vertical and diagonal) as well as stress due to bending, so their orientation is not as critical.

If the flange splices are cut transversely to the long axis of the plate, the preferred action is to replace them. If this is not possible due to material availability or scheduling (e.g., the splices have already been drilled in assembly and the steel is needed at the jobsite), CVN samples may be necessary with the specimen taken from material known to be from the same plate, and cut along its transverse direction (i.e., "V" notch parallel to the primary rolling direction).

Commentary:

Engineering experience would play a big part in determining whether or not the piece could be salvaged depending on its function and location within the structure.

Fracture toughness, as demonstrated by CVN specimens, for rolled steel plate is highest in its longitudinal direction. This is due to the grain refinement that occurs during rolling. When making a plate, a steel slab is initially rolled in both directions, but the majority of rolling occurs in the plate's long dimension, so better properties result along that axis. The through-thickness direction has the poorest toughness.

In situations where the material properties are crucial (e.g., a splice in a fracture critical member), questionable properties would be unacceptable and would require replacement of the piece.

7 - MATERIAL DEFECTS, NICKS, AND GOUGES

The QC/QA inspectors should refer to A 6 and D1.5 for tolerances, repair procedures, or rejection procedures. Approval for repairs in accordance with A 6 and/or D1.5 is permitted with the application of an approved WPS. Repairs discussed within this section are drawn largely from these two references.

7.1 Material Surface Quality

Issue:

Structural steels are normally furnished in the "as rolled" condition. Non-injurious surface or internal imperfections or both may be present in the steel as delivered and may require conditioning.

Recommendation:

Section 9 in A 6 addresses material quality (9.1), conditioning (9.2 - 9.4), and repair by welding (9.5). Section 3 in D1.5 addresses workmanship, including preparation of base metal (3.2), control of distortion and shrinkage (3.4), and dimensional tolerances (3.5).

Commentary:

Plate defects occur in a variety of forms including, but not limited to, burn gouges from cutting, deformations due to handling, and mill plate defects, such as rolling marks, "fish scales," or "beer tabs."

7.2 Base Metal Defects

Issue:

A base metal defect requires repair for non-fracture-critical applications.

Recommendation:

To repair surface defects, follow the requirements of Section 9.2, "Plate Conditioning," in A 6.

Commentary:

Base metal defects can occur from a variety of reasons. Such a defect could include a laminar discontinuity, either internal or at an edge of the base metal, fins, handling damage, or other surface defects detrimental to the coating system.

For surface defects, no welding is required if the thickness of the plate is not reduced more than the permissible minimum thickness. When determining permissible thickness, take into consideration that plate ordered by bridge manufacturers is ordered by thickness rather than weight. Tolerances for plate ordered by thickness are small, as shown in Table 1 of A 6. Plate received from the mills is usually over-thickness around 0.02" [0.5 mm]. Note that the tolerances for plates ordered by weight and rolled shapes are quite different, as shown in Tables 2, 16, and 17 of A 6.

Section 9.2.1 in A 6 provides direction on plate conditioning for shallow defects. An example of the type of repair that could be specified follows. Refer directly to A 6 for specifics:

1. Grind surface to a smooth (i.e., roughness not exceeding 64 μin [2 μm]), bright finish free of irregularities.
2. Grinding shall transition to adjacent surfaces at a 10:1 slope.
3. Final marks shall be parallel to primary stress.

4. MT the area to assure removal of all discontinuities.

Section 9.2.2 in A 6 provides direction on the deposition of weld metal following the removal of surface defects. An example of the type of repair that could be specified follows. Refer directly to A 6 for specifics:

1. Remove the defective area by grinding, gouging, or chipping. Final grind the excavation to create a uniform cross-section for welding.
2. At a minimum, groove excavations shall satisfy the requirements of joint configuration C-P6 (i.e., for the process used) in Figure 2.5 of D1.5, and the ends shall transition to the surface at a 1:1 slope.
3. MT the area to assure removal of all discontinuities.
4. Perform welded restoration using an owner-approved weld procedure and grind the surface to satisfy Section 3.2.2 in D1.5, based on the weld location.
5. NDE per Section 3.2.3 in D1.5 and other governing specifications. All weld passes must receive 100% VT.

If deficiencies are excessive (i.e., surface defects exceeding the limits of Section 9.2 in A 6 or internal defects exceeding the limits of Section 3.2 in D1.5) or occur in a highly restrained area, the affected portion of the member may have to be restored by special welding techniques or replaced entirely.

Internal laminar discontinuities are not usually discovered unless exposed at a cut edge or found by UT prior to repairing a nearby edge defect. If the contract stipulates "UT Quality Plate" or mandates volumetric NDE for plates highly stressed in the through-thickness direction, the Contractor is obligated to order plate with proper testing and/or inspect for internal discontinuities. Any rejectable discontinuities discovered must be repaired to the Owner's satisfaction or the material must be replaced. If CJP welds are used at cruciform joints (i.e., plates on opposite sides of a common plate, forming a ╬), their residual stress may cause laminations to open. These tightly closed laminar discontinuities open and become delaminations. Residual stress at cruciform joints may also cause internal rupture of previously sound material in the center plate by lamellar tearing (see Section 6.4, "Rolling Direction Not Parallel to Primary Stress"). If delaminations or ruptures are discovered during UT of weld joints, thickness checks, or other investigations, the QCI and QAI should document their extent and report them to the Owner. If a critical joint is involved, remedial actions may be considered, including adding high-strength bolts perpendicular to the defect, sandwiching the area between bolted plates, or replacement. Such remedies, if outside the provisions of the original contract, must be negotiated between the Owner and the Contractor before proceeding.

