S 2.1 Steel Bridge Fabrication Guide Specifications

2nd Edition

American Association of State Highway
and Transportation Officials

National Steel Bridge Alliance

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 standard represents the consensus of a diverse group of professionals.

It is intended that Owners adopt and implement Collaboration standards in their entirety to facilitate the achievement of standardization. It is understood, however, that local statutes or preferences may prevent full adoption of the document. In such cases Owners should adopt these documents with the exceptions they feel are necessary.

DISCLAIMER

The information presented in this publication has been prepared in accordance with recognized engineering principles and is for general information only. While it is believed to be accurate, this information should not be used or relied upon for any specific application without competent professional examination and verification of its accuracy, suitability, and applicability by a licensed professional engineer, designer, or architect.

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INTRODUCTION

SPECIFICATION

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

This standard is intended to be used in close tandem with AASHTO/NSBA Steel Bridge Collaboration S 4.1, *Steel Bridge Fabrication QC/QA Guide Specification*.

For newly painted steel bridges using a zinc-rich primer system, Owners are encouraged to adopt AASHTO/NSBA Steel Bridge Collaboration S 8.1, *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 S 10.1, which Owners are encouraged to adopt.

In this standard, imperatives are directed to the Contractor and Fabricator. Many references are made to the *AASHTO/AWS D1.5M/D1.5 Bridge Welding Code*, referred to in this standard as the *Bridge Welding Code*. Specific section numbers are based on the 2002 edition, and subsequent editions may modify section numbers or content, but the *Bridge Welding Code* current at time of contract advertisement should apply.

COMMENTARY

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

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

Variations among projects in the shop would be minimized because Fabricators would not need different practices, procedures, and operations for each state, and minimizing variation improves quality and reduces errors.

Economy in bridge fabrication would improve because Fabricators would not have to change their methods and production variables from state to state.

Expertise in steel bridge fabrication could be shared among states, resulting in a well-rounded, consistent fabrication standard.

Owners would be able to share their resources, minimizing the effort each would otherwise have to expend to maintain a bridge fabrication specification.

This Guide Specification was written by experienced representatives from a number of fabricators, state DOTs, consultants, and the FHWA. The work was based on existing state specifications, the *Bridge Welding Code*, and the AASHTO bridge design and construction manuals.

SECTION 1

DEFINITIONS AND RESPONSIBILITIES

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

1.1—CONTRACTOR

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

1.2—FABRICATOR

In this document, “Fabricator” refers to the facility or facilities performing such shop activities as cutting, welding, drilling, punching, cleaning, and painting of structural steel. “Fabricator” also includes any agents of the Fabricator such as subcontract fabricators. 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 these 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 Quality Assurance Inspector (QAI). The Designer, Engineer, and QAI may be employees either of the Owner or of professional firms contracted for the work. In this standard, “QAI” 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*, Fourth Edition as “A member designed to carry the internal forces determined from an analysis.”

C1.4

Typically, primary members will include:

- Webs and flanges of plate, tub, and box girders;
- Rolled beams and their cover plates;
- Floor beam webs and flanges;
- Arch ribs, ties, and hangers;
- Main truss members (chords, verticals, and diagonals of truss panels, transverse wind bracing, floor system);
- Pier diaphragm members for tub girders;
- Splice plates for primary members; and
- Columns.

Transverse bracing (diaphragms, crossframes, etc.) for curved plate girders or beams also fits the AASHTO definition but does not require shop assembly. See Section 7.1.5.

The list given above is for informational purposes only; it is not the Fabricator’s responsibility to understand and apply design principles. For this reason, the Designer should note all primary members on the plans, as recommended in Collaboration standard G 1.2, *Design Drawing Presentation Guidelines*.

For further clarity, “primary members” is used throughout this Specification and use of “main members” is avoided. Users of this Specification should be careful to avoid conflicts if they use such terms in other parts of their contract.

Owners are cautioned against using the term “primary members” to designate which materials require Charpy (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 in the category appropriate for the type of work being performed. The categories include:

- Simple Steel Bridge Structures (Sbr) —Required for highway sign fabrication, bridge parts such as cross frames, and unspliced rolled beam bridges.
- Major Steel Bridges (Cbr) —Required for all bridges other than unspliced rolled beam bridges.
- Fracture Critical Endorsement (F)—Required for any fabrication conducted on fracture critical members (FCMs).
- Sophisticated Paint Endorsement (P)—Required for any shop and/or field painting performed on steel 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. AISC administers the certification program and provides the following description of their program:

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

More information, including a list of all AISC-certified fabricators, is available from the AISC website at http://www.aisc.org. Information may also be obtained by writing to AISC, One East Wacker Drive, Suite 700, Chicago, IL 60601-1802.

