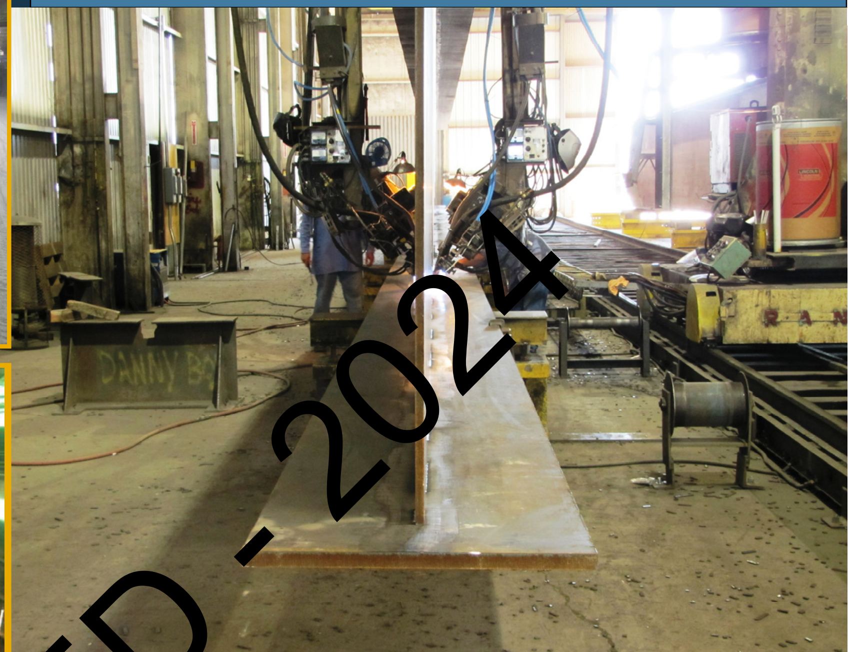
Steel Bridge Fabrication Guide Specification

S2.1-2018



American Association of State Highway and Transportation Officials
AASHTO

National Steel Bridge Alliance
Founded 1995

American Association of State Highway and Transportation Officials
National Steel Bridge Alliance
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 bridge design and construction of the highest quality and value through standardization of the design, fabrication, and erection processes. Each document represents the consensus of a diverse group of professionals.

Owners should adopt Collaboration standards to achieve standardization and to help ensure that their steel bridge practices reflect the state-of-the-art in technology and the most cost-effective solutions. The standards are written with mandatory language that may be adopted by direct reference from standard specifications or project specifications.

DISCLAIMER

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2018–2019

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INTRODUCTION

SPECIFICATION

This document 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 document is intended to be used in close tandem with AASHTO/NSBA Steel Bridge Collaboration S4.1, *Steel Bridge Fabrication QC/QA Guide Specification.*

For new painted steel bridges using a zinc-rich primer system, Owners are encouraged to adopt AASHTO/NSBA Steel Bridge Collaboration S8.1, *Guide Specification for Application of Coating Systems with Zinc-Rich Primers to Steel Bridges.*

The Collaboration also publishes a guide specification for steel bridge erection, AASHTO/NSBA Steel Bridge Collaboration S10.1*, Steel Bridge Erection Guide Specification,* which Owners are encouraged to adopt.

In this document, imperatives are directed to the Contractor and Fabricator. Many references are made to the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*. Specific section numbers are based on the 2015 edition, and subsequent editions may modify section numbers or content, but the AASHTO/AWS D1.5M/D1.5 current at time of contract advertisement should apply.

COMMENTARY

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

Historically, state Departments of Transportation (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 document was written by experienced representatives from a number of fabricators, state DOTs, consultants, and the Federal Highway Administration (FHWA). The work was based on existing state specifications, AASHTO/AWS D1.5M/D1.5, the *AASHTO LRFD Bridge Design Specifications,* and the *AASHTO LRFD Bridge Construction Specifications.*

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SECTION 1
DEFINITIONS AND RESPONSIBILITIES

Terms used in this document 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. 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 performed by the Fabricator, but which could also be done by the 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 Verification Inspector. The Designer, Engineer, and Verification Inspector may be employees either of the Owner or of professional firms contracted for the work. In this document, "Verification 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

The term “primary member” is defined by the *AASHTO LRFD Bridge Design Specifications* (Eighth Edition) as follows: “A steel member or component that transmits gravity loads through a necessary as-designed load path. These members are therefore subjected to more stringent fabrication and testing requirements; considered synonymous with the term ‘main member’.” Primary and secondary members are further defined in AASHTO LRFD Table 6.6.2.1-1, which is repeated for reference in Appendix A.

C1.4

Transverse bracing (diaphragms, crossframes, etc.) for curved plate girders or beams is defined by AASHTO as primary but does not require shop assembly. See Section 7.1.5.

Because it is not the Fabricator’s responsibility to understand and apply design principles, AASHTO requires the Designer to note all primary members on the plans as well as their state of stress (tension or compression).

Owners are cautioned against using the term “primary members” to designate which materials require Charpy V-notch (CVN) testing requirements because doing so may result in CVN requirements for compression-only portions of members and end diaphragms.

SECTION 2
PREFABRICATION

2.1 FABRICATOR CERTIFICATION

2.1.1

Certification from the AISC Quality Certification Program is required for Fabricators to the standard appropriate for the type of work to be performed. The standards include:

- AISC 205, *AISC Certification Program for Structural Steel Fabricators—Standard for Steel Bridges*—required for vehicular steel bridge superstructures
- AISC 420/SSPC-QP 3, *Certification Standard for Shop Application of Complex Protective Coating Systems*—required for any shop painting performed on steel bridges

The base level of certification to AISC 205 is for simple bridges, which consist of unspliced rolled sections. The following supplements to AISC 205 are also required:

- *Fracture Critical Endorsement*—required for any fabrication conducted on fracture-critical members (FCMs)
- *Intermediate Bridges*—required for bridges in the following categories:
  1) rolled beam bridges with field or shop splices, either straight or with a radius over 500 ft;
  2) built-up I-shaped plate girder bridges with constant web depth (except for dapped ends), with or without splices, either straight or with a radius over 500 ft;
  3) a built-up I-shaped plate girder with variable web depth (e.g., haunched), either straight or with a radius over 1000 ft; or
  4) other bridges determined by the Owner to require a similar level of skill as categories (1)–(3).
- *Advanced Bridges*—bridges that do not fall under the definition of simple or intermediate bridges.

C2.1.1

Certification is intended to help ensure that the Fabricator has the needed expertise and commitment to quality to achieve a successful project.

Coating certification is addressed in S8.1, 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.

The Owner is advised not to require certification endorsements for categories of a higher complexity than the structure warrants. The AISC endorsement requirements have been written to be pertinent to specific structure types and may not be relevant for simpler bridges.

An example of what might be designated an intermediate bridge that does not meet the typical categories (1)–(3) listed in Section 2.1.1 would be a "prefabricated" truss bridge, which would be entirely or substantially pre-assembled at the certified facility and shipped in only a few sub-assemblies. Because most of the connections are made in the shop and not the field, the geometric control required to ensure fit of field connections is not as rigorous as a truss bridge fully assembled in the field. Furthermore, most fabricators of "prefabricated" truss bridges produce only this sort of bridge, and thus would not be able to qualify for the advanced bridge endorsement by fulfilling the prerequisite of producing an intermediate bridge.

Examples of typical advanced bridges include tub or trapezoidal box girders, closed box girders, large or non-preassembled trusses, arches, bascule bridges, cable-supported bridges, moveable bridges, and bridges with particularly tight curve radius.

The Owner should also consider accepting the Certification Program for Bridge and Highway Metal Component Manufacturers for the fabrication of non-main-member components such as crossframes for straight non-skewed bridges or other bracing subassemblies.

More information, including a list of all AISC-certified fabricators, is available from the AISC website at http://www.aisc.org.

Information about the SSPC certification program is available from the SSPC website, http://www.sspc.org.

2.1.2

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

2.2 COMMUNICATION

C2.2

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 considerations:

- 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 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.
- After shop drawings have been approved, the Fabricator should go through the Owner's representative responsible for acceptance.
- Holding a prefabrication meeting with all concerned parties is recommended. See Section 2.4.