7.3 Repair of Base Metal Cut Edges

Issue:

Repair is required for material edge defects exposed by thermal cutting.

Recommendation:

To repair base metal cut edges, follow the requirements of Section 3.2.3, "Visual Inspection and Repair of Base Metal Cut Edges," in D1.5.

Commentary:

Laminar material defects may be exposed by thermal cutting as shown in Figure 3.1 of D1.5. Those laminar discontinuities often cause burn gouges by deflecting the cutting jet, and such gouges can be corrected as described in Section 7.4, “Gouges or Erroneous Cuts,” after the laminar defect is corrected per Section 3.2.3, “Visual Inspection and Repair of Base Metal Cut Edges,” in D1.5.

7.4 Gouges or Erroneous Cuts

Issue:

Repair is required of a gouge or an unintended cut in non-quenched and tempered material for a non-fracture critical application.

Recommendation:

To repair gouges or erroneous cuts, follow the requirements of Section 3.2, “Preparation of Base Metal,” in D1.5.

Commentary:

Gouges and other damage can occur from a variety of reasons. Examples include, but are not limited to, a torch blow hitting some imperfection when cutting, irregular travel speed of torch, or simple carelessness with the torch.

Section 3.2 in D1.5 addresses base metal preparation. Examples of repairs that could be specified for shallow, medium, and deep gouges follow. Refer directly to D1.5 for specifics.

To repair gouges 3/16" [5 mm] or less in depth, but not exceeding 2% loss of cross-sectional area:

1. Grind in the direction of applied stress in the member.
2. Material shall be faired to the material edge at a slope not to exceed 10:1.

To repair gouges over 3/16" [5 mm] to a maximum of 7/16" [11 mm] deep:

1. Grind gouge to satisfy the requirements of joint configuration C-P6 (i.e., for the process used) in Figure 2.5 of D1.5.
2. Attach run-off tabs with no tack welds outside the repair area. Tack-welds must be consumed by the final weld.
3. Perform welded restoration using a weld procedure approved by the Owner for the situation.
4. Remove the run-off bars and grind smooth.
5. NDE per governing specifications.

To repair gouges deeper than 7/16" [11 mm] or through-thickness cuts:

1. Use an applicable owner-approved weld procedure.
2. Grind burn gouges at a 60º bevel. Prepare through-thickness cuts for CJP welds in accordance with an owner-approved weld procedure. Large gaps may require steel backing, which is removed before second-side welding, while smaller gaps may use fast-freeze electrodes for first-side root passes.
3. Attach run-off tabs with tack-welds inside the repair area. Tack-welds must be consumed by the final weld.
4. Perform welded restoration on the first side.
5. Backgouge the second side to sound weld metal for CJP welding.
6. Perform welded restoration on the second side.
7. Remove the run-off tabs and grind to a required radius or surface finish.
8. NDE per governing specifications.

If the gouging is frequent, indicating defective material, improperly functioning equipment, and/or generally poor workmanship, or occurs in a high tensile stress zone, the affected portion of the member may have to be replaced.

Erroneous cuts can occur due to a variety of reasons. Such cuts may be burnt in the top or bottom of the web from incorrectly cutting copes or weld access holes.

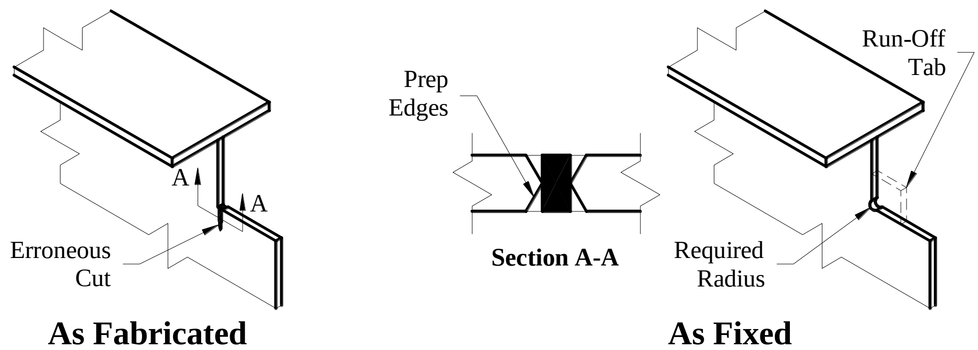

Figure 17: Erroneous Cuts

7.5 Arc Strikes

Issue:

An arc strike occurs on a non-fracture-critical application.

Recommendation:

To correct arc strikes:

1. Grind to bright metal to remove the defect area approximately 1/16" [2 mm] below the surface.
2. MT the area to assure the absence of cracks.

Commentary:

Arc strikes occur when an electrode briefly contacts the work. Extreme worker carelessness, such as pulling a cable to drag a “stinger” (i.e., energized SMAW weld rod and holder) across the piece or “walking” the arc to the weld groove to avoid lifting the welding hood, both leave a string of arc strikes. Other sources include sporadic touches of the weld gun, cracked insulation on welding cables, and poorly attached grounding. The arc melts base metal and may deposit a small amount of electrode (or copper at a poor ground) that freeze instantly, creating high localized residual stresses and possible microcracks. Since these can combine to initiate cracks, they must be removed before the structure is exposed to cyclic stress. However, arc strikes that will subsequently be covered by welding need not be removed. The effects usually only extend about 1/32" [1 mm] to 1/16" [2 mm] deep, so shallow grinding will suffice and no additional repairs are needed. Hardness testing may be inconclusive due to the shallow penetration of arc strikes, unlike mislocated welds (see 4.2, “Mislocated Plate Tacked In Place”). If arc strikes occur on Gr. 100W [690W] quenched and tempered material, the QAI and the Owner must be informed and additional NDE may be required.