2.1.2

Qualification under SSPC-QP3, *Standard Procedure for Evaluating the Qualifications of Shop Painting Contractors*, may be substituted for the Sophisticated Paint Endorsement.

C2.1.2

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

> SSPC's Painting Contractor Certification Program is a national pre-qualification service developed by SSPC for facility Owners and others who hire industrial painting contractors. The program evaluates painting contractors in two categories:
>
> 1. SSPC-QP 1, *Standard Procedure for Evaluating Qualifications of Painting Contractors: Field Application to Complex Structures*.
> 2. SSPC-QP 2, *Standard Procedure for Evaluating the Qualifications of Painting Contractors to Remove Hazardous Paint*.

These standards were developed through the SSPC consensus process.

SSPC recently introduced a new quality certification program designed to evaluate the qualifications of firms involved in surface preparation and shop paint application. The shop certification, or QP 3, program is based on criteria established in SSPC-QP 3, *Standard Procedure for Evaluating the Qualifications of Shop Painting Contractors*. More information about this program is available from the SSPC website, http://www.sspc.org, or from SSPC: The Society for Protective Coatings, 40 24th Street, Pittsburgh, PA 15222-4656, phone (412) 281-2331, fax (412) 281-9992. Coating certification is addressed in the Collaboration's guide specification for coatings. The requirements of Collaboration standards S 2.1 and S 4.1 are intended to work in conjunction with quality control requirements in the AISC Quality Certification Program.

2.1.3

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

2.2—COMMUNICATION

2.2.1

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

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

C2.2.1

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.

2.2.2

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

2.2.3

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

C2.2.3

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

Important rules of thumb:

- Establish lines of communication agreed to by all parties, including the Designer, the Engineer responsible for quality assurance, the Engineer responsible for erection, the Fabricator, and the 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.

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

2.2.4

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

2.3—SHOP DRAWINGS

2.3.1

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

2.3.2

Prepare and submit shop drawings in accordance with AASHTO/NSBA Steel Bridge Collaboration G 1.3, *Shop Detail Drawing Presentation Guidelines*, or another equivalent system.

C2.3.2

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.

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.

2.3.3

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

C2.3.3

The Owner's review of shop drawings is a form of quality assurance 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.

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, G 1.1, *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.

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 QA inspection representatives as appropriate.

2.4.3

The Fabricator representatives will 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

Review these aspects of the job:

- Progress on shop drawing submittal and approval;
- Plant and personnel certification;
- Organizational structure and primary (lead) plant personnel;
- Handling of MTRs;
- Traceability of materials;
- Fabrication procedures, especially shop assembly, welding, and painting;
- Supply and sampling of paint, bolts, and other materials, if applicable;
- Work schedule;
- Availability and advance notification of quality assurance inspectors (QAIs);
- Inspector's office;
- Appropriate lines of communication;
- Project-specific areas of concern for fabrication and inspection, including any special applications of non-destructive examination and testing (NDE/NDT);
- Handling of non-conformance and repair issues;
- Special requirements, especially any exceptions to this specification;
- Project details, requirements, or processes that have caused prior difficulties; 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 QA inspection will be accomplished, including identification of inspectors, the intended inspection schedule, and any special inspection or hold points.
- Clear lines of communication should be established between all parties.
- The shop drawing review and fabrication schedules should be discussed and mutually understood.

The Owner should have at least one designer, one acceptance representative, and one QA inspector present at the meeting. The Fabricator should have representatives from production, engineering, quality control, and general management. The 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

2.5.1

Written procedures must be maintained for the fabrication processes listed below. These are subject to the Owner's review and acceptance.

- Material traceability;
- Cutting and fitting;
- 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; and
- Blast cleaning and painting.

C2.5.1

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.

2.5.2

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

2.5.3

The procedures may be standardized and not require resubmittal and approval for each project.

2.6—COMMENCEMENT OF WORK

Provide a written advance notice to the Owner a minimum of two weeks before fabrication begins.

C2.6

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

2.7—EVALUATION OF THE WORK

2.7.1

The QAI will evaluate the work and accept fabricated components that satisfy the requirements of the contract documents.

C2.7.1

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

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

2.8.1

Perform QC inspection using trained and qualified personnel in accordance with AASHTO/NSBA Steel Bridge Collaboration S 4.1, *Steel Bridge Fabrication QC/QA Guide Specification*.

C2.8.1

The AASHTO/NSBA Steel Bridge Collaboration S 4.1, *Steel Bridge Fabrication QC/QA Guide Specification* describes quality-related actions for both the Owner and the Fabricator. For the Owner, it provides a detailed inspection practice that may be adopted and implemented. For the Fabricator, it provides requirements and guidelines for writing a quality control plan. These guidelines parallel many of the requirements the Fabricator must already satisfy in order to achieve AISC plant certification.

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 QC/QA disagreements, determine current status of completed and in-progress work, clarify unusual or altered contract requirements, discuss current or potential problems and their resolution, and monitor the anticipated production and completion schedule.

2.10—SAFETY

Perform work in accordance with industry codes for safety, including OSHA regulations and any other applicable codes or restrictions.

C2.10

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

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, until the 2000 edition of the AASHTO standards, a stand-alone version (e.g., ASTM A36 or AASHTO M 183). Hence, for a given type of steel, there may be four different specifications. As of the 2000 edition of the *AASHTO Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, the stand-alone structural steel specifications have been withdrawn, so owners should specify M 270M/M 270 steel if they use AASHTO specifications.

Table C1 provides a summary of these materials and their associated specifications.

Table C3.1.1-1— Typical Steel Bridge Materials and Associated Specifications

	Specifications			
Material	ASTM	ASTM	AASHTO	AASHTO
Carbon Steel, Grade 36	A709 Grade 36	A36	M 270 Grade 36	M 183*
High Strength, Low Alloy Steel, Grade 50	A709 Grade 50	A572	M 270 Grade 50	M 223*
Structural Steel Shapes, Grade 50S	A709 Grade 50S	A992	M 270 Grade 50S	
High Strength, Low Alloy Steel, Weathering, Grade 50W	A709 Grade 50W	A588	M 270 Grade 50W	M 222*
High Performance Steel (HPS), Grade 50W	A709 Grade HPS 50W		M 270 Grade HPS 50W	
High Performance Steel (HPS), Grade 70W (Q&T and TMCP)	A709 Grade HPS 70W		M 270 Grade HPS 70W	
High Performance Steel (HPS), Grade 100W (Q&T)	A709 Grade HPS 100W		M 270 Grade HPS 100W	

* Not included in the AASHTO material specifications after the 2000 edition.

There is no designated grade for HPS made by thermo-mechanically controlled processing (TMCP). The high performance steels initially recognized by A709/A709M and M 270M/M 270 were manufactured only by quenching and tempering (Q&T). Subsequent ASTM and AASHTO material specifications permit TMCP processing. 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 allow substitutions.

For both ASTM A709/A 709M and AASHTO M 270M/M 270, Charpy requirements are supplementary, so the Designer must mandate CVN testing for the specific materials where toughness is required.

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

3.1.2

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

3.2—CERTIFICATIONS AND VERIFICATION

3.2.1

Provide MTRs for all steel materials used in fabrication, including plates, bars, shapes, and fasteners. MTRs must originate from the producer of the material and not from a supplier, unless the Owner permits specific supplemental testing of stock or service-center-supplied material for toughness or other parameters.

C3.2.1

Requirements for MTRs are generally found in the material specification or an associated specification. For example, ASTM A709 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. Note that ASTM A6 does not require a signature or certification of domestic production on the MTR, but these may be required under "Buy America" mandates.

3.2.2

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 U.S. 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 U.S. Applicable requirements will be in the contract.

3.3—IDENTIFICATION AND TRACEABILITY

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 paint stick or other suitable method not detrimental to the member's function.

C3.3.2

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.

3.3.3

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

3.4—HANDLING, STORAGE, AND SHIPMENT

3.4.1

Handle, store, and ship raw and fabricated materials in a manner that protects them from damage, facilitates subsequent inspections, and does not compromise the safety of personnel.

C3.4.1

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

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.

C3.4.3

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.

3.4.4

Protect materials from detrimental corrosion or coating deterioration.

C3.4.4

Concentrated corrosion or pitting due to prolonged damp storage must be avoided, especially for stacked plates and unpainted splices loosely assembled for shipping.

3.4.5

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 so that traceability is maintained.

3.4.6

Store paint in accordance with manufacturer's recommendations.