Note 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.

On design–build jobs or other situations where the design is not complete when the Fabricator is engaged, it is particularly useful for the Designer and Fabricator to discuss complex details as they are developed in the design.

2.2.1

Prior to beginning work, Owner and Fabricator 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) by Fabricator and Quality Verification by Owner; and
- Engineering, including the Designer, the Fabricator's Project Engineer, and the Owner's Engineer 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 verification, 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.

C2.2.3

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

C2.3

The Owner's review of shop drawings is a form of quality verification and not quality control. The Owner reviews shop drawings to check 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 should be expedient when reviewing shop drawings. Fabricators plan the flow of work and placement of jobs months in advance. They need to 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 obtain 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.*

In some cases, the Fabricator decides to begin fabrication before receiving approved shop drawings, and the Owner may consider requests from the Fabricator to proceed without approved shop drawings. However, 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.

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* or equivalent.

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, non-destructive testing and examination (NDT/NDE), or partial disassembly/ reassembly to satisfy the Owner's quality verification.

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, construction, and verification inspection representatives as appropriate.

2.4.3

The Fabricator representatives should include the plant manager; engineering, production, and quality control inspection personnel as appropriate; and, if applicable, subcontractor representatives (e.g., painter, subcontract fabricators, or testing agencies).

2.4.4

The review should include, at a minimum, the following aspects of the job:

- Progress on shop drawing submittal and approval;
- Plant and personnel certification;
- Organizational structure and primary (lead) plant personnel;
- Handling of MTRs;
- Traceability of materials;
- Fabrication procedures, especially shop assembly, welding, and painting;
- Supply and sampling of coatings bolts, and other materials, if applicable;
- Work schedule and milestones;
- Availability and advance notification of verification inspectors;
- Hold points;
- Inspector's office;
- Appropriate lines of communication;
- Planned coverage of shop operations by QC personnel;
- 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 document;
- Project details, requirements, or processes that have previously caused difficulties; and
- Loading and shipping.

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 verification 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 verification inspector present at the meeting. The Fabricator should have representatives from production, engineering, quality control, and general management. The 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.

2.5 PROCEDURES

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 helps 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 of 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.

2.5.1

Written procedures must be maintained for the fabrication processes listed below.

- Material traceability;
- Heat-assisted and cold bending;
- Welding (welding procedure specifications and supporting documentation must have Engineer approval before they can be used);
- Cambering and heat-curving, including temperature measurement, patterns, and sequences (must have Engineer approval to be used);
- 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 coating; and
- Inspection and testing procedures as required by AASHTO/AWS D1.5M/D1.5.

2.5.2

Each procedure must define how tasks will be performed, evaluated, and accepted by both the quality control inspector and the verification inspector (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.

C2.6

Owners generally provide some level of quality verification 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.

2.7 EVALUATION OF THE WORK

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 quality verification 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.

2.7.1

The verification inspector 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 CONTROL

Perform QC inspection using trained and qualified personnel in accordance with applicable contract documents.

C2.8

AASHTO/NSBA S4.1 provides the Fabricator with 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.

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.

C2.9

Progress meetings can be used to resolve disagreements over quality requirements, 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.

SECTION 3
MATERIAL CONTROL

3.1 QUALITY

3.1.1

Provide materials that satisfy contract requirements.

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.

The AASHTO and ASTM steel specifications used for bridge construction have a bridge steel version (e.g., ASTM A709/A709M Grade 36 or AASHTO M 270M/M 270 Grade 36) and, for certain grades, an equivalent ASTM stand-alone version (e.g., ASTM A36). Hence, for a given type of steel, there may be three different specifications. The ASTM "standalone" versions are identical to the A709/A709M grades except for mandatory toughness testing, which would need to be specified for the non-A709/A709M steels if they are used in tension in bridges. The non-A709/A709M grades are typically most likely to be used where steel greater than 4 in. thick is needed, because A709/A709M or M 270M/M 270 is only available up to 4 in. thick.

Table C3.1.1-1 provides a summary of ASTM A709/A709M or AASHTO M 270M/M 270 grades and associated standalone ASTM specifications.

Table C3.1.1-1—Typical Steel Bridge Specifications

ASTM A709/A709M or AASHTO M 270M/M 270 Grade		ASTM Standalone
U.S. Customary	Metric	
36	250	A36
50	345	A572
50S	345S	A992
50W	345W	A588

There are no separate designated grades for high performance steel (HPS) made by thermo-mechanically controlled processing (TMCP) or quenching and tempering (Q&T). TMCP material is available up to 2 in. (50 mm) 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 the Designer should not require one process over the other.

As of the 2008 editions of ASTM A709/A709M and AASHTO M 270M/M 270, Charpy requirements are no longer supplementary but required within the body of the standard for components in tension. Thus the Designer need no longer specify supplementary requirements for CVN testing for the specific materials where toughness is required, but should indicate tension and fracture-critical components and temperature zone.

Welding consumables—Information about welding consumables is available from AASHTO/AWS D1.5M/D1.5, the AWS D1.1 *Structural Welding Code,* and associated AWS filler metal specifications. These documents are available from AWS.

3.1.2

Material meeting equivalent AASHTO and ASTM specifications may be supplied under either specification.

3.2 CERTIFICATIONS AND VERIFICATION

C3.2

Requirements for MTRs (material test reports) are generally found in the material specification or an associated specification. For example, ASTM A709/A709M requires that MTRs be in accordance with ASTM A6. In turn, ASTM A6 provides specific details about the information that must be present in the MTR. ASTM A6 does not require a signature or certification of domestic production on the MTR, but these may be required under "Buy America" mandates.

3.2.1

Provide MTRs (material test reports) 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, except that reports for supplemental tests for toughness or other parameters may come directly from the party that performs the supplemental testing.

3.2.2

Check the MTRs for 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 certifying that the materials were melted and manufactured in the United States and that all applicable requirements are satisfied.

C3.2.4

Fabricators should be aware that most steel bridges constructed in the US are governed by 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 United States. Applicable requirements will be in the contract.

3.3 IDENTIFICATION AND TRACEABILITY

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.

3.3.1

Ensure that all structural steel materials are identified in accordance with ASTM A6 or other applicable code requirements.

3.3.2

Maintain heat numbers on all primary bridge materials until the material is permanently joined into a piece-marked member. Use a paint stick or other suitable method not detrimental to the member's function.

3.3.3

Document all primary member material identification for shop records and provide copies of this documentation to the verification inspector for the Owner's records.

3.4 HANDLING, STORAGE, AND SHIPMENT

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.

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.

SECTION 4
WORKMANSHIP

4.1 CUTTING, SHEARING, AND MACHINING

C4.1

AASHTO/AWS D1.5M/D1.5 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 AASHTO/AWS D1.5M/D1.5, these may be repaired using a procedure approved by the Engineer. AASHTO/AWS D1.5M/D1.5 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 $^1/_2$ in. (12 mm) deep and are not too frequent. The inspector can best judge what frequency is reasonable.

The AREMA (American Railway Engineering and Maintenance of Way Association) 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 Section 4.1.1, the word “exposed” means any sheared surfaces that are 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).

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 $^5/_8$ in. (15 mm), plane $^3/_{16}$ in. (5 mm) 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, except that the direction of rolling may be in either direction for web splice plates unless otherwise shown on the contract drawings; and
- Cut flanges to within $+^1/_4$ in., $-^1/_8$ in. (+6 mm, −3 mm) of the specified width.

C4.1.1

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.

4.1.2

Machine (grind, mill, plane, etc.) in accordance with the contract requirements and applicable codes, specifications, and accepted industry practices.

4.1.3

For steel that will be coated, ensure that hardening of thermal cut edges does not prevent achieving the required surface profile. Grind, machine, or heat if necessary to eliminate a hardened layer.

C4.1.3

Thermal-cut surfaces do not always have the same surface condition as uncut surfaces after blast cleaning. These surfaces may have been affected by the cutting process; there may be striations or marks that interfere with the use of replica tape or have a condition (such as increased hardness) which inhibits the blast media from producing the same surface profile as uncut surfaces. Determination of adequate profile is best performed visually with magnification and a profile comparator.

