

A Guide to

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# Engineering and Quality Criteria for Steel Structures

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Common Questions Answered



Fourth Edition

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by

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## Preface

This publication replaces and expands upon the information available previously in *Quality Criteria and Inspection Standards* (AISC, 1988), which has been updated by the AISC Committee on Manuals, Textbooks, and Codes as a general construction and design reference. It is not intended to preclude the use of other rational procedures or alternatives.

This reference has been prepared to: (1) familiarize engineers, architects, fabricators, erectors, inspectors, and owners with the techniques and tolerances of standard steel fabricating and erecting practice and with the economic effect of special workmanship requirements; and, (2) to provide engineering guidance in answer to common questions in steel design and construction. Some common problems and conflicts of interpretation of standards involving fabrication tolerances and procedures are discussed; the AISC recommendation for their clarification and resolution is provided. Such recommendations should be used together with the existing specifications and codes to advance the goals of achieving steel structures that are safe, economical, and serviceable. While some recommendations offer helpful guidance, they may not be based upon code requirements.

A wide variety of resource information exists in the literature. Specifications and codes such as AISC's *LRFD Specification for Structural Steel Buildings* and *Code of Standard Practice for Steel Buildings and Bridges* provide a practical basis for engineering and quality standards. Other organizations also provide specifications that cover steel-related products; the American Welding Society (AWS), the Research Council on Structural Connections (RCSC), the American Society for Testing and Materials (ASTM), the Steel Structures Painting Council (SSPC), and the American Association of State Highway and Transportation Officials (AASHTO), among others.

These commonly accepted standards for fabricating and erecting structural steel framing provide satisfactory criteria for design, workmanship, fit, and appearance at minimum cost for the vast majority of buildings and bridges. The imposition of more restrictive criteria or tolerances, while sometimes justified by specific conditions of usage, can significantly increase the cost of fabrication. Furthermore, accuracy or perfection of finish that serves no useful function only increases the cost, not the quality, of steel construction.

In addition to these documents, many other sources of information exist. Peer-reviewed papers covering a wide variety of steel design, fabrication, and erection concerns are published in the *AISC Engineering Journal* and the *ASCE Journal of Structural Engineering*. The *Proceedings* of AISC's National Steel Construction Conference, published annually, contain many papers of interest. While primarily non-technical in nature, AISC's *Modern Steel Construction Magazine* regularly contains articles of interest in engineering decision-making.

This guide is generally based upon the provisions of the 1993 AISC LRFD Specification for Structural Steel Buildings (referred to herein as the LRFD Specification). Most

information herein can also be used with the 1989 AISC ASD Specification for Structural Steel Buildings. Where this is not the case, it has been so noted in the text.

In many cases, statements are quoted from the various referenced specifications, codes, and standards and other sources. In some cases, editorial clarification has been made herein to minimize the volume of verbiage. When this is the case, square brackets surround the editorialization.

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## Abbreviations and Definitions

The following terms and abbreviations appear throughout the text of this guide. In general, defined terms are capitalized in the text.

- AASHTO* American Association of State Highway and Transportation Officials.
- ACI* American Concrete Institute.
- AESS* Architecturally exposed structural steel, as defined in *AISC Code of Standard Practice* Section 10.
- AISC* American Institute of Steel Construction.
- AISI* American Iron and Steel Institute.
- ANSI* American National Standards Institute.
- API* American Petroleum Institute.
- Applicable Building Code* The code under which the building was designed.
- ASCE* American Society of Civil Engineers.
- ASTM* American Society of Testing and Materials.
- AWS* American Welding Society.
- CMTR* Certified mill test report.
- HSS* Hollow structural section.
- Mill* The steel material manufacturer.
- NAAMM* National Association of Architectural Metals Manufacturers.
- NDE* Non-destructive Examination.
- PQR* Procedure Qualification Record.
- RCSC* Research Council on Structural Connections.
- SER* Structural Engineer of Record.
- SSPC* Steel Structures Painting Council.
- Statically Loaded Structures* Structures subject to loading that characteristically is slowly applied and removed, as would be typical in building, sign, and tower structures; dead, live, wind and similar loads are generally considered to be static.
- Cyclically Loaded Structures* Structures subject to loading that is applied and/or removed at a rate that cannot be considered to be static and requires consideration of fatigue, as would be typical in bridge structures and crane runways.
- WPS* Welding Procedure Specification.





## CHAPTER 1

### MILL PRODUCTION AND TOLERANCES

ASTM A6/A6M covers mill requirements for structural steel, including dimensional tolerances on the cross-section of structural shapes, the quality requirements and the type of mill conditioning permitted. This commentary includes a discussion of portions of these provisions and the work required either when supplied material does not meet the tolerances specified or when more restrictive tolerances are specified.

#### 1.1. Cross-sectional and Straightness Tolerances

##### 1.1.1. *Where are the (mill) dimensional tolerances for structural shapes and plates given?*

Permissible variations for structural shapes and plates as received from the mill are established in ASTM A6/A6M Section 13. These historically developed standard tolerances define the acceptable limits of variation from theoretical dimension for the cross-sectional area, flatness, straightness, camber, and sweep for rolled sections. It should be noted that cross-sectional tolerances are expressed as a percentage of weight or area, not as tolerances on dimensions such as the flange and web thicknesses.

Generally, standard fabrication practice accommodates these structurally acceptable variations. In special cases such as high-rise construction, the accumulation of mill tolerances may require consideration in design by the SER. If more restrictive tolerances are required they must be specified in the contract documents.

#### 1.2. Surface Condition

##### 1.2.1. *Where are the permissible variations in surface condition for structural shapes defined?*

ASTM A6/A6M Section 9 defines the permissible variations in the surface condition for structural shapes and plates in the as-rolled condition. It should be recognized that surface imperfections, such as seams and scabs, within these specified limits may be present on material received at the fabrication shop; particularly on heavyweight cross-sections because of higher finishing temperatures and production difficulties. Certain steel chemistries, such as that for ASTM A588, will also exhibit a higher incidence of surface imperfections.

Special surface-condition requirements must be specified in the contract documents. Material purchased to meet the requirements of ASTM A6/A6M is usually subject to acceptance or rejection based upon visual inspection both at the rolling mill and at the time of receipt by the fabricator, although more extensive inspection methods may be used. This inspection is important because mills normally limit their contractual liability to replacement or credit. Because occasional surface imperfections may be discovered after the fabricator's acceptance of mill material, particularly after blast cleaning, any requirements for remedial work should also be specified in the contract documents.

*1.2.2. What corrective procedures are available to the mill when variations in surface condition do not meet specified tolerances?*

ASTM A6/A6M Section 9 specifies limited conditioning that the mill may perform when as-rolled material does not meet specified tolerances. Note that it further states that “conditioning of imperfections beyond the [specified] limits ... may be performed by [the fabricator] at the discretion of [the fabricator]”.

Unless required in the contract documents, code-compliant surface imperfections generally need not be repaired or removed if they are not detrimental to the strength of the member. When required, they may be repaired by grinding or welding. The responsibility for any required repairs should be assigned in the contract documents so that it is clearly understood by all parties involved, including the owner’s representative (e.g., general contractor), fabricator, erector, and painter.

*1.2.3. How should edge discontinuities in mill material be treated?*

Non-injurious edge discontinuities in Statically Loaded Structures need not be removed or repaired, unless otherwise specified in the contract documents. Injurious defects, such as a longitudinal discontinuity that will be subjected to through-thickness loading, should be repaired by welding and/or grinding. The requirements for treatment of such edge discontinuities must be clearly specified in the contract documents and the repair procedure should be approved by the SER.

In Cyclically Loaded Structures, the provisions of AWS D1.1 Section 5.15.1.2 for edges that are to be welded are appropriate for non-welded edges, except that:

1. With the approval of the purchaser, discontinuities need not be explored to a depth greater than 1 in. When the depth of a discontinuity exceeds 1 in., the discontinuity should be gouged out to a depth of 1 in. and beyond its intersection with the surface and repaired by the deposition of weld metal as indicated in AWS D1.1 Section 5.15.1.1.
2. For discontinuities over 1-in. long, with depth exceeding 1/8-in. but not greater than 1 in., the discontinuity must be removed and repaired, but no single repair should exceed 20 percent of the length of the edge repaired.

### **1.3 Ordering Steel**

*1.3.1. What information is required to be reported in a Certified Mill Test Report (CMTR)?*

The information required to be reported in a CMTR is as given in ASTM A6/A6M Section 18. This includes but is not limited to the steel grade and nominal sizes supplied and tension test results. This document may be in written form or, per ASTM A6/A6M Section 18.8, transmitted electronically.

*1.3.2. What must the specifier indicate when material is subject to a domestic purchasing requirement?*

When a domestic purchasing requirement is in effect for a given project, the specifier must indicate in the contract documents and purchase order that material must be melted and manufactured in the United States of America.

*1.3.3. When a project is subject to a metric design requirement, what shapes are available?*

ASTM A6M, the metric equivalent of ASTM A6, covers the metric series of structural shapes that is in use in the United States. Because it is a soft<sup>1</sup> metric conversion, the metric series is physically identical to the inch-pound-unit shape series. The dimensions are given in millimeters (mm) with mass expressed in kilograms (kg); note that the mass must be multiplied by the acceleration of gravity 9.81 m/s<sup>2</sup> to obtain kilonewtons (kN). Metric properties and dimensions of the ASTM A6M shape series can be found in AISC's *Metric Properties of Structural Shapes*.

*1.3.4. To what ASTM Specifications are hollow structural sections (HSS) ordered?*

ASTM A500 and A847 are appropriate when specifying square, rectangular, and round HSS. These specifications cover cold-formed production of both welded and seamless HSS; ASTM A847 offers atmospheric corrosion resistance properties similar to that of ASTM A588 for W-shapes. Pipe-size rounds (P, PX, and PXX) are also available in ASTM A53 grade B material.

#### **1.4. Other General Information**

*1.4.1. Color combinations are commonly used to indicate various steel grades. Where are these color combinations established?*

Colors that identify the various grades of structural steel are established in ASTM A6/A6M Section 12.6.3; for example, green and yellow for ASTM A572 grade 50 steel and blue and yellow for ASTM A588 steel.

*1.4.2. Where are chemistry requirements for structural steel specified?*

Chemistry limitations and requirements are specified in the ASTM specifications for structural steels, such as ASTM A36, A572, A588, etc. Steel producers are required to report steel chemistry for each heat of steel produced on a CMTR (see 1.3.1).

*1.4.3. Structurally, is there a difference between a ½ × 4 bar and a ½ × 4 plate?*

Structurally, none; furthermore, plate is becoming a universally applied term today. However, the historical classification system for such structural material would suggest the following physical difference: all four sides of a ½ × 4 bar would be rolled edges, i.e., the mill rolled it to that thickness and width. A ½ × 4 plate will have been cut from a ½-in. plate of greater width either by shearing or flame cutting.

*1.4.4. What are the common length limits on structural steel members as ordered from the mill?*

Common mill lengths range from 30 ft to 65 ft in 5-ft increments. However, because individual mill practices and standards vary, it is best to consult with individual mills directly. When steel is purchased from a warehouse, the selection

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1. A soft conversion is made by directly converting the U.S. customary unit value to a metric equivalent, for example, 1 in. equals 25.4 mm; conversely, a hard conversion is made by selecting new values in round metric increments, such as replacing 1 in. with 25 mm.

of available lengths may be further limited. Additionally, the method of shipment may also limit the available length.

## CHAPTER 2

### GENERAL FABRICATION

The AISC LRFD *Specification for Structural Steel Buildings* and *Code of Standard Practice for Steel Buildings and Bridges* cover the requirements for fabrication of structural steel. This commentary includes a discussion of portions of these provisions and subsequent recommendations.

#### 2.1. Material Identification and Traceability

##### 2.1.1. *What is required for the identification of material?*

Identification means the ability to determine that the specified material grade and size is being used. An identification system is required in AISC LRFD Specification Section M5.5: “...the fabricator shall be able to demonstrate by a written procedure and by actual practice a method of material application and identification, visible at least through the “fit-up” operation, of the main structural elements of a shipping piece. The identification method shall be capable of verifying proper material application as it relates to:

1. Material specification designation
2. Heat number, if required
3. [CMTR] for special requirements [if required].”

##### 2.1.2. *What is the difference between traceability and identification of material?*

Traceability means the ability to identify a specific piece of steel in a structure, throughout the life of the structure, and its specific CMTR. As such, traceability requirements are significantly more expensive than the identification requirements in 2.1.1. The owner should clearly understand the differences, limitations, and relative costs involved.

Traceability is not a requirement in the AISC LRFD Specification and, when required, must be clearly specified in the contract documents prior to the ordering of material. The following elements of traceability should be selected only as needed:

1. *Lot traceability vs. piece-mark traceability vs. piece traceability:* Lot traceability means that the materials used in a given project can be traced to the set of CMTR’s for that project. Piece-mark traceability means that the heat number can be correlated for each piece mark, of which there can be many individual pieces. Piece traceability means that the heat number can be correlated for each piece, which effectively demands separate piece marks for each piece.

Each of these three successive levels of traceability adds significant costs. Piece traceability, the most expensive option, is necessary only in critical applications, such as the construction of a nuclear power facility. Piece-mark traceability is often specified for main members in bridges. Lot identification is most common in other applications where traceability is required.

2. *Main-material traceability vs. all-material traceability:* Main-material traceability means that beams, columns, braces, and other main structural members are traced as specified above. All-material traceability means that connection and detail materials are also traced as specified above.

All-material traceability, the more expensive option, is necessary only in critical applications, such as the construction of a nuclear power facility. In other cases, main-material traceability is sufficient, when traceability is a requirement.

3. *Consumables traceability* means that lot numbers for consumables such as bolts, welding electrodes, and paint can be traced. This is necessary only in critical applications, such as the construction of a nuclear power facility.

4. *Required record retention* defines the level of detail required in documenting traceability (who, what, when, where, how, etc.).

5. *Fool-proof record retention vs. fraud-proof record retention:* Fool-proof record retention means internal verification of records. Fraud-proof record retention means external certification of records. Fraud-proof record retention is necessary only in critical applications, such as the construction of a nuclear power facility. In other cases, foolproof record retention is sufficient, when traceability is a requirement.