SECTION 4

WORKMANSHIP

4.1—CUTTING, SHEARING, AND MACHINING

4.1.1

Cut and shear materials in accordance with *Bridge Welding Code* tolerances and with the following:

- For primary member plate components thicker than $^5/_8$ in., plane $^3/_{16}$ in. off sheared edges that remain exposed after fabrication;
- Cut and fabricate steel plates for primary member components and splice plates with the direction of rolling parallel to the direction of primary stresses. The primary stress for web plates is assumed to be parallel to flanges unless otherwise shown on the approved shop drawings. The Engineer may permit the rolling direction of web splice plates to be perpendicular to the flanges, based on anticipated horizontal and vertical load components and the splice plate sizes required.

C4.1.1

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

Torch cutting notches should be minimized but may still occur. In accordance with the *Bridge Welding Code*, these may be repaired using a procedure approved by the Engineer. The *Bridge Welding Code* also provides guidelines that should be followed in the repair and NDE. The Owner should allow the Fabricator to develop preapproved procedures for common repairs. Preapproved procedures should apply provided the notches are less than $^1/_2$ in. 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 for 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 this Section, 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).

4.1.2

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

C4.1.2

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.

4.2—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 1, unless otherwise noted in the contract.

Table 4.2.1-1—Maximum Permitted Surface Roughness

Steel slabs	ANSI 2000 μin.
Heavy plates in contact with shoes to be welded	ANSI 1000 μin.
Milled ends of compression members, milled or ground ends of stiffeners or rockers	ANSI 500 μin.
Bridge rollers and rockers	ANSI 250 μin.
Sliding bearings	ANSI 125 μin.
Pins and pin holes	ANSI 125 μin.

4.2.2

For I-girders, ensure that flanges are square to webs (within specified tolerances) before attachment of stiffeners or connection plates. For skewed web-to-flange orientations, such as trapezoidal box girders, proper orientation is required before attachment of stiffeners or connection plates.

4.3—COLD BENDING

4.3.1

Do not cold-bend fracture-critical materials.

C4.3.1

The bending radius limits in the guide specification are also intended to avoid initiating fracture during bending. The limits in Table 4.3.2-1 are based on the work conducted by Roger Brockenbrough for the AISI Transportation and Infrastructure Committee published June 28, 1998.

4.3.2

When possible, orient bent plates for connections so that the bend line will be approximately perpendicular to the direction of rolling. If the bend line must be approximately parallel to the direction of rolling, multiply the suggested minimum radii in Table 1 by 1.5.

Table 4.3.2-1—Minimum Cold-Bending Radii

Material	Inside Radius in Terms of Plate Thickness, ***t*** in.		
ASTM A709 Grade	$t \leq 1$	$1\ (25\ ) < t \leq 2$	$2 < t$
36 (250)	1.5*t*	1.5*t*	2.0*t*
50 (345), 50S (345S), 50W (345W), HPS 50W (HPS 345W)	1.5*t*	2.0*t*	2.5*t*
HPS 70W (485W)	1.5*t*	2.5*t*	3.0*t*
100 (690), 100W (690W) and HPS 100W (HPS 690W)	2.25*t*	4.5*t*	5.5*t*

4.3.3

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

C4.3.3

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.

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.

4.3.4

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

4.3.5

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

4.3.6

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

4.3.7

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

C4.3.7

Rounding the corners of the plate reduces the likelihood of cracking during bending.

4.3.8

Suggested minimum bend radii for cold bending, measured to the concave face of the plate, are given in Table 4.3.2-1. If a smaller radius is required, see Section 5.6.

C4.3.8

The bending radius limits in the guide specification are also intended to avoid initiating fracture during bending. The limits in Table 4.3.2-1 are based on the work conducted by Roger Brockenbrough for the AISI Transportation and Infrastructure Committee published June 28, 1998.

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

4.5.1

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

C4.5.1

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

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

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

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

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

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

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 is available about AWS from their website at www.aws.org.

Only welds shown on the approved drawings or otherwise allowed by the Owner should be permitted in the structure. This includes applications for erection. Unapproved welds can result in a number of problems, including the introduction of fatigue-sensitive details that compromise long-term performance.

Many Fabricators show a welding procedure number or numbers in the tail of the weld symbol. This is information provided for the welder. The Fabricator may use a procedure other than the one indicated in the symbol as long as the procedure is suitable for the application and has been approved by the Engineer.

4.5.2

Weld tubular structural elements in accordance with AWS D1.1, *Structural Welding Code—Steel*.

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

4.6.1

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

C4.6.1

The authority on high strength, slip-critical bolted connections is the Research Council on Structural Connections (RCSC) of the Engineering Foundation, known informally as the Bolt Council. The American Institute of Steel Construction (AISC) endorses the Bolt Council's work and publishes the Council's specification, "Specification for Structural Joints Using ASTM A325 or A490 Bolts," in the *Manual of Steel Construction*. The RCSC specification is also available separately from http://boltcouncil.org. The bolting provisions in the AASHTO *Standard Specifications for Highway Bridges* are also based on the Bolt Council's work and recommendations.

The bolt hole criteria in this Specification help 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 are usually by spot checks of dimensions. Simple templates may be used to verify compatibility of components for field assembly. Variations in rolled section geometry and straightness, visually positioned drills, minor fluctuations in component assemblies, misread details, and other small cumulative shop errors may be caught by such checks. When other subcontractors produce items such as seismic and pot bearings, finger and modular expansion joints, or other fixtures (lights, signs, drains) that attach to steel members, coordination is essential. Paper templates, electronic files, or other interactive information transfer can avoid major problems at the job site.

Edge distances and bolt spacing, both minimum and maximum, are established by the applicable AASHTO design specifications and often create problems for detailers and fabricators. Whenever reasonable, the contract plan details should be satisfied but these may leave some latitude for the Fabricator. If the design calls for the minimum edge distances permitted by the AASHTO Specifications, the Fabricator should first verify where increases are possible without affecting the design. Flange splice bolts can usually be moved farther from the splice center, but web bolt patterns and their distance from the center of the splice affect design parameters. Web plates may already be at maximum depth for the member, and if inside splice plates are used on narrow flanges, there may not be room to widen them. The Fabricator should contact the Owner with a detailed proposal for any modifications desired, specifically noting any changes in bolt spacing or pattern. If the Fabricator discovers that plan details either do not satisfy applicable specifications or are not possible because of interference, this should also be conveyed to the Owner. Theoretical gaps between joined members at a splice (other than compression members in bearing) are usually specified on the plans. These dimensions, preferably evenly divisible by two for detailing, vary in actual fabrication, but may not be substantially changed on shop drawings without the Owner's approval.

4.6.2

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

- Square to splice plates within $^1/_{20}$.
- No tears, cracks, fins, dirt, loose rust, burrs, or other anomalies that could impede intimate contact or concentrate stress.
- Round within $\pm^1/_{32}$ in.
- Within $+^1/_{32}$ /–0 in. of the specified size.
- For sub-size holes, aligned and sized so that a pin $^1/_8$ in. smaller than the sub-size holes can pass through all assembled plies in at least 75 percent of the holes before reaming and a pin $^3/_{16}$ in. smaller than the hole can pass through 100 percent of the holes before reaming.
- Thermal-cut holes or portions of slots ground as required to provide maximum surface roughness of ANSI 1000 μin. and remove gouges.

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 size will be the same as the die size. The following have historically been considered to be appropriate thickness limits for hole punching:

Grade 36 (250):	$^3/_4$ in.
Grade 50/50W/50S/HPS 50W (345/345W/345S/HPS 345W):	$^5/_8$ in.
Grade HPS 70W (HPS 485W):	$^1/_2$ in.

4.6.3

Full-size punched holes may be used as follows:

- Do not punch holes full size in longitudinal primary members, transverse floor beams, or any component designated as fracture-critical. These holes must be drilled full-size, unless subpunched or subdrilled and then reamed full size.
- Crossframes and diaphragm connection plates in longitudinal primary members may be punched full size.

C4.6.3

The restrictions of this Section acknowledge the 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 and stiffeners in curved members.

4.6.4

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. greater in width or $^1/_{16}$ in. greater in length than specified.

4.6.5

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

4.6.6

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 C7.4).

4.6.7

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.

4.6.8

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

4.6.9

Locate standard size bolt hole centers no closer to the nearest edge than the minimum distances given in Table 1. For oversize or slotted holes, provide a minimum clear distance between the hole and the edge of one bolt diameter.

C4.6.9

For some tolerances, Fabricators should consider requesting permission from the Designer to position the holes slightly further from edges 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 of the plate. Contract plans typically show bolt holes with AASHTO minimum distances. Therefore, if the designer details the holes for the same clearance, there is essentially zero tolerance for mislocating holes closer to the edge of the member or splice plate. The designer should preferably detail the holes at a distance slightly greater than the AASHTO minimums, especially at field splice centerlines. Fabricators may also wish to increase edge distances from those presented in the design. If so, the Fabricator should convey this to the Owner and reflect the modified details in the shop drawings to be approved.

Minimum edge distances in Table 1 are approximately 1.75 times the bolt hole diameter for sheared edges and 1.25 times the bolt diameter for rolled or thermally cut edges. These criteria come from the AASHTO LRFD Code, and are similar to those in the AISC Code but less conservative than the earlier AASHTO LFD Code. Since the LRFD allows smaller edge distances, the Owner has less latitude to accept holes made closer to edges than specified.

4.6.10

The maximum fabricated edge distance is the lesser of eight times the thickness of the thinnest outside plate or 5 in.

4.6.11

The tolerance for bolt hole spacing is ± $^{3}/_{16}$ in.

C4.6.11

Bolt spacing tolerance is not to be used to reduce minimum required edge or end distance.

4.6.12

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

C4.6.12

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.

4.6.13

Hot-dip galvanized slip-critical faying surfaces shall be free of high spots and hand wire brushed to remove heavy oxides and provide some roughness.

4.6.14

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

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 = 0.33) 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. 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.

4.6.15

Ensure that faying surfaces are free of dirt, lubricants, and other foreign or loose material at the time of bolting, including metal shavings.

Table 4.6.9-1—Minimum Fabricated Edge Distances for Standard Holes

Fastener Size (in.)	Sheared Edges (in.)	Rolled or Gas-Cut Edges (in.)
$^5/_8$	$1^1/_8$	$^7/_8$
$^3/_4$	$1^1/_4$	1
$^7/_8$	$1^1/_2$	$1^1/_8$
1	$1^3/_4$	$1^1/_4$
$1^1/_8$	2	$1^1/_2$
$1^1/_4$	$2^1/_4$	$1^5/_8$
$1^3/_8$	$2^3/_8$	$1^3/_4$

4.7—BOLTING

4.7.1

Perform rotational capacity (RC) tests prior to installation of permanent fasteners in primary connections. Test in accordance with Collaboration standard S 10.1, Appendix 1, "Rotational Capacity Test (Long Bolts in Tension Calibrator)," or Appendix A1, "Rotational Capacity Test (Bolts Too Short to Fit In Tension Calibrator)," as applicable.

C4.7.1

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

Rotational capacity (RC) testing is required at two levels. ASTM requires RC testing in the manufacture of zinc coated bolts and FHWA requires that DOTs conduct field RC testing for fastener assemblies (bolt, nuts, and washers) used in structures. The appendices to S 10.1 referenced in this Section reproduce the requirements of FHWA report FHWA-SA-91-031 with 1994 revisions.

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, Owners should check the Material Test Report to be sure that this test requirement has been satisfied.

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

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

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

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

4.7.2

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.

C4.7.2

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 the DOT conduct verification inspection.

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

4.8.1

Provide an SSPC-SP 6 blast in the shop to all fascia surfaces of unpainted weathering steel beams. 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.

C4.8.1

The aim of the weathering steel surface preparation requirements is to give a relatively consistent appearance from the ground without requiring a perfectly uniform finish. If part of the girder will be painted, other levels of surface preparation may be required.

4.8.2

Do not mark fascia surfaces. Use one of the following methods as soon as possible to remove any markings or any other foreign material 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*

4.8.3

Do not use acids to remove stains or scales. Feather out touched-up areas over several feet.

SECTION 5

HEAT APPLICATION

5.1—HEATING PROCESS AND EQUIPMENT

5.1.1

Do not exceed the maximum allowable temperatures given in Table 5.1.1-1 when applying heat to steel.

C5.1.1

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. The guide specification provides a number of rules that must be followed; these should be incorporated into the procedures. The Fabricator should also incorporate checks and verifications to be performed by the QC inspectors and any non-destructive testing that may be necessary.

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.

Table 5.1.1-1—Maximum Temperature Limits for Heat Application

Grade	Maximum Temperature °F
36	1200
50, 50S, 50W, HPS 50W	1200
HPS 70W, Q&T and TMCP	1100
100/100W and HPS 100/100W	1100

5.1.2

Complete heating before painting.

5.1.3

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

5.1.4

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

C5.1.4

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.

5.1.5

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

C5.1.5

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.), two torches are needed to heat both sides simultaneously.

5.1.6

Manipulate heating torches to avoid overheating.

C5.1.6

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.

5.1.7

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

5.1.8

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

5.1.9

Avoid buckling when heating relatively thin, wide plates.

C5.1.9

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.

5.1.10

Routinely monitor temperatures with temperature-sensitive crayons, pyrometers, or infrared non-contact thermometers. Measure the temperature 5–10 seconds after the heating flame leaves the area to be tested.

5.1.11

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

5.1.12

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

5.1.13

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

5.1.14

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

5.2—HEAT-CURVING FOR SWEEP OF BRIDGE MEMBERS

5.2.1

Use an approved procedure that describes the method of supporting, loading, or both, and also provides calculations, if applicable, that satisfy the preload limits of Section 5.1.3.

C5.2.1

The term "heat-curving" is usually used to describe the shaping by heat of bridge members to the curve shape required in the structure. It is often more practical to fabricate I-shape members straight and then curve them rather than building them curved. However, there are limits to what can be effectively curved, depending upon the properties of the member and how tight the radius will be. The heat-curving formula in this specification is intended to provide a conservative limit.

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., 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.

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 5.3.

5.2.2

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

C5.2.2

Usually, it is best to fabricate a girder "shell", or flanges and webs, then perform curving, and then add parts like stiffeners. Stiffeners may be added before heat curving, but then the stiffener-to-flange welds should be done after curving. Longitudinal stiffeners are added after curving to avoid twisting of the member due to asymmetry.

5.2.3

When the radius is less than 1000 ft, heat-curve only with the web in the horizontal position or preload to induce stress prior to heating. (See Sections 5.3.2 and 5.4.1)

C5.2.3

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.

5.2.4

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.

5.2.5

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

C5.2.5

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.

5.2.6

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.

5.3—MINIMUM RADIUS FOR HEAT-CURVING

5.3.1

Do not heat-curve beams or girders when the horizontal radius of curvature measured to the centerline of the member web is less than either value calculated using Eqs. 5.3-1 and 5.3-2, or when the radius is less than 150 ft at any cross section throughout the length of the member.

$$R = \frac{14bD}{\sqrt{F_y \psi t}} \tag{5.3-1}$$

C5.3.1

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

or

$$R = \frac{7500b}{F_y \psi} \tag{5.3-2}$$

where:

F_y = specified minimum yield point in ksi of the member web

ψ = ratio of the total cross-sectional area to the cross-sectional area of both flanges

b = width of the widest flange in in.

D = clear distance between flanges in in.

t = web thickness in in.

R = radius in in.

5.3.2

Do not heat-curve portions of members where the required radius of curvature is less than 1,000 ft, and the flange thickness exceeds 3 in. or the flange width exceeds 30 in.

5.4—HEAT-CAMBERING

5.4.1

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

C5.4.1

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

5.4.2

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

5.5—HEAT-STRAIGHTENING DAMAGED STRUCTURAL STEEL

5.5.1

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.

C5.5.1

See Section C5.1.1 for FHWA resources on heat-straightening.

5.6—HEAT-ASSISTED BENDING (MECHANICAL HOT BENDING)

If a smaller radius than that allowed in Table 4.3.2-1 is required or heavy plate thickness exceeds the shop's capacity to cold-bend standard radii, materials may be bent with heat assistance. Apply heat to obtain uniform temperature throughout the plate thickness before jacking pressure is applied. Heat-assisted bending shall be done below the maximum temperature limits of Table 5.1.1-1.

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

C5.6

"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). (See Section 11.4.3.3.3 in Division II of the AASHTO *Standard Specifications for Highway Bridges* or in the AASHTO *LRFD Bridge Construction Specifications*.)

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.

5.7—HEAT TREATMENT

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

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.

C5.7

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

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 the *Bridge Welding Code* tolerances and as described below.

6.1.3

Rolled or fabricated sections of equal or slightly greater dimensions than the section specified may be proposed for the Engineer's acceptance. For changes that affect splice design or may significantly alter deflection, provide complete design calculations.

6.2—SUBSTRUCTURE MEMBERS

6.2.1

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.

C6.2.1

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 Specification does not provide tolerances for bearings oriented perpendicular to each other or in different planes because the amount of offset allowable is a function of the torsional stiffness of the box and many other factors. Proper seating may be further complicated by field conditions. Together, the Contractor and Fabricator are responsible for the fit of the structure in the field. Therefore, this Specification requires that the completed structure satisfy the design requirements, but does not provide specific fabrication tolerances.

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

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

Fabricate pins and pinholes so that the pinhole diameter does not exceed the pin diameter by more than 0.015 in. for pins 5 in. or less in diameter, or $^{1}/_{32}$ in. for larger pins.

SECTION 7

BRIDGE GEOMETRY

7.1—ASSEMBLY

C7.1

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 ft, 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 computer-numerically-controlled 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.

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 AASHTO. In the early days of steel bridges when members were shorter, entire girder lines would be laid down in the shop. Then, as members got longer, the norm became five, and then, finally, three. For many steel bridges, even three members 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, i.e., cross frames, rolled shape diaphragms, plate diaphragms, lateral bracing, etc., is typically need not 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 using low- or mini-stress steel stamps. Provide diagrams showing match marking method and location on the approved shop drawings.

7.2—BOLTED SPLICES

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

- Eight-five percent of the bolt holes in any adjoining group vary no more than $^1/_{32}$ in. 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.,$+^1/_8$, $-\,^3/_{16}$.

C7.2

A zero 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—WELDED FIELD SPLICES

7.3.1

For field-welded splices, prepare the ends of beams and girders in accordance with Figure 1.

C7.3.1

These provisions address shop-required work for field-welded connections in primary members. The actual accomplishment of the connections is beyond the scope of this Specification. The webs may be restrained by hand-installed devices while checking compliance with the tolerances of Sections 7.3.2 and 7.3.3.

7.3.2

Assemble girders to demonstrate that workmanship requirements of the *Bridge Welding Code* are satisfied.

7.3.3

Prepare joints in accordance with the alignment tolerances of *Bridge Welding Code,* Section 3.3.4. Prepare the access hole so that the sloping transition is tangent to the joint bevel.

7.4—ALTERNATE GEOMETRY CONTROL METHODS

Fabricators may propose alternate methods of geometry control for continuous girder bridges based on demonstrated accuracy that precludes the necessity for assembly.

C7.4

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

7.5—TRUSSES, ARCHES, AND FRAMES

7.5.1

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

C7.5.1

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

7.5.2

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

7.5.3

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

C7.5.3

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.

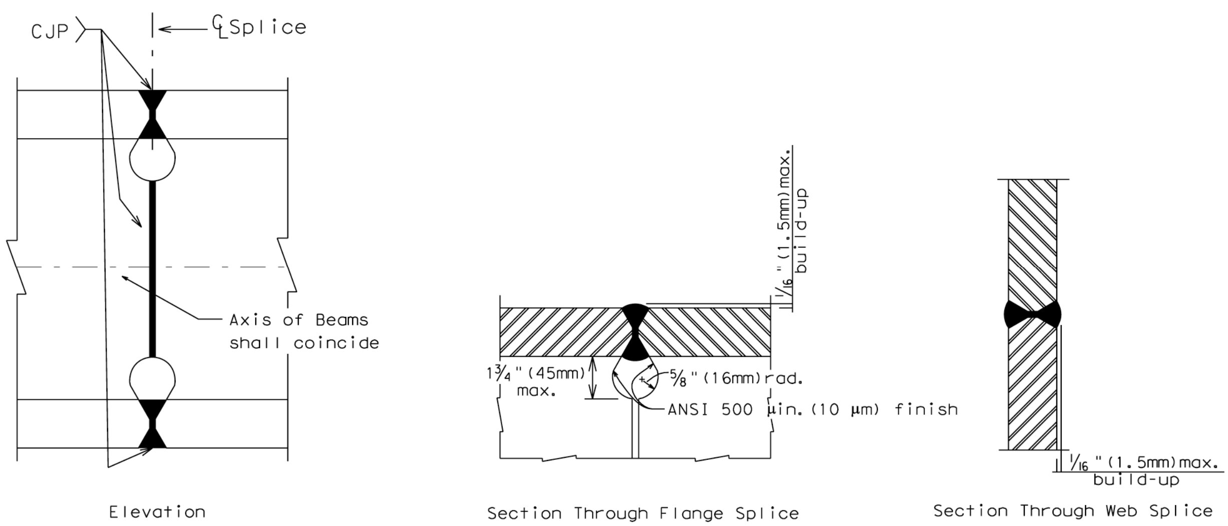

Figure 7.3.1-1— Field Welded Splice Details

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