Normal blasting practices will typically allow the formation of anchor profile adequate for paint, but the hardness of thermal-cut edges may be too much to allow the formation of the deeper and more angular profile required for thermal-sprayed metallic coatings. In such cases, the hardness may be reduced by machining, grinding, or a local application of heat. Heating should be performed in accordance with a procedure that has been submitted and approved by the Engineer, and in accordance with Section 5 of this document. Grinding and machining should remove sufficient material to eliminate the hardened surface (typically no more than $^1/_8$ in. [3 mm]), but not enough to reduce the width of the plate beyond allowable tolerances.

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 4.2.1-1, unless otherwise noted in the contract.

Table 4.2.1-1—Maximum Permitted Surface Roughness

Milled ends of compression members, milled or ground ends of stiffeners or rockers	ANSI 500 µin. (10 µm)
Bridge rollers and rockers	ANSI 250 µin. (5 µm)
Sliding bearings	ANSI 125 µin. (3 µm)
Pins and pin holes	ANSI 125 µin. (3 µm)

4.2.2

Ensure that flange orientation is within tolerances before attachment of stiffeners or connection plates. Finish stiffeners and connection plates to the required fit before installation, and do not use them to change the flange-to-web angle.

4.3 COLD BENDING

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 are also intended to avoid initiating fracture during bending and are based on work by Peter Keating published by the Texas Transportation Institute in 2012 (Report No. FHWA/TX-10/0-4624-2).

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.

Cold cambering, or introduction of camber by cold bending, is a customary means of achieving camber in rolled beams. While all steel bending operations, heated and cold, alter steel base material strength and toughness to some extent, the relatively small strains associated with cold cambering result in minimal effect on material properties. To avoid impact damage to the steel, it is appropriate to introduce bending pressure in a controlled fashion.

4.3.1

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 minimum radii given in Section 4.3.6 by 1.5.

4.3.2

Do not use material with non-specified kinks or sharp bends, cracks, large dents, or visible reduction of section (necking).

4.3.3

Visually inspect all deformed areas and die contact points, and check any suspected damage by magnetic particle testing (MT).

4.3.4

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.

4.3.5

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

4.3.6

The minimum bending radius is given by AASHTO as 5 times the plate thickness, except for cross-frame or diaphragm connection plates to $^3/_4$ in. (20 mm) thick, for which the minimum bending radius may be reduced to 1.5 times the thickness. Plates may be bent hot or cold. When heat is used, see Section 5.

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.

4.5 WELDING

C4.5

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:

1. *The Procedure Handbook of Welding*, The Lincoln Electric Company
2. *The Welding Handbook*, Volumes 1–3, AWS
3. *Design of Weldments*, Lincoln Electric Company
4. *Design of Welded Structures*, Lincoln Electric Company
5. *Welding Metallurgy*, Volume I, George Linnert/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 about AWS is available 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.

4.5.1

Weld built-up plate and open rolled-shape structural elements in accordance with AASHTO/AWS D1.5M/D1.5.

4.5.2

Weld tubular structural elements as specified in the contract.

C4.5.2

AASHTO/AWS D1.5M/D1.5 does not address welding of tubular members. Welding of tubular members is covered in AWS D1.1, *Structural Welding Code—Steel,* but a simple reference to that code for tubular bridge members is not sufficient. It leaves open the question of whether all provisions of AASHTO/AWS D1.5M/D1.5 are waived when tubular members are joined, or, if not, which provisions remain in place and which are to be substituted with provisions from AWS D1.1. The Engineer should consider the following issues, among others, when specifying tubular members:

- Welding tubular members often involves the kind of one-sided weld prohibited or discouraged by AASHTO/AWS D1.5M/D1.5, and additional testing per AASHTO/AWS D1.5M/D1.5 clause 5.7.5 is needed to qualify such a weld as complete joint penetration (CJP). Partial joint penetration welds are not subject to the same NDE criteria as CJP welds unless additional NDE is specified. D1.1 requires very little NDE, other than visual examination, unless NDE is specified by the contract.
- The longitudinal seam in welded pipe is not made to AASHTO/AWS D1.5M/D1.5 criteria, nor even to AWS D1.1 criteria.
- AASHTO/AWS D1.5M/D1.5 requires the use of ASTM A709/A709M steel, which does not include tubular sections. Use of other steels requires additional consideration by the Engineer, per AASHTO/AWS D1.5M/D1.5 clause 5.4.3.
- The dimensions of tubular material are often such that the standard AASHTO/AWS D1.5M/D1.5 test specimens for welding procedure qualification cannot be made. Alternative specimens or tests will be needed.
- ASTM A709/A709M and the *AASHTO LRFD Bridge Design Specifications* require Charpy V-notch (CVN) impact testing of base metal in tension. ASTM A501 is a specification for tubular material that also requires CVN testing, but this steel is not readily available. A much more commonly used material is ASTM A500, which does not require CVN testing. ASTM A1085 is a specification for hollow structural sections with improved properties, tighter controls on wall thickness, and required CVN testing.

The following suggested requirements, modified and expanded from the Florida Department of Transportation *Standard Specifications for Road and Bridge Construction,* are a potential starting point but not an all-inclusive list of provisions needed to address welding of tubular members adequately:

- For welding procedure qualification, use ASTM A709/A709M Gr. 50W meeting the provisions of AASHTO/AWS D1.5M/D1.5 clause 5.4.2.
- For TYK joints between round tubes, use AWS D1.1 welder qualification for tubular welding. For all other joints, use AWS D1.5 welder qualification.
- Follow AWS D1.1 provisions for design and qualification or prequalification of flare groove welds.
- If the material specification for the tubular component does not require CVN testing, provide

material meeting the CVN testing requirements of ASTM A1085.

- Perform ultrasonic testing on all tubular complete joint penetration groove welds in main members in accordance with AWS D1.1, at the following frequency:
  1. 100% of each joint subject to tension or reversal of stress.
  2. 25% of each joint subject to compression or shear. If unacceptable discontinuities are found in the joint, the remainder of the joint shall be tested.
- Perform magnetic particle testing on fillet and partial penetration groove welds at the frequency given in AASHTO/AWS D1.5M/D1.5.

4.5.3

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.

C4.5.3

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 evaluate any negative impact on the bridge's fatigue resistance. Appropriate welding practices, removal methods, and NDE must be employed, even for temporary fixtures.

4.6 BOLT HOLES

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 High-Strength Bolts," in the *Manual of Steel Construction*. The RCSC specification is also available separately from http://boltcouncil.org. The bolting provisions in the AASHTO specifications are also based on the Bolt Council's work and recommendations.

The bolt hole criteria in this document help maintain bolt hole alignment 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 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 is 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.

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 High-Strength Bolts.* Use dimensions and tolerances based on the actual fasteners provided, whether they are in U.S. customary or SI units.

4.6.2

Ensure that all bolt holes meet the following criteria:

- Hole axis is square to faying surface within 1 in 20.
- No tears, cracks, fins, burrs, or other anomalies that could result in stress concentration or impede intimate contact at the faying surface
- Round within $\pm^{1}/_{32}$ in. (1 mm)

C4.6.2

Holes should not be punched in material that is thicker than the diameter of the hole. If the material thickness exceeds the punch diameter, the punch may fail if the die diameter is not increased. However, increasing the die diameter will result in holes being oversized at the die side of the hole because the hole

- For bolt holes in primary members, within $+^{1}/_{32}/ -0$ in. (+1/−0 mm) of the specified size For bolt holes in secondary members or in crossframes or diaphragm connection plates, within $+^{1}/_{16}/-0$ in. (+2/−0 mm) of the specified size throughout the depth of the hole

Thermal-cut portions of slots ground as required to provide maximum surface roughness of ANSI 1000 μin. (25 μm) and remove gouges.

size will be the same as the die size. The thicknesses shown in Table C4.6.2-1 have historically been considered to be appropriate thickness limits for hole punching.

Table C4.6.2-1—Historical Thickness Limits for Punching

Grade 36 (250):	$^{3}/_{4}$ in. (20 mm)
Grade 50/50W/50S/HPS 50W (345/345W/345S/HPS 345W):	$^{5}/_{8}$ in. (16 mm)
Grade HPS 70W (HPS 485W):	$^{1}/_{2}$ in. (12 mm)

4.6.3

Holes in longitudinal primary members must be drilled full-size, or else made subsize by other means and then reamed full size. Align and size the subsize holes so that a pin $^{1}/_{8}$ in. (3 mm) smaller than the subsize holes can pass through all assembled plies in at least 75% of the holes before reaming, and a pin $^{3}/_{16}$ in. (5 mm) smaller than the hole can pass through 100% of the holes before reaming.

C4.6.3

The restrictions of Section 4.6.3 acknowledge the findings of research conducted by the University of Texas, Austin (reference Report 0-4624, "Performance and Effects of Punched Holes and Cold Bending on Steel Bridge Fabrication," Frank, Brown, Cekov, Lubitz, Christian, and Keating, 2006). Full-size, punched holes reduce ductility and fatigue strength and therefore should not be used in flanges and webs without an associated design strength factor. However, such reduction is not considered significant for the performance of members such as bracing and crossframes and their connection plates, including crossframes in curved members.

4.6.4

Holes in secondary members or in crossframes or diaphragm connection plates may be made full-size by drilling, punching, plasma-cutting, or water-jetting, as long as all geometric and finish requirements are met.

C4.6.4

University of Texas research (reference Report 0-4624, "Performance and Effects of Punched Holes and Cold Bending on Steel Bridge Fabrication," Frank, Brown, Cekov, Lubitz, Christian, and Keating, 2006) on bolt holes produced by non-conventional means, such as plasma and laser, resulted in hole peripheries exhibiting a reduction in fatigue resistance equivalent to punched holes. Where the owner's standards and specifications allow the use of full size punched holes (without reaming), typically in secondary and miscellaneous components, permitting holes produced by alternative methods should also be considered. There is no reason to believe that holes produced by the waterjet method would not perform similarly.

The quality (geometry, size, location accuracy, peripheral smoothness) of holes produced by some techniques might not consistently satisfy all performance standards. Therefore, before wholesale acceptance of a new or unproven process, the Owner/Engineer should obtain verification by requiring that a fabricator demonstrate its capabilities to consistently produce holes meeting geometric tolerances and finishing standards. Even if other fabricators are successfully using a process, a shop initiating new equipment with recently trained employees may justify verification testing.

When a proposed method is not specifically permitted by the contract, the Owner/Engineer is obligated to ensure permitting its use will not be detrimental. When a fabricator requests an alternative technique, it is logical to require that they demonstrate that acceptable results will be consistently produced.

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 $^{1}/_{32}$ in. (1 mm) greater in width or $^{1}/_{16}$ in. (2 mm) greater in length than specified.

4.6.6

Do not thermally cut holes in quenched and tempered steel, unless subsequent processes will remove all material within $^{1}/_{16}$ in. (2 mm) of the cut surface. Thermally cut holes in other material must be free of gouges and other cutting defects.

C4.6.6

To avoid gouges in thermally cut holes, initiate cutting with the material that will be removed.

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 C7.3).

4.6.8

Holes in floor beam to primary member connections and continuous 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 standard size bolt hole centers no closer to the nearest edge or end than the minimum distances given in Table 4.6.10-1 or 4.6.10-2. For oversize or slotted holes, provide a minimum clear distance of one bolt diameter between the outside of the hole and the edge or end.

C4.6.10

For some tolerances, Fabricators should consider requesting permission from the Designer to position the holes slightly further from edges or ends than the distance shown on the plans. Edge and end distances are important because a minimum amount of material is needed between the bolt hole and the edge or end 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 or end of the member or splice plate. The designer should preferably detail the holes at a distance slightly greater than the AASHTO minimums, especially at field splice centerlines. Fabricators may also wish to increase edge or end 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 or end distances in Table 4.6.10-1 or 4.6.10-2 are approximately $1^1/_4$ times the bolt diameter. These criteria come from the AASHTO *LRFD Bridge Design Specifications* (starting with the 2016 Interim Revisions to the 7th Edition), and are similar to those in the AISC Code but less conservative than the earlier AASHTO *LRFD Bridge Design Specifications* and *Standard Specifications for Highway Bridges.* Since the specifications allow smaller edge and end distances, the Owner has less latitude to accept holes made closer to edges or ends than specified.

Table 4.6.10-1—Minimum Fabricated Edge or End Distances for Standard Holes (U.S. customary units)

Fastener Size, d (in.)	Edge Distance (in.)
$^5/_8$	$^7/_8$
$^3/_4$	1
$^7/_8$	$1^1/_8$
1	$1^1/_4$
$1^1/_8$	$1^1/_2$
$1^1/_4$	$1^5/_8$
Over $1^1/_4$	$1^1/_4 \times d$

Table 4.6.10-2—Minimum Fabricated Edge or End Distances for Standard Holes (SI units)

Fastener Size, d (mm)	Edge Distance (mm)
16	22
20	26
22	28
24	30
27	34
30	38
Over 30	$1^1/_4 \times d$

4.6.11

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

C4.6.11

Limiting the maximum edge or end distance is intended for sealing against moisture.

4.6.12

The tolerance for bolt hole spacing is $\pm^{3}/_{16}$ in. (5 mm), as long as edge and end distance requirements are satisfied.

4.6.13

When slip-critical faying surfaces are to be coated, use a coating and dry film thickness that is certified to provide the required slip coefficient.

C4.6.13

When bridges with bolted connections are painted, the preferred practice is to prime-coat all faying surfaces. This maintains the continuity of the prime coat and thus offers better corrosion protection. However, in slip-critical connections, the primer must provide enough friction to transfer the applied loads. The contract specifies the required coating. If the minimum required slip coefficient is not specified in the contract, the Engineer can provide this information (see Section C4.6.14). 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 RCSC Specification, Appendix A, "Testing Method to Determine the Slip Coefficient for Coating Used in Bolted Joints."

Though it is common and preferred that faying surfaces be primed, it is not recommended that other coatings, such as epoxy or urethane intermediate or top coats, be applied to the faying surfaces. The same holds true for the surfaces beneath bolt heads and washers at slip-critical connections.

According to Publication No. FHWA-HRT-14-083, "Slip and Creep of Thermal Spray Coatings," "unsealed zinc/aluminum alloy TSCs [thermal spray coatings] had no problems passing Class B slip performance requirements in accordance with the RCSC specification." TSCs (thermal spray coatings, also called "metallizing") are required to be applied over a blast-cleaned surface with an angular "anchor tooth" profile to achieve acceptable adhesion.

An unsealed metallized surface allows the fabricator to avoid masking off the areas of the faying surfaces, a time-consuming step that adds cost to the overall fabrication of the bridge.

4.6.14

Provide an SSPC-SP 6 cleaning for non-painted faying surfaces, but do not power wire-brush them. Prepare painted faying surfaces in accordance with the RCSC Specification.

C4.6.14

The surface preparation required by RCSC will depend on the design assumption for the mean slip coefficient (μ) of the faying surface. If μ is not noted in the contract, or approved coatings are not specified, contact the Engineer for further information. Class A (μ = 0.30) requires the removal of all loose material but permits clean, tightly adhering mill scale to remain. Class B (μ = 0.50) requires the removal of all mill scale, essentially an SSPC-SP 6 surface preparation, or the use of a coating certified to a slip coefficient of μ = 0.50 in accordance with Appendix A of RCSC. Class C (μ = 0.30) applies only to hot-dip galvanized surfaces. Class D (μ = 0.45) requires the use of a coating certified to a slip coefficient of μ = 0.45 in accordance with Appendix A of RCSC. Surface rust at the time of bolting may slightly exceed an SP 6 condition, but the faying surfaces must be free of any loose material. If loose material is present, it can typically be removed by a power wash just before bolting. Wire brushing is not permitted because this will "polish" the surface and reduce its slip resistance. Prior to 2017, the roughening of galvanized faying surfaces was required because it was assumed this improved the slip performance, but subsequent research ("Slip Coefficients for Galvanized Surfaces," Donahue, Helwig, and Yura. University of Texas at Austin, 2014) determined that trying to roughen the surface was more likely to polish it and be detrimental to performance. ASTM A123 has provisions addressing galvanizing quality and protrusions or excess thickness that may affect the performance of the product.

4.6.15

Ensure that faying surfaces are free of dirt, lubricants, metal shavings, burrs, and other foreign or loose material at the time of bolting that could prevent intimate contact.

4.7 BOLTING

C4.7

Quality requirements and tensioning requirements do not apply to "shop fasteners"—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.

ASTM F3125 was introduced in 2015 as a combined standard for structural bolts, replacing ASTM A325, A325M, A490, A490M, F1852, and F2280. The primary technical change from the previous bolt standards was an increase in tensile strength for A325 bolts larger than 1 in. in diameter.

The RCSC 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."

The RCSC 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 inspection be conducted.

Rotational capacity (RC) testing is required at two levels. ASTM requires RC testing in the manufacture of zinc-coated bolts, and FHWA requires that RC testing be conducted for fastener assemblies (bolt, nuts, and washers) used in structures. The result is a rotational capacity lot which consists of a specific combination of production lots of nut, bolt, and washer. Typically nuts and washers will be paired with a variety of bolt lengths. ASTM F3125 Annex A2 is essentially the FHWA RC test, with updated requirements for number of required turns.

ASTM requires RC testing for zinc-coated fasteners because when fasteners are galvanized, the manufacturer removes a certain amount of additional material from the nut thread (overtapping) 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 verify 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, the Material Test Report should be checked to be sure that this test requirement has been satisfied.

The RC field test mandate for all structural fasteners used on bridges 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.

There are two tests that may be performed in the field and that must be done for every combination of bolt and nut heats. Both the RC test and the preinstallation verification (PIV) test give information about the bolt condition. The PIV test verifies that the proposed installation method will reach the required tension. The RC test verifies that excessive torque is not needed in order to achieve the proper tension, and also that the bolts will survive "overtightening." Although the 1991 FHWA mandate required that the RC test be performed in the field, in the years and decades following the initial research, concerns about counterfeit bolts have diminished, and having manufacturers or distributors perform the RC test has become more acceptable. In practice, the FHWA "field" RC test is often performed upon receipt of the bolts, rather than "at the point of bolting" as originally intended; some owners even require that the fabricator perform this test before the bolts are shipped to the jobsite. Even further, some owners require testing both by the fastener manufacturer or distributor and by the bolt installer. In addition, the PIV test is supposed to be done at the time of bolting as well.

In order to reduce this redundancy in testing, as of the 2016 edition of S2.1, bolts are required to be ordered RC-tested from the manufacturer or distributor; if the fasteners are maintained in good condition thereafter, there is no reason to do a "field" RC test.

The PIV test is intended not only to verify acceptable fastener condition but also to verify that the wrench operator is following a procedure that will achieve the required result. Because of this, it is important to have the wrench operator rather than an inspector perform the test. Effectively, the test verifies that the wrench operator's idea of "snug" leads to the required installation tension. If the same wrench operator is repeatedly installing the same bolts, there should be no need to repeat the verification test for turn-of-the-nut or DTI installation. Daily verification is required for the calibrated wrench method to ensure that the equipment is still accurate. Note that "snug tight" for the FHWA RC test is different from the "snug tight" prescribed for installation and the PIV test.

Where daily testing is not required, it is recommended that tests on particular sizes, lengths, and grades be performed shortly before those particular fasteners are installed, rather than testing all fasteners at the beginning of the project. Testing each fastener type as it comes up for installation will help keep the crew familiar with that particular fastener assembly. Too long a time interval between testing and installation may be considered reason to question the understanding of the crew. The Owner and Contractor should agree on an appropriate interval, taking into account the complexity of the project and the experience of the crew.

See further discussion in the commentary to the bolting section of S10.1.

4.7.1 BOLT TENSION

Where the RCSC Specification is specified herein, substitute Table 4.7.1-1 for RCSC Table 7.1 and Table 4.7.1-2 for RCSC Table 8.1.

C4.7.1

The introduction of ASTM F3125 in 2015 came just after the publication of the 2014 edition of the RCSC Specification, and so the tension tables in RCSC are inconsistent with F3125. Tables 4.7.1-1 and 4.7.1-2 are a temporary correction until RCSC is updated.

Table 4.7.1-1—Minimum Verification Tension

Bolt Size (in.)	Verification Tension (kips)	
	A325 or F1852	A490 or F2280
1/2	13	16
5/8	20	25
3/4	29	37
7/8	41	51
1	54	67
1 1/8	67	84
1 1/4	85	107
1 3/8	102	127
1 1/2	124	155

Table 4.7.1-2—Minimum Bolt Installation Tension

Bolt Size (in.)	Installation Tension (kips)	
	A325 or F1852	A490 or F2280
$^{1}/_{2}$	12	15
$^{5}/_{8}$	19	24
$^{3}/_{4}$	28	35
$^{7}/_{8}$	39	49
1	51	64
$1^{1}/_{8}$	64	80
$1^{1}/_{4}$	81	102
$1^{3}/_{8}$	97	121
$1^{1}/_{2}$	118	148

4.7.2 ROTATIONAL CAPACITY TESTING

Have the manufacturer or distributor perform rotational capacity (RC) tests on ASTM F3125 Grade A325, A490, A325M, and A490M fastener assemblies. Test in accordance with ASTM F3125 Annex A2. Additionally, perform the RC test at the point of installation if the condition of the fasteners is in question.

4.7.3 PREINSTALLATION VERIFICATION TESTING

4.7.3.1—Turn-of-the-Nut and Tension-Control Bolts

When bolts are tensioned using the turn-of-the-nut method or when tension-control (“twist-off”) bolts are used, verify bolt installation method prior to bolt installation, in accordance with the RCSC Specification, Section 7, “Pre-Installation Verification”. Have the verification test performed by each wrench operator for each combination of grade, length, and diameter that the wrench operator will be installing, and whenever the condition of the fasteners or the knowledge or practice of the wrench operator is in question.

4.7.3.2—Direct Tension Indicators (DTIs)

When bolts are tensioned using DTIs, verify bolt installation method prior to bolt installation, in accordance with S10.1 Appendix C, “Direct Tension Indicators (DTI) (Verification Test Procedure)”. Have the verification test performed by each wrench operator and for each combination of grade and diameter that the wrench operator will be installing, and whenever the condition of the fasteners or the knowledge or practice of the wrench operator is in question.

4.7.3.3—Calibrated Wrench

When bolts are tensioned using the calibrated wrench method, verify bolt installation method prior to bolt installation, in accordance with the RCSC Specification, Section 7, "Pre-Installation Verification". Have the verification test performed daily by each wrench operator for each combination of grade, length, and diameter that the wrench operator will be installing, and whenever the condition of the bolts or the knowledge or practice of the wrench operator is in question.

4.7.4 INSTALLATION

Install fasteners in accordance with the RCSC Specification, Item 8.2, "Pretensioned Joints." If special fasteners not addressed by the RCSC Specification are required, install them in accordance with the manufacturer's recommendations. Ensure that no loose mill scale, dirt, metal shavings, or other foreign material that would preclude solid seating of the parts or frictional transfer of load is present on faying surfaces at time of installation.

4.8 SURFACE PREPARATION OF UNPAINTED WEATHERING STEEL (NON-FAYING SURFACES)

C4.8

The aim of the weathering steel surface preparation requirements is to give a relatively consistent appearance in service without requiring a perfectly uniform finish. If part of the girder will be painted, other levels of surface preparation may be required. See Section 4.6.14 for preparation of faying surfaces.

4.8.1

Provide an SSPC-SP 6 blast in the shop to all fascia surfaces of unpainted weathering steel beams to remove mill scale, in-process markings, or foreign material. Fascia surfaces include:

- Outward-facing sides and bottom flanges of exterior plate girders and rolled beams,
- All outer surfaces of tub girders (both webs and bottom flange) and box girders (both webs and both flanges),
- All surfaces of truss members,
- Webs and undersides of bottom flanges of plate diaphragms for tub structures,
- Any other surfaces designated as "fascia" on the plans.

4.8.2

During fabrication, use any of the following methods, as needed, to remove any markings or any other foreign material on fascia surfaces that adheres to the steel during fabrication and that could inhibit the formation of oxide film:

- SSPC-SP 1, Solvent Cleaning
- SSPC-SP 2, Hand Tool Cleaning
- SSPC-SP 3, Power Tool Cleaning
- SSPC-SP 7, Brush-Off Blast Cleaning

Where marks are removed, provide a gradual transition to the adjacent surface.

4.8.3

Do not use acids to remove stains or marks.

C4.8.2

Typically, grease sticks or crayons are likely to leave behind a residual film that may affect the appearance of the weathered steel in service, even after blast cleaning, while paint sticks, soapstone, or chalk are more easily removed.

SECTION 5
HEAT APPLICATION

5.1 HEATING PROCESS AND EQUIPMENT

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.

Fabricators should develop standard procedures for heat application, and Owners are encouraged to allow their use in appropriate situations. This document 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 manual is available from the FHWA Office of Bridge Technology at http://www.fhwa.dot.gov/bridge/heat.htm. Further information is also available in AWS C4.4, *Recommended Practices for Heat Shaping and Straightening*.

5.1.1

The measured temperature of the steel shall not exceed the maximum permitted in Table 5.1.1-1 when applying heat to steel.

Table 5.1.1-1—Maximum Temperature Limits for Heat Application

Grade	Maximum Temperature °F (°C)
36 (250)	1,200 (650)
50 (345), 50S (345S) 50W (345W), HPS 50W (HPS345W)	1,200 (650)
HPS 70W (HPS 485W), Q&T and TMCP	1,100 (600)
HPS 100W (690W)	1,100 (600)

5.1.2

Complete heating before painting.

5.1.3

Use multi-orifice or single-orifice heating tips (i.e., not a cutting head), and proportion tip size to the thickness of the material.

C5.1.3

Single- or multiple-orifice ("rosebud") heating or brazing tips lack a center oxygen jet for cutting, which is inappropriate for heating. As plate thickness increases, heat applied must increase to rapidly bring the through-thickness to desired temperature. For thick plates (over $1^1/_2$ in. [38 mm]), two torches are needed to heat both sides simultaneously.

5.1.4

Manipulate heating torches to avoid overheating.

C5.1.4

Heating tips can overheat metal, causing significant reduction in toughness or ductility, so they must be moved in a prescribed manner to attain results without damage.

5.1.5

Bring steel within the planned temperature as rapidly as possible without overheating.

5.1.6

Take steps to avoid excessive distortion of steel while heating.

C5.1.6

Means by which buckling might be avoided include heating near stiffeners, adding temporary bracing, and adjusting the shape and spacing of heat patterns. The heating procedure required in Section 2.5.1 should include the methods used to minimize buckling.

5.1.7

Routinely monitor temperatures with temperature-sensitive crayons, pyrometers, or infrared non-contact thermometers. Measure the temperature within 5 seconds after the heating flame leaves the area to be tested.

5.1.8

Cooling with dry compressed air after the steel has cooled to below 600°F (315°C) is permitted. Do not cool the steel with water or mist.

5.2 HEAT-SHRINK OR UPSET-SHORTENING METHODS

5.2.1

Limit stresses due to preload (including loads induced by member weight) to $0.5F_y$ at the extreme fiber, where F_y is the nominal yield strength of the material.

5.2.2

When jacks are used, apply and lock off load before applying heat.

5.2.3

When vee or rectangular heat patterns are used, mark the patterns on the steel prior to heating.

5.2.4

Allow the steel to cool to below 250° F (120° C) before applying another set of heating patterns.

5.2.5

When curving or cambering by vee heat, reheat a location only after at least three sets of heating patterns at other locations.

5.2.6

Do not handle, support, or load the member in a manner that causes material to yield without the application of heat.

5.3 HEAT-CURVING FOR SWEEP OF BRIDGE MEMBERS

C5.3

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 document is intended to provide a conservative limit. Stiffeners should be added before heat curving to prevent buckling of the web during curving, but then the stiffener-to-flange welds should be done after curving. Longitudinal stiffeners are required per Section 5.3.2 to be added after curving to avoid twisting of the member due to asymmetry, although heat-curving of members slender and deep enough to warrant longitudinal stiffeners is not recommended.

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 $1^{1}/_{4}$ in. (30 mm), 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, to achieve horizontal curvature is not prohibited; cutting flanges is necessary to achieve curvature for radii that are smaller than allowed under Section 5.4.

5.3.1

Use an approved procedure that describes the method of supporting and/or loading, and also provides calculations, if applicable, that satisfy the preload limits of Section 5.2.1.

5.3.2

Heat-curve prior to the attachment of longitudinal stiffeners and painting.

5.3.3

When the radius is less than 1000 feet (300 meters), heat-curve only with the web in the horizontal position or preload to induce stress prior to heating. (See Sections 5.4 and 5.5.1).

5.3.4

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, and so that camber will not significantly change.

5.3.5

Maintain intermediate "catch" blocks as needed to prevent buckling and excessive or concentrated deformations.

5.3.6

Plan and apply the heating patterns along the length of the member to produce the specified curvature, either using enough patterns to avoid visually obvious chording effects and produce a relatively uniform geometry, or heating flanges full-length with automated equipment for uniform heating.

5.4 MINIMUM RADIUS FOR HEAT-CURVING

AASHTO LRFD Bridge Design Specifications require the Engineer to indicate where heat-curving is permitted, based on the equations in Article 11.4.12.2.2 of the *AASHTO LRFD Bridge Construction Specifications*, which are reproduced for reference in Appendix B. Adjustment of cut-curved flanges by heat is not subject to the radius limitations on heat-curving.

C5.4

The AASHTO minimum radii for heat curving are based on the following research:

Sause, R., H. Ma, and J. M. Kulicki. 2013. "Residual Stresses in Heat-Curved I-Girders and Associated Limits on Radius of Curvature," ATLSS Report No. 13-01, Center for Advanced Technology for Large Structural Systems, Lehigh University, Bethlehem, PA, April 2013.

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 3 in. or 75 mm), 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 is required to be accomplished by cutting the flanges to the prescribed curve.

5.5 HEAT CAMBERING

C5.5

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.

5.5.1

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

5.5.2

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.

5.6 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, temperature monitoring, cooling, verifying final dimensions, and non-destructive examination.

C5.6

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

5.7 HEAT-ASSISTED BENDING (MECHANICAL HOT BENDING)

5.7.1

If heat-assisted bending is used:

- Follow approved procedures;
- Keep the temperature above 700°F (370°C) during bending, but below the maximum temperature limits of Table 5.1.1-1; and
- Apply heat to obtain uniform temperature throughout the plate thickness before jacking pressure is applied.

C5.7.1

"Heat-assisted 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 heat-assisted bending is forming flanges for haunch girders. Steels, including Q&T steels, may be readily bent with heat-assisted bending, provided the temperature limits of Table 5.1.1-1 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 heat-assisted bending. If the material is bent too quickly, it will crack or have severe local distortions (necking or mushrooming).

Depending on composition and manufacture, steel may exhibit brittle behavior within certain temperature ranges below the maximum limits given. To prevent unnecessary damage during heat assisted bending, the fabricator should obtain necessary guidance and employ appropriate temperature controls. The minimum temperature given in Section 5.7.1 is intended to prevent bending in the "blue brittle" temperature range.

5.7.2

When quenched and tempered (Q&T) steels are bent with heat assistance, perform MT or dye penetrant testing (PT) after the steel has cooled to verify that no surface cracks resulted from the procedure.

5.8 HEAT TREATMENT

5.8.1

When thermal stress relief is required by the contract or requested by the Fabricator and approved for the project, follow AASHTO/AWS D1.5M/D1.5 requirements.

5.8.2

When normalizing and annealing are required, follow the requirements of ASTM E44. 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 120°F (50°C).

C5.8

Heat treatment is not usually stipulated in fabrication specifications, though AASHTO/AWS D1.5M/D1.5 has a procedure for stress relief of weldments. If heat treatment other than the stress relief of weldments provided in AASHTO/AWS D1.5M/D1.5 is required, it should be fully defined in the contract.

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 A6 or other applicable code requirements.

6.1.2

Fabricate built-up members in accordance with AASHTO/AWS D1.5M/D1.5 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 SUBSTRUCTURE MEMBERS

Fabricate steel pier caps and other substructure elements based upon mutual agreement between the Contractor and the Engineer regarding bearing plane and twist tolerances, with proper regard for erection requirements.

C6.2

Box girders used as bent caps generally sit on two or more bearings, and framing beams may in turn sit on bearing surfaces that are part of the girders. The box girder bearing surfaces must be true to each other for proper fit in the field. This document 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 document requires that the completed structure satisfy the design requirements, but does not provide specific fabrication tolerances.

6.3 SPECIALTY STRUCTURES

6.3.1

Fabricate component parts of specialty structures, such as bascule, arch, suspension, cable-stayed, and truss bridges, to the preceding tolerances as applicable.

6.3.2

At a prefabrication meeting with the Contractor, Owner, and Erector, establish critical dimensions and tolerances required for proper installation and performance of the structure.

C6.3.2

A prefabrication meeting should be held to establish critical dimensions and tolerances necessary to meet 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.

6.4 PINS, PINHOLES, AND ROCKERS

6.4.1

Bore pinholes true to the specified diameter, smooth to ANSI 125 μin. (3 μm), at right angles with the axis of the member, and parallel with each other.

6.4.2

Fabricate pins and pinholes so that the pinhole diameter does not exceed the pin diameter by more than 0.02 in. (0.5 mm) for pins 5 in. (125 mm) or less in diameter, or $^{1}/_{32}$ in. (1 mm) for larger pins.

SECTION 7
BRIDGE GEOMETRY

7.1 ASSEMBLY

C7.1

Assembly—Drilling or reaming connection holes to final size with members in assembly has historically been used to ensure proper fit in the field. However, using advanced technology and techniques, some fabricators can achieve accurate field fit without shop assembly. The Engineer should consider waiving requirements for shop assembly if the Fabricator can consistently achieve proper fit of the members by other documented, demonstrated methods. Periodic check assemblies may be mandated to verify continuing accuracy, especially with highly complex structures.

The following discussion of assembly methods is to facilitate communication between owners, contractors, and subcontractors. The Owner should seek input from the construction community before requiring a special complete structure assembly.

Progressive Beam, Girder Arch Rib or Truss Assembly—Successive assemblies include at least one "carry-over" longitudinal segment (truss panel, arch section, or longitudinal member) of the previous assembly, repositioned for accurate alignment (i.e., providing the advancing assembly the proper relative rotation, horizontal, and vertical position), plus one or more longitudinal segments at the advancing end. For entire structures with lengths up to 150 feet [45 m], assembling the entire line or truss side is recommended.

Normally, transverse members are not included in the longitudinal assembly unless required in the contract documents or they are an integral part of the longitudinal assembly. If the contract requires shop-assembling specific transverse elements, either to complete their own connections (e.g., a rigid frame steel pier), or for connections involving longitudinal members (e.g., full-depth diaphragms for box girders creating an integral pier), separate subassemblies including only directly affected longitudinal elements should be permitted. Account for end rotations and deflections as necessary.

Progressive Chord Assembly—Similar to progressive truss assembly, except that the holes in truss connections are located to provide the final desired geometry. Vertical and diagonal truss panel members have connections to each truss chord made separately, based on calculated deflections, so top and bottom chords are not placed in a concurrent shop assembly. This requires that the truss members, when erected in a supported condition, must be forced to fit the end conditions. This condition introduces an initial reverse secondary stress that theoretically disappears when the structure carries its own weight and members become straight.

Special Complete Structure Assembly—When required in the contract documents, this will include simultaneously shop-assembling all structural steel, including the diaphragms, cross frames, integral steel substructure, and floor components. Miscellaneous components are not included unless specified in the contract documents. The Contractor establishes procedures for each structure or structure type including consideration of expected field conditions such as incremental erection, temporary support locations, stage construction, and final tightening of field connections.

Computer-Numerically-Controlled (CNC) Drilling with Progressive Girder, Truss, or Chord Assembly—If the Fabricator chooses to drill full-size holes in all plies of primary connections using CNC drilling procedures, assembly including both adjacent members should not be required, subject to the following:

1. Before continuing the practice, perform a check fit of the first three panels, segments or longitudinal chords; or of the entire first bent, tower face, or rigid frame produced to verify the accuracy of the CNC procedures and equipment.
2. As selected by the Engineer and before acceptance for shipping, perform another check fit of a second assembly to verify that the accuracy of the CNC procedures and equipment is being maintained.

If either of the above fails to meet the requirements, determine the source of the problem and verify correction by additional check assemblies to the satisfaction of the Owner, or revert to traditional assembly techniques. If problems are found by the second check fit, previously completed connections shall be checked to define the extent of the problem and correct errors to the Owner's satisfaction.

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.

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.

Temperature effects should be considered during assembly/laydown. The industry standard is to reference measuring devices to 68°F (20°C).

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.

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 the *AASHTO LRFD Bridge Construction Specifications*. 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.

7.1.3

Complete welding (except shear connectors) and cutting of individual pieces prior to assembly.

C7.1.3

Studs must often be applied in the field because of local safety restrictions and OSHA regulations.

7.1.4

Assemble continuous rolled beam, I-girder, and box girder lines to the required geometry and prepare primary member splices.

7.1.5

Include primary members in assembly, except for transverse bracing (diaphragms, crossframes, etc.) for curved plate girders or beams. Including transverse bracing or secondary members in the assembly is not required unless mandated by the contract.

C7.1.5

Transverse bracing, e.g., cross frames, rolled shape diaphragms, plate diaphragms, or lateral bracing, typically does not need to be assembled with the primary members. Generally, the connections of these members are planar, and the accuracy to which they are built is sufficient to maintain fit. The primary members are set up for grade and line. This validates the geometry of the primary members. The stiffeners (or other connections) are generally accurate in their placement and manufacture. If the primary members are built and assembled within tolerance, and the transverse bracing is fabricated within tolerance, this will generally produce pieces that will erect with no problems.

There are exceptions to this rule. Lateral bracing that is connected to gusset plates bolted to the flange of the girders typically uses oversized holes in one ply of the connection. This is to mitigate the difference in tolerances in sweep between girders. If two adjacent bays of cross frames or diaphragms are connected to the girder flange and each other via a moment plate, assembly of this connection, but not necessarily the entire structure, may be prudent. This also may vary with the use of oversized holes. Full-depth plate diaphragms generally do not require assembly with the primary members. With the advent of CNC machinery and the use of hardened bushing templates, fit-up problems with these connections have greatly diminished. Specifying the method of hole placement (CNC or drill templates) is preferred over assembling the connection. The exception to the full-depth plate diaphragm assembly would be if the diaphragm is used as the terminus of a discontinuous girder, or if the diaphragm connects to both flanges. Again, the strategic use of oversized holes may eliminate the need for assembly.

7.1.6

Support 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

Use drift pins to align parts, 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. Provide diagrams showing match marking method and location on the approved shop drawings. Produce match marks using low- or mini-stress steel stamps. Other means may be used if they have been demonstrated by test to meet Fatigue Category B.

C7.1.9

To facilitate the low-stress condition, die stamp marks should not be too deep but need to be deep enough such that the marks are readily legible under typical paint systems.

There is no defined radius for a "low stress" die stamp, but accomplishing marks with stamps that have a radius instead of a sharp point is suitable. Examples of stamps that are considered to be low-stress include dot, vibration, and rounded-V stamps. MIL-STD-792F(SH) provides a definition of low-stress die stamp in terms of tip radius and impression width and depth and describes other methods of marking as well.

It is known that surface imperfections can compromise fatigue performance of the otherwise smooth plate or rolled section. However, experience demonstrates that die stamp marks are innocuous for steels typically used for new bridges, particularly when precautions are taken to ensure the marks are not sharp. Modern computer-controlled stamping equipment has demonstrated the capacity to provide markings with fatigue strengths equal to or exceeding Category B.

7.2 BOLTED SPLICES

Provide holes for primary member connections that satisfy these workmanship tolerances:

- 85 percent of the bolt holes in any adjoining group vary no more than $^{1}/_{32}$ in. (1 mm) between adjacent thicknesses of metal and a bolt of the size specified for the connection can be inserted in every hole.
- The gap between ends of continuous girders or beams is $^{1}/_{4}$ in., $+^{1}/_{8}$ in., $-\,^{3}/_{16}$ in. (6 mm, +3 mm, −4 mm).

C7.2

A zero gap between members would impede field assembly, especially if steel is subsequently primed, and is not permitted. If members are hot-dip galvanized after splices are drilled, holes may need to be reamed or otherwise cleaned and drips or runs on member ends may need to be ground to avoid interference during assembly.

7.3 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.

C7.3

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.

7.4 TRUSSES, ARCHES, AND FRAMES

C7.4

Field-bolted frame structures may have shop connections successfully accomplished with sequential partial assemblies. For example, a truss panel in the horizontal (no load) position may have all verticals and diagonals drilled or reamed while joined with the top chord elements, and then the verticals and diagonals drilled or reamed in a separate assembly with the bottom chord elements. Aligning the elements to compensate for anticipated displacements within the erected truss permits the verticals and diagonals to be straight in the final, full dead-load condition. This will require diagonal members to be distorted during erection, but will significantly reduce residual stress in those members. As for other structure types, use of CNC equipment and other advanced methods may preclude the need for physical assembly altogether.

7.4.1

Fabricate abutting truss chord joints considered to be close joints so that no openings are larger than $^1/_4$ in. (6 mm).

7.4.2

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

7.4.3

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

C7.4.3

For trusses, provide an assembly procedure that ensures components are aligned within tolerances under the steel's self-weight. For trusses and arches, simultaneously shop-assemble as many sections as practical, providing positive, documented geometric controls to ensure subsequent carry-over assemblies will fit within applicable tolerances.

APPENDIX A (INFORMATIONAL)
AASHTO DESIGNATION OF PRIMARY AND SECONDARY (LRFD DESIGN TABLE 6.6.2.1-1)

This information was current as of the 2017 publication of the *AASHTO LRFD Bridge Design Specifications*. It may have been changed in subsequent revisions.

Member or Component Description	Member or Component Designation
Girders, beams, stringers, floorbeams, bent caps, bulkheads, and straddle beams	Primary
Truss chords, diagonals, verticals, and portal and sway bracing members	Primary
Arch ribs and built-up or welded tie girders	Primary
Rigid frames	Primary
Gusset plates and splice plates in trusses, arch ribs, tie girders, and rigid frames	Primary
Splice plates and cover plates in girders, beams, stringers, floorbeams, bent caps, and straddle beams	Primary
Bracing members supporting arch ribs	Primary
Permanent bottom-flange lateral bracing members and mechanically fastened or welded bottom-flange lateral connection plates in straight and horizontally curved bridges	Primary
Top flange lateral bracing members or struts and top flange lateral connection plates in straight and horizontally curved bridges	Secondary
Diaphragm and cross-frame members and mechanically fastened or welded cross-frame gusset plates in straight bridges	Secondary
Diaphragm and cross-frame members and mechanically fastened or welded cross-frame gusset plates in horizontally curved bridges	Primary
Diaphragm and cross-frame members, and mechanically fastened or welded cross-frame gusset plates and bearing stiffeners at supports in bridges located in Seismic Zones 3 or 4	Primary
Bearings, filler plates, sole plates, and masonry plates	Secondary
Mechanically fastened or welded longitudinal web and flange stiffeners	Primary
Mechanically fastened or welded transverse intermediate web stiffeners, transverse flange stiffeners, bearing stiffeners, and transverse-connection plates	Secondary
Mechanically fastened or welded batten plates and stay plates, lacing, and continuous nonperforated or perforated plates in built-up members	Primary
Eyebars and hanger plates	Primary
Miscellaneous structural components or attachments not mentioned above joining two primary members	Primary
Miscellaneous nonstructural components or attachments (e.g., expansion dams, drainage material, brackets, other miscellaneous attachments)	Secondary

APPENDIX B (INFORMATIONAL) AASHTO REQUIREMENTS FOR HEAT-CURVING (LRFD CONSTRUCTION SECTION 11.4.12.2.2)

This information was current as of the 2017 publication of the *AASHTO LRFD Bridge Construction Specifications*. It may have been changed in subsequent revisions.

Rolled beams and constant depth welded I-section plate girders (AASHTO Section 11.4.12.2.2a)**:**

Heat curving is allowed if:

- R > 1,000 ft if (t_f > 3.0 in.) or (b > 30.0 in.), otherwise R > 150 ft
- $\Psi \leq 2.0$
- $\Psi_f \geq 0.20$
- $t_{nf} \leq t_f$

in which:

$$\psi = \frac{b_{nf}t_{nf} + bt_f + D_w t_w}{b_{nf}t_{nf} + bt_f} \qquad \text{(Eq. B1)}$$

$$\psi_f = \frac{b_{nf}t_{nf}}{bt_f} \leq 1.0 \qquad \text{(Eq. B2)}$$

where:

R = horizontal radius of curvature measured to the centerline of the girder web (in.)

b = width of wider flange (in.)

b_{nf} = width of narrower flange (in.)

D_w = clear distance between flanges (in.)

t_f = thickness of wider flange (in.)

t_{nf} = thickness of narrower flange (in.)

t_w = web thickness (in.)

Doubly-symmetric beams and girders (AASHTO Section 11.4.12.2.2b):

Heat curving is allowed if the horizontal radius of curvature R measured to the centerline of the girder web, in inches, is not less than the following:

- $If\ \frac{D_w}{t_w} > \frac{592}{\sqrt{F_{yw}}}$, then :

$$R = 0.0365\frac{b}{\psi}\left(\frac{D_w}{t_w}\right)^2 \quad \text{(Eq. B3)}$$

- Otherwise:

$$R = \frac{12,800b}{F_{yw}\psi} \quad \text{(Eq. B4)}$$

where:

F_{yw} = specified minimum yield strength of a web (ksi)

Singly-symmetric beams or girders (AASHTO Section 11.4.12.2.2c)**:**

Heat curving is allowed if the horizontal radius of curvature R measured to the centerline of the girder, in inches, is not less than the values calculated from Eqs. B3 and B4. Additionally, for singly-symmetric girders with ψ greater than or equal to 1.46 and with ψ_f less than ψ_{fo}, the radius shall not be less than that determined as:

$$R = \left[1.43\psi\left(1 - \frac{\psi_f}{\psi_{fo}}\right)^2 + 1\right]\left(\frac{12800b}{F_{yw}\psi}\right) \quad \text{(Eq. B5)}$$

in which:

$$\psi_{fo} = 0.68\psi - 0.79 \quad \text{(Eq. B6)}$$

Hybrid girders (11.4.12.2.2d)**:**

Heat curving allowed if:

- $\eta_w \leq \eta_f$, and
- $\eta_w = \eta_f$ if $\eta_f < 1$
- $b_{yf} \geq b_{yfw}$ if $\eta_f < 1$

in which: (Eq. B7)

$$\eta_f = \frac{F_{yfw}}{F_{yf}}$$

$$\eta_w = \frac{F_{yw}}{F_{yf}} \quad \text{(Eq. B8)}$$

where:

F_{yfw} = yield stress of flange with lower yield stress (ksi)

F_{yf} = yield stress of flange with higher yield stress (ksi)

F_{yw} = yield stress of web (ksi)

b_{yfw} = width of flange with lower yield stress (in.)

b_{yf} = width of flange with higher yield stress (in.)

For hybrid sections with $\eta_f = 1$ and $\eta_w < 1$, the horizontal radius of curvature R measured to the centerline of the girder, in inches, shall not be less than the minimum radius determined for singly symmetric and doubly symmetric beams and girders above, with F_{yf} substituted for F_{yw} in Eq. B5.

For hybrid sections with $\eta_f < 1$, the horizontal radius of curvature R measured to the centerline of the girder, in inches, shall not be less than the minimum radius determined for doubly symmetric beams and girders above. Additionally, for girders with ψ_{fo} greater than or equal to $0.2\sqrt{\eta_f}$ and with ψ_f less than $\frac{\psi_{fo}}{\sqrt{\eta_f}}$, the radius shall not be less than that determined as:

$$R = \left(1.43\left(1 - \frac{\psi_f\sqrt{\eta_f}}{\psi_{fo}}\right)^2 \cdot \psi + 1\right)\left(\frac{12{,}800\ b}{F_{yw}\psi}\right) \qquad \text{(Eq. B9)}$$

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