If arc strikes occur frequently or if a welder “drags” or “walks” the arc, the QCI must take appropriate corrective actions and remove careless workers until properly trained.

7.6 Bending and Restraint-Induced Fractures

Issue:

Stress-induced fractures have occurred in a plate during fabrication.

Recommendation:

Dye-penetrant or MT the area suspected of having stress fractures to determine their extent and size.

If the defect is caused by bending in an area that will have low in-service stress and the fractures are small, less than 1/4" [6 mm] long, consider repair per Section 7.2, “Base Metal Defects.”

For fractures caused by high residual stress due to welding and/or large fractures, more than 1/4" [6 mm] long, caused by plate bending, either material replacement or welded repairs are required. Welded repairs must utilize owner-approved procedures and NDE, and post-weld stress relief may be necessary to avoid subsequent re-cracking in service.

Commentary:

Due to fatigue concerns, stress-induced fractures in high-stress areas and fractures more than 1/4" [6 mm] long are normally not acceptable in bridge members. Even secondary bracing members with initial cracks may have long-term problems due to small cyclic stresses.

To avoid stress fractures during fabrication, all bending of material should be done in accordance with AASHTO and AISC guidance, including minimum bending radii for direction of rolling. Grind outside corners and sheared edges before bending to eliminate stress-concentrating defects, and for thick plates, consider limited heating (i.e., approximately 600º F [315º C]) near edges to lower yield stress.

High restraint forces may cause fractures during or following welding. Cruciform joints (i.e., plates welded to opposite sides of a common plate, forming a ╬) using CJP welds or welding a plate between two thick, highly restrained plates may lead to fractures in the welds or lamellar tearing inside a plate loaded in the through-thickness direction (see the commentary in Section 7.2, "Base Metal Defects"). If the weldment must be repaired, the QCI, QAI, the Fabricator's engineer, and the Owner must jointly discuss the situation and mutually agree on methods and what constitutes "success". Lamellar tearing cannot be repaired per se, but additional reinforcement or alternate load paths may be possible. Repair welding may require higher preheat, special consumables (e.g., high nickel wire for greater ductility), post heat, and/or stress relief. If additional similar assemblies are needed, consider changing the assembly sequence to reduce restraint, reorienting plates, changing CJP welds to fillets, adding intermediate plates between components, changing weld procedures, or UT of plates before welding and stress relief.

7.7 Material Substitution

Issue:

The Fabricator inquires about substitution of a different material for the original material specified.

Recommendation:

This is not arbitrarily permitted, unless otherwise written into the specifications.

Commentary:

In most cases, if substituting a higher strength material, approval is granted. However, if welding is required, quenched and tempered material cannot usually be substituted for lower grades.

A request may be to substitute an "equivalent" material with a different designation, such as ASTM A709 Gr. 50W (345W) for Gr. 50 (345), ASTM A36 for AASHTO M270 Gr. 36, ASTM A847 for A500 Gr. C rectangular tubing, or ASTM A992 for A709 Gr. 50

rolled shapes. The Owner should consider these favorably as long as any required properties (e.g., strength, CVN testing, or weathering) are met and weld procedures are appropriate. ASTM A709 or AASHTO M270 Gr. 50W (345W) weld procedures must be qualified with plate chemistry satisfying Section 5.4.2 in D1.5.

If the Fabricator requests substituting a lower grade, this may only be permitted if the specified strength is not required. If the contract stipulates, “all plates shall be Grade 50,” and the Fabricator requests using Grade 36 for interior stiffeners, this could be considered, since these are based on size effects, not strength. If the Fabricator mistakenly installs a lower grade and then requests it be allowed to remain, the Owner may consider design requirements and the actual plate strength, based on MTRs, to avoid potential damage caused by replacing the element, but the option of requiring replacement remains.

7.8 Raw Material Out of Tolerance

Issue:

Steel products received by the Fabricator are found to be out of tolerance.

Recommendation:

A 6 specifies the tolerance for steel products supplied to the Fabricator. If feasible, out-of-tolerance raw material would be rejected by the Fabricator and returned to the supplier. If this approach is not feasible, consider conditioning the materials as specified in A 6. Problematic material must be addressed before it is fabricated into a member.

Commentary:

A 6 governs tolerances for steel products shipped from the mill or regional steel warehouse. Once the steel has moved into fabrication, D1.5 governs the fabrication tolerances.

Although it seems straightforward to resolve out-of-tolerance raw material issues by refusing to accept the shipment, this is often not practical. Mills may ship material that fabricators do not view as meeting A 6. However, usually because of time constraints, the Fabricator may not be in a bargaining position to return the material.

The Owner must recognize that A 6 tolerances are significantly different for plates and rolled shapes (see the commentary in Section 7.2, "Base Metal Defects"). Table 16 in A 6 sets tolerances for non-parallelism, overall depth, flange width, and web thickness of rolled I- and C-shapes (W-beams, H piles, channels, etc.), but it does not specifically address flange thickness. Therefore, surface defects in beam flanges need to be evaluated based on the entire section, not just the flange thickness at that location.

8 - HEAT APPLICATION

Heat is a very valuable tool for fabrication. S2.1 provides an excellent discussion of heat application used during normal fabrication, in particular, Section 5 and its relevant commentary. For additional information associated with heat treatment of structural materials, see Section 6 in A 6. Also, refer to Section 16 in A 6 for information regarding retreatment of material. For the heating processes discussed in this section, all work should be performed in accordance with D1.5.

There is a great deal of literature available on the use of heat in steels. "Heat-straightening Repair of Damaged Bridges," Report No. FHWA-IF-99-004 (October, 1998), is a comprehensive work on steel bridges and is available from the FHWA Office of Technology Applications (400 7th St. SW, Washington, DC 20590, 202-366-7900). Dr. Richard Avent of Louisiana State University conducted the research and although the work is focused on repairs, it covers the basics and provides useful information for fabrication.

This section is limited to the application of heat to repair fabrication errors. To prevent problems that arise as a result of improper application of heat, it is imperative that personnel have proper training and follow an approved written procedure to avoid overheating the work piece. Monitoring equipment typically includes, but is not limited to, temperature-indicating crayons (i.e., for 50º F [10º C] above and below the maximum limit as well as other desired temperatures), surface pyrometers, or infrared non-contact thermometers. Depending on the steel grade used, heat should not exceed 1100º F [590º C] or 1200º F [650º C]. Refer to Table 5.1 of S2.1 for the maximum temperature limits for heat application on typical grades of bridge steel.

Ensuring that overheating does not occur is much more cost-effective than proving to the Owner that possibly overheated material is acceptable (see Section 8.5,"Improper Use or Monitoring of Heat Application"). For this reason, it is particularly important to provide appropriate QC/QA, before and during heat application.

8.1 Correcting Girder Camber

Issue:

The final camber after assembly of a girder is outside permitted tolerances.

Recommendation:

A suggested procedure for correcting plate girder or rolled-section camber follows. Some owners limit corrections by this procedure to twice the tolerance permitted by D1.5 for camber:

1. Girder will be heated with the web in the vertical position and with the flange intended to move toward the girder centerline (i.e., the flange to be concave) placed on top. This uses the girder's self-weight to assist movement. The girder's self-weight plus any additional loads applied to assist movement must not cause calculated stresses to exceed 50% of the material's nominal yield stress at ambient temperature. Vertical blocking shall be provided to prevent excessive deflection of the member. The girder shall also be secured to prevent lateral movement and provide stability during the process.
2. Heat shall be applied to adjust the girder's camber to meet contractual requirements. Heat shall not be applied to areas heated during previous sequences. A "sequence" includes all heating patterns applied before allowing

the girder to cool to verify displacement achieved. Heat patterns may need to overlap within a sequence.

3. All work shall be performed within temperature limitations of D1.5. Appropriate QC supervision is mandatory during the heating process.
4. Triangular (Deep-Vee) heating patterns shall be established at locations throughout the length of the member, based on the amount of movement needed. Optimally, these may be at stiffener or connection plate locations to reduce web deformation. The apex of the heating triangle shall be located in the web at a point not less than 2/3 the depth of the member from the flange that will be concave after cambering.
5. Heating shall be performed using an appropriate size multi-orifice “rosebud” type tip to promote efficiency while minimizing out-of-plane distortion and excessive temperatures.
6. Heating shall begin at the apex of the heating pattern and progress toward the flange in a serpentine motion with the included angle not exceeding 20º. At the flange to move toward the girder centerline (i.e., the flange to be concave), the base shall be a maximum of 10 inches [250 mm]. The heating torch shall not be returned toward the apex of the heating triangle. A strip of flange equal to the width of the pattern at the base of the web shall be heated either before or after the web, based on relative dimensions and thicknesses, to obtain the desired results. For flanges over 1-1/4" [30 mm] thick, inside and outside faces shall be heated concurrently.
7. Heating shall be confined to the pre-planned and marked patterns. The steel shall be brought to a temperature between 1000º F [540º C] and 1150º F [620º C] as

rapidly as possible. Temperature-indicating crayons manufactured to indicate 600, 1000, 1100 and 1200º F [315, 540, 590, and 650º C] shall be utilized. All heat measurements shall be taken between 3 and 8 seconds after the torch has been removed from the steel. Any heating that results in a steel temperature in excess of 1150º F [620º C] and causing the 1200º F [650º C] crayon to melt shall be considered destructive and may result in the rejection of the steel. See Section 8.5, "Improper Use or Monitoring of Heat Application."

8. Quenching with water or a combination of air and water is not permitted. Cooling with dry compressed air is permitted after the steel has been allowed to cool naturally to 600º F [315º C].
9. MT of all fillet welds in heated areas plus other designated areas (e.g., outside face of cold bends, re-entrant cuts, etc.) shall be performed by the Contractor after all heating is completed and the material returns to ambient temperature.

Commentary:

This procedure has similar requirements and limitations to that in Section 8.2, "Sweep Adjustment," with the heat patterns changed to be suitable for camber correction.

8.2 Sweep Adjustment

Issue:

The sweep in the fabricated member is outside tolerance.

Recommendation:

A suggested procedure for adjusting sweep follows:

1. Girder will be heated with the web in the horizontal position and with the flange edge intended to move toward the web (i.e., the flange edge to be concave) placed on top. This uses the girder's self-weight to assist movement. The girder's self-weight plus any additional loads applied to assist movement must not cause calculated stresses to exceed 50% of the material's nominal yield stress at ambient temperature. Flanges will be supported at each end and at additional locations as required to avoid either overstressing the member or exceeding the desired movement. Bracing and catch blocks shall be provided to ensure stability and prevent any localized buckling or sagging during the process.
2. Heat shall be applied to adjust the girder's sweep to meet contractual requirements. V-heats shall not be applied to areas heated during previous sequences. A "sequence" includes all heating patterns applied before allowing the girder to cool to verify displacement achieved. V-heat patterns may need to overlap within a sequence.
3. All work shall be performed within temperature limitations of D1.5. Appropriate QC supervision is mandatory during the heating process.
4. Establish heat patterns at locations throughout the length of the member. This may employ either linear strip or triangular V-heating patterns. Before heating begins, the desired metal temperature and anticipated pattern placement shall be calculated based on the displacement required and the properties of the member. If available, the publications "Fabrication Aids for Girders Curved with V-Heats" and "Fabrication Aids for Continuously Heat-Curved Girders," prepared by Roger Brockenbrough for U.S. Steel, provide basis for such calculations. Heat is applied concurrently to the top and bottom flanges in either strips along each

flange or wedge shaped areas having their base at the flange edge and spaced at appropriate intervals along each flange. For flanges more than 1-1/4" [30 mm] thick, both the outside and inside faces of the flange must be heated so through-thickness heating is quickly achieved. Heating both faces of the flange concurrently provides the maximum effect.

5. For the V-type heating, to provide uniform radii less (tighter) than 1000 feet [300 m], V-patterns on the outside of the flange will extend past the web centerline by 3 inches [75 mm] or 1/8 of the flange width, whichever is less. If the inside face of one or both flanges must be heated, the apex of the truncated triangular area on the inside surface occurs just before the flange-to-web fillet weld on plate girders, or before the rolled fillet on rolled beams. If correcting abrupt local distortions (e.g., bends or kinks) by methods in Report No. FHWA-IF-99-004 (see the introduction to Section 8, "Heat Application") and the proposed V-pattern width at the web centerline is more than 1-1/4" [30 mm] wide, an equal width strip of web and the included flange-to-web welds should also be heated to facilitate movement during cooling. Unless heating such a strip is required, do not apply heat to the web or web-to-flange fillet to avoid undesirable distortion and residual stress.

6. For the continuous heat method, a strip of heat about 2 inches [50 mm] wide is simultaneously applied to each flange (i.e., outside face or outside and inside face for over 1-1/4" [30 mm] thick) about 2 inches [50 mm] below the upper edge. For small radii, a wider strip may be needed, but heat should not be directly applied to the edge of the flange. Torches may be hand-held or carriage-

mounted, but motor-driven mounts are recommended, especially for long, heavy flanges, to assure more uniform heating.

7. Heating shall be performed using an appropriate size multi-orifice “rosebud” type tip to promote efficiency while minimizing distortion and excessive temperatures.
8. For the truncated heating pattern, provide an included angle of 15º to 30º, but do not exceed 10 inches [250 mm] for the base of the triangle. The heating torch shall progress from the apex end toward the base in a serpentine pattern and not be returned toward the apex.
9. Heating shall be confined to the pre-planned and marked patterns. The steel shall be brought to a temperature between 1000º F [540º C] and 1150º F [620º C] as rapidly as possible. Temperature-indicating crayons manufactured to indicate 600, 1000, 1100 and 1200º F [315, 540, 590, and 650º C] shall be utilized. All heat measurements shall be taken between 3 and 8 seconds after the torch has been removed from the steel. Any heating that results in a steel temperature in excess of 1150º F [620º C] and causing the 1200º F [650º C] crayon to melt shall be considered destructive and may result in the rejection of the steel. See Section 8.5, “Improper Use or Monitoring of Heat Application.”
10. Quenching with water or a combination of air and water is not permitted. Cooling with dry compressed air is permitted after the steel has been allowed to cool naturally to 600º F [315º C].
11. MT of all fillet welds in heated areas plus other designated areas (e.g., outside face of cold bends, re-entrant cuts, etc.) shall be performed by the Contractor after all heating is completed and the material returns to ambient temperature.

Commentary:

This procedure has similar requirements and limitations to that in Section 8.1, "Correcting Girder Camber," with the heat patterns changed to be suitable for sweep adjustment.

8.3 Correcting Flange Distortion or Tilt

Issue:

During the process of welding the flange to the web, the flange distorts or tilts out of square to the web.

Recommendation:

One type of distortion involves flange edges being pulled toward the web, sometimes referred to as "cupping", by weld shrinkage. This is most common when attaching light flanges (i.e., up to about 7/8" [22 mm] thick) to webs with 5/16" [8 mm] or larger fillets or by CJP welds. To reduce existing flange cupping, apply a strip heat approximately 1-1/2" [40 mm] wide, centered over the web (see Figure 18). To enhance effects, install jacks or other restraints between flanges to apply restraint before and during heating.

If cupping is observed or anticipated, the Fabricator can take pro-active steps to reduce effects on subsequent girders. These pro-active steps include, but are not limited to, pre-bending of flanges before installation to compensate for expected movement, increasing weld preheat to reduce shrinkage stresses, and inquiring if the flange-to-web weld size may be decreased to the minimum per Table 2.1 in D1.5.

Another type of distortion is flange tilt, leaving the flange non-perpendicular to the web (see Figure 18). This usually occurs with heavy flanges and large fillet or CJP flange-to-web welds. The flange is stiff enough to resist cupping, and the distortion results from shrinkage of the first side weld rotating the flange. Shrinkage stresses during second side welding are not large enough to overcome the first side's restraint. To correct flange tilt, partial or complete removal of the weld metal between the flange and the web on the obtuse side is normally required. For fillet welds, the obtuse flange-to-web weld is completely removed and enough web material is excavated to permit the flange to rotate back to "square". For CJP welds, excavate the weld and part of the web from the obtuse side. For both fillet and CJP flange-to-web welds, preheat both the remaining weld and the side to be welded to reduce restraint and subsequent residual stresses and apply jacking force to the acute side of the flange. Preheat will typically be higher than normal, but should not exceed about 400º F [200º C] to avoid hot-working (i.e., plastically deforming the metal) the weld fusion and HAZ areas. Jacking alone must not be used to correct the tilt, since that could cause cracking or distort the web, and subsequent welding could over-correct the initial problem.

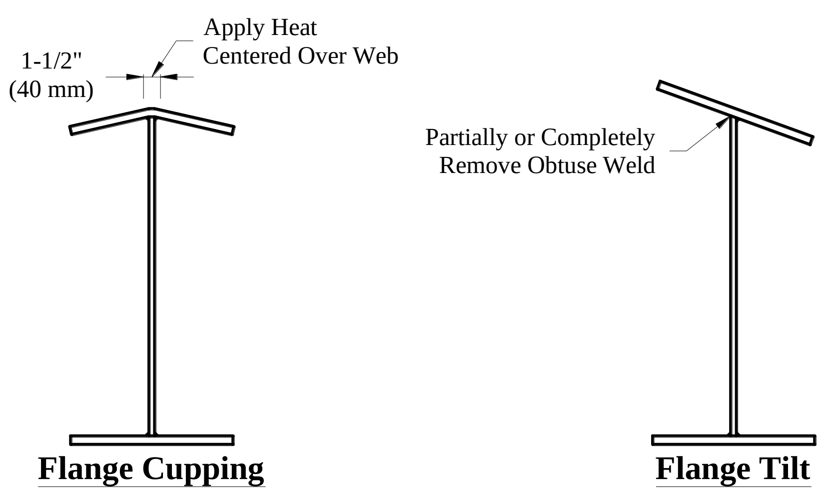

Figure 18: Flange Cupping and Tilt

If tilt is observed or anticipated, problems should be reduced or avoided on subsequent girders by pre-positioning flanges to anticipate rotation and by changing welding procedures and sequences to balance forces.

For both types of distortion, depending on severity, location, and tolerance, correction may be required along the full length of the girder, or only at the bearing and splice locations.

Commentary:

Distortion or tilting in a flange can cause significant problems at field splices and bearings. D1.5 has stringent criteria for flanges at bearings. Attempting to force flanges into position by jacking or driving in bearing stiffeners can cause severe web distortion and residual stresses that may later buckle stiffeners.

Both flange cupping and tilt are caused by weld shrinkage, showing the extremely high residual stresses this phenomenon can cause when movement is restrained. If jacks or other restraints are used during welding to restrict flange motion, additional NDE may be needed to verify hot cracking did not occur in the weld or fusion zone. If the QCI and QAI cannot agree on alternate procedures or sequences to correct the problem, the Fabricator's engineer and the Owner should discuss possible alternatives, including different weld configurations or consumables.

8.4 Correcting Web Distortion

Issue:

Web distortion in the form of waviness, otherwise known as "oil-canning", is beyond permitted tolerances and requires correction.

Recommendation:

"Oil-canning" is caused by weld shrinkage around the perimeter of a web "panel", bounded by the flanges and interior stiffeners or connection plates. It is common for thin (i.e., less than 1/2" [12 mm] thick) webs, especially if 5/16" [8 mm] and larger welds are used. The amount of distortion is dependent on the web thickness, welding sequence, and panel dimensions. Reducing it to within allowable tolerances requires correction by judicious use of mechanical methods and/or heat patterns. Additional vertical stiffeners can be welded or bolted to the member. The web is moved back before attaching the stiffeners, usually to the concave side of the web (bowing inward) for welded or convex side for bolted.

Heating the web entails application of perimeter and/or spot heats to the web while limited jacking pressure is applied to the convex side. Excessive jacking can lead to local buckling, which is a much worse problem than the "oil-canning". Heat patterns should be planned based on Report No. FHWA-IF-99-004 (see the introduction to Section 8, "Heat Application") and the Owner should review and accept the Fabricator's plan before implementation (the Fabricator remains responsible). Heat patterns may be used in conjunction with added stiffeners, depending on the amount and location of distortion.

Commentary:

Web distortion happens frequently on relatively thin webs, especially when the Fabricator uses high heat input tandem/parallel SAW welding processes.

The preferred technique to resolve this problem is the application of heat to the web. Sometimes, corrections will result in distortion appearing in another panel section and it may take several attempts to correct the problem. D1.5 is fairly lenient on the amount of web deflection (i.e., localized lateral bowing) allowed.

If “oil-canning” is observed or anticipated, it may be reduced on subsequent girders by changing the welding sequence (e.g., alternating sides to “balance” stresses), modifying the weld procedure (e.g., modifying preheat or adding post heat), or determining if smaller welds may be employed. Production workers should never apply heat to webs without controls or a firm plan, since distortions may increase or more serious defects may result.

8.5 Improper Use or Monitoring of Heat Application

Issue:

The temperature was not properly monitored or there is reason to suspect that the material was overheated during heat application to a member.

Recommendation:

Overheating may reduce ductility and toughness, initiate or propagate discontinuities, cause local plastic distortion and high residual stresses, and adversely affect material properties of quenched and tempered steel. Changes to material properties are essentially irreversible, so a repair may not be possible without partial or complete replacement.

If the Fabricator desires to use the item, the physical properties of the heated areas must be assessed to determine if the material is still acceptable to the Owner. Subject to prior agreement between the Owner and the Fabricator, specific tests may validate the material’s acceptability. Hardness tests, such as Brinell, Knoop, Vickers, or Rockwell, and NDE, including, but not limited to, MT and UT, on both heated and unheated areas may provide an indication of whether damage occurred. Yield, ductility, and/or Charpy V-notch toughness tests can also be done, but they require material removal from the suspect area and careful preparation of specimens, insuring the primary rolling direction is maintained. A metallographic analysis can determine if the microstructure has changed, but this is more difficult and less conclusive than ductility and toughness testing. Tests are the Fabricator's responsibility, and must be conducted by a qualified, third-party testing agency.

The material may be accepted if the mutually agreed testing provides adequate assurance of its properties. Otherwise, the material must be replaced unless the Contractor's engineer can propose a scheme acceptable to the Owner to salvage the piece by supplemental strengthening. If the specimen tests are utilized, the Fabricator must develop an acceptable method to restore the areas where material was extracted.

If the concern is not metallurgical damage, but rather high residual stresses due to constraint in a heated area, stress relief may be appropriate, but this is not an overheating issue. The Owner and the Fabricator should discuss that situation and determine both its cause and potential remedies. If the design details lead to the condition, then the Owner may be responsible for corrections.

Commentary:

Possible visual indicators that the material has been overheated include, but are not limited to:

1. A bright ("cherry") red color visible in the base metal after heat application. However, a dull red that fades away within seconds after the torch is removed is usually acceptable, subject to verification by temperature-indicating devices. Ambient lighting is critical to interpretation, since any red visible in full daylight after heat is removed implies overheating, but when heating in a dimly lit shop, a dull red "glow" may persist a few seconds in steel heated to 1100º F [590º C]. Also, a red color directly under the torch flame does not constitute overheating,

as long as it disappears as the flame moves. This is especially true if thin mill scale or rust is present.

2. Local distortion/melting of the base metal. Star-burst surface cracks or actual displacement (flow) of the metal indicate temperatures far in excess of those permitted. Melting of thin mill scale is not suitable evidence of overheating, since the torch may rapidly elevate it without bringing the base metal above the maximum permitted levels. Oxy-fuel flame temperatures vary from about 5000º F [2760º C] for natural gas to 5700º F [3150º C] for acetylene, so surface temperatures directly under the flame will exceed 1200º F [650º C] in order to quickly bring the through-thickness temperature above 1000º F [540º C]. Therefore, temperature testing is delayed 3 to 8 seconds after the torch leaves the area.
3. Discoloration of the material's surface. Based on the chemistry (especially carbon content) of the steel, the color of "bluing" can provide an indication of the maximum temperature reached.

Hardness testing of questionable areas has limited value in determining if temperatures exceeded specified limits, since most modern bridge steels have very low carbon equivalents, and consequently are not very hardenable. This is a "good news - bad news" situation. The good news is that these steels are usually not significantly affected by overheating a few hundred degrees, especially if permitted to cool gradually. The bad news is that proving whether properties were diminished is more difficult than it would have been for more hardenable steels. Requiring removal of material to perform physical testing should be viewed as a last option, probably reserved for material in highly stressed situations.

Ironically, that means inflicting substantial repairs to restore an area where additional welds are undesirable. Any specimens should be taken from an edge to avoid welding material back into a highly restrained condition, surrounded by solid metal.

The QCI should provide written documentation to the QAI, indicting what training and production safeguards would be enacted to prevent recurrences of overheating.

Refer to Table 5.1 of S2.1 for the maximum temperature limits for heat application on typical grades of bridge steel.

9 - REPAIR DATABASE

A database of corrective actions can provide guidance and improve confidence in solutions to unusual but non-unique problems. When DOTs and fabricators within a geographic region use a sharable and well-reasoned repair database, standardized solution procedures expedite fabrication and reduce fabrication costs, which result in a reduction of costs to the DOTs and the public.

The AASHTO/NSBA Steel Bridge Collaboration Task Group 5 is assembling a repair database in the context of an FHWA pooled-fund research project at the University of Kansas. The repair database software, **F**abrication error **I**ndexed e**X**amples and **S**olutions (**FIXS**), recommends corrective action for fabrication errors or damage to steel bridge members during construction. FIXS provides solutions and examples to steel bridge fabrication errors with graphical and instructive explanations based on both rule-based and case-based reasoning. The type of error, the reasoning behind the trial solution, and the outcome of the solution are all included in the FIXS database so that when the same or similar type of error occurs in the future, engineers can benefit from or improve upon previous solutions. To make the software easily available by using the World Wide Web, a web site < http://www.ceae.ku.edu/fixs.html > is under development. The website is intended to provide access to the case-base, including mechanisms for adding and discussing cases.

REFERENCES

[A 6] Annual Book of ASTM Standards - 2002.
Section 1: Iron and Steel Products.
Volume 01.04: Steel - Structural, Reinforcing, Pressure Vessel, Railway.

This collection is annually revised by ASTM, which is one of the largest voluntary standards development systems in the world. The standard that is of interest and referenced often within this document is A 6/A 6M, "Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling."

[AISC] AISC LRFD Manual of Steel Construction. 3rd Edition.

This widely used steel manual is tailored to building design, but is an all-around good reference whenever dealing with steel construction. The sections consulted for this document were Section 9, specifically Pages 9-15 and 9-16, for coping practices, Tables 7-3a and 7-3b for clearance requirements for high-strength bolts, and Section 16.5, "Specification," for Structural Joints Using ASTM A325 or A490 Bolts (see [RCSC]).

[D1.1] AWS D1.1/D1.1M:2002 - Structural Welding Code – Steel.
An American National Standard.

This publication of the American Welding Society (AWS) is widely used. It covers the welding requirements for any type of welded structure made from commonly used carbon and low-alloy constructional steels.

[D1.5] AASHTO/AWS D1.5M/D1.5:2002 - Bridge Welding Code.
An American National Standard.

This joint publication of the American Association of State Highway and Transportation Officials (AASHTO) and the American Welding Society (AWS) is widely used. It covers the welding requirements for AASHTO welded highway bridges made from carbon and low-alloy constructional steels.

[FHWA] Avent, Richard R. and David Mukai.
Report No. FHWA-IF-99-004. October 1998.
Heat-straightening Repair of Damaged Bridges.
A Manual of Practice and Technical Guide.

The purpose of this manual is to provide comprehensive guidelines on heat straightening repair techniques for damaged steel bridge members. The manual is designed to be used in conjunction with a multimedia instructional computer program and video produced as part of this project. This document is available from the FHWA Office of Technology Applications, 400 7th St. SW, Washington, DC 20590, phone (202) 366-7900.

[Fisher] Fisher, John W.
Fatigue and Fracture in Steel Bridges - Case Studies.
A Wiley-Interscience Publication.
John Wiley & Sons, Inc. Copyright © 1984.

John W. Fisher, Professor of Civil Engineering and Associate Director of the Fritz Engineering Laboratory at Lehigh University, is a recognized expert in the fatigue and fracture resistance of riveted, bolted, and welded connections. This book presents several case studies of cracking in steel bridges since the late 1960's. Included are pictures and evidence of the cracking or failures related to fatigue of steel bridge structures. The section of interest for this document was Chapter 7, "Welded Holes."

[LFD] AASHTO Standard Specifications for Highway Bridges.
Sixteenth Edition - 1996.

This highway bridge design reference is used throughout the industry. The sections of interest for this document are Section 10.24, "Fasteners (Rivets and Bolts)," and Section 10.34.4, "Transverse Intermediate Stiffeners."

[LRFD] AASHTO LRFD Bridge Design Specifications.
Second Edition - 1998.

This highway bridge design reference is based on Load and Resistance Factored Design or LRFD principles. The sections of interest for this document are Section 6.3.2.6, "Edge clearances," Section 6.10.8, "Stiffeners," and Section 6.13, "Connections and Splices."

[Owens, et al.] Owens, Graham W., Peter R. Knowles, and Patrick J. Dowling.
Steel Designers' Manual - Fifth Edition.
The Steel Construction Institute.
Blackwell Scientific Publications. Copyright © 1992.

This manual was consulted for a guideline of maximum gap permitting shims to reduce gaps in full contact bearings. The section of interest for this document was Section 31.5.6, "Shimming full contact bearing splices." The reference cites tests commissioned by AISC that showed that it is not prudent to permit shimming for gaps exceeding 6 mm.

[RCSC] Research Council on Structural Connections - *Specification for Structural Joints Using ASTM A325 or A490 Bolts*. June 23, 2000.

This specification is a companion to AISC 350-99 that extends coverage to the use of ASTM A325, F1852 and A490 high-strength bolts in steel-to-steel structural connections, including materials, design, installation, and inspection. It is available as a standalone document, which can be downloaded at the following URL: < http://www.boltcouncil.org >. It is also located in Section 16.5, "Specification," of the 3rd Edition of the AISC LRFD Manual of Steel Construction (see [AISC]).

[S2.1] AASHTO/NSBA Steel Bridge Collaboration S2.1 - 2002.
Steel Bridge Fabrication Guide Specification, May 2002.
Task Group 2, Fabrication.

This specification was written by experienced representatives from a number of fabricators, state DOTs, consultants, and the FHWA. The work was based on existing state specifications, the Bridge Welding Code, and the AASHTO bridge design and construction manuals. This standard was written with the intent to be used in close tandem with Collaboration standard S4.1. It is available as a standalone document, which can be downloaded at the following URL: < http://www.steelbridge.org/standards.htm >

[S4.1] AASHTO/NSBA Steel Bridge Collaboration S4.1 - 2002.
Steel Bridge Fabrication QC/QA Guide Specification, May 2002.
Task Group 4, QC/QA.

This document establishes and defines the functions, operations, requirements, and activities needed to achieve consistent quality in steel bridges. The responsibilities of each party are laid out and discussed. The document concentrates on the Quality Control and Quality Assurance aspects of the fabrication process. It is available as a standalone document, which can be downloaded at the following URL:< http://www.steelbridge.org/standards.htm >

[Wang & Roddis] Wang, Xiuzhen and W.M. Kim Roddis.
Fabrication error Indexed eXamples and Solutions: FIXS.
K-TRAN Project No. KU-98-8. MATC/KU 98-2.
University of Kansas Center for Research, Inc.
Lawrence, Kansas. June 1999.

This reference is a report on research sponsored by the Kansas Department of Transportation (KDOT), the Mid-America Transportation Center (MATC), and the National Steel Bridge Alliance (NSBA). This reference focuses on the cases documented and used in the FIXS project. Numerous cases of fabrication errors occurring during bridge construction are discussed with figures provided for most.

[WHB] AWS WHB-1.9 - Welding Handbook.
Volume 1 - Welding Science & Technology.

This publication of the American Welding Society (AWS) is the best-known welding handbook in the industry. Written in descriptive, everyday language, it offers ample detail to satisfy the welding professional while providing understanding to students and those new to the field.