### 2.1.3 *How does a fabricator maintain traceability, when it is required?*

Each heat of steel produced by the mill is tested for chemical content and mechanical properties and the results are recorded on a CMTR, which is provided to and maintained in the records of the fabricator. Each piece that is rolled from this heat is then labeled with an identification mark that relates to the corresponding CMTR. The fabricator applies an identification mark to each piece. Because this piece mark remains with the piece throughout the fabrication and erection process, the material is traceable back to the CMTR for that individual piece.

Many connecting elements and similar fittings are too small to accommodate the marks to identify the piece from which they were cut. Additionally, such items are commonly made from stock materials with marks that may have inadvertently been abraded or lost during years of storage. In such cases, the fabricator provides written certification that the stock material meets the contract requirements.

Manufacturers of consumables such as bolts, welding electrodes and paint provide documentation as to the content and specification compliance of their products. This documentation is provided to and maintained in the records of the fabricator. The packaging in which the products are shipped is referenced to this documentation.

In some cases, the fabricator may purchase materials through a warehouse. When this is the case, the warehouse must transmit the necessary documentation from the manufacturer to the fabricator.

## 2.2. Cutting and Finishing Steel

### 2.2.1. *What methods are available for cutting steel and what is the corresponding range of utility for each?*

The following methods are commonly used to cut steel:

1. Friction sawing, which is performed with a high-speed rotary blade, is commonly used by mills and is limited only by machine size. This cutting method, however, is no longer commonly used in fabrication shops.
2. Cold sawing, whether by rotary saw, hack saw, or band saw, is limited only by machine size.
3. Oxygen-acetylene (and related fuel) flame cutting, which can be mechanically or hand-guided, is commonly used for general cutting and edge preparation operations, such as coping, beveling, notching, etc.; its utility is virtually unlimited.
4. Plasma cutting, which is mechanically guided, is generally useful for cutting plate of up to  $\frac{3}{4}$ -in. thickness.
5. Laser cutting, which is mechanically guided, is generally useful for cutting plate; thickness limitations vary.
6. Shearing, which is performed with mechanical presses, is generally useful for cutting plates and angles and is limited only by machine size and capacity.

Additional minor material removal and finishing may also be accomplished by one of the methods listed in 2.2.2.

- 2.2.2. *What methods are commonly used to provide finished surfaces, when required?*  
Some of the cutting methods in 2.2.1 result in surfaces that are finished without further treatment; see 2.2.3 and 2.2.4. When this is not the case, the following methods are commonly used to provide finished surfaces:
1. Milling, which is commonly used to bring members to their required length and end finish.
  2. Face machining, which can be used to finish large areas to exact dimensions.
  3. Planing.
  4. Grinding, which is commonly used for edge preparation, including treatment of flame-cut edges, removal of burrs, etc. when required.
- 2.2.3. *Can the end of a column, as received from the rolling mill, be considered to be a finished surface?*  
Yes, provided the mill cut is at right angles to the column axis and meets the surface roughness requirements in ANSI/ASME B46.1 (see 2.2.4).
- 2.2.4. *Is it commonly necessary to mill bearing surfaces after sawing?*  
No. As stated in AISC LRFD Specification Section M2.6, "compression joints which depend on contact bearing ... shall have the bearing surfaces of individually fabricated pieces prepared by milling, sawing, or other suitable means." AISC *Code of Standard Practice* Section 6.2.2 states that "any fabricating technique such as friction sawing, cold sawing, milling, etc. that produces [a finished surface with maximum ANSI roughness height value of 500] may be used." Cold-sawing equipment produces cuts that are more than satisfactory.
- 2.2.5. *What constitutes acceptable thermal cutting practice?*  
Structural steel preferably should be thermally cut by mechanically guided means. However, mechanically guided cutting may not be feasible in some cases, such as the cutting of copes, blocks, holes for other than bolt holes (see 2.4.1 and 2.4.2), and similar cuts. Accordingly, hand-guided thermal cutting should be

allowed as an alternative. Regardless, thermally cut surfaces must meet the appropriate roughness limitations as summarized in 2.2.6.

2.2.6. *What are the appropriate roughness limitations for thermally cut edges?*

Inadvertent notches or gouges of varying magnitude may occur in thermally cut edges, depending upon the cleanliness of the material surface, the adjustment and manipulation of the cutting head, and various other factors. When thermally cut edges are prepared for the deposition of weld metal, AISC LRFD Specification Section M2.2 and AWS D1.1 Section 5.15.1.1 provide acceptance criteria that consider the effect of discontinuities that are generally parallel to the applied stress on the soundness of welded joints. Additionally, correction methods for defects of various magnitudes are stipulated therein. When thermally cut edges are to remain unwelded, the following surface condition guidelines are recommended:

1. If subjected to a calculated tensile stress parallel to the edge, edges should, in general, have a surface roughness value not greater than 1,000 as defined in ANSI/ASME B46.1.
2. Mechanically guided thermally cut edges not subjected to a calculated tensile stress should have a surface roughness value not greater than 2,000 as defined in ANSI/ASME B46.1.
3. Hand-guided thermally cut edges not subjected to a calculated tensile stress should have a roughness not greater than  $\frac{1}{16}$ -in.
4. All thermally cut edges should be free of notches<sup>2</sup> and reasonably free of gouges.<sup>3</sup> Occasional gouges not more than  $\frac{3}{16}$ -in. deep are permitted.

Gouges greater than  $\frac{3}{16}$ -in. deep and all notches should be repaired as indicated in 2.2.7.

2.2.7. *When surface roughness for thermally cut edges does not meet the limitations in 2.2.6, how is the surface repaired?*

Roughness exceeding the criteria in 2.2.6 and notches not more than  $\frac{3}{16}$ -in. deep should be removed by machining or grinding and fairing-in at a slope not to exceed 1:2½. The repair of notches or gouges greater than  $\frac{3}{16}$ -in. deep by welding should be permitted. The following criteria are recommended:

1. The discontinuity should be suitably prepared for good welding.
2. Low-hydrogen electrodes not exceeding  $\frac{5}{32}$ -in. diameter should be used.
3. Other applicable welding requirements of AWS D1.1 should be observed.
4. The repair should be made flush with the adjacent surface with good workmanship.
5. The repair should be inspected to assure soundness.

2.2.8 *To what profile must re-entrant corners, such as corners of beam copes, be shaped?*

Re-entrant corners should provide a smooth transition between adjacent surfaces, but generally need not be cut exactly to a circular profile. The recommendation

2. defined as a V-shaped indentation or hollow.

3. defined as a groove or cavity having a curved shape.



in the AISC LRFD Manual (page 8-225) is that an approximate minimum radius of  $\frac{1}{2}$ -in. is acceptable. However, the primary emphasis should be that square-cut corners and corners with significantly smaller radii do not provide the smooth transition that is required. From AISC LRFD Specification Section J1.6, it is acceptable to provide radius transitions by drilling (or hole sawing) with common-diameter drill sizes (not less than  $\frac{3}{4}$ -in.) as suggested in LRFD Specification Commentary Figure C-J1.2.

When the corner of a cope has been square-cut, a common solution is to flame-cut additional material at the corner to provide a smooth transition as illustrated in Figure 2.2.8-1. Note that the sides of the cope need not meet the radius transition tangentially. Any notches that occur at re-entrant corners should be repaired as indicated in 2.2.7.

### 2.3. Use of Heat in Fabrication

#### 2.3.1. *Is it permissible to use controlled heat to straighten, curve, or camber structural steel shapes?*

Yes. AWS D1.1 Section 5.26.2 permits heat-straightening of members that are distorted by welding and stipulates rules for this procedure. These rules are equally applicable for all heat straightening or curving. Furthermore, AISC LRFD Specification Section M2.1 and the discussion in the AISC LRFD Manual (page 1-12), provide a sound basis for the use of controlled heat to straighten, curve, camber, and form structural steel. The proper control of heat application generally involves the use of rosebud tips on torches to disperse the applied flame and temperature indicating crayons or similar devices to monitor the induced temperature.

#### 2.3.2. *Is it permissible to accelerate cooling of structural steel after the application of controlled heat?*

Yes, provided heated steel for Cyclically Loaded Structures is first allowed to cool ambiently to 600 degrees F. Because the maximum temperature permitted

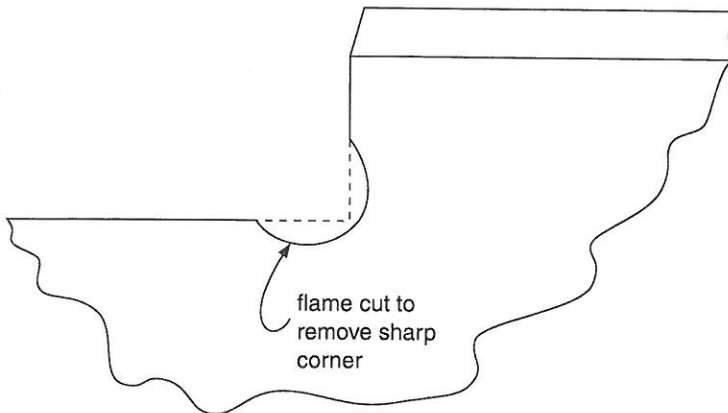


Fig. 2.2.8-1. Correction of square-cut copes.

by AISC LRFD Specification Section M2.1 for heating operations is below any critical metallurgical temperature for the material being heated, the use of compressed air, water mist, or a combination thereof should be permitted to accelerate the final cooling of the heated. For members to be used in cyclically loaded structures (i.e., where fatigue and toughness are design issues) it is recommended that such accelerated cooling not begin until the temperature has dropped below 600 degrees F. This limitation is more historical than technical in nature. As a fair balance between the desires of the fabricator and the concerns of the owner, it provides an added safeguard to prevent the abuse of excessive cooling and undesirable residual stresses should accepted procedures not be strictly monitored.

## 2.4. Bolt Holes

### 2.4.1. *What are the acceptable methods for making bolt holes?*

Acceptable methods for making bolt holes include:

1. Punching
2. Sub-punching and reaming
3. Drilling
4. Hole sawing
5. Flame piercing and reaming
6. Flame cutting, subject to surface quality requirements as discussed in 2.4.2.

See also 5.1.3.

### 2.4.2. *What variation in profile is generally acceptable for bolt holes?*

The slightly conical hole that naturally results from punching operations is acceptable. From RCSC Specification Section 3(c), "holes not more than  $\frac{1}{32}$ -in. larger in dimension than the true decimal equivalent of the nominal dimension that may result from a drill ... of the corresponding nominal diameter are considered to be acceptable." The width of slotted holes that are produced by flame-cutting, or a combination of punching or drilling and flame-cutting should generally be not more than  $\frac{1}{32}$ -in. greater than the nominal width except that gouges not more than  $\frac{1}{16}$ -in. deep are permitted. In Statically Loaded Structures, the flame-cut surface need not be ground smooth; for Cyclically Loaded Structures, the flame-cut surface must be ground smooth.

### 2.4.3. *Must burrs be removed in bolted connections?*

From RCSC Specification Section 3(c), "Burrs extending  $\frac{1}{16}$ -in. or less above the plate surface are permitted for connections that are not slip-critical. Burrs that would prevent solid seating of the connected parts in the snug-tight condition shall be removed from slip-critical connections." This longstanding provision is augmented by more recent test results as reported in RCSC Bolt Bulletin No. 5: "... burrs are not detrimental to the performance of bearing connections. [In slip-critical connections] if burrs are so small that they are flattened during the snugging, it is not necessary that they be removed." It is further stated therein that larger burrs can remain if extra care is taken in the bolt installation process to achieve the proper bolt tension.

## 2.5. Correction of Fabrication Errors

### 2.5.1. *Must fabrication errors always be repaired?*

No. Because the human element is involved in all phases of structural steel fabrication, material inadvertently may be cut to the wrong length, holes may be misplaced, parts may be located incorrectly, or notches or gouges may occur. However, many such errors or deviations need not be altered or repaired and are acceptable without change or penalty to the structure or its end use. Furthermore, some repair work may be more detrimental, such as that which creates higher residual stresses. In general, the SER should evaluate the deviation and whether it would be detrimental to the end use of the product.

In some cases, repair will be required and can usually be made so that the member will meet all performance criteria. Corrective measures to meet the requirements of shop drawings and specifications may generally be made by the fabricator during the normal course of fabrication, using qualified personnel and procedures that meet AISC and AWS specifications. Such action is considered to be a part of the fabricator's quality control program and should not require either notification of, or approval from, the owner or SER. However, in cases where major work is involved (cutting or removal of welded members from a welded assembly, modification of design, deviation from critical dimensions, etc.), the SER must be consulted and a plan of corrective action agreed upon.

### 2.5.2. *What repair is appropriate for material that is cut too short?*

When material is short of the minimum required length, welded splices or deposited weld metal, when applied with appropriate welding procedures and specified material, should be permitted with the approval of the SER.

### 2.5.3. *What repair is appropriate for mislocated bolt holes?*

Generally, mislocated fastener holes are not detrimental to the strength of a member if the remaining effective net section is adequate for the loads. As such, they may be left open, filled with bolts, or plug welded in accordance with AWS D1.1 Section 5.26.5 with the approval of the SER. Ultrasonic inspection is not generally required for plug-welded fastener holes. Alternatively, if a bolt hole is mislocated by a small amount, say less than a bolt diameter, it is often possible to adjust the connection material to accommodate the error.

### 2.5.4. *What repair is required when a minor member mislocation occurs?*

When detail parts are placed in error, minor mislocations should be investigated to determine if relocation is necessary. When relocation is necessary, such as when dimensions are critical, the error is major, or the incorrectly placed part is visually unacceptable under an AESS requirement, the incorrectly placed part should be removed. For a welded detail, flame cutting, gouging, chipping, grinding, or machining may be required. Care should be taken to avoid damage to the main material of the associated member. The surface of the main material should be ground smooth and repaired, if necessary, as indicated in 2.2.6 and 2.2.7.

### 2.5.5. *What is "moderate reaming" as indicated in AISC Code of Standard Practice Section 7.12?*

During the course of erection, it occasionally becomes necessary to ream holes so fasteners can be installed without damage to the threads, resulting in a hole

that is larger than normal or elongated. The hole types recognized by the AISC and RCSC Specifications are standard, oversized, short-slotted, and long-slotted, with nominal dimensions as given in AISC LRFD Specification Table J3.3. From RCSC Specification Section 3(c), “holes not more than  $\frac{1}{32}$ -in. larger in dimension than the true decimal equivalent of the nominal dimension that may result from a ... reamer of the corresponding nominal diameter are considered to be acceptable.” Note that reamed holes must meet the provisions for minimum edge distance and minimum spacing in AISC LRFD Specification Sections J3.3 and J3.4, respectively.

When more major misalignments occur, it is indicated in AISC *Code of Standard Practice* Section 7.12 that they are “... reported immediately to the owner and fabricator by the erector, to enable whoever is responsible to either correct the error or to approve the most efficient and economical method of correction to be used by others.”

## 2.6. Other General Information

### 2.6.1. *What precautions are required when cold bending material with sheared or flame-cut edges?*

When cold bending plates or performing other operations involving cold bending and a sheared or flame-cut edge, care must be taken to preclude the initiation of cracks at the edge. Minimum radii for cold bending plates of various steel grades are indicated in AISC LRFD Manual Table 9-13 (page 9-219). It is indicated in the corresponding text therein that the tabular values may have to be increased when bend lines are parallel to the direction of final rolling or longer than 36 in. Additionally, “Flame cut edges of hardenable steels should be machined or softened by heat treatment. Nicks should be ground out and sharp corners should be rounded.”

### 2.6.2. *What are the common length limits on fabricated structural steel members?*

The maximum length of a fabricated assembly is primarily limited by shipping and erectability concerns, such as overall length, depth, and total weight. However, because individual practices and capabilities vary, it is best to consult with the fabricator directly.

The common solution to a member length concern is a splice, which may be necessary and/or desirable for fabrication, shipping, and/or erectability considerations. When approved by the SER, fabricator-initiated splices in members are acceptable.

### 2.6.3. *Common steel items, such as metal deck and open-web steel joists, are not considered to be structural steel in the AISC Code of Standard Practice. Why?*

Even though items such as metal deck and open-web steel joists may be provided by the structural steel fabricator, they are not considered to be structural steel because they are neither manufactured nor fabricated by the structural steel fabricator. As such they are listed in Section 2.2 as “other steel or metal items”. Items that are normally part of the fabricator’s work are listed as structural steel items in Section 2.1.

## CHAPTER 3

# FABRICATION AND ERECTION TOLERANCES

The AISC *LRFD Specification for Structural Steel Buildings*, the AISC *Code of Standard Practice for Steel Buildings and Bridges*, AWS D1.1, and other existing specifications and codes cover tolerance requirements for the fabrication and erection of structural steel. This commentary includes a discussion of portions of these provisions and subsequent recommendations.

The structural steel fabrication industry has traditionally achieved a remarkable degree of dimensional accuracy in the fabrication and erection of steel structures. This is particularly evident when considering the variety and levels of skills essential to coordinate and perform the planning, detailing, fabrication, and erection of many unique and complex steel buildings, bridges, and other special structures.

### 3.1. Member Cross-sectional Tolerances

#### 3.1.1. *Can out-of-tolerance mill material be adjusted by the fabricator so that it conforms to the appropriate tolerances?*

Sometimes. Infrequently, material is discovered after delivery to be beyond mill tolerances. When material received from the rolling mill does not conform to the requirements of ASTM A6/A6M or more restrictive tolerances that are specified in the contract documents, the fabricator can use controlled heating, mechanical straightening, or a combination of both methods, consistent with manufacturer recommendations, to adjust cross-section, flatness, straightness, camber, and/or sweep.

#### 3.1.2. *What is the tolerance on depth for built-up girders and trusses?*

The appropriate tolerances for the welded cross-section are specified in AWS D1.1 Section 5.23. However, at bolted splices for such members, AWS D1.1 Section 5.23 is silent on this subject. AISC recommends that the permissible deviations for girder depth in AWS D1.1 Section 5.23.9 be applied to depth at bolted splices. Any differences within the prescribed tolerances at such joints should be taken up, if necessary, by shimming.

#### 3.1.3. *What is the flatness tolerance for webs of built-up girders?*

For members in Statically Loaded Structures, web flatness does not affect the structural integrity of a girder because it primarily resists shear. Accordingly, neither the AISC LRFD Specification nor the AISC *Code of Standard Practice* includes a limitation on the out-of-flatness of girder webs. Such a tolerance is specified for welded plate girders, however, in AWS D1.1 Section 5.23.6.2.

Shrinkage of web-to-flange welds and/or welds that attach stiffeners to the web can create operational difficulties in girder webs, particular those that are less than  $\frac{5}{16}$ -in. thick. Accordingly, the dimensional tolerance for deviation from flatness of a girder web less than  $\frac{5}{16}$ -in. thick, with or without stiffeners, in Statically Loaded Structures should be determined as the larger of  $\frac{1}{2}$ -in. or the value determined in AWS D1.1 Section 5.23.6.2. In Cyclically Loaded Structures,

the value in AWS D1.1 Section 5.23.6.3 should be observed. If architectural considerations require a more restrictive flatness tolerance, it should be specified in the contract documents. In all cases, the web thickness specified should be adequate to minimize such distortion.

### 3.2. Member Straightness Tolerances

#### 3.2.1. *How are the permissible deviations from straightness described in Chapter 1 accounted for in fabrication and erection?*

In most cases, deviations from true straightness and dimension of individual members (within the tolerances specified in ASTM A6/A6M) are compensated for during erection by the relative flexibility of the individual members compared to that of the overall structural steel frame they comprise. In some structures using heavy, rigid cross-sections, however, the stiffness of the member may preclude any adjustment of out-of-straightness that, although within acceptable limits, can prevent tight fit-up of connections. This situation is most likely to occur with multi-story building columns and may cause difficulty in erecting the floor framing members.

Although normal detailing practices may compensate in part for this problem, special shop layout practices are essential for heavy, rigid framing. A straight theoretical working line should be established between member ends as defined by the AISC *Code of Standard Practice* Section 7.11.2(d).

See also 3.5.3.

#### 3.2.2. *What tolerance is applicable for the camber ordinate when beam camber is specified?*

As indicated in AISC *Code of Standard Practice* Section 6.4.5, for members less than 50-ft long, the camber tolerance is minus zero/plus 1/2-in; an additional 1/8-in. per each additional 10 ft of length (or fraction thereof) is allowed for lengths in excess of 50 ft. An exception is also included: members received from the rolling mill with 75 percent of the specified camber require no further cambering. Furthermore it is specified that camber be inspected in the fabricator's shop in an unstressed condition.

#### 3.2.3. *What is the tolerance on sweep for curved girders?*

Permissible variations in sweep for horizontally curved welded plate girders are specified in AWS D1.1 Section 5.23.5. However, because the method of measurement for this sweep dimension is not defined, the tolerance is sometimes misapplied. The permissible variation specified is the deviation of the theoretical mid-ordinate from a chord through the ends of a single fabricated girder section.

If it is required to hold the ordinate of additional points along the beam within a certain tolerance, these requirements should be specified in the contract documents. Note, however, that most girders have sufficient lateral flexibility to easily permit the attachment of diaphragms, cross-frames, lateral bracing, etc., without damaging the structural member or its attachment.

#### 3.2.4. *What is the tolerance on twist of welded box members?*

As stated in AWS D1.1 Section 5.23.11.4, "...[the tolerance on] twist of box members ... shall be individually determined and mutually agreed upon by the

contractor and the owner with proper regard for erection requirements.” In the absence of a specified tolerance, an attempt is sometimes made to apply the provisions of ASTM A500 or ASTM A6/A6M. However, the provisions of these material specifications should not be applied to fabricated box members.

In an unspliced member, the necessary tolerance on twist is generally a matter of serviceability or aesthetics. In a member that will be spliced, twist must be kept within limits that will allow safe and uncomplicated erection. Shop assembly of the entire member by the fabricator may be necessary to accomplish this. It is recommended that the fabricator and erector mutually agree on the means and methods necessary to achieve installation of an acceptable member in the completed structure (see 3.5.1). Connection details for fabricated box members should accommodate twist in the completed member.

In any case, the required twist tolerance should be specified in the contract documents. Note, however, because of high torsional strength and stiffness, correction of twist in a closed box or similar shape is nearly impossible and carries the potential for damage. If the actual twist of a fabricated member exceeds a specified tolerance, whether to attempt correction should be a case-by-case decision made by the SER.

### 3.3. Element Location Tolerances

#### 3.3.1. *Is a tolerance on hole or hole pattern location specified in the AISC Code of Standard Practice?*

No. Neither the  $\pm 1/16$ -in. tolerance, where applicable, on overall length of members framed to other steel parts, nor the  $1/16$ -in. clearance on size of standard holes, should be construed as implying that the tolerance  $\pm 1/16$ -in. also applies either to the maximum tolerance on hole location within a pattern of holes or to the position of intermediate connections.

#### 3.3.2. *What is the tolerance on location of intermediate and longitudinal stiffeners?*

When intermediate stiffeners are spaced at a distance that is approximately equal to the girder depth, weld shrinkage up to  $3/8$ -in. in a 100-ft-long girder is not uncommon. Furthermore, thermal expansion or contraction in a like length of girder due to a temperature differential of 50 degrees F. can cause a change in length of approximately  $3/8$ -in. In view of these and other factors, there is a need for a tolerance on the location of longitudinal stiffeners. Because AWS D1.1 Section 5.23 is silent on this subject, AISC recommends the following criteria:

1. Intermediate stiffeners may deviate from their theoretical location  $\pm 2$  in. as measured from the girder end.
2. Diaphragm and other connection stiffeners may deviate from their theoretical location by no more than twice the thickness of the stiffener.
3. Longitudinal stiffeners may deviate from their theoretical location by a distance equal to 1 percent of the girder depth.
4. If longitudinal stiffeners are interrupted by vertical stiffeners, the ends should not be offset by more than half the thickness of the longitudinal stiffeners.

3.3.3. *When forces are to be transferred by contact bearing, is a gap allowed between the contact surfaces?*

From AISC LRFD Specification Section M4.4, "...lack of contact bearing not exceeding a gap of  $\frac{1}{16}$ -in., regardless of the type of splice used [i.e., welded or bolted] is permitted." If the gap exceeds  $\frac{1}{16}$ -in., but is less than  $\frac{1}{4}$ -in., and an engineering investigation shows that the actual area in contact (within  $\frac{1}{16}$ -in.) is adequate to transfer the load, then the gap is acceptable. Otherwise, from AISC LRFD Specification Section M4.4, the gap must be packed with non-tapered steel shims. Similarly, a tolerance of  $\frac{1}{16}$ -in. for bearing stiffeners is allowed in AWS D1.1 Section 5.23.11.1. Such a gap would presumably be closed under load, bringing the stiffener into full contact bearing.

### 3.4. Erection Tolerances

3.4.1. *How do individual member deviations impact the alignment and erected position of the overall structural steel frame?*

In many cases, individual member deviations that exceed established tolerances will have no adverse effect on the overall structural steel frame. However, in other instances, individual member deviations may accumulate and cause the overall structural steel frame to substantially exceed the overall permissible tolerances for plumbness, level, and line. It is essential that the effect of individual member tolerances on the overall structural steel frame be recognized and accounted for with practical detailing and fabrication techniques that permit compliance with overall tolerances.

### 3.5. Other General Information

3.5.1. *How are tolerances determined if they are not addressed in the applicable standards?*

The fabrication and erection tolerances in the AISC *LRFD Specification for Structural Steel Buildings*, the AISC *Code of Standard Practice for Steel Buildings and Bridges*, AWS D1.1, and other existing specifications and codes have evolved over nearly three-quarters of a century. Although these standards generally present a workable format for the fabricator and erector, they tend to address individual members, rather than the role of individual members in the completed structure.

Tolerances for assemblies, such as those on shop-assembled bents, frames, platforms, pairs of girders, etc., are not covered by any code or standard. AWS D1.1 Section 5.23.11.4 states that "... other dimensional tolerances of members not covered by [Section] 5.23 shall be individually determined and mutually agreed upon by the contractor and the owner with proper regard for erection requirements." This practice is recommended in all cases. The agreed upon tolerances should account for the erection tolerances specified in the AISC Code of Standard Practice.

3.5.2. *If special or more restrictive tolerances are required for the overall structural steel frame, can they be met?*

Possibly, but at a higher cost. Special clearances or tolerances may be difficult or impossible to achieve because of considerations such as temperature change,



fabrication and construction procedures, and erection stresses. When specified, such requirements must be identified in the contract documents. The additional cost of special or more restrictive tolerance requirements should be justified.

3.5.3. *How can the accumulation of mill, fabrication, and erection tolerances be economically addressed?*

While individual member tolerances are usually self-compensating and of minor significance in the overall structure, the possibility exists that these tolerances may accumulate and lead to misalignments that are difficult to correct in the field. As an example of the effect individual member tolerances may have on the total structure, consider the tolerances on columns and beams. Individual column and beam members are shown with their respective permissible tolerances in Figure 3.5.3-1. These tolerances come from several sources: permissible camber and

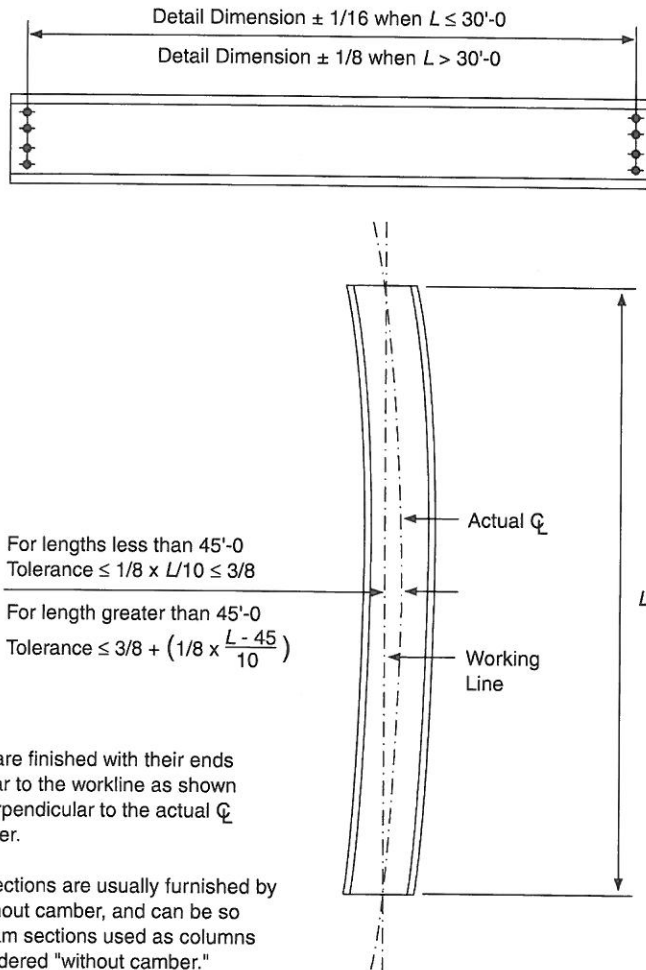


Fig. 3.5.3-1. Beam and column fabrication tolerances

sweep are specified in ASTM A6/A6M and AWS D1.1; permissible variation from detailed length for members framed to other steel parts is specified in the AISC *Code of Standard Practice*; mill tolerances on the cross-section are illustrated in AISC *Code of Standard Practice* Commentary Figure 1. The foregoing example involves a possible but highly unlikely scenario.

A case where individual members fabricated within permissible tolerances could make it impossible to erect a heavy two-story column within the plumbness tolerance of  $\pm 1:500$  is illustrated in Figure 3.5.3-2. Although the condition shown would be unusual and represents the worst case with all member tolerances maximized and accumulated in one direction, it is evident that the accumulation of tolerances requires special consideration. Other possible examples include double-angle and end-plate connections to columns, attached shelf or spandrel

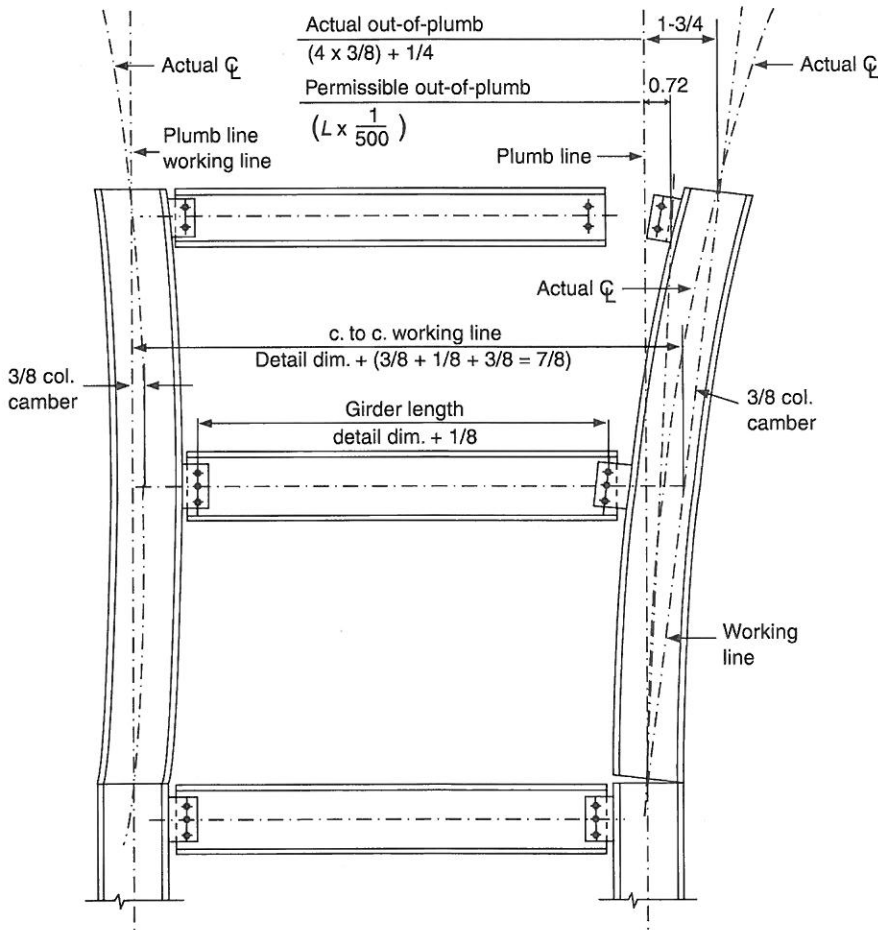


Fig. 3.5.3-2. Possible (but unlikely) accumulation of tolerances when details are located from actual centerlines.

angles, large plan dimensions in which many pieces line up, long bracing, expansion joints, and vertical systems such as stairs and multi-story wall panels. Deflections of cantilevered members and tolerance accumulation on complex framing systems involving a long series of connections before the load is in the column (causing accumulation of vertical tolerances) should also be considered.

Details for material supported by the steel framing must provide for the standard tolerances. For example, in buildings with large plans, it is beneficial to develop special details that accommodate the accumulation of fabrication tolerances. Note that building expansion joints cannot be adjusted to proper position without a provision for this adjustment.

The use of oversized holes, short-slotted holes, and long-slotted holes, provided a satisfactory method for achieving erection within tolerances as illustrated in Figures 3.5.3-3 and 3.5.3-4. Other satisfactory methods include the use of finger

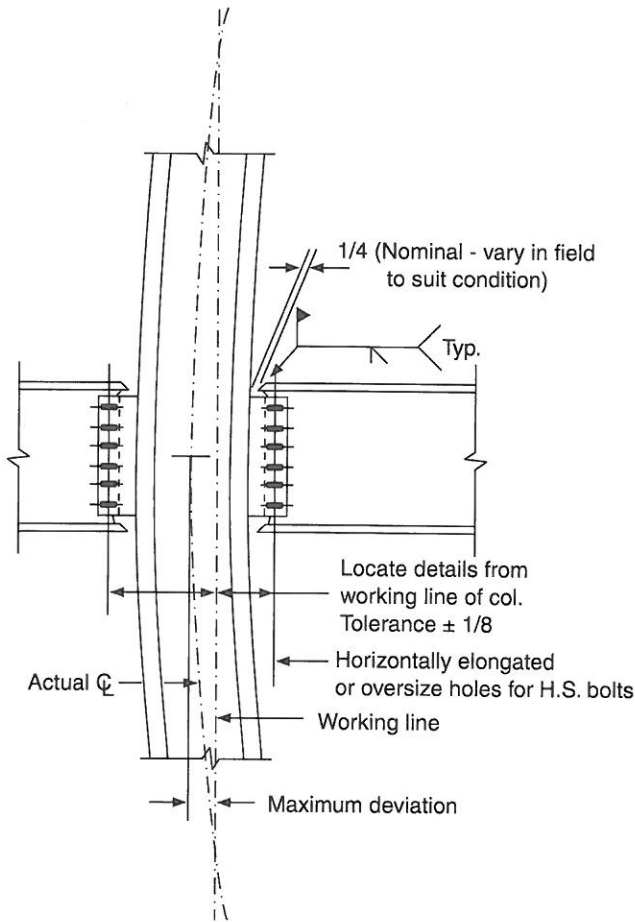


Fig. 3.5.3-3. Adjustments for column curvature in beam-to-column connections.

shims, shop layout to theoretical working lines, and recognition of tolerance accumulation in details for finishes, such as the curtain wall or stonework attachments.

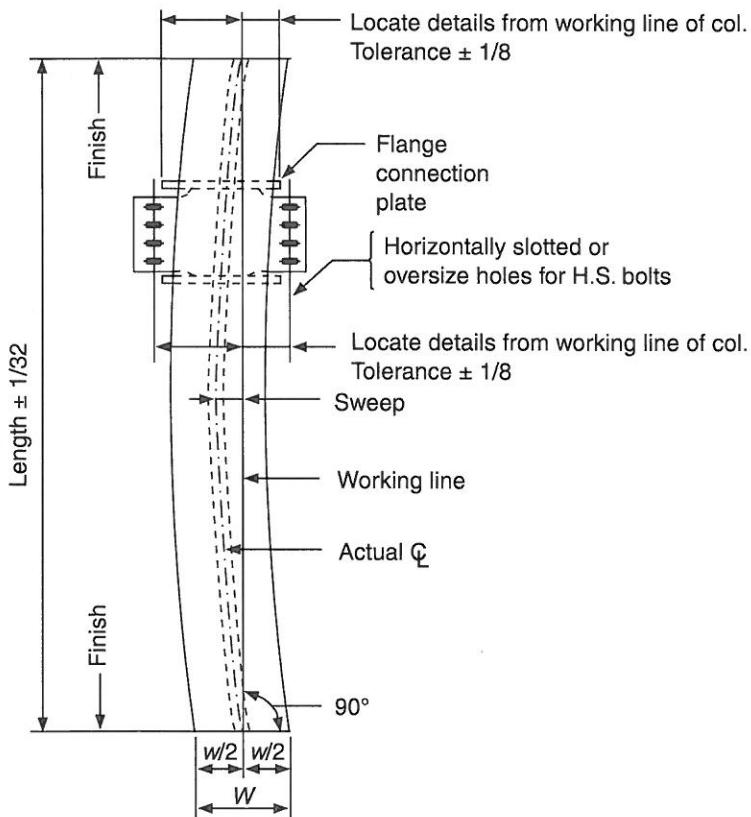


Fig. 3.5.3-4. Adjustments for column sweep in beam-to-column connections.

## CHAPTER 4

### MEMBER DESIGN

The AISC LRFD *Specification for Structural Steel Buildings* covers requirements for the design of structural steel members. This commentary includes a discussion of portions of these provisions and subsequent recommendations. Additional information on specific topics of interest can be found in the AISC Design Guide Series listed in Appendix B.

#### 4.1. AISC Specification Requirements

##### 4.1.1. *Can the AISC LRFD Specification be used to design structural shapes not listed in Part 1 of the Manual of Steel Construction?*

Yes. AISC's *Manual of Steel Construction* is written based upon the *AISC LRFD Specification for Structural Steel Buildings* and provides design aids and charts for commonly used structural shapes: those listed in ASTM A6/A6M as well as HSS (square, rectangular, and round), which are listed in their corresponding specifications. This coverage is not intended to exclude the use of other shapes that may be available. Other shapes and combinations thereof may be designed and used within the requirements of the AISC LRFD Specification.

##### 4.1.2. *Why are deflection and drift limits not specified in the AISC LRFD Specification?*

Deflection and drift are examples of serviceability criteria. Serviceability limitations are not mandated other than the requirement in AISC LRFD Specification Section L3.1 that "Deformations ... shall not impair the serviceability of the structure." Summarizing from AISC LRFD Specification Commentary Section L3.1, any specific serviceability limit would depend on the function and components of the structure. Guidance can be found in AISC Design Guide #3 *Serviceability Design Considerations for Low-Rise Buildings* (Fisher and West, 1990).

##### 4.1.3. *It is indicated in the AISC LRFD Specification Section B7 that the slenderness ratio of a tension member should preferably not exceed 300. Why? Is this requirement strength related?*

From AISC LRFD Specification Commentary Section B7, this recommended limit is "based on professional judgment and practical considerations of economics, ease of handling, and care required to minimize inadvertent damage during fabrication, transport and erection." It is further indicated that this requirement is not strength related.

Essentially, this rule also ensures that a non-pretensioned structural brace will be stiff enough that significant lateral deflection of the structure will not be required to activate the strength of the brace. As such, this requirement does not pertain to rod bracing, cable bracing and other light bracing systems that are fabricated and installed with draw to activate the brace.

- 4.1.4. *What is the difference in design philosophy between a building structure that has been designed to meet the AISC LRFD Specification for Structural Steel Buildings and a building that has been designed to meet the AISC Seismic Provisions for Structural Steel Buildings?*

A building designed to the *AISC LRFD Specification for Structural Steel Buildings* is one that possesses adequate strength to resist all design loads, primarily through nominally elastic behavior. A building designed to the *AISC Seismic Provisions for Structural Steel Buildings*, contains additional provisions for dissipating large magnitude seismic input energy through controlled inelastic deformations in discreet locations in the structure, such as through hinging of beams in moment frames, buckling of braces in concentrically braced frames, and shear (or flexural) yielding of the link in eccentrically braced frames to preclude structural collapse under high overload conditions that may occur. Obviously, a higher cost is associated with designing to the latter specification and achieving this level of ductility.

## 4.2. Requirements in Other Related Specifications and Codes

- 4.2.1. *As defined in AISC Code of Standard Practice Section 7.9, what is the difference between a self-supporting and a non-self-supporting steel frame?*

A non-self-supporting steel frame requires interaction with other elements (a precast concrete shear wall system, masonry infill, metal deck diaphragms, etc.) for stability while resisting gravity and/or lateral loads, even when completely erected. A self-supporting steel frame, once completely erected, is stable while resisting gravity and/or lateral loads. Note that structural steel and non-structural elements are as defined in *AISC Code of Standard Practice* Section 2.0; see also 2.6.3.

- 4.2.2. *When design loads are not specified in the Applicable Building Code, from where can they be determined?*

The provisions in *ANSI/ASCE 7-95 Minimum Design Loads for Buildings and Other Structures* (ASCE, 1995) cover most loads in building structures, including design live loads and lateral loads.

- 4.2.3. *To what standards are non-structural steel items such as metal stairs, handrails, and catwalks designed?*

Design force requirements for these items are commonly specified in the *Applicable Building Code*. Also, *ANSI A1264.1* is available and the *National Association of Architectural Metals Manufacturers (NAAMM)* publishes standards that cover the design of miscellaneous and certain other non-structural steel elements.

## 4.3. Stability Bracing

- 4.3.1. *What constitutes a lateral brace for a beam?*

The designer may use one or both of two general options to provide a beam brace: (1) brace the flange subject to compression directly or (2) prevent twist of the cross section. A direct brace may be provided for a beam by a properly attached floor system itself or by a secondary framing member or brace. Generally, a brace connection, such as the simple shear connection for an infill beam, that is located within the one-third depth of the beam web that is closest to the compression

flange can be considered to provide a direct brace. If this is not the case, transverse stiffening can be provided to prevent twist and transfer the bracing effectiveness from a properly attached floor system to the compression flange.

A long-standing rule of thumb is to provide bracing for two percent of the compressive force in the flange or member being braced. Although it lacks an explicit consideration of the required bracing stiffness, this approximation is typically conservative. A task committee of the AISC Committee on Specifications is currently developing specific strength and stiffness criteria for beam bracing, which will be included in a future revision of the AISC LRFD Specification. Additional information on this and other bracing topics can be found in Yura and Helwig (1995).

*4.3.2. For a continuous crane runway girder with multiple spans and cantilevered ends, what bottom-flange laterally unbraced length must be assumed in design?*

The use of continuous crane runway girders should be discouraged because they are subject to many problems and a short fatigue life (see 4.8.4). However, if such girders must be made continuous, three alternatives are available in the literature: (1) from ANSI (1981), the unbraced length should be taken as twice the cantilevered distance or the distance between the supports, whichever is greater; (2) Tanner (1985) gives an alternative and possibly less conservative solution that considers the ratio of overhanging span to the adjacent interior span in the developed equations; and, (3) for doubly symmetric cross-sections, Galambos (1988) gives cantilever unbraced lengths for use in a corresponding equation for nominal lateral torsional buckling strength.

#### **4.4. Special Design Considerations**

*4.4.1. When should notch toughness properties be specified?*

Toughness is the ability of a material to deform inelastically in the presence of flaws and under conditions of high stress concentration. A tough material will resist the initiation of a crack at points of stress concentration. Although, under certain abnormal conditions, the stress may rise locally to such levels that a crack is initiated, a tough material will better resist the propagation of the crack. The consideration of fracture mechanics principles may be necessary in some instances.

Some examples of when notch toughness should be specified include:

1. dynamic or impact loading
2. fatigue loading
3. service temperatures below 25 degrees F.

Additionally, selected welding details require notch toughness, such as welding of heavy shapes (see LRFD Specification Section A3.1c) and CJP groove welds in high-seismic systems (see 1997 AISC Seismic Provisions). The use of bolted alternatives will often relieve or eliminate the need for special notch-toughness properties.

*4.4.2. What special considerations are required for structures in low-temperature service?*

As indicated in 4.4.1, notch toughness properties should be specified.

4.4.3. *What special considerations are required for structures in elevated-temperature service?*

The reduction in yield strength as temperature is increased must be considered in design for service at elevated temperature. An excellent discussion of the effect of elevated temperature on mechanical properties as well as creep and other effects can be found in Brockenbrough and Johnston (1981).

Alternatively, the steel can be shielded from the source of heat. For example, a ¼-in. steel plate shield with a 2-in. air space is sometimes provided for crane girders that span above furnaces in mill buildings.

4.4.4. *When a beam with a web penetration requires reinforcement, what is the most efficient location for this reinforcement?*

As indicated in AISC Design Guide #2 *Steel and Composite Beams with Web Openings* (Darwin, 1990), horizontal reinforcement above and below the web penetration as illustrated in Figure 4.4.4-1 is the most efficient and effective, when reinforcement is required.

4.4.5. *AISC LRFD Specification Appendix K covers constant amplitude fatigue. What about variable amplitude fatigue loading?*

An effective stress range can be calculated using Miner's rule (Schilling and Kleppstein, 1978).

## 4.5. Composite Design

4.5.1. *What is the minimum concrete cover required above the head of a shear stud connector?*

At one time, such a requirement did exist in the AISC Specification. However, the strength of a shear stud connector depends upon the strength of the connector itself and the cone of concrete under the head, not the concrete over it. Accordingly, the cover requirement was eliminated. The reader is cautioned, however, to allow sufficient slab thickness to accommodate possible variations in composite beam shape, such as that due to camber, which could otherwise cause shear stud connectors to protrude through the top of the slab.

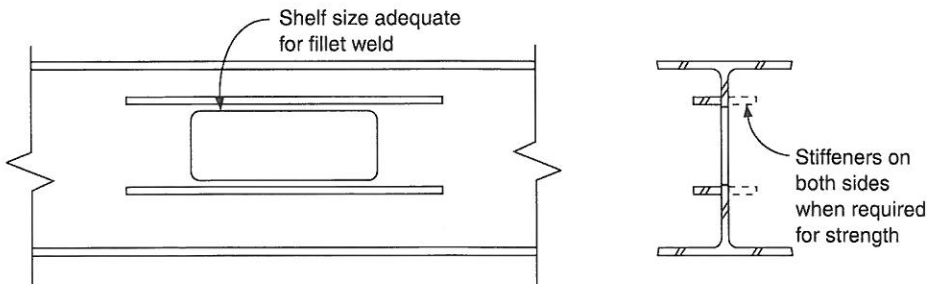


Fig. 4.4.4-1. Web penetration reinforcement.



4.5.2. *Can a shear stud connector be welded to the beam flange at any location across its width as long as there is adequate space for the weld?*

Yes, however, from AISC LRFD Specification Commentary Section I5.6, “[the diameter of] studs not located directly over the web of a beam ... is limited to [a maximum of] 2.5 times the flange thickness.”

4.5.3. *What is meant when a shear stud is said to be located in the strong or weak position?*

Modern steel deck profiles generally have a reinforcing rib at the center of the corrugation that prevents the welding of shear stud connectors directly in the center of the rib. When a stud is welded on the side of the rib that is away from the point of maximum moment, it is said to be in the strong position (see Figure 4.5.3-1) because more load can be transmitted to it. Conversely, a stud that is welded on the side of the rib that is toward the point of maximum moment is said to be in the weak position. For further information, refer to Easterling et al. (1993).

4.5.4. *What is the impact on design strength of shored vs. unshored construction in composite beam design?*

Research has shown that the ultimate strength of a composite beam is unaffected by the construction (shored or unshored). The ultimate flexural strength of the cross-section is unaffected by the initial load present because the applicable limit states in AISC LRFD Specification Chapter I occur in the inelastic range (i.e., after stress is no longer proportional to strain). The acceptability of unshored construction then is entirely a question of serviceability and constructability.

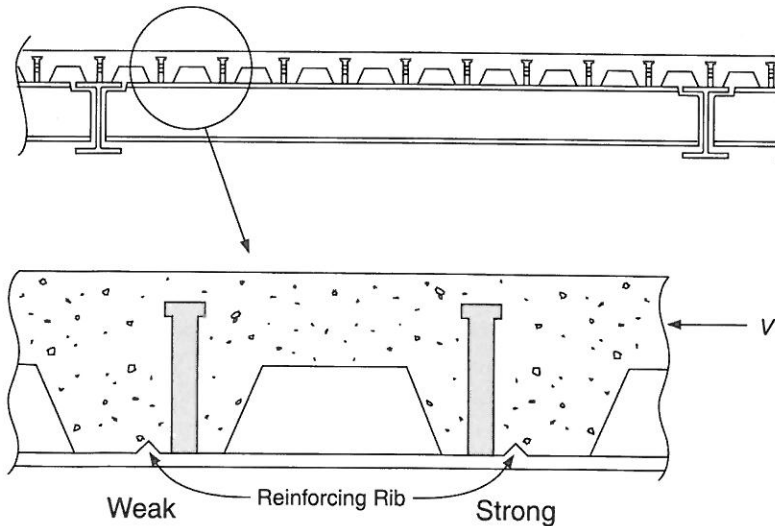


Fig. 4.5.3-1. Shear stud connectors in the strong and weak positions.

- 4.5.5. *Can an existing non-composite steel beam and concrete slab be made to act compositely by adding shear stud connectors through cored holes in the slab?*

Yes. Cored holes must be of a diameter that is sufficient to allow stud placement and grouting. A shrink-compensating cementitious grout with strength at least equal to that of the existing slab should be used. As stated in 4.5.4, the design strength of the resulting composite beam is independent of the initial load present.

#### 4.6. Built-up Member Design

- 4.6.1. *In a built-up I-shaped cross-section, how are welds connecting the plates designed?*

Assuming that continuous fillet welds are used, the welds may be minimum size per AISC LRFD Specification Table J2.4 (fillet welds) if the member is subjected only to axial compression or tension. If the member is subjected to flexure, the shear flow (kips/in.) can be calculated from the beam shear  $V_u$  as  $V_u Q / I$  and the weld sized to provide for this required strength;  $Q$  is the first moment about the neutral axis of the flange area and  $I$  is the moment of inertia of the entire cross-section.

- 4.6.2. *Can built-up sections be designed and fabricated with elements of different grades of steel, such as ASTM A572 grade 50 flanges and an ASTM A36 web?*

Yes. This common practice results in a member that is known as a hybrid girder.

- 4.6.3. *What is tension-field action?*

Tension-field action is the post-buckling development of diagonal tensile stresses in slender plate-girder web panels and compressive forces in the transverse stiffeners that border those panels. When tension-field action is considered in design, AISC LRFD Specification Appendix G3 provisions apply; otherwise AISC LRFD Specification Section F2 is used.

#### 4.7. Design of Angles

- 4.7.1. *When a pair of angles is noted SLBB or LLBB, what does this mean?*

SLBB stands for "short legs back-to-back." Alternatively, LLBB stands for "long legs back-to-back."

- 4.7.2. *When a single angle is noted SLV or LLV, what does this mean?*

SLV stands for "short leg vertical." Alternatively, LLV stands for "long leg vertical."

- 4.7.3. *How is a masonry shelf-angle designed?*

From Tide and Krogstad (1993), "Factors such as deformation of lower courses of masonry during construction and the rigidity of masonry walls after the mortar begins to set cause shelf angles to be loaded by a combination of uniform and concentrated loads acting near the back edge of the masonry. Restraint provided by friction and the masonry ties reduces torsional forces and deflection and provides lateral support for the shelf angles." A design procedure that accounts for these factors can also be found in Tide and Krogstad (1993).

#### 4.8. Other General Information

4.8.1. *When a stiffening element is attached to a plate, what width of plate is effective in resisting load with the stiffening element?*

A general rule of thumb that is used in American Petroleum Institute (API) and American Iron and Steel Institute (AISI) standards for liquid storage tanks is that 16 times the plate thickness is effective on either side of a stiffening element, but not greater than:

1. The actual projecting width.
2. One-half the distance to the next stiffening element.
3. One-quarter of the stiffener span.
4. The width based upon the limiting width-thickness ratios in AISC LRFD Specification Table B5.1 for compression elements.

4.8.2. *How are stepped columns designed?*

This subject is addressed in AISC Design Guide #7 *Industrial Buildings—Roofs to Column Anchorage* (Fisher, 1993; page 67) and Anderson and Woodward (1972).

4.8.3. *Should weep holes be used at the bottoms of HSS columns?*

If the HSS assembly is fabricated as an airtight enclosure, weep holes need not be provided because any moisture in the contained air will quickly be used and corrosion cannot progress. When non-airtight HSS columns are exposed to the weather or to temperature changes that can cause interior condensation, weep holes should be provided. If, however, a column is protected from the elements and is neither subject to drastic temperature change nor a humid environment, weep holes may not be necessary.

4.8.4. *What special considerations should be noted in crane-girder design?*

The following special design considerations should be noted:

1. Adequate vertical and lateral stiffness are of primary importance.
2. Simple-span construction should be used.
3. The cross-section should be proportioned on the basis of its elastic flexural strength.
4. Lateral stability should be provided at the bearing ends without inhibiting end rotations.
5. Vertical and lateral impact loads should be considered.
6. Lateral forces due to trolley acceleration and braking, runway misalignment, crane skew, and other sources should be considered.
7. Fatigue should be considered.
8. The class of service to which the crane will be subject should be considered.
9. The local strength of the web under crane wheel loads should be checked.
10. Biaxial bending of the top flange should be checked.
11. Bottom-flange bracing should be provided when required; bottom-flange bracing is recommended for spans over 36 ft. in AISE (1991).
12. For built-up crane-girders, the weld between the top flange and the web should be a CJP groove weld.
13. Suitable provision should be made for the crane-rail attachment system.

14. Suitable provision should be made for the electrification of the crane system.

Further information is available in the following references: AISE (1991), Fisher (1993), and Ricker (1982).

## CHAPTER 5

### CONNECTIONS

The AISC LRFD *Specification for Structural Steel Buildings* covers requirements for the design of structural steel connections. Additional recommendations can be found in the AISC LRFD Manual, Volume II-Connections. This commentary includes a discussion of portions of these provisions and subsequent recommendations with regard to general issues in connection design, fabrication, and erection. For bolting- and welding-specific issues, refer to Chapters 6 and 8, respectively.

#### 5.1. Bolt Holes

5.1.1. *Maximum hole sizes for bolts are specified in LRFD Specification Table J3.3. What if an actual hole dimension is between two of the values?*

AISC LRFD Specification Table J3.3 is based upon RCSC Specification Table 1 and contains the maximum dimensions of standard, oversized, short-slotted, and long-slotted holes. If an actual dimension exceeds the tabulated maximum, it must be treated as the next larger hole size. For example, a  $1\frac{1}{16}$ -in. by  $1\frac{1}{4}$ -in. slotted hole for a  $\frac{3}{4}$ -in. diameter bolt must be treated as a long-slotted hole because it exceeds the maximum short-slotted hole size ( $1\frac{1}{16}$ -in. by 1 in.). Note that RCSC Specification Section 3(c) allows a  $\frac{1}{32}$ -in. tolerance on these maximum hole sizes as discussed in 2.4.2 and 2.5.5.

5.1.2. *Alternatives are provided in AISC LRFD Specification Section J3.10 for the calculation of bearing strength at bolt holes with deformation considered or not considered. What is the philosophical difference between these options?*

When deformation is a design consideration, the design strength is limited to the force at which the hole edge has deformed by a maximum of  $\frac{1}{4}$ -in. When deformation is not a design consideration, larger hole ovalization is permitted as the material attains its maximum bearing strength.

5.1.3. *Does flame-cutting of bolt holes affect connection strength and performance?*

Generally, no. Iwankiw and Schlaflly (1982) investigated the performance of double-lap joints with holes made by punching, punching with burrs removed, sub-punching and reaming, drilling, flame-cutting, and flame-cutting and reaming. The comparison of 18 samples using  $\frac{1}{2}$ -in.-thick ASTM A36 steel plates with standard holes indicated that there is no significant variation in connection strength according to the method of hole formation under static load. Additional considerations may be warranted for much thicker plates, steel grades other than those tested, and Cyclically Loaded Structures.

## 5.2. Single-plate Connections

5.2.1. *In the design procedure for single-plate connections in the AISC LRFD Manual, the plate thickness is limited to a maximum of  $\frac{1}{16}$ -in. larger than half the bolt diameter and the fillet weld size is required to be three-quarters of the plate thickness. What does this accomplish?*

Generally, it results in a plate that is the critical element of the connection. Bolt bearing deformation will occur before bolt shear and plate yielding will occur before weld rupture; thereby, a ductile limit-state will control the strength of the connection.

5.2.2. *Are through-plates always required for single-plate connections to HSS columns?*

No. Sherman and Ales (1991) demonstrated that local yielding of the support was not a concern due to the self-limiting nature of simple-shear connection end rotation and that the compressive strength of the HSS column was unaffected by the associated local deformations. However, this same research indicated that punching shear may be of concern for relatively thin supporting material thicknesses. Punching shear can be prevented by selecting an HSS with a wall thickness  $t_w$  that meets the following criteria:

$$t_w \geq \frac{(F_y t)_{pl}}{F_{tw}} \quad (5.1)$$

where

$F_{ypl}$  = yield strength of the single plate

$t_{pl}$  = thickness of the single plate

$F_{tw}$  = tensile strength of the HSS wall

Note that this equation differs slightly from that given in Sherman and Ales (1991). Here, the expression is derived at the design strength level ( $\phi$  factors included) whereas it was previously derived at the nominal strength level (no  $\phi$  factors).

If the actual maximum stress is known, it can be substituted for  $F_{ypl}$  in the above equation for a less conservative result.

The above minimum thicknesses would also be applicable to a welded plate tension connection (uniform stress distribution assumed). However, for cantilevered bracket connections, which do not have self-limiting rotations; yielding must also be checked.

## 5.3. Other General Information

5.3.1. *In pin-connected plates, it is required in the AISC LRFD Specification that the net area beyond the pin hole be greater than or equal to the net area required for strength across the pin hole. Why?*

As stated in AISC LRFD Specification Section D3, "In pin-connected plates other than eyebars, the minimum net area beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than two-thirds of the net area required for strength across the pin hole." As illustrated in Figure 5.3.1-1, this ensures that the end distance  $a$  will be adequate to prevent a tearout of the pin through the end of the plate.

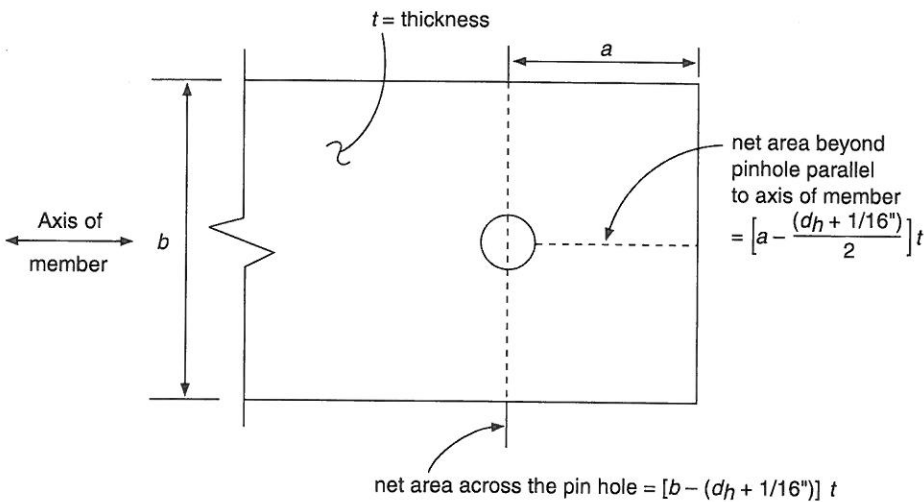
5.3.2. *Are shop assembly requirements, such as sub-punching and reaming or reaming to a template, necessary in contract documents?*

The use of modern punching and drilling equipment consistently produces and duplicates hole patterns with excellent dimensional accuracy. Some specifications fail to recognize this capability and still require that matching hole patterns be produced by drilling or reaming through a steel template.

In lieu of a template or assembly drilling or reaming, the fabricator should be allowed to demonstrate the capability to fabricate component structural members to the tolerance and accuracy specified so that further shop assembly to assure proper fit can be eliminated. In some cases, however, such as large trusses or plate girders, shop assembly may be advisable to reduce the occurrence of field fit-up problems. In any case, responsibility for final fit inevitably rests with the fabricator.

5.3.3. *How much of a joint must be in contact to be considered to be in full contact?*

Projecting elements of bolted connection attachments, such as clip-angles or end-plates, often are not flat in the plane of the connection because of profile variations due to as-rolled mill tolerances or welding distortions. In double-angle connections, for example, the outstanding legs tend to bend back toward the



From LRFD Specification Section D3:

$$\left[ a - \frac{(d_h + 1/16")}{2} \right] t \geq \frac{2}{3} [b - (d_h + 1/16")] t$$

$$a \geq \frac{2}{3} b - \frac{1}{6} (d_h + 1/16")$$

Fig. 5.3.1-1. Pin-connected plates.

centerline of the span. Any resulting gaps are usually drawn together when the bolts are installed, except in relatively thick material.

In bearing connections, this is of little concern. In slip-critical connections, the full slip resistance of the connection will be developed regardless of the initial position of such projecting elements if the following conditions are met:

1. Some part of the connection is in contact with the support before the bolts are tensioned.
2. The bolts are subsequently tensioned in accordance with the RCSC Specification.
3. The faying surfaces are drawn into contact at the bolts within the area of the bolt head or nut as illustrated in Figure 5.3.3-1.

Accordingly, it is stated in AISC *Code of Standard Practice* Section 6.3.1 that “projecting elements of connection attachments need not be straightened in the connecting plane if it can be demonstrated that installation of the connectors or fitting aids will provide reasonable contact between faying surfaces.”

5.3.4. *What lamellar tearing considerations are required for through-thickness loading?*

Lamellar tearing concerns affect material, design, fabrication, testing and inspection, and erection in conjunction with welded details. When material is loaded in the through-thickness direction and the strength of the component is dependent upon soundness of the material in this direction, understanding of and attention to the special problems involved are required. Lack of such understanding and attention on the part of the designer may make the task of the fabricator impossible. Conversely, lack of attention and substandard welding workmanship may result in difficulties with good designs and details.

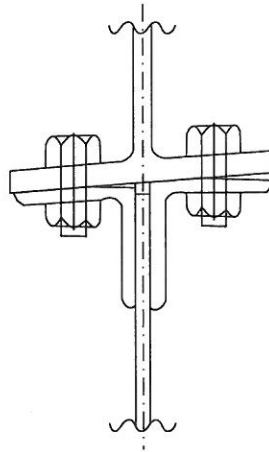


Figure 5.3.3-1.



From AISC LRFD Manual page 1-8, "It should be noted that elongation and area reduction values may well be significantly lower in the through-thickness direction than in the planar direction. This inherent directionality is of small consequence in many applications, but does become important in the design and fabrication of structures containing massive members with highly restrained welded joints. With the increasing trend toward heavy welded-plate construction, there has been a broader recognition of the occurrence of lamellar tearing in some highly restrained joints of welded structures, especially those using thick plates and heavy structural shapes. The restraint induced by some joint designs in resisting weld deposit shrinkage can impose tensile strain sufficiently high to cause separation or tearing on planes parallel to the rolled surface of the structural member being joined. The incidence of this phenomenon can be reduced or eliminated through greater understanding by designers, detailers, and fabricators of: (1) the inherent directionality of construction grades of steel, (2) the high restraint developed in certain types of connections, and (3) the need to adopt appropriate weld details and welding procedures with proper weld metal for through-thickness connections. Further, steels can be specified to be produced by special practices and/or processes to enhance through-thickness ductility and thus assist in reducing the incidence of lamellar tearing. Steels produced by such practices are available from several producers. However, unless precautions are taken in both design and fabrication, lamellar tearing may still occur in thick plates and heavy shapes of such steels at restrained through-thickness connections. Some guidelines in minimizing potential problems have been developed (AISC, 1973). See also [AISC LRFD Manual page 8-113 and ASTM A770]."

5.3.5. *What is shear lag and when must it be considered?*

Shear lag describes behavior at an end connection of a tension member where some but not all of the cross-sectional elements are connected; the area that is effective in resisting tension may be less than the full calculated net area. Procedures for treatment of shear lag and determination of the effective net area in bolted and welded connections are provided in AISC LRFD Specification Section B3. Alternatively, shear lag concerns can be addressed by selecting a connection length that mobilizes the entire load-transmitting capability.

5.3.6. *What column stiffening requirements apply at beam-to-column-flange moment connections?*

Column stiffening requirements are covered in AISC LRFD Specification Section K1 for concentrated flange forces and panel zone shear. Additionally, column stiffening is covered in the AISC LRFD Manual (pages 10-35 through 10-55). Generally, the use of larger columns to eliminate column stiffening, particularly web doubler plates, is recommended. For seismic applications, see the 1997 AISC Seismic Provisions.

5.3.7. *In many design examples in the 2nd Edition LRFD Manual of Steel Construction, yielding and buckling in a gusset plate or similar fitting are checked on a Whitmore section. What is a Whitmore section?*

A Whitmore section identifies a theoretically effective cross-sectional area at the end of a connection resisting tension or compression, such as that from a brace-to-gusset-plate connection or similar fitting. As illustrated in Figure 5.3.7-1

for a WT hanger connection, the effective length for the Whitmore section  $L_w$  is determined using a spread-out angle of 30 degrees along both sides of the connection, beginning at the start of the connection. It is applicable to both welded and bolted connections.

5.3.8. *How can adequate flexibility be maintained in double-angle simple shear connections subjected to combined shear and tension load?*

As the tensile force component increases in a double-angle shear connection subjected to combined shear and tension, prying action and/or bending require that the fitting thickness be increased or the bolt gage be decreased, thereby decreasing the available flexibility. Thornton (1995) assesses the ductility of bolts in the outstanding legs of double-angle and similar simple-shear connections. This study validates the long-standing AISC Manual recommendation that maximum angle thickness be limited to  $\frac{5}{8}$ -in. for usual gages (4½-in. to 6½-in.) in double-angle simple-shear connections. For welded connections, a parallel examination can be made as illustrated in Thornton (1996). It should be noted that an alternative connection detail, such as a single-plate connection, may be more feasible for shear-tension applications.

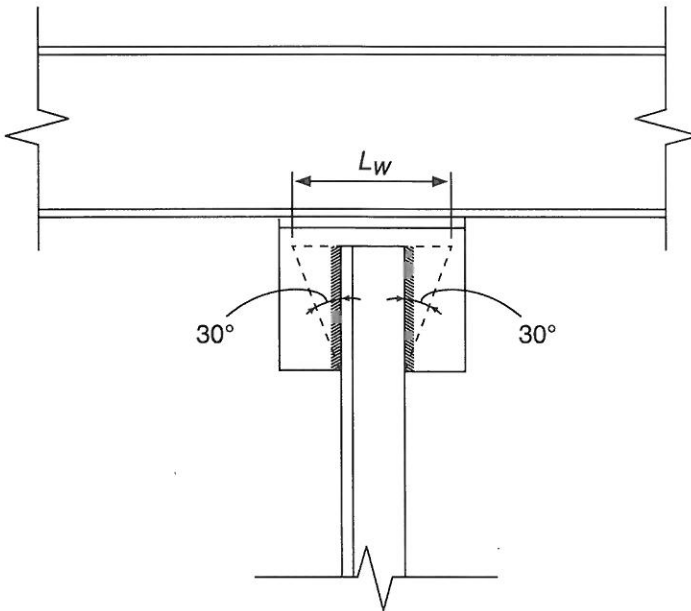


Fig. 5.3.7-1. Whitmore section.

## CHAPTER 6

### BOLTING

The AISC LRFD *Specification for Structural Steel Buildings* and the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts* cover requirements for the use of bolts in structural steel connections. This commentary includes a discussion of portions of these provisions and subsequent recommendations. Some of the discussion is taken from *Bolt Bulletins* published by RCSC.

#### 6.1. Economical Suggestions

##### 6.1.1. *Why should bolt diameters be limited to a 1-in. maximum when possible?*

The industry standard 3/4-in., 7/8-in., and 1-in. bolt diameters provide adequate design strength for the vast majority of connections in steel structures. Accordingly, commonly available bolt installation equipment has been designed with a capacity to fully tension 1-in. diameter ASTM A490 bolts, when required; larger bolts will usually require special equipment and/or effort. Additionally, bolt diameters larger than 1 in. require larger clearances, edge distances, and spacings than are standard. Therefore, bolt diameters larger than 1 in. should be avoided, when possible, to avoid potential bolt tensioning difficulties.

##### 6.1.2. *Why should mixed use of ASTM A325 and A490 bolt grades be limited to different diameters?*

If ASTM A325 and A490 bolts of the same nominal diameter are mixed on a project, there exists the possibility that the A325 bolts might be installed where the A490 bolts were required. Therefore, when ASTM A325 and A490 bolts are used on the same project, quality assurance is simplified if different diameters are used for different grades.

#### 6.2. Ordering Bolts

##### 6.2.1. *What quality requirements must high-strength bolts, nuts, and washers meet?*

The manufacturing quality requirements for high-strength bolts, nuts, and washers are covered in the following specifications: for high-strength bolts, ASTM A325/A325M and ASTM A490/A490M; for nuts, ASTM A563/A563M or ASTM A194/A194M; for washers, ASTM F436/F436M; and for direct tension indicators, ASTM F959/F959M. These fasteners should be specified and accepted based upon the criteria established therein.

##### 6.2.2. *What information must be included in the purchase order for high-strength bolts?*

From RCSC Bulletin No. 3, the purchase order for high-strength bolts must include the ASTM grade (A325 or A490), the type (1 or 3), a copy of the project specification for the manufacturer or vendor, and the "Ordering Information" as required by ASTM, Volume 15.08 *Fasteners*, pages 56 and 98. Additionally, the purchase order should require the following:

1. That the vendor provide certification that the bolts, nuts, and washers furnished conform to all requirements of the referenced ASTM specifications.
2. That certified manufacturer's mill test reports be supplied that clearly show the applicable ASTM mechanical and chemical requirements together with the actual test results for the lot of supplied fasteners.
3. That the bolt heads and the nuts of the supplied fasteners be marked with the manufacturer's identification mark and the ASTM grade and type as specified in ASTM specifications.
4. For projects requiring slip-critical connections, that the lubricated bolt, nut, and washer be pre-assembled to ensure proper fit of the bolt and nut and that the lot be tested for strength prior to shipment to the purchaser to meet the requirements of RCSC Specification Table 4.

6.2.3. *When must high-strength bolts be ordered as a bolt/nut assembly from a single manufacturer?*

As indicated in the RCSC Specification Commentary, there are two cases in which bolts and nuts must be treated as a manufactured matched assembly: when bolts are galvanized (Section C2) and when "tension-control" bolts are specified (Section C8). In the former case, because nut-thread over-tapping to accommodate the added thickness of galvanizing may reduce the nut stripping strength, ASTM A325 requires that the galvanized assembly be lubricated and tested by the manufacturer to ensure adequate rotational capacity. In the latter case, some of the negative aspects of this torque-controlled installation method are minimized through good quality control in the matched assembly.

6.2.4. *Is it acceptable to substitute ASTM A449 bolts for ASTM A325 bolts?*

In general, no. While ASTM A449 seems to offer the same strength as ASTM A325, the use of A449 material is restricted in AISC LRFD Specification Section A3.3 to bolt diameters larger than 1½-in. for non-slip-critical connections. This is because, A449 bolts are not produced to the same inspection and quality assurance requirements as ASTM A325 bolts. Also, A449 bolts are not produced to the same heavy-hex head dimensions as are A325 bolts.

6.2.5. *Is it acceptable to substitute SAE J429 grades 5 and 8 bolts for ASTM A325 and A490 bolts, respectively?*

No. The strength properties of SAE J429 grade 5 bolts and ASTM A325 bolts are identical; likewise, SAE J429 grade 8 bolts are the strength equivalent of ASTM A490 bolts. These material specifications differ, however, in that ASTM A325 and A490 specify thread length and head size, whereas SAE J429 does not. Additionally, quality assurance and inspection requirements for ASTM A325 and A490 bolts are more stringent.

6.2.6. *What is an ASTM A325T bolt?*

The "T" in the designation ASTM A325T invokes supplement S1 in ASTM A325, which allows for full-length threading (ASTM A325T). This provision may be specified for ASTM A325 bolts of length less than or equal to four times the bolt diameter only; there is no similar provision in ASTM A490. The fully threaded bolt allows the fabricator the option to use a single-length fastener in the majority

of bolting applications, if desired. Note that if ASTM A325T bolts are specified, it is impossible to exclude the threads from the shear plane and the design must be based upon the “threads included” strength values.

### 6.3. Verification Testing

#### 6.3.1. *What constitutes evidence of material conformity for high-strength bolts, washers, and nuts?*

In its Bulletin No. 1, RCSC recommends that the supplier’s certification that bolts, nuts, and washers furnished comply with all of the appropriate requirements of the applicable specifications, and that complete manufacturers mill test reports (manufacturers inspection certificate) be accepted as evidence of conformity. Certification numbers must appear on the product containers and correspond to the identification numbers on the mill test reports. Additionally, the manufacturers symbol and grade markings must appear on all bolts and nuts.

#### 6.3.2. *What testing should be performed on high-strength bolts upon delivery from the manufacturer?*

To ensure that the delivered fasteners comply with the purchase requirements and furnished documents, upon receipt, the purchaser should verify that:

1. The bolts and nuts are marked as specified.
2. The manufacturer’s mill test report shows a chemistry that meets the requirements for the type of bolts and nuts specified.
3. Certification numbers appear on the product containers and correspond to the certification numbers on the mill test reports for the fasteners.
4. Mill test reports are supplied to both the purchaser and the testing laboratory responsible for quality control.
5. If the fasteners are galvanized, rotational capacity test results and nut lubrication should be verified to be in compliance with the appropriate ASTM specification requirements.

A bolt tension calibration device (see 6.9) should be available in the shop and at the jobsite at the beginning of bolting start-up. From RCSC Bulletin No. 1, regardless of installation requirements for the project, whenever high-strength bolts are to be installed, not less than three bolt, nut, and washer assemblies from each lot supplied should be tested in a tension measuring device. Such testing must demonstrate that the bolts and nuts, when used together, can develop tension not less than that provided in RCSC Specification Table 4. The bolt tension should be developed by tightening the nut. A representative of the manufacturer or supplier should be present, if required by the SER, to assure that the fasteners are properly used, and to demonstrate that the fastener assemblies supplied satisfy the specification requirements. Additionally, the inspector should be present.

#### 6.3.3. *Why must the purchaser test high-strength bolts if the manufacturer has already done so?*

ASTM Specifications are adequate and appropriate for the strength and quality of the separate components, but may not be representative of the factors and conditions that determine performance of the fastener assembly during installation and service as it is used in construction. For one example, the bolt itself is

tested for strength by the manufacturer by screwing the bolt into a standard testing fixture and subjecting it to pure tension. However, in usual applications, bolt tension is induced by torque on the nut and the bolt is subject to combined tension and torque, which may cause the bolt to fail at a load less than its strength in pure tension. Testing is performed to ensure that the highly variable torque component of the combined stress is not so large that, for bolts required to be fully tensioned, it prevents the development of the minimum tension specified in RCSC Specification Table 4.

#### **6.4. Handling and Storage**

##### **6.4.1. *Should bolts and nuts be cleaned of all grease, wax or other lubricant prior to installation?***

No. Bolts are intentionally lubricated to facilitate installation. Accordingly, it is stated in RCSC Specification Section 8(a) that "Fasteners shall not be cleaned of lubricant that is present in [the] as-delivered condition." Note however that provision is also made in this section for fasteners that accumulate rust or dirt resulting from job site conditions, which, if used, "...shall be cleaned and [properly] relubricated prior to installation."

##### **6.4.2. *What storage requirements apply to high-strength bolts, nuts, and washers?***

All fastener components must be stored in a manner that affords complete protection from moisture, heat, and dirt contamination. These precautions are necessary to avoid corrosion, loss of lubricant effectiveness, and dirt contamination that will both alter the required installation torque and increase the corresponding scatter of installed tension.

Each day, upon removal from storage, each bucket of fasteners should be visually inspected for corrosion, lubricant condition, and dirt contamination; any fastener found to be corroded, lacking lubrication, or dirty is unacceptable for installation, but may be cleaned, re-lubricated with an approved lubricant, and re-tested. Such fastener assemblies that cannot be demonstrated to have an installed tension that is 5 percent greater than the minimum tension required in RCSC Specification Table 4 should be deemed unacceptable for use.

Only the number of fasteners that are required for work to be done that day should be removed from storage. At the end of the work day, all fasteners that are not installed should be returned to storage.

#### **6.5. Bolt Installation**

##### **6.5.1. *What can be done to prevent the nut from loosening?***

In general, when properly installed, the high-strength bolt-nut assembly will not loosen. When snug-tight bolts are used, the loading will be such that loosening of a nut will not occur. When fully tensioned bolts are required, as for slip-critical connections subjected to vibratory or fatigue loading, the installed tension and the attendant friction on the threads will prevent the nut from loosening.

In some other cases, such as nuts on anchor rods (for which full-tensioning is generally inappropriate), further consideration may be required. In such cases, an additional jamb-nut or second nut may be provided. Alternatively, the threads can be spiked or marred or the nut can be tack-welded to the base metal to prevent it

from turning. Note that the latter two solutions are permanent actions. There also exist proprietary nut devices with locking features to prevent the nut from backing off.

6.5.2. *What is the definition of snug-tight bolt installation and when is it allowed?*

RCSC defines snug-tight as “the [bolt] tightness that exists when all plies of a joint are in firm contact.” Note that no specific level of installed tension is required to achieve this condition, which is commonly attained after a few impacts of an impact wrench or the full effort of an ironworker with an ordinary spud wrench. It is a simple analogy to say that a snug-tight bolt is installed in much the same manner as the lug nut on the wheel of a car; each nut is turned to refusal and the pattern is cycled and repeated so that all fasteners are snug. Essentially, snug-tight bolts utilize the higher shear/bearing strength of high-strength bolts with installation procedures similar to those used for ASTM A307 common bolts, which are never fully tensioned (see 6.6.2).

6.5.3. *When must bolts be fully tensioned?*

Snug-tight high-strength bolts are permitted for all bearing joints except when fully tensioned (bearing or slip-critical) bolts are required per AISC LRFD Specification Section J1.11 and RCSC Specification Sections 5 through 8. For example, high-strength bolts must be fully tensioned for:

1. Slip-critical connections (see 6.5.4).
2. Connections where the bolts are subject to direct tension loading.
3. Column splice connections in tier structures that are 200 ft or more in height, 100 to 200 ft in height if the least horizontal dimension is less than 40 percent of the height, or less than 100 ft in height if the least horizontal dimension is less than 25 percent of the height.
4. In structures over 125 ft in height, connections of beams and girders to columns and of any other beams and girders upon which the bracing of columns is dependent.
5. In structures carrying cranes over five ton capacity, roof-truss splices and connections of trusses to columns, columns splices, column bracing, knee braces, and crane supports.
6. Connections for supports of running machinery or of other live loads that produce impact or stress reversal.
7. Other connections stipulated as fully tensioned on the design plans.

6.5.4. *When should bolted connections be specified as slip-critical?*

Slip in bolted connections is not a structural concern for the majority of connections in steel building structures. RCSC Specification Commentary Section C4 states that “The maximum amount of slip that can occur in [any] connection ... is, theoretically, an amount equal to two hole clearances. In practical terms, it is observed to be much less than this. In laboratory tests it is usually about one-half a hole clearance. This is because the acceptable inaccuracies in the location of holes within a pattern of bolts would usually cause one or more bolts to be in bearing in the initial unloaded condition. Furthermore, ... even with perfectly positioned holes, the usual method of erection would cause the weight of the connected elements to put the bolts into direct bearing at the time the member is

supported on loose bolts and the lifting crane is unhooked. Subsequent gravity loading could not cause additional connection slip.”

In some cases, slip resistance is required. The AISC and RCSC Specifications list cases where connections must be designated by the SER as slip-critical:

1. Connections with oversized holes
2. Connections with slotted holes when the direction of the slot is not perpendicular to the direction of the load, unless slip is the intended function of the joint.
3. Connections subject to fatigue or significant load reversal.
4. Connections in which welds and bolts share in transmitting shear loads at a common faying surface.
5. Other connections stipulated as such on the design plans (i.e., From RCSC Specification Commentary Section C4, “... those cases where slip could theoretically exceed an amount deemed by the Engineer of Record to affect the suitability for service of the structure by excessive distortion or reduction in strength or stability ...”).

One special case also exists. A nominal amount of slip resistance is required at the end connections of bolted built-up compression members so that the individual components will act as a unit in column buckling. As specified in AISC LRFD Specification Section E4, “the end connection shall be ... fully tensioned bolted with clean mill scale or blast-cleaned faying surfaces with Class A coatings.” In other words, the end connection can be proportioned as a bearing connection as long as the faying surfaces offer at least a Class A slip coefficient.

## 6.6. Methods for Fully Tensioned Installation

### 6.6.1. *What torque is required to fully tension a high-strength bolt?*

Torque is an invalid measure for fully tensioned installation, unless it is calibrated. In 1951, the first RCSC Specification incorporated a table of standard torque values for the installation of fully tensioned high-strength bolts. However, depending upon the condition of the threads, it was demonstrated that the resulting installed tension varied by as much as plus or minus 40 percent. It is now known that clean, well lubricated threads result in tensions that are higher than required (and probably a few broken bolts), whereas, rusted, dirty, or poorly lubricated threads result in tensions that are below the minimum required. Therefore, recognition of these standard torque values has long been withdrawn. Accepted procedures for fully tensioning high-strength bolts can be found in RCSC Specification Section 8 (see also 6.6.3.). If torque is to be used as in the calibrated wrench method as described in RCSC Specification Section 8(d)(2), it must be calibrated on a daily basis for the lot, diameter, and condition of bolts being installed.

### 6.6.2. *Can an ASTM A307 bolt be fully tensioned?*

No. ASTM A307 is the bolting strength equivalent of the ASTM A36 steel specification. As such, it is a mild steel material that is suitable only for use in snug-tight bearing connections. Note that ASTM A307 bolts are seldom used in



structural connections today, except perhaps for the end connections of purlins and girts, incidental sub-framing, and as anchor rods.

6.6.3. *What are the accepted procedures for fully tensioning high-strength bolts?*

Provisions in RCSC Specification Section 8 include four methods for the installation of high-strength bolts in fully tensioned bearing and slip-critical connections: turn-of-nut method, calibrated wrench method, alternative design bolt method, and direct tension indicator method. When used properly, each method can produce properly tensioned high-strength bolts. The RCSC Specification gives the complete requirements for each method. The use of these methods is further described in 6.6.4 through 6.6.7.

6.6.4. *How should the turn-of-nut method be used?*

At the start of work, the bolt installation crew should demonstrate in a bolt tension calibration device that the routine installation procedure properly establishes both the snug-tight condition and the subsequent rotation required to fully tension the bolts.

Installation should commence at the tightest part of the joint and progress toward the free edges, both in the initial stage of snug-tightening and the final stage of fully tensioning; several cycles may be needed. Bolt tensioning to the proper final tension may be accomplished with greatest assurance by match-marking the outer face of the nut with the protruding bolt point after the initial snug-tightening operation.

The inspector should observe the testing of representative samples and verification demonstrations of the method to be used, as well as monitor actual work to assure that the proven method is routinely followed and that all bolts are properly tensioned.

6.6.5. *How should the calibrated wrench method be used?*

Hardened washers must be used under the element to be turned in tightening (to control the frictional resistance).

At least once each working day, using the bolt tension calibration device and representative samples of the fasteners to be installed that day, the erector should calibrate the wrenches and demonstrate that the installation procedure and equipment being used achieves a tension that is at least 5 percent higher than that specified in RCSC Specification Table 4. Standard torque values determined by formula, table, or other uncalibrated means are invalid and should not be used. The installation procedure should be revalidated or recalibrated any time a component of the installation process, such as the operator, impact wrench, compressor, or fastener component combination, is changed.

Installation should commence at the tightest part of the joint and progress toward the free edges, both in the initial stage of snug-tightening and the final stage of fully tensioning; several cycles may be needed.

The inspector should observe the calibration of wrenches and/or required installation torque, as appropriate, for the fastener assemblies to be used and use the torques so determined to check the tightening of bolts in the work and to assure that all bolts are properly tensioned.

6.6.6. *How should the alternative design bolt method be used?*

The sheared-off splined end of an individual bolt indicates only that, at the time the splined end was torqued off, enough torque had been applied to the bolt to fracture the break-neck. Proper tension is assured for all bolts in a connection only if the bolts have been systematically snug-tightened and subsequently fully tensioned as specified.

Installation should commence at the tightest part of the joint and progress toward the free edges, both in the initial stage of snug-tightening and the final stage of fully tensioning (fracturing the break-neck); several cycles may be needed in the snugging operation. Installation to the full tension and shearing of the break-neck should not be accomplished in a single, continuous operation, especially in large joints.

The greatest assurance of properly installed and tensioned bolts will result if the inspector monitors work in progress to assure that all bolts are initially properly snug-tightened and subsequently properly fully tensioned, i.e., progressing from the most rigid part of the connection to the free edges before the break-neck is sheared. Note that specific and proper lubrication of “tension-control” or “twist-off” bolts is essential to the reliable installation of these fasteners.

6.6.7. *How should the direct-tension-indicator method be used?*

Strict adherence to the manufacturers installation instructions is required with direct-tension indicators (DTI).

At the start of work, representative samples should be checked in a bolt tension calibration device to demonstrate that the DTI's supplied are within the installed tolerances specified in ASTM F959 Table 3. The presence of a DTI with protrusions that are compressed to the specified gap merely indicates that at some time an adequate load was applied.

Tightening of individual bolts with DTI's to final gap should not be accomplished in a single continuous operation. First, all fasteners should be tightened so as to compress the DTI protrusions to approximately twice the specified final gap. The subsequent stage of full tensioning should progress from the most rigid part of the connection to the free edges until the calibrated torque is reached for all bolts in the group; several cycles may be needed.

The greatest assurance of properly installed and tensioned bolts will result if the inspector observes work in progress to assure that all bolts are initially properly snug-tightened and subsequently properly fully tensioned.

6.6.8. *What is the upper limit on the installed tension of high-strength bolts?*

As stated in RCSC Specification Section 9(a), installed tensions in excess of those given in RCSC Specification Table 4 shall not be cause for rejection. Accordingly, there is no specified upper limit on the installed tension of high-strength bolts. This supports the long-standing rule of thumb that as long as the bolt is not broken during installation, the bolt is adequate for service.

This general rule applies because the bolt is subjected to combined stress (tension and torque) during installation. Once installed, however, the torque is relaxed and the bolt is essentially subject only to a tensile stress that is always less than the combined stress. Thus, even if the bolt were on the verge of failure

during installation, it would be subject to a less demanding state of stress (simple pretension) during service.

## 6.7. Faying Surfaces

### 6.7.1. *When is paint permitted on the faying surfaces of bolted connections?*

In snug-tight and fully tensioned bearing connections, paint is unconditionally permitted on the faying surfaces. In slip-critical connections, however, if paint is present, it must be a qualified paint. A qualified paint is one that has been tested in accordance with RCSC Specification Appendix A and offers a defined slip-coefficient. Other paints that do not offer a defined slip-coefficient are not permitted on the faying surfaces of slip-critical connections, even when due to inadvertent over-spray.

### 6.7.2. *Both the AISC and RCSC Specifications require that paint on the faying surfaces of slip-critical connections be qualified (providing a minimum slip coefficient) or that such surfaces remain unpainted. Does this requirement apply to the surfaces under the bolt head and nut?*

No. In a slip-critical connection, the faying surfaces are those that resist relative movement (or slip) of the plies. This occurs on the contact surfaces between the plies, not those surfaces under the bolt head or nut.

### 6.7.3. *What is the difference between the surface preparation requirements for Class A, B, and C slip resistance in slip-critical connections?*

With uncoated faying surfaces, clean mill scale provides a Class A slip resistance ( $\mu = 0.33$ ), whereas blast cleaning is required to obtain the higher Class B slip coefficient  $\mu = 0.50$ . With painted faying surfaces, the slip resistance is determined by the tested performance of the paint system as meeting Class A, B, or some other intermediate slip coefficient and the steel to be painted must be blast-cleaned in all cases. Class C surfaces are roughened (see 6.7.4) hot-dip galvanized surfaces that provide a slip coefficient  $\mu = 0.35$ . This current value, taken from the RCSC Specification, is lower than the historically permitted slip coefficient of 0.40.

### 6.7.4. *As required in RCSC Specification Section 3(b)(5), galvanized surfaces in slip-critical connections must be roughened by means of hand wire brushing. What treatment constitutes roughening?*

Kulak et al. (1987) indicates that the galvanized surface must be visibly altered without disrupting the continuity of the galvanizing. This is usually accomplished by wire brushing. As indicated in RCSC Specification Commentary Section C3, such treatment must be controlled to achieve the necessary roughening or scoring. Power wire-brushing is generally not acceptable because it tends to polish the surface rather than roughen it. Note that an acceptable result can be achieved with a variable-speed power tool with a stiff wire brush when used within a speed range that is comparable to that for hand wire brushing.

## 6.8. Inspection

### 6.8.1. *What should the inspector observe when bolts are installed?*

From RCSC Bulletin No. 1, the inspection procedures should be in accordance with RCSC Specification Section 9. The inspector should confirm that the materials meet the requirements of the contract documents and that they are properly cared for.

When connections are required to be slip-critical, the inspector should confirm that the faying surfaces have been properly prepared before the connections are assembled. When bolts are required to be fully tensioned, the inspector should observe the specified job-site testing and calibration and confirm that the installation procedure to be used does provide the required tension. Additionally, the inspector must monitor the work to assure that these procedures are routinely followed on the joints that are specified to be fully tensioned. Such surveillance by the inspector provides the greatest assurance of proper bolt installation. The inspector should confirm that the bolt holes are properly made and ready to receive the bolt and that the proper type and size of bolt is installed.

As stated in RCSC Specification Section 8(c), snug-tightened and fully tensioned bearing connections need not [and should not] be subject to inspection testing to determine the actual level of bolt tension; the inspector need only confirm that all bolts in the group have been adequately visited during the installation. For fully tensioned bolts in slip-critical connections and connections subject to direct tension, however, RCSC Specification Section 9(a) indicates that “the Inspector shall monitor installation of bolts to determine that all plies of the material have been drawn together and that the selected procedure has been used to tighten all bolts ...”

### 6.8.2. *How is a dispute over installed bolt tension in slip-critical connections resolved?*

When disputes arise, an arbitration procedure utilizing a calibrated torque wrench is covered in RCSC Specification Section 9(b). The reader is also referred to the supporting information in RCSC Commentary Sections C8 and C9. As discussed in 6.6.1, published standard torque values are not acceptable for use in lieu of actual calibrated torque values.

## 6.9. Bolt Tension Calibration

### 6.9.1. *The RCSC Specification discusses a “calibration device capable of indicating bolt tension.” What is an example of such a bolt tension calibration device?*

One such device is the Skidmore-Wilhelm Bolt Tension Calibrator, manufactured by the Skidmore-Wilhelm Manufacturing Company, Cleveland, OH, 216/481-4774. When a sample bolt is installed in the “Skidmore,” the tension is measured on a dial gauge. Thus, the appropriate torque for use in the calibrated wrench installation method may be determined, or the proper tension resulting from the turn-of-nut, alternative design bolt, or direct tension indicator methods may be verified. It is not intended that the use of other similar devices be excluded by this discussion.

6.9.2. *When short bolts will not fit in the bolt tension calibration device how can they be tested?*

Because devices such as the Skidmore have a minimum bolt length, testing of shorter bolts can be accomplished in any convenient steel plate by the use of a washer-type direct tension indicator (DTI). A similar DTI must first be tested using a longer bolt in the bolt tension calibration device to verify that they are neither under nor over strength. Alternatively, a calibrated torque may be determined using a bolt tension calibration device and a longer bolt with a hardened washer under the turned element. This torque may then be used for testing shorter bolts with a hardened washer under the turned element in a steel plate, provided lubrication and condition of threads for the long and short bolts are similar.

## 6.10 Washer Requirements

6.10.1. *When are washers required in bolted connections?*

The cases in which  $\frac{5}{32}$ -in.-thick ASTM F436 washers must be used with ASTM A325 and A490 bolts are indicated in RCSC Specification Section 7(c). Such washers are not required for these high-strength bolts in standard, oversized, and short-slotted holes except:

1. Under the turned element when the bolt is fully tensioned by the calibrated wrench method.
2. Under the bolt head and nut when ASTM A490 bolts are used in combination with material with a specified yield strength below 40 ksi.
3. To cover an oversized or short-slotted hole in an outer ply. For ASTM A490 bolts over 1 in. in diameter through an oversized or short-slotted hole in an outer ply, the washer must be  $\frac{5}{16}$ -in. thick; two  $\frac{5}{32}$ -in.-thick washers do not meet this requirement.

Some alternative design fasteners provide a bearing circle on the bolt head and/or nut of diameter that is equal to or greater than the diameter of an ASTM F436 washer. When such fasteners are used, the need for washers in cases 1 and 2 above is eliminated, except where it is necessary to eliminate the potential for galling.

When high-strength bolts are used with long-slotted holes in an outer ply, a plate washer with standard holes that completely covers the slot must be provided. The plate washer must be made from structural grade material with a minimum thickness of  $\frac{5}{16}$ -in., but need not be hardened. For ASTM A490 bolts over 1 in. in diameter through long-slotted holes in an outer ply, single  $\frac{5}{16}$ -in.-thick ASTM F436 washers must be used; two  $\frac{5}{32}$ -in.-thick washers do not meet this requirement.

In some cases, the combination of grip and selected bolt length may be such that the nut would jam on the thread run-out. In such cases, the use of washer(s) is a common solution.

6.10.2. *When are beveled washers required?*

To assure proper bolt performance, it is required in RCSC Specification Section 7(c)(1) that the surfaces against which the head and nut bear have a slope not greater than 1:20 with respect to the plane normal to the bolt axis. American

standard beams (S-shapes) and channels are rolled with beveled flanges that exceed this limit. Because bolt holes are made perpendicular to the outside face of these flanges, a beveled washer must be used at the inside face to provide the required parallelism. Beveled washers are made square or rectangular so that they can more easily be prevented from turning to assure that the bevel is oriented in the proper direction.

### 6.11. Other General information

#### 6.11.1. *Why is the design strength of a bolt calculated in the AISC LRFD Specification on the basis of the nominal cross-sectional area rather than the net tensile area that remains after threading?*

The ratio of stress area to nominal bolt area ranges from 0.75 for  $\frac{3}{4}$ -in. diameter to 0.79 for  $1\frac{1}{8}$ -in. diameter (Kulak et al., 1987). Accordingly, to simplify calculations, the lower bound reduction of 0.75 is incorporated in AISC-tabulated nominal strength values for use with nominal bolt areas.

#### 6.11.2. *When is it permissible to reuse high-strength bolts?*

High-strength bolts that have been previously installed in the snug-tight condition are suitable for reuse. However, high-strength bolts that have been fully tensioned, either in bearing or slip-critical connections, may or may not be suitable for reuse as follows.

As stated in RCSC Specification Section 8(f), ASTM A490 bolts and galvanized ASTM A325 bolts are never suitable for reuse if they have once been fully tensioned in accordance with the procedures in RCSC Specification Section 8(d). Reuse of non-galvanized ASTM A325 bolts is acceptable if approved by the SER. As indicated in AISC LRFD Manual page 8-19, such reuse is acceptable, regardless of previous use, if the nut can be placed on the threads and run down the full length of the thread by hand. This simple rule-of-thumb gives a good indication of the amount of plastic deformation that has occurred on the shank of the fastener. Other references suggest that one or two reuses is acceptable.

Note the qualification in the RCSC Specification that “touching-up or re-tightening previously snug-tightened bolts that may have been loosened by the snugging of adjacent bolts shall not be considered to be a reuse.” Similarly, fit-up bolts (which are snug-tight when initially installed) may be left in place and subsequently fully-tensioned, if required, as permanent bolts in the connection.

#### 6.11.3. *What minimum stick-through is required for high-strength bolts?*

None. As defined in RCSC Commentary Section C2, full thread engagement is achieved when “...the end of the bolt at least flush with the face of the nut.” Some contract documents include a stick-through requirement (minimum protrusion of the bolt point beyond the nut). However, because the threaded length for any given bolt diameter is constant regardless of the bolt length, a stick-through requirement (which may require a longer bolt) increases the risk of jamming the nut on the thread run-out.<sup>4</sup> Because a stick-through requirement does not enhance the performance of the bolt, its specification is discouraged.

4. Nut jamming is not a concern for fully threaded ASTM A325T bolts. See 6.2.6.

Note that there is no specified maximum limitation on bolt stick-through. However, in order to properly tension high-strength bolts, sufficient thread must be available. The use of additional flat washers under the head and/or nut is a common solution when there is a risk of jamming the nut on the thread run-out. Multiple washers are permitted under either or both the head and the nut.

6.11.4. *When an extended end-plate moment connection is specified as slip-critical, must the slip resistance of the bolts at the tension flange be reduced for the tension present?*

No. Because the tensile and compressive flange forces are equal and opposite, any loss of slip resistance adjacent to the tension flange of the beam is compensated for by an increase in slip resistance adjacent to the compression flange. This is further discussed in RCSC Specification Commentary Section C5.

6.11.5. *What ply thickness is required to exclude threads from the shear plane in high-strength bolted connections?*

A common rule of thumb given in RCSC Specification Commentary Section C2 is as follows: with no washers, the threads will always be excluded from the shear plane (regardless of the grip) for  $\frac{3}{4}$ -in. and  $\frac{7}{8}$ -in. diameter bolts if the ply thickness closest to the nut is not less than  $\frac{3}{8}$ -in.; the same is true for 1-in. and  $1\frac{1}{8}$ -in. diameter bolts if the ply thickness closest to the nut is not less than  $\frac{1}{2}$ -in. With one washer under the nut, these values may be reduced by  $\frac{1}{8}$ -in. Because this rule of thumb is based upon the worst case combination of grip and bolt length, lesser thickness of the ply closest to the nut is often acceptable. Refer to Carter (1996).

6.11.6. *As indicated in AISC LRFD Specification Table J3.2, when the pattern of fasteners in a bolted joint exceeds 50 in. in length, tabulated design strengths should be reduced by 20 percent. Why?*

As indicated in Kulak et al. (1987) the average shear strength per bolt varies with the number of bolts in the joint due to the non-uniformity of force distribution; see Figure 5.28 on p. 107 therein. To simplify joint design, bolt shear strengths in the RCSC and AISC LRFD Specifications incorporate a 20 percent reduction to allow the use of a consistent per-bolt design strength for joints up to 50 in. in length. However, if joint length exceeds 50 in., the designer must further reduce the design strength by another 20 percent. This phenomenon is a by-product of shear lag in the connection.

6.11.7. *How do hot-dip galvanizing and mechanical galvanizing processes differ?*

In the hot-dip galvanizing process, the piece is first degreased and cleaned with a combination of caustic and acidic solutions. After rinsing, the piece is dipped into a tank of molten zinc for a specified period of time. The full process is described in ASTM A153.

In the mechanical galvanizing process, the piece is similarly cleaned and rinsed. The piece is then tumbled in a mixture of various-sized glass beads and a predetermined amount of water, with small amounts of chemicals and powdered zinc added periodically. Collisions between the glass beads, zinc, and the piece causes a cold-welding process that applies the zinc coating. Powdered zinc is added until the specified thickness is attained. The full process is described in ASTM B695.





## CHAPTER 7

### ANCHOR RODS, BASE PLATES AND EMBEDDