Steel Bridge Fabrication Guide Specification

AASHTO/NSBA Steel Bridge Collaboration
Preface

This document is a standard developed by the AASHTO/NSBA Steel Bridge Collaboration. The primary goal of the Collaboration is to achieve steel bridges of the highest quality and value through standardization of the design, fabrication, and erection processes. Each standard represents the consensus of a diverse group of professionals.

As consensus documents, the Collaboration standards represent the best available current approach to the processes they cover. It is intended that Owners adopt and implement Collaboration standards in their entirety to facilitate the achievement of standardization, but it is understood that local statutes or preferences may prevent full adoption for some. In such cases Owners should adopt these documents with the exceptions they feel are necessary.

Disclaimer

All data, specifications, suggested practices presented herein, are based on the best available information and delineated in accordance with recognized professional engineering principles and practices, and are published for general information only. Procedures and products, suggested or discussed, should not be used without first securing competent advice respecting their suitability for any given application.

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Introduction

This Guide Specification governs steel bridge fabrication. Provisions are intended to result in the achievement of high quality and value. It is intended to be included in Contract documents by reference from the Owner’s specifications. Parts designated as “commentary” are not contractual.

This standard is intended to be used in close tandem with AASHTO/NSBA Steel Bridge Collaboration S4.1, “Steel Bridge Fabrication QC/QA Guide Specification”, as required in Section 2.8 of this document.

For new painted steel bridges using an inorganic zinc-rich primer system, Owners are encouraged to adopt AASHTO/NSBA Steel Bridge Collaboration S8.1, “Guide Specification for Coating Systems with Inorganic Zinc-Rich Primer”.

The Collaboration is also developing a guide specification for steel bridge erection, AASHTO/NSBA Steel Bridge Collaboration S10.1. Owners are encouraged to adopt the erection standard when it becomes available.

In this standard, imperatives are directed to the Contractor and Fabricator. Many references are made to the current AASHTO/AWS D1.5M/D1.5 Bridge Welding Code, referred to in this standard as the Bridge Welding Code.
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Section 1
Definitions and Responsibilities

Terms used in this standard are in accordance with the AASHTO/NSBA Steel Bridge Collaboration standards. Terms significant specifically to this document are defined below.

1.1 Contractor
The Contractor is responsible for proper completion of all tasks required by the Contract documents. Subcontractors, including fabricators, erectors, and field painters, may be used by the Contractor, but the Contractor retains responsibility for material, operations, and the final product. The Contractor may 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. The Contractor may permit or reject the changes.

1.2 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 subcontract fabricators and those who prepare Shop Detail Drawings. In some cases the Fabricator may also be the Contractor, but usually the Fabricator is subcontracted by the Contractor. In this document, the term “Fabricator” is used to describe roles usually held by the Fabricator, but this could also be the prime contractor.

1.3 Owner
1.3.1 In this document, “Owner” refers to the entity paying the Contractor to fulfill the terms of the Contract. The Owner encompasses both those preparing the Contract documents, including the Designer responsible for the structure’s adequate design, and those representing the Owner during construction, commonly called the Engineer and the Inspector. The Designer, Engineer, and Inspector may be employees either of the Owner or of professional firms contracted for the work. In this standard, “Inspector” and “Engineer” are used when describing those specific responsibilities, and “Owner” is used when the role could be either.
1.3.2 An “approved procedure” in this document means one approved by the Engineer.

1.4 Primary members
Primary members are elements designed to carry loads.
Steel Bridge Fabrication Guide Specification

Section 2
Prefabrication

2.1 Fabricator certification
2.1.1 Certification from the AISC Quality Certification Program is required for Fabricators in the category appropriate for the type of work being performed. The categories include:

- Simple Steel Bridge Structures (Sbr) - Required for highway sign fabrication, bridge parts such as cross frames, and unspliced rolled beam bridges
- Major Steel Bridges (Cbr) - Required for all bridges other than unspliced rolled beam bridges.
- Fracture Critical Endorsement (F) - Required for any fabrication conducted on fracture critical structures
- Sophisticated Paint Endorsement (P) - Required for any shop and/or field painting performed on steel bridges

2.1.2 SSPC-QP3, “Standard Procedure for Evaluating the Qualifications of Shop Painting Contractors”, may be substituted for the Sophisticated Paint Endorsement.

2.1.3 Allow the Owner to review the certification records upon request. Resolve all findings noted during this review prior to fabrication.

2.2 Communication
2.2.1 Prior to beginning work, Owner and Contractor representatives shall identify individuals who are responsible for the following functions:

- Preparation, submittal, review, approval, and distribution of shop drawings
- Submittal and control of material test reports (MTRs)
- Quality Control (QC) and Quality Assurance (QA)
- Engineering, including the Designer, the Fabricator’s Project Engineer, and the Owner’s Engineers for technical submittals during fabrication.

2.2.2 The Owner will identify individuals or agents responsible for handling shop detail drawings, approving welding procedures, providing quality assurance, and accepting proposed repairs.

2.2.3 During the project, maintain effective communications with the Owner’s representatives. Address problems and concerns as early as possible in the work.

2.2.4 On complex projects, start communication about special aspects of the job, including tolerances or other requirements, very early in the project.

2.3 Shop drawings
2.3.1 Provide separate shop drawings for each steel structure on a project. Dual (twin) bridges shall have separate drawings for future reference.

2.3.2 Prepare and submit shop drawings in accordance with AASHTO/NSBA Steel Bridge Collaboration G1.3, “Shop Detail Drawing Presentation Guidelines”.

2.3.3 Do not begin fabrication until drawings are approved or approved-as-noted. Work performed prior to shop drawing approval is at the Fabricator’s risk, and may require additional inspection, NDT/NDE, or partial disassembly/reassembly to satisfy the Owner’s QA.
2.4 Prefabrication meeting

2.4.1 Before work begins, a prefabrication meeting may be held at the discretion of the Owner or if requested by the Fabricator or Contractor.

2.4.2 The Owner will provide design, engineering, and QA inspection representatives.

2.4.3 The Fabricator representatives will include the plant manager; engineering, production, and quality control personnel; and, if appropriate, subcontractor representatives (e.g., painter, subcontract fabricators, or material suppliers).

2.4.4 Review these aspects of the job:
- Progress on shop drawing submittal and approval
- Plant and personnel certification
- Organizational structure and primary (lead) plant personnel
- Handling of Material Test Reports (MTRs)
- Traceability of materials
- Fabrication procedures, especially shop assembly, welding, and painting
- Supply and sampling of paint, bolts, and other materials, if applicable
- Work schedule
- Availability and advance notification of quality assurance inspectors (QAIs)
- Inspector’s office
- Appropriate lines of communication
- Project-specific areas of concern for fabrication and inspection, including any special applications of non-destructive examination and testing (NDE/NDT)
- Handling of non-conformance and repair issues
- Special requirements, especially any exceptions to this specification
- Project details, requirements, or processes that have caused prior difficulties
- Loading and shipping

2.5 Procedures

2.5.1 Written procedures must be maintained for the fabrication processes listed below. These are subject to the Owner’s review and acceptance.
- Material traceability
- Hot and cold bending
- Welding (welding procedure specifications must be approved by the Engineer)
- Cambering and heat-curving, including temperature measurement, patterns and sequences (must be approved by the Engineer)
- Shop assembly/laydown, including drilling and punching
- Postheat and stress-relieving procedures
- Shop installation of fasteners, with rotational capacity (RC) test, if applicable
- Blast cleaning and painting

2.5.2 Each procedure must explain how tasks will be accomplished, evaluated and accepted by both the quality control inspector (QCI) and the QAI (as applicable) prior to subsequent operations.

2.5.3 The procedures may be standardized and not require resubmittal and approval for each project.
2.6 **Commencement of work**
Provide a written advance notice to the Owner a minimum of two weeks before fabrication begins.

2.7 **Evaluation of the work**
2.7.1 The QAI will evaluate the work and accept fabricated components that satisfy the requirements of the contract documents.
2.7.2 The Engineer may accept fabricated components that do not fully conform to the contract provided the Engineer is satisfied that alternate practices or work proposed by the Fabricator will not compromise the durability, performance, or integrity of the structure.

2.8 **Quality assurance/quality control**
2.8.1 Perform QC inspection using trained and qualified personnel in accordance with AASHTO/NSBA Steel Bridge Collaboration S4.1, “Steel Bridge Fabrication QC/QA Guide Specification”.
2.8.2 The Owner will provide QA inspection in accordance with AASHTO/NSBA Steel Bridge Collaboration S4.1, “Steel Bridge Fabrication QC/QA Guide Specification”.

2.9 **Progress meetings**
Progress meetings may be held during the course of the work at the discretion of the Owner or at the request of the Fabricator or Contractor.

2.10 **Safety**
Perform work in accordance with industry codes for safety, including OSHA regulations and any other applicable codes or restrictions.
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Section 3
Material Control

3.1 Quality
3.1.1 Provide materials that satisfy contract requirements.
3.1.2 Material meeting equivalent AASHTO and ASTM specifications may be supplied under either specification.

3.2 Certifications and verification
3.2.1 Provide certified mill tests reports (MTRs) for all steel materials used in fabrication, including plates, bars, shapes, and fasteners. MTRs must originate from the producer of the material and not from a supplier, unless the Owner permits specific supplemental testing of stock or service-center-supplied material for toughness or other parameters.
3.2.2 Check the MTRs to ensure that material is in conformance with the applicable material specification, including actual values from required tests.
3.2.3 Use material from stock only if it can be positively identified, if the appropriate documentation is provided, and if the direction of rolling, when required, can be established.
3.2.4 When “Buy America” restrictions apply, provide MTRs that demonstrate that the materials were melted and manufactured in the United States and certify in writing that all applicable “Buy America” requirements are satisfied.

3.3 Identification and traceability
3.3.1 Ensure that all structural steel materials are identified in accordance with ASTM A 6.
3.3.2 Maintain heat numbers on all primary bridge materials until the material is permanently joined into a piece-marked member. Use paint stick or other suitable medium that does not damage the member.
3.3.3 Maintain documentation of all primary member material for shop records and provide this documentation to the QAI for the Owner’s records.

3.4 Handling, storage, and shipment
3.4.1 Handle, store, and ship raw and fabricated materials in a manner that protects them from damage, facilitates subsequent inspections, and does not compromise the safety of personnel.
3.4.2 Place raw and fabricated materials above the ground on platforms, skids, or other supports.
3.4.3 Keep materials free from dirt, grease, and other foreign matter, and provide proper drainage for materials stored outside.
3.4.4 Protect materials from detrimental corrosion or coating deterioration.
3.4.5 Organize bulk materials such as fasteners and studs into separate production lots and store them so that they are protected from adverse environmental conditions and that traceability is maintained.
3.4.6 Store paint in accordance with manufacturer's recommendations.
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Section 4
Workmanship

4.1 Cutting, shearing and machining
4.1.1 Cut and shear materials in accordance with Bridge Welding Code tolerances and with the following:
- For primary member plate components thicker than 15 mm (5/8 in.), plane 5 mm (3/16 in.) off sheared edges that remain exposed after fabrication,
- Cut and fabricate steel plates for primary member components and splice plates with the direction of rolling parallel to the direction of primary stresses. The primary stress for web plates is assumed to be parallel to flanges unless otherwise shown on the approved shop drawings. The Engineer may permit the rolling direction of web splice plates to be perpendicular to the flanges, based on anticipated horizontal and vertical load components and the splice plate sizes required.
4.1.2 Machine in accordance with the contract requirements and applicable codes, specifications and accepted industry practices.

4.2 Contact and bearing surfaces
4.2.1 Finish bearings, base plates, and other contact surfaces to the ANSI surface roughness requirements defined in ANSI B46.1, "Surface Roughness, Waviness and Lay", Part I, given in Table 4.1, unless otherwise noted in the contract.

<table>
<thead>
<tr>
<th>Table 4.1 ANSI Surface Roughness Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel slabs</td>
</tr>
<tr>
<td>ANSI 50 µm (2000 micro-inches)</td>
</tr>
<tr>
<td>Heavy plates in contact with shoes to be welded</td>
</tr>
<tr>
<td>ANSI 25 µm (1000 micro-inches)</td>
</tr>
<tr>
<td>Milled ends of compression members, milled or</td>
</tr>
<tr>
<td>ground ends of stiffeners or rockers</td>
</tr>
<tr>
<td>ANSI 10 µm (500 micro-inches)</td>
</tr>
<tr>
<td>Bridge rollers and rockers</td>
</tr>
<tr>
<td>ANSI 5 µm (250 micro-inches)</td>
</tr>
<tr>
<td>Sliding bearings</td>
</tr>
<tr>
<td>ANSI 3 µm (125 micro-inches)</td>
</tr>
<tr>
<td>Pins and pin holes</td>
</tr>
<tr>
<td>ANSI 3 µm (125 micro-inches)</td>
</tr>
</tbody>
</table>

4.2.2 For I-girders, ensure that flanges are square to webs prior to attachment of stiffeners or connection plates. For skewed web-to-flange orientations, such as trapezoidal box girders, ensure proper orientation prior to attachment of stiffeners or connection plates.

4.3 Cold bending
4.3.1 Do not cold-bend fracture-critical materials.
4.3.2 When possible, orient bent plates for connections so that the bend line will be approximately perpendicular to the direction of rolling. If the bend line must be approximately parallel to the direction of rolling, multiply the suggested minimum radii in Table 4.2 by 1.5.
4.3.3 Reject material with non-specified kinks or sharp bends, cracks, large dents, or visible reduction of section (necking).

4.3.4 Visually inspect all load points, and check any suspected damage by magnetic particle testing (MT).

4.3.5 For bent plates, use the largest bend radius that the finished part will permit and ensure that the surfaces of dies, rams, restraints or other tools are smooth. Use a width across the shoulders of the female die of approximately 8 times the plate thickness for Grade 250 (36) material. Higher strength steels may require larger die openings.

4.3.6 Where the concave face of a bent plate must uniformly contact an adjacent surface, ensure the male die is thick enough and has the proper radius to result in the required curvature.

4.3.7 Before bending, break (slightly chamfer or radius) corners in the area to be bent.

4.3.8 Suggested minimum bend radii for cold bending, measured to the concave face of the plate, are given in Table 4.2. If a smaller radius is required, see Section 5.6.

<table>
<thead>
<tr>
<th>Material</th>
<th>Grade</th>
<th>Radius in Terms of Plate Thickness, t mm (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM specification</td>
<td>Grade</td>
<td>t &lt; 25 (1)</td>
</tr>
<tr>
<td>A36/A36M</td>
<td>--</td>
<td>1.5t</td>
</tr>
<tr>
<td>A572/A572M</td>
<td>290 (42)</td>
<td></td>
</tr>
<tr>
<td>A709/A709M</td>
<td>250 (36)</td>
<td></td>
</tr>
<tr>
<td>A572/A572M</td>
<td>345 (50)</td>
<td>1.5t</td>
</tr>
<tr>
<td>A588/A588M</td>
<td>--</td>
<td></td>
</tr>
<tr>
<td>A709/A709M</td>
<td>345 (50), 345W (50W)</td>
<td>1.5t</td>
</tr>
<tr>
<td>A572/A572M</td>
<td>380(55) HPS 485W (70W)</td>
<td>1.5t</td>
</tr>
</tbody>
</table>

4.4 Straightening
4.4.1 Straighten bridge member parts, such as plates, angles or shapes, before the parts are assembled.

4.4.2 If materials are cold-straightened, follow the applicable provisions of Section 4.3.

4.4.3 If heat is to be used for straightening, apply the provisions of Section 5.5.

4.5 Welding
4.5.1 Weld built-up plate and open rolled-shape structural elements in accordance with the Bridge Welding Code.

4.5.2 Weld tubular structural elements in accordance with AWS D1.1, Structural Welding Code.

4.5.3 Weld HPS materials in accordance with the requirements of Appendix A of the AASHTO/AISI Guide for Highway Bridge Fabrication with HPS70W Steel.

4.5.4 Do not weld or tack brackets, clips, shipping devices, or other materials not required by the contract to any member unless permitted by the Engineer and shown on the approved shop drawings.
4.6 Bolt holes

4.6.1 Fabricate bolt holes to the workmanship requirements of the latest edition of the Research Council on Structural Connections (RCSC) Specification for Structural Joints Using ASTM A325 or A490 Bolts. Use dimensions and tolerances based on the actual fasteners provided, whether US customary or metric.

4.6.2 Ensure that bolt holes in primary members meet the following criteria:

- Square to splice plates within \( \frac{\sqrt{2}}{20} \).
- No tears, cracks, fins, dirt, loose rust, burrs, or other anomalies that could impede intimate contact or concentrate stress.
- Round within \( \pm 1 \text{ mm} \) (\( \frac{\sqrt{2}}{32} \text{ in.} \)).
- Within \( +1 \text{ / } - 0 \text{ mm} \) (\( +\frac{\sqrt{2}}{32} \text{ / } - 0 \text{ in.} \)) of the specified size.
- For subsize holes, able to pass a pin 3 mm (\( \frac{\sqrt{16}}{32} \text{ in.} \)) smaller than the subsize holes through all assembled plies in at least 75\% of the holes prior to reaming.
- Thermal-cut holes or portions of slots ground to ANSI 25 \( \mu \text{m} \) (1000 micro-inches).

4.6.3 Do not punch holes full size in primary members.

4.6.4 Apply these maximum thickness limits when punching:

- Grade 250 (36): 20 mm (\( \frac{3}{8} \text{ in.} \)).
- Grade 345/345W (50/50W): 16 mm (\( \frac{5}{32} \text{ in.} \)).
- Grade HPS 70W: 12 mm (\( \frac{1}{8} \text{ in.} \)).

4.6.5 When slotted holes are required by the contract:

- Use AASHTO short slotted holes if the contract calls for slotted holes but does not provide dimensions.
- Make slots by a single punch or by joining two adjacent drilled or punched holes using guided thermal cutting.
- Do not make slotted holes more than 1 mm (\( \frac{\sqrt{16}}{32} \text{ in.} \)) greater in width or 2 mm (\( \frac{1}{64} \text{ in.} \)) greater in length than specified.

4.6.6 Do not thermally cut holes in quenched and tempered steel.

4.6.7 Assess bolt hole quality in primary members with the members and splice plates assembled, except in cases where the use of computer numerically controlled (CNC) drilling equipment or bushed templates are allowed (see Section 7.4).

4.6.8 Holes in floor beam to primary member connections and stringer to floor beam connections do not require shop assembly verification unless specified by the contract.

4.6.9 Do not use temporary welds to secure materials while drilling or reaming through multiple plies.

4.6.10 Locate bolt hole centers no closer than the minimum as-fabricated distances from the center of a bolt hole to an edge given in Table 4.3 or 4.3a.

4.6.11 The maximum fabricated edge distance is the lesser of 8 times the thickness of the thinnest outside plate or 125 mm (5 in.).

4.6.12 The tolerance for bolt hole spacing is \( \pm 5 \text{ mm} \) (\( \frac{1}{64} \text{ in.} \)).

4.6.13 When slip-critical faying surfaces are to be primed, ensure that the coating is certified to provide the needed friction.

4.6.14 Prepare non-painted faying surfaces in accordance with RCSC Specification requirements.
### Table 4.3 Minimum Fabricated Edge Distances (metric units)

<table>
<thead>
<tr>
<th>Fastener Size (mm)</th>
<th>Sheared Edges (mm)</th>
<th>Rolled or Gas-Cut Edges (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>28</td>
<td>22</td>
</tr>
<tr>
<td>20</td>
<td>34</td>
<td>26</td>
</tr>
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<td>38</td>
</tr>
<tr>
<td>36</td>
<td>64</td>
<td>46</td>
</tr>
</tbody>
</table>

### Table 4.3a Minimum Fabricated Edge Distances (US customary units)

<table>
<thead>
<tr>
<th>Fastener Size (in.)</th>
<th>Sheared Edges (in.)</th>
<th>Rolled or Gas-Cut Edges (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>¾</td>
<td>1 ⅛</td>
<td>¾</td>
</tr>
<tr>
<td>⅝</td>
<td>1 ⅛</td>
<td>1</td>
</tr>
<tr>
<td>⅞</td>
<td>1 ½</td>
<td>1 ⅛</td>
</tr>
<tr>
<td>1</td>
<td>1 ⅝</td>
<td>1 ¼</td>
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<tr>
<td>1 ⅝</td>
<td>2</td>
<td>1 ½</td>
</tr>
<tr>
<td>1 ⅞</td>
<td>2 ⅛</td>
<td>1 ¾</td>
</tr>
<tr>
<td>1 ⅝</td>
<td>2 ⅜</td>
<td>1 ¾</td>
</tr>
</tbody>
</table>

### 4.7 Bolting

4.7.1 Perform rotational capacity (RC) tests prior to installation of permanent fasteners in primary connections. Test in accordance with *High Strength Bolts for Bridges*, Appendix A1, “Procedure for Performing Rotational Capacity Tests”, FHWA report number FHWA-SA-91-031.

4.7.2 Install fasteners in accordance with the RCSC Specification, Item 8(d), "Joint Assembly and Tightening of Slip-Critical and Direct Tension Connections". If special fasteners not addressed by the Bolt Council specification are required, install them in accordance with the manufacturer’s recommendations.
5.1 Heating process and equipment
5.1.1 Do not exceed the maximum allowable temperatures given in Table 5.1 when applying heat to steel.

<table>
<thead>
<tr>
<th>Grade</th>
<th>Maximum temperature ° C (° F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>250 (36)</td>
<td>650 (1200)</td>
</tr>
<tr>
<td>345 (50), 345W (50W), HPS 345W (HPS50W)</td>
<td>650 (1200)</td>
</tr>
<tr>
<td>HPS 485W (HPS 70W), Q&amp;T and TMCP</td>
<td>600 (1100)</td>
</tr>
<tr>
<td>690/690W (100/100W) and HPS 690/690W (100/100W)</td>
<td>600 (1100)</td>
</tr>
</tbody>
</table>

5.1.2 Complete heating before painting.
5.1.3 Limit stresses due to preload (including loads induced by member weight) to 0.5 $F_y$, where $F_y$ is the nominal yield strength of the material.
5.1.4 When jacks are used, apply and lock off load before applying heat.
5.1.5 Use only orifice tips, and proportion tip size to the thickness of the material.
5.1.6 Manipulate heating torches to guard against overheating.
5.1.7 When vee or rectangular heat patterns are used, mark the patterns on the steel prior to heating.
5.1.8 Bring steel within the planned temperature as rapidly as possible without overheating.
5.1.9 Guard against buckling when heating relatively thin, wide plates.
5.1.10 Closely monitor temperatures with temperature-sensitive crayons, pyrometers, or infrared non-contact thermometers. Measure the temperature 5-10 seconds after the heating flame leaves the area to be tested.
5.1.11 Cooling with dry compressed air after the steel has cooled to below 315° C (600° F) is permitted. Do not cool the steel with water or mist.
5.1.12 Allow the steel to cool to below 120° C (250° F) before applying another set of heating patterns.
5.1.13 When curving or cambering by vee heat, reheat a location only after at least three sets of heating patterns at other locations.
5.1.14 Do not handle, support, or load the member in a manner that causes material to yield without the application of heat.

5.2 Heat-curving for sweep of bridge members
5.2.1 Achieve the required sweep at the plant (as opposed to the field).
5.2.2 Use an approved procedure that describes the method of supporting and loading and provides design computations that satisfy the preload limits of Section 5.1.3.
5.2.3 Heat-curve prior to the attachment of longitudinal stiffeners and painting.
5.2.4 When the radius is less than 300 meters (1000 feet), heat-curve only with the web in the horizontal position or preload to induce stress prior to heating.
5.2.5 When heat-curving with the web in the vertical position, support the member so that the tendency of the member to deflect laterally during the heat-curving process will not cause the member to overturn or twist.
5.2.6 Maintain intermediate safety catch blocks to prevent buckling and excessive or concentrated local deformations.

5.2.7 Plan and apply the heating patterns along the length of the member to produce the specified curvature, using enough patterns to eliminate visually obvious chording effects.

5.3 Minimum radius for heat-curving

5.3.1 Do not heat-curve beams or girders when the horizontal radius of curvature measured to the centerline of the member web is less than either value calculated using Equations Section 5.1 and Section 5.2, or when the radius is less than 45 meters (150 feet) at any cross section throughout the length of the member.

\[ R = \frac{14bD}{\sqrt{F_y(\psi)}} \]  

(Section 5.1)

or

\[ R = \frac{7500b}{F_y \psi} \]  

(Section 5.2)

where

- \( F_y \) = specified minimum yield point in ksi of the member web
- \( \psi \) = ratio of the total cross-sectional area to the cross-sectional area of both flanges
- \( b \) = width of the widest flange in inches
- \( D \) = clear distance between flanges in inches
- \( t \) = web thickness in inches
- \( R \) = radius in inches

5.3.2 Do not heat-curve portions of members where the required radius of curvature is less than 300 meters (1,000 feet), and the flange thickness exceeds 75 mm (3 inches) or the flange width exceeds 750 mm (30 inches).

5.4 Heat-cambering

5.4.1 Use an approved procedure that addresses support conditions, preloading (if any), and heat application and control.

5.4.2 Support members to be heat-cambered with the web vertical, and space supports to take maximum advantage of dead load in the member before heat is applied.

5.5 Heat-straightening damaged structural steel

For heat-straightening damaged steel, use approved procedures that describe the distortion to be corrected and all steps for preloading, heating, cooling, verifying final dimensions, and non-destructive examination.

5.6 Heat-assisted bending

5.6.1 If a smaller radius than that allowed in Table 4.2 is required, materials may be hot-bent. Apply heat uniformly through the plate thickness and observe the temperature limits of Table 5.1.

5.6.2 When Q&T steels are hot-bent, perform MT or dye penetrant testing (PT) after the steel has cooled to ensure that no surface cracks resulted from the procedure.
5.7 Heat treatment

5.7.1 When thermal stress relief is required by the contract or requested by the Fabricator and approved for the project, follow Bridge Welding Code requirements.

5.7.2 When normalizing and annealing are required, follow the requirements of ASTM E 44. Maintain temperature uniformly throughout the furnace during heating and cooling so that the temperatures at all points on the member do not differ by more than 50° C (120° F).
Steel Bridge Fabrication Guide Specification

Section 6
Member Geometry

6.1 General
6.1.1 As-received rolled shapes, plates, bars, and other applicable items must satisfy the quality requirements and dimensional tolerances in ASTM A 6.
6.1.2 Fabricate built-up members in accordance with the Bridge Welding Code tolerances and as described below.
6.1.3 Rolled or fabricated sections of equal or slightly greater dimensions than the section specified may be proposed for the Engineer’s acceptance. For changes that affect splice design or may significantly alter deflection, provide complete design calculations.

6.2 Pier cap members
6.2.1 Fabricate steel pier elements based upon mutual agreement between the Contractor and the Engineer regarding bearing plane and twist tolerances, with proper regard for erection requirements.
6.2.2 Fabricate beam support planes true to the box girder bearing to 2 mm (1/6 in.) in the short direction and true to the vertical axis of the nesting girders to 2 mm.

6.3 Trapezoidal bridge members
6.3.1 Fabricate trapezoidal bridge members (tub girders) to applicable Bridge Welding Code dimensional tolerances.
6.3.2 Verify girder camber with the girder in its upright position, supported to avoid dead load deflections.

6.4 Specialty structures
6.4.1 Fabricate component parts of specialty structures, such as bascule, arch, suspension, cable-stayed, and truss bridges, to the preceding tolerances as applicable.
6.4.2 At a prefabrication meeting with the Contractor, Owner, and Erector, establish critical dimensions and tolerances required to ensure proper installation and performance of the structure.

6.5 Shoes
Assemble fabricated pin-and-rocker type expansion shoes and ensure the following:
- The axis of rotation coincides with the central axis of the pin.
- As the top bolster travels through its full anticipated range, no point in the top bolster plane changes elevation by more than 2 mm (1/6 in.) and the top bolster does not change inclination by more than 1 degree, for the full possible travel.

6.6 Pins, pinholes, and rockers
6.6.1 Bore pinholes true to the specified diameter, smooth to 3 μm (ANSI 125), at right angles with the axis of the member, and parallel with each other.
6.6.2 Fabricate pins and pinholes so that the pinhole diameter does not exceed the pin diameter by more than 0.5 mm (0.015 in.) for pins 125 mm (5 in.) or less in diameter, or 1 mm (1/6 in.) for larger pins.

6.7 Welded studs
Weld shear studs in accordance with the Bridge Welding Code.
Steel Bridge Fabrication Guide Specification

Section 7
Bridge Geometry

7.1 Assembly
7.1.1 Follow an approved procedure that complies with the camber or blocking diagram shown on the approved shop drawings and describes the full or progressive assembly sequence.
7.1.2 Assemble members from bearing to bearing at one time unless another method of sequential geometry control is described in the approved procedure.
7.1.3 Complete welding and cutting of individual pieces prior to assembly.
7.1.4 Assemble continuous beam bridges, I-girders, and box girder lines to the required geometry and prepare main member splices.
7.1.5 Include primary members in assembly. Assembly of secondary members (diaphragms, lateral bracing, etc.) is not required unless mandated by the contract.
7.1.6 Put members in a no-load condition unless compensation for member dead loads is described in the approved procedure.
7.1.7 Bring members into proper alignment, satisfying the camber or blocking diagram, and secure all parts prior to drilling or reaming.
7.1.8 Drift pin to bring parts into position, but do not enlarge the holes or otherwise distort the metal.
7.1.9 When it is necessary to retain splice or fill plates in specific positions and orientations, such as in connections reamed or drilled in assembly, match-mark all components prior to disassembly using low- or mini-stress steel stamps. Provide diagrams showing match marking method and location on the approved shop drawings.

7.2 Bolted splices
Accomplish bolted connections in accordance with these workmanship tolerances:
- 85% of the bolt holes in any adjoining group vary no more than 1 mm (\(\frac{1}{32}\) in) between adjacent thicknesses of metal.
- The gap between ends of continuous girders or beams is 6 mm, + 3, - 4 (\(\frac{3}{4}\) in., + \(\frac{3}{8}\), - \(\frac{3}{16}\)).

7.3 Welded field splices
7.3.1 For field-welded splices, prepare the ends of beams and girders in accordance with Figure 7.1.
7.3.2 Assemble girders to demonstrate that workmanship requirements of the Bridge Welding Code and item 7.3.3, below, are satisfied.
7.3.3 Prepare joints so that the centerlines of land of opposing web and flange bevels do not deviate from each other by more than 2 mm (\(\frac{7}{64}\) in.) and root faces do not vary by more than 2 mm from contact.
7.3.4 Prepare the access hole so that the sloping transition is tangent to the joint bevel.

7.4 Alternate geometry control methods
Fabricators may propose alternate methods of geometry control for continuous girder bridges based on demonstrated accuracy that precludes the necessity for assembly.
7.5 **Trusses and frames**

7.5.1 Fabricate abutting truss chord joints considered to be close joints so that no openings are larger than 6 mm (¼ in.).

7.5.2 Bring milled and compression abutting joints in truss chords into bearing and demonstrate that 75% of the abutting surfaces are in full bearing.

7.5.3 Shop-assemble entire units or propose an alternate assembly procedure.

Figure 7.1
Introduction

The primary objective of this guide specification is to achieve quality and value in the fabrication of steel bridges. The Collaboration’s intent is for transportation authorities to adopt this guide specification by direct reference in their standard specifications. This will help standardize steel bridge fabrication across the nation.

Historically, DOTs have written their specifications based on AASHTO standards and their own individual experiences. Though this approach has worked fairly well, many agencies and Fabricators recognized that all would benefit from a common specification because:

- Variations among projects in the shop would be minimized because Fabricators would not need different practices, procedures, and operations for each state, and minimizing variation improves quality and reduces errors.
- Economy in bridge fabrication would improve because Fabricators would not have to change their methods and production variables from state to state.
- Expertise in steel bridge fabrication could be shared among states, resulting in a well-rounded, consistent fabrication standard.
- Owners would be able to share their resources, minimizing the effort each would otherwise have to expend to maintain a bridge fabrication specification.

This guide 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.

C1.4 Primary members

Terms like “primary members” and “main members” have long been used in specifications to help express requirements – i.e., “primary members must be assembled” or “mill certifications must be provided for main members”. However, such terms have not been clearly defined, and this has lead to conflicts. Hence, a definition is provided and so applied within this specification. The definition chosen for use in this standard reflects the simplest and most consistent use of the term. For further clarity, “primary members” is used throughout this specification and use of “main members” is avoided. Users of this specification should be careful to avoid conflicts if they use such terms in other parts of their contract.

Owners are cautioned against using the term “primary members” to describe which materials have Charpy (CVN) testing requirements because, by the definition used in this standard, doing so would result in CVN requirements for compression-only portions of members and end diaphragms.

C2.1 Certification

Certification is intended to help ensure that the Fabricator has the needed expertise and commitment to quality to achieve a successful project. AISC administers the certification program and provides the following description of their program:

The purpose of the AISC Quality Certification Program is to confirm to the construction industry that a Certified structural steel fabricating plant has the personnel, organization, experience, procedures, knowledge, equipment, capability and commitment to produce fabricated steel of the required quality for a given category of structural steel work.
The AISC Quality Certification Program is not intended to involve inspection and/or judgement of product quality on individual projects. Neither is it intended to guarantee the quality of specific fabricated steel products.

More information, including a list of all AISC-certified fabricators, is available from the AISC website at <http://www.aisc.org>. Information may also be obtained by writing to AISC, One East Wacker Drive, Suite 3100, Chicago, IL 60601-2001.

The following information is taken from the SSPC website and describes their certification program:

SSPC’s Painting Contractor Certification Program is a national pre-qualification service developed by SSPC for facility Owners and others who hire industrial painting contractors. The program evaluates painting contractors in two categories:


These standards were developed through the SSPC consensus process.

SSPC recently introduced a new quality certification program designed to evaluate the qualifications of firms involved in surface preparation and shop paint application. The shop certification, or QP 3, program is based on criteria established in SSPC-QP 3, “Standard Procedure for Evaluating the Qualifications of Shop Painting Contractors”. More information about this program is available from the SSPC website, <http://www.sspc.org>, or from SSPC: The Society for Protective Coatings, 40 24th Street, Pittsburgh, PA 15222-4656, phone (412) 281-2331, fax (412) 281-9992.

Coating certification is addressed in the Collaboration’s guide specification for coatings. The requirements of Collaboration standards S2.1 and S4.1 are intended to work in conjunction with quality control requirements in the AISC Quality Certification Program.

C2.2 Communication - During the course of the work, the Fabricator may propose changes to the structure that slightly or significantly deviate from the structure’s design. In such cases, the Owner may prefer that the Fabricator approach the Designer directly, that the Fabricator go through the Owner responsible for acceptance, or that some other procedure be followed.

Important rules of thumb:

- Establish lines of communication agreed to by all parties, including the designer, the engineer responsible for quality assurance, the engineer responsible for erection, the Fabricator, and the prime contractor.
- Keep the lines of communication as simple and direct as possible.
- Always keep the Designer in the loop when decisions are made affecting the structure’s performance or appearance.
- Allow the Fabricator to contact the Designer directly prior to shop drawing approval.
Steel Bridge Fabrication Guide Specification

- After shop drawings have been approved, the Fabricator should go through the Owner’s representative responsible for acceptance.

Note, however, that if a consultant designer is involved, the consultant might charge for time spent on the project, so the Fabricator must coordinate with the Owner before any submittal to a consultant.

Effective communication between the Owner, the Fabricator, and the Contractor is essential to a successful project. Before work begins, these parties should establish the simplest, most direct lines of communication possible and make sure that all parties understand and use them. This is an important function of the pre-fabrication meeting.

C2.3 The Owner’s review of shop drawings is a form of quality assurance and not quality control. This Owner reviews shop drawings to ensure that they accurately reflect the design, but the Owner does not check every detail and calculation. Regardless of the Owner’s review and approval, the accuracy of shop drawings remains the Contractor’s responsibility.

Owners must be expedient when reviewing shop drawings. Fabricators plan the flow of work and placement of jobs months in advance. They must be able to start work on schedule to keep production moving and to satisfy field delivery requirements. Because shop drawings must be approved before work starts, the Owner will delay the Fabricator if checking the drawings takes too long. Conversely, Fabricators must provide complete, legible, and accurate shop drawings to the Owner to facilitate prompt return. Submitting shop drawings in packages according to a mutually agreeable review schedule also facilitates prompt return.

To expedite the review process and to help ensure consistency in shop drawing review, the AASHTO/NSBA Steel Bridge Collaboration has developed a standard for shop and erection detail drawing approval, G1.1, “Shop Detail Drawings Review/Approval Guidelines”. G1.1 has been endorsed by AASHTO.

In some cases the Fabricator must begin fabrication before receiving approved shop drawings, and the Owner may consider requests from the Fabricator to proceed without approved shop drawings. However, the Fabricator must understand that work done without approved shop drawings may have to be changed based on final, approved shop drawings. No work done without approved drawings should be concealed by subsequent work before drawings are approved.

Except for emergency situations, work should not be allowed to proceed before shop drawings have been submitted for review.

Use of electronic drawings has proven effective and should be considered if all parties agree on the system requirements, review and approval authentication, and the storage and handling of electronic drawings.

C2.4.4 A prefabrication meeting may avert many of the problems that may complicate or delay fabrication. At the prefabrication meeting:

- The Owner and Fabricator should review the project and discuss specific concerns.
- The Fabricator should describe the expected approach to the project, including milestones or specialized work in detail.
- The Owner should describe any unusual requirements for the project.
• The Owner should describe how QA inspection will be accomplished, including identification of inspectors, the intended inspection schedule, and any special inspection or hold points.
• Clear lines of communication should be established between all parties.
• The shop drawing review and fabrication schedules should be discussed and mutually understood.

The Owner should have at least one designer, one acceptance representative, and one QA inspector present at the meeting. The Fabricator should have representatives from production, engineering, quality control, and general management. The prime contractor, other subcontractors, and suppliers may be included. All parties should be given the opportunity to ask questions or express concerns.

It is not necessary to have a prefabrication meeting before every project, especially if the Owner and Fabricator work together on a regular basis.

C2.5 Procedures are intended to facilitate understanding between the Owner and the Fabricator about how various aspects of the work will progress. These procedures may be included in the documentation reviewed during the AISC Certification process. Having these procedures ensures that the Fabricator’s employees understand requirements, and providing copies for review by the Owner helps minimize conflicts once the work has begun. Most procedures reflect the Fabricator’s standard practices, so they do not need to be resubmitted for routine jobs unless a specific aspect of work needs particular attention. Written procedures provide more specific guidance than the specification will, but the Owner should not use written procedures to introduce requirements beyond the intent the specification. For repairs, the Fabricator and Owner should reach an understanding about NDE methods, scheduling and the advance notice needed to coordinate quality control and quality assurance inspections.

Procedures should convey how the Fabricator’s process will satisfy specification requirements. Information presented on a shop drawing may suffice in lieu of formally submitting a written procedure.

C2.6 Owners generally provide some level of QA during bridge fabrication, and they often have projects underway at a number of locations. When the Fabricator provides the Owner with an anticipated work schedule, this allows planning and preparation for inspection. The earlier notification is provided to the Owner the better, so Fabricators should provide schedule information as soon as possible and not simply follow the prescribed minimum lead times. The Fabricator can initially provide a general estimate to the Owner and then provide more precise details as the commencement date approaches.

C2.7 Generally Owners have one representative who oversees fabrication of steel bridge members; this may or may not be the same individual responsible for review of shop drawings. In order for fabrication to proceed smoothly, the Owner should clearly identify the individuals responsible for shop drawing review and approval, for QA during fabrication, and for questions about contract requirements and changes. When a consultant is responsible for the design or shop drawing review, an Owner’s employee should act as the intermediary to coordinate inquiries or disagreements between the fabricator and consultant.

C2.8 The AASHTO/NSBA Steel Bridge Guide Specification for Quality Assurance and Quality Control describes quality-related actions for both the Owner and the Fabricator. For the Owner, it provides a detailed inspection practice that may be adopted and implemented. For the Fabricator, it provides requirements and guidelines for writing a quality control plan. These guidelines
parallel many of the requirements the Fabricator must already satisfy in order to achieve AISC plant certification.

**C2.9** Progress meetings can be used to resolve QC/QA disagreements, determine current status of completed and in-progress work, clarify unusual or altered contract requirements, discuss current or potential problems and their resolution, and monitor the anticipated production and completion schedule.

**C2.10** Worker safety is a critical issue in steel bridge fabrication and erection. However, there are many national and state standards that address safety concerns for industry and apply to fabrication shops by law and civil statute. It would be difficult to provide a comprehensive list of all applicable safety requirements in this standard. Therefore, specifics about worker health and safety are not addressed in this standard.

**C3.1.1** Almost all steel bridges in the United States use a small group of steel material specifications. Each of the steels typically used in bridges is available in both AASHTO and ASTM specifications, but there are virtually no differences between the two specifications for each material. However, some Owners prefer to use AASHTO specifications, while others prefer to use ASTM specifications, so both sets of specifications continue to be maintained.

Each AASHTO and ASTM steel specification has a bridge steel version (e.g., ASTM A 709/A709M Grade 36 or AASHTO M 270M/M270 Grade 36) and, until the 2000 edition of the AASHTO standards, a stand-alone version (e.g., ASTM A 36 or AASHTO M 183). The exceptions are the HPS steels, which were developed after M 270 and A 709 became available and do not have stand-alone specifications. Hence for a given type of steel, there may be four different specifications. As of the 2000 edition of the AASHTO Material Specifications, the stand-alone structural steel specifications have been withdrawn, so owners should specify M 270M/M270 steel if they use AASHTO specifications.

Tables C3.1 and C3.2 provide a summary of these materials and their associated specifications.

**Table C3.1 Typical Steel Bridge Materials and Associated Specifications, Metric**

<table>
<thead>
<tr>
<th>Material</th>
<th>Specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon Steel, Grade 250</td>
<td>A709M Grade 250</td>
</tr>
<tr>
<td></td>
<td>A36M</td>
</tr>
<tr>
<td></td>
<td>M 270M Grade 250</td>
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<tr>
<td></td>
<td>M 183M</td>
</tr>
<tr>
<td>High Strength, Low Alloy Steel, Grade 345</td>
<td>A709M Grade 345</td>
</tr>
<tr>
<td></td>
<td>A572M</td>
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<tr>
<td></td>
<td>M 270M Grade 345</td>
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<td>M 223M</td>
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<td>High Strength, Low Alloy Steel, Weathering, Grade 345W</td>
<td>A709M Grade 345W</td>
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<tr>
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<td>A 588M</td>
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<tr>
<td></td>
<td>M 270M Grade 345</td>
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<tr>
<td></td>
<td>M 222M</td>
</tr>
<tr>
<td>High Performance Steel (HPS), Grade 345W (Q&amp;T and TMCP)</td>
<td>A709M Grade HPS485W</td>
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<td></td>
<td>M 270M Grade PS485W</td>
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### Table C3.2 Typical Steel Bridge Materials and Associated Specifications, US Customary

<table>
<thead>
<tr>
<th>Material</th>
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<tbody>
<tr>
<td>Carbon Steel, Grade 36</td>
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</tr>
<tr>
<td>ASTM A709 Grade 36</td>
<td>ASTM A36</td>
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<tr>
<td>AASHTO M 270 Grade 36</td>
<td>AASHTO M 183</td>
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<tr>
<td>High Strength, Low Alloy Steel, Grade 50</td>
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</tr>
<tr>
<td>ASTM A709 Grade 50</td>
<td>ASTM A572</td>
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<tr>
<td>AASHTO M 270 Grade 50</td>
<td>AASHTO M 223</td>
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<td>High Strength, Low Alloy Steel, Weathering, Grade 50W</td>
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<td>ASTM A709 Grade 50W</td>
<td>ASTM A588</td>
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<tr>
<td>AASHTO M 270 Grade 50W</td>
<td>AASHTO M 222</td>
</tr>
<tr>
<td>High Performance Steel (HPS), Grade 70W (Q&amp;T and TMCP)</td>
<td>ASTM A709 Grade HPS70W</td>
</tr>
</tbody>
</table>

There is no designated grade for HPS made by thermo-mechanically controlled processing (TMCP). The high performance steels initially recognized by A709/A709M and M270M/M270 were manufactured only by quenching and tempering (Q&T). Subsequent ASTM and AASHTO material specifications permit TMCP processing. TMCP material is available up to 50 mm (2”) thick. Heat treatment temperatures must be documented on the test reports when the material is Q&T. There is virtually no performance difference between materials made from either process, so designers should allow substitutions.

For both ASTM A 709/A709M and AASHTO M 270M/M270, Charpy requirements are supplementary, so the designer must mandate CVN testing for the specific materials where toughness is required.

**Welding consumables** - Information about welding consumables is available from the *Bridge Welding Code*, the AWS D1.1 *Structural Welding Code*, and associated AWS filler metal specifications. These documents are available from AWS.

C3.2 Requirements for Material Test Reports (MTRs) are generally found in the material specification or an associated specification. For example, ASTM A 709 requires that MTRs be in accordance with ASTM A 6. In turn, ASTM A 6 provides specific details about the information that must be present in the MTR. Note that ASTM A 6 does not require a signature or certification of domestic production on the MTR, but these may be required under “Buy America” mandates.

C3.2.4 Fabricators should be aware that most steel bridges constructed in the US fall under federal or state requirements that all manufacturing processes for steel or iron materials and application of coatings to steel or iron materials must occur in the US. Applicable requirements will be in the Contract.

C3.3 Material traceability must be maintained throughout fabrication for all materials. The fabricator should have an effective method for material control in the Quality Control Plan. Identification information (heat number, project number, material grade, plate dimensions) should be kept visible on stockpiled material by staggered stacking whenever possible.

C3.4 Plate sagging between dunnage is not detrimental as long as the material is not kinked or permanently deformed.

Storing material outside is acceptable, provided the material is protected from water ponding, damage, and salt spray or other deleterious substances. Concentrated corrosion or pitting due to
prolonged damp storage must be avoided, especially for stacked plates and unpainted splices loosely assembled for shipping.

**C4.1** The *Bridge Welding Code* addresses cutting of materials. Proper cutting and surface preparations are important for fatigue resistance. Special care must be exercised when cutting and repairing quenched and tempered steels.

Torch cutting notches should be minimized but may still occur. In accordance with the *Bridge Welding Code*, these may be repaired using a procedure approved by the Engineer. The *Bridge Welding Code* also provides guidelines that should be followed in the repair and NDE. The Owner should allow the Fabricator to develop preapproved procedures for common repairs. Preapproved procedures should apply provided the notches are less than 12 mm (½ in.) deep and are not too frequent. The inspector can best judge what frequency is reasonable.

The AREMA commentary provides this explanation concerning the need of planing of sheared members:

> Any sheared edge may have incipient cracks resulting from the shearing operation, which literally tears the material apart. Since such cracks might be harmful, the requirements for edge planing of sheared material have been included in these [AREMA] and other specifications for many years. The planing requirements need not be applied to thin A36 material because the shearing operation does not produce structurally damaging defects therein.

Under the first bullet of 4.1.1, the word “exposed” means any sheared surfaces that is still visible after fabrication is complete, as opposed to sheared edges that are enclosed by welds and therefore are no longer exposed (e.g., web plates).

Plates and shapes generally have superior properties in the direction of rolling. Further, the direction of rolling is normally prescribed for material property tests. Therefore, the direction of rolling must be the direction of the primary design stress for main components. Orientation with stress is a lesser issue with webs than with flanges, since web splices carry longitudinal (bending) stress and vertical (shear) stress. Obtaining small quantities of wide plate may be uneconomical, so permitting either direction of rolling for web splice plates is recommended. Web splice plates may then be ordered with the direction of rolling parallel to either their vertical or horizontal axes.

**C4.3** Steel is very formable by bending, and bending is often the best way to produce certain geometries. As a rule of thumb, if the steel has been bent without kinks or perceptible necking and no fracture has occurred, it is probable that the integrity of the steel has not been compromised. Repeated bending (back and forth) is an exception to this rule.

Rounding the corners of the plate reduces the likelihood of cracking during bending. The bending radius limits in the guide specification are also intended to avoid initiating fracture during bending. The limits in Table 4.2 are based work conducted by Roger Brockenbrough for the AISI Transportation and Infrastructure Committee published June 28, 1998.

After bending, steel springs back slightly. The amount of springback depends upon a number of factors, including the grade of the material. For higher grades, more springback will occur.

**C4.5** This specification requires that welding of plates and open shapes (angles; channels; W, M, S & HP I-shapes; etc.) be performed in accordance with the AASHTO/AWS D1.5M/D1.5 *Bridge...*
Welding Code. The AWS D1.1 Structural Welding Code governs welding of structures using round and rectangular tubular members.

The Bridge Welding Code is a joint document of the American Association of State Highway and Transportation Officials (AASHTO) and the American Welding Society (AWS). It was developed in response to a need for a common steel bridge welding specification. Before it was published, most transportation authorities used the AWS D2.0 Specifications for Welded Highway and Railway Bridges from 1963 to 1974, and the AASHTO Standard Specifications for Welding of Structural Steel Highway Bridges, which modified the AWS D1.1 Structural Welding Code, from 1974 to 1989. These, together with individual or regional modifications, were used by Owners to govern steel fabrication. The confusion caused by the proliferation of additional requirements resulted in the recognition of the need for a single document that could increase standardization in bridge fabrication, while at the same time addressing the issues of structural integrity and public safety.

The committee responsible for the Bridge Welding Code consists of AASHTO and AWS members. The AASHTO members make up Task Force 2, which reports to T-17, Technical Committee for Welding, of the AASHTO Subcommittee for Bridges and Structures. The subcommittee, together with AWS members representing industry, is referred to by AWS as Subcommittee 10 and follows the AWS Rules of Operation for conducting its activities.

All revisions to the Bridge Welding Code must first be approved by AWS Subcommittee 10 and then satisfy two separate adoption processes. For AWS approval, the AWS D1 Main Committee, then the Technical Activities Committee, and then the Technical Committee must approve all revisions. For AASHTO approval, T-17, then the Subcommittee for Highway Bridges and Structures, and then the Standing Committee on Highways (SCOH) must approve revisions.

Comments and inquiries pertaining to the Bridge Welding Code are welcomed by AWS and should be sent to the Secretary of the AWS Structural Welding Committee or the Chair of the AASHTO Technical Committee for Welding. The process for submitting comments and inquiries may be found in the Forward of the code.

There are a number of good welding resources available for the Owner. The following volumes are suggested for an Owner responsible for structural welding in a department of highways or transportation:

- The Procedure Handbook of Welding, The Lincoln Electric Company
- The Welding Handbook, Volumes 1 - 3, AWS
- Design of Welding, Lincoln Electric Company
- Design of Welded Structures, Lincoln Electric Company
- Welding Metallurgy, Volume I, George Linner/AWS

Other excellent resources include the journal Welding Innovation, published by the James F. Lincoln Arc Welding Foundation, and Modern Steel Construction, a monthly magazine published by AISC. More information is available about AWS from their website at www.aws.org.

Only welds shown on the approved drawings or otherwise allowed by the Owner should be permitted in the structure. This includes applications for erection. Unapproved welds can result in a number of problems, including the introduction of fatigue-sensitive details that compromise long-term performance.
Many Fabricators show a welding procedure number or numbers in the tail of the weld symbol. This is information provided for the welder. The Fabricator may use a procedure other than the one indicated in the symbol as long as the procedure is suitable for the application and has been approved by the Engineer.

**C4.5.4** When the Contractor or Fabricator wishes to attach temporary or permanent hardware for lifting or other purposes, the Engineer should consider the location and orientation to ensure that there is no negative impact on the bridge’s fatigue resistance. Appropriate welding practices, removal methods, and NDE must be employed, even for temporary fixtures.

**C4.6** The authority on high strength, slip-critical bolted connections is the Research Council on Structural Connections (RCSC) of the Engineering Foundation, known informally as the Bolt Council. The American Institute of Steel Construction (AISC) endorses the Bolt Council’s work and publishes the Council’s specification, “Specification for Structural Joints Using ASTM A 325 or A 490 Bolts,” in the *Manual of Steel Construction*. The bolting provisions in the *AASHTO Standard Specifications for Highway Bridges* are also based on the Bolt Council's work and recommendations.

The bolt hole criteria in this specification help ensure that bolt holes align when the bridge is erected. Bolt holes are best examined with all involved members assembled. However, some Fabricators use advanced fabrication methods, such as CNC drilling equipment, to produce bolt holes that are just as accurate without conducting assembly. See Section Section 7 for more information.

Connections using oversized or slotted holes in at least one ply do not usually require shop assembly for drilling or reaming. Unless the Contract or Owner requires a complete or partial check assembly of the components, verification of hole patterns are usually by spot checks of dimensions. Simple templates may be used to verify compatibility of components for field assembly. Variations in rolled section geometry and straightness, visually positioned drills, minor fluctuations in component assemblies, misread details, and other small cumulative shop errors may be caught by such checks. When other subcontractors produce items such as seismic and pot bearings, finger and modular expansion joints, or other fixtures (lights, signs, drains) that attach to steel members, coordination is essential. Paper templates, electronic files or other interactive information transfer can avoid major problems at the job site.

Edge distances and bolt spacing, both minimum and maximum, are established by the applicable AASHTO design specifications and often create problems for detailers and fabricators. Whenever reasonable, the contract plan details should be satisfied, but these may leave some latitude for the Fabricator. If the design calls for the minimum edge distances permitted by the AASHTO Specifications, the Fabricator should first verify where increases are possible without affecting the design. Flange splice bolts can usually be moved farther from the splice center, but web bolt patterns and their distance from the center of the splice affect design parameters. Web plates may already be at maximum depth for the member, and if inside splice plates are used on narrow flanges, there may not be room to widen them. The Fabricator should contact the Owner with a detailed proposal for any modifications desired, specifically noting any changes in bolt spacing or pattern. If the Fabricator discovers that plan details either do not satisfy applicable specifications or are not possible because of interference, this should also be conveyed to the Owner. Theoretical gaps between joined members at a splice (other than compression members in bearing) are usually specified on the plans. These dimensions, preferably evenly divisible by two for detailing, vary in actual fabrication, but may not be substantially changed on shop drawings without the Owner’s approval.
C4.6.10 For some tolerances, Fabricators should consider requesting permission from the designer to position the holes slightly further from edges. Edge and end distances are important because a minimum amount of material is needed between the bolt hole and the edge of the plate. Contract plans typically show bolt holes with AASHTO minimum distances. Therefore, if the designer details the holes for the same clearance, there is essentially zero tolerance for mislocating holes closer to the edge of the member or splice plate. The designer should preferably detail the holes at a distance slightly greater than the AASHTO minimums. Fabricators may also wish to increase edge distances from those presented in the design. If so, the Fabricator should convey this to the Owner and reflect the modified details in the shop drawings to be approved.

Minimum edge distances in Tables 4.3 and 4.3a are approximately 1.75 times the bolt hole diameter for sheared edges and 1.25 times the bolt diameter for rolled or thermally cut edges. These criteria come from the AASHTO LRFD Code, and are similar to those in the AISC Code but less conservative than the earlier AASHTO LFD Code. Since the LRFD allows smaller edge distances, the Owner has less latitude to accept holes made closer to edges than specified.

C4.6.13 When bridges with bolted connections are painted, the preferred practice is to include the prime coat on all faying surfaces. This maintains the continuity of the prime coat through the splice and thus offers better corrosion protection. However, in slip-critical connections, the primer must provide enough friction to transfer the applied loads. The designer specifies the minimum required friction classification for the connection and can provide this information. The paint manufacturer can provide the coefficient of friction for the paint and the range of dry film thickness and curing conditions (temperature, humidity and minimum curing time) needed to achieve the required friction. If joints are bolted before the primer is properly cured, the performance of the connection may be compromised. The primer must also meet the creep characteristics required for coatings on faying surfaces, and the paint supplier will provide a certification attesting to this. More information about coating of faying surfaces is available in the Bolt Council Specification, Appendix A, “Testing Method to Determine the Slip Coefficient for Coating Used in Bolted Joints”.

Though it is common and preferred to apply primer to faying surfaces, it is not recommended that other coatings, such as epoxy or urethane coatings, be applied to the faying surfaces. The same holds true for the surfaces beneath bolt heads and washers.

C4.7 The term “shop fasteners” addresses bolts that are used strictly in the shop to aid in fabrication or assembly of the members and do not become a permanent part of the project. Because they will not actually be part of the bridge, quality requirements and tightening requirements do not apply for these bolts.

Punching is not suitable for thicker, high-strength materials because they are difficult to punch without tearing out the surrounding material. The material thickness should not exceed the punch diameter, or the punch may fail.

The Bolt Council Specification and associated commentary provide useful information about the installation of fasteners in structures. Experience in bridges has shown that two problems persist:

- Fasteners are often installed without regard for proper tightening procedures.
- There is often disagreement about what is meant by “snug-tight”.

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The Bolt Council Specification provides instructions on how to achieve the snug-tight condition during installation. Proper fastener installation requires that the contractor have trained personnel installing the bolts and that the DOT conduct verification inspection.

Rotational capacity (RC) testing is required at two levels. ASTM requires RC testing in the manufacture of zinc coated bolts, and FHWA requires that DOTs conduct field RC testing for fastener assemblies (bolt, nuts, and washers) used in structures.

When fasteners are galvanized, the manufacturer removes a certain amount of additional material from the nut thread (overlapping) to make room for the zinc on the bolt. There tend to be variations in the amount of zinc accumulated on the bolt threads, so it is important to ensure that galvanized bolts will be capable of developing the required strength when they are installed. This is a primary reason for the manufacturer’s test. When purchasing galvanized fasteners, Owners should check the Material Test Report to be sure that this test requirement has been satisfied.

The RC field test mandate was a result of research conducted by the FHWA into problems that occurred with bolts in the 1980s. Complaints from the field about bolt failures prompted the study. The FHWA began by surveying inspectors, Owners, Fabricators, manufacturers and suppliers about the problems observed, and they narrowed the results down to two primary problems:

- Bolts were sometimes supplied which were not actually represented by the paperwork supplied to the Owner.
- Lack of proper lubrication led to tightening problems.

When proper lubrication is not present, a high degree of friction results between the nut and the fastener and this makes it very difficult (or impossible) to turn the nut at all. Further, the nut can feel tight long before it is properly tightened. Fasteners must elongate to provide clamping force, and if a bolt is not properly lubricated but feels tight, the installer may be misled into thinking that the bolt has been properly tightened. Another problem exists in the plastic behavior of improperly lubricated nuts. When a bolt is properly lubricated and is tightened beyond its yield point, it demonstrates a great deal of ductility so that the nut may be turned beyond what is required without compromising the connection. But when the fastener is not properly lubricated, the fastener may be twisted, resulting in poor ductility and rupture before proper bolt tension is attained.

Fasteners must be properly lubricated, and the field RC test is intended to demonstrate that the proper condition exists at the time of bolting. The federal mandate requires that each combination of bolt heat, washer heat, and nut heat be tested in the field at the time of bolting. “Time of bolting” can be subjective and must be determined by the Owner; it can be once per lot, or it can be every day. Each combination of lots must be tested at least once; further RC testing may not be necessary unless the contractor does not take proper care of the fasteners, if there is a significant interruption in bolting, or there is any other reason to question the lubricated condition of the fasteners. Note that “snug tight” for RC testing has different criteria from the “snug tight” prescribed for installation.

C5.1 Steel may be readily straightened, curved, or bent by force, by heat, or by a combination of both. If heat is used for fabrication or geometry correction, carefully planned and controlled procedures are required to avoid compromising the properties of the steel. When the rules are followed, there is little concern about changes in the material’s integrity after heating. Because heat is a very valuable tool for fabrication, its proper use should not be unnecessarily limited.
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Fabricators should develop standard procedures for heat application, and Owners are encouraged to allow their use in appropriate situations. The guide specification provides a number of rules that must be followed; these should be incorporated into the procedures. The Fabricator should also incorporate checks and verifications to be performed by the QC inspectors and any non-destructive testing that may be necessary.

When heat and force are used together for straightening, bending, or curving, extra care must be taken. The load must be calculated to avoid overloading the steel; then the load must be applied and “locked off” so that the load being applied to the materials does not increase because of external factors. Preventing the externally applied load from increasing is intended to keep the materials from reaching their plastic limit and fracturing or buckling. The plastic limit stress decreases as the temperature of the steel rises.

There is a great deal of literature available about the use of heat in steels. The most recent comprehensive work for steel bridges is “Heat-Straightening Repair for Damaged Bridges”, available from the FHWA. Dr. Richard Avent of Louisiana State University conducted the research. Though this work is focused on repairs, it covers the basics and provides useful information for fabrication. This document is available from the FHWA Office of Technology Applications, 400 Seventh St, SW, Washington, DC 20590, phone (202) 366-7900.

C5.1.5 Single or multiple tips may be used when heating.

C5.2 The term “heat-curving” is usually used to describe the shaping by heat of bridge members to the curve shape required in the structure. It is often more practical to fabricate I-shape members straight and then curve them rather than building them curved. However, there are limits to what can be effectively curved, depending upon the properties of the member and how tight the radius will be. The heat-curving formula in this specification is intended to provide a conservative limit. Usually, it is best to fabricate a girder “shell”, or flanges and webs, then perform curving, and then add parts like stiffeners. Stiffeners may be added before heat curving, but then the stiffener-to-flange welds should be done after curving. Longitudinal stiffeners are added after curving to avoid twisting of the member due to asymmetry.

The two methods usually employed for heat-curving are vee and strip heating. Under vee heating, “V” shaped patterns of heat are applied to the flanges with the wide end of the vee on the side of the girder that will be inside the curve. These should be spaced as necessary to achieve the required curve. Under the strip heating method, heat is applied along a strip near the edge of the flanges on the side that will be inside the curve. Heat is not actually applied directly to the edge, but rather to one or both surfaces of the flange. If the flanges are thicker than 30 mm (1 3/4 in.), both surfaces of each flange should be heated. After heating, flanges must be allowed to cool completely so that results may be evaluated before any additional series of heats are applied.

Girders may be heat-curved with the web in either the vertical or the horizontal position. When the web is in the horizontal position, the girder’s weight may be used to contribute to the curving process. If so, limiting supports should be used to make sure the girder will remain within the required curve. When heat curving is conducted on members in the vertical position, supports are vital because as the member changes shape, its center of gravity moves, and the member can become unstable or fall over.

Though heat curving is used very effectively for I-girders, it is not as effective for box girders, either trapezoidal or rectangular. Note that the cutting of flanges, as opposed to heating flanges,
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to achieve horizontal curvature is not prohibited; cutting flanges is necessary to achieve curvature for radii that are smaller than allowed under Section 5.3.

C5.3 The minimum radii for heat curving in this guide specification are based on the original AASHTO requirements that reflect practical limits. Curving outside of these limits will likely result in distorted members or, in the case of curving members with very thick flanges, will not be possible. When flanges are very thick (over 75 mm or 3 in.), the heat tends to escape through the member too quickly to realize an effective localized heat zone. When the radius is too tight for heat curving, the curve should rather be accomplished by cutting the flanges to the prescribed curve.

C5.4 Note that the cutting of webs, as opposed to heating flanges, to achieve camber is generally the preferred means of achieving camber in built-up members. When used, cover plates are welded to beams either before or after heat cambering.

C5.5 See C5.1 for FHWA resources on heat-straightening.

C5.6 “Hot-bending” refers to first applying heat and then sufficient force to bend a plate about a die. The work is shaped by the force, which is aided by the heat. This is converse to heat-curving, where a limited amount of preload is introduced, then the work is heated, and the work is shaped by the heating and cooling, with the force as an aid. A typical application of hot-bending is forming flanges for haunch girders. Steels, including Q&T steels, may be readily hot-bent, provided the temperature limits of Table 5.1 are observed and the load is not applied too quickly. Quick load applications may fracture the material. Through-thickness heating is also essential for avoiding cracks during hot-bending. If the material is bent too quickly, it will crack or have severe local distortions (necking or mushrooming).

C5.7 Heat treatment is not usually stipulated in fabrication specifications, though the Bridge Welding Code has a procedure for stress relief of weldments. If heat treatment other than the stress relief of weldments provided in the Bridge Welding Code is required, it should be fully defined in the contract.

C6.2.1 Box girders used as bent caps generally sit on two or more bearings. The box girder bearing surfaces must be true to each other for proper fit in the field. The specification does not provide tolerances for bearings oriented perpendicular to each other or in different planes because the amount of offset allowable is a function of the torsional stiffness of the box and many other factors. Proper seating may be further complicated by field conditions. Together, the Contractor and Fabricator are responsible for the fit of the structure in the field. Therefore, this specification requires that the completed structure satisfy the design requirements, but does not provide specific fabrication tolerances.

C6.3 Like I-girder bridges, trapezoidal steel bridges are designed and detailed in the no-load condition, so camber should be checked in the no-load condition. However, unlike I-girders, which can be laid on their sides, tub girders cannot readily be positioned in the no-load condition. The most suitable alternative is to support the girder so that dead loads are balanced and no deflections result. For straight, symmetric girders, this means supporting the girders at points 0.225 times the girder length from either end of the girder. For curved or non-symmetrical girders, the Engineer should be consulted on how best to support the girders for camber checks.

C6.4.2 A prefabrication meeting should be held to establish critical dimensions and tolerances necessary to ensure erection and design requirements. This helps ensure final acceptance after
construction. Special requirements or tolerances not fully defined in the contract can be resolved at the meeting.

C6.7 This specification requires that fabrication be complete before members are assembled. However, this does not apply to shear studs. In fact, studs must often be applied in the field because of local safety restrictions and OSHA regulations.

C7.1 Putting girders into assembly has long been the established way of ensuring proper fit of the members in the field. However, by using advanced measurement technology or fabrication techniques, some Fabricators can achieve good fit of the structure in the field without shop assembly. The Engineer should consider waiving the requirement for shop assembly only if the Fabricator can demonstrate the ability to achieve proper fit of the members without it. Periodic check assemblies may be mandated to verify continuing accuracy, especially with highly complex structures.

C7.1.1 “Laydown” is a term used to describe the process of assembling members to match their theoretical, undeflected geometry (as opposed to the geometry of individual pieces). The term originated from the way I-girders are usually handled, with girders lying on their sides, thereby avoiding dead-load deflections. However, it is not mandatory that girders be horizontal during laydown, as long as they are supported in the no-load condition. Tub girders, for example, are generally assembled upright.

C7.1.2 Complete shop assembly is generally only necessary for very complex or precise structures, but not for routine simple or continuous span girder structures. “Complex” may include structurally indeterminate frames and ballast-plated through-girder railroad bridges. “Precision” structures may include moveable bridges, such as bascule and swing spans requiring exact alignment for proper functioning. When the Engineer considers complete assembly to be necessary, this should be fully defined in the contract. The Engineer should contact local Fabricators for help in determining when complete assembly may be necessary.

Owners often require a three-girder assembly, incorporating at least three members in each assembly. This requirement comes from AASHTO. In the early days of steel bridges when members were shorter, entire girder lines would be laid down in the shop. Then, as members got longer, the norm became five, and then, finally, three. For many steel bridges, even three is difficult, especially for curved girder bridges, for which the assembly of just two members may require extensive shoring and vertical or horizontal clearance. The number of girders in a laydown is not important as long as the Fabricator has a system to accurately maintain proper geometry for key points in each assembly.

C7.3 These provisions address shop-required work for field-welded connections in primary members. The actual accomplishment of the connections is beyond the scope of this specification.

C7.4 Fabricators may use CNC equipment, “virtual assembly”, or other formalized methods to establish member geometry and prepare connections so that shop assembly may be avoided or reduced. Avoiding assembly offers many production benefits, but the Engineer should be satisfied that proper fit will be achieved before authorizing alternate methods. Accuracy may be verified by assembling the first elements drilled and periodically checking assemblies thereafter, or by successful accomplishment of other work. The number of verification assemblies should be based on the variety of connection details and member sizes in a project, and on previously demonstrated success with the equipment, software and shop personnel. See Section 4.6 for
quality requirements. Whether or not assembly is performed, the Contractor remains responsible for the fit of the structure in the field.

C7.5 Frame structures may be successfully accomplished with sectional assembly. For example, a truss panel may have all verticals and diagonals drilled or reamed in a geometrically controlled assembly with the top chord elements, and then, after disassembly, have a separate assembly of the verticals and diagonals with the bottom chord elements. This can permit the diagonals to be straight in the final, full dead-load condition by pre-compensating for the truss panel’s in-plane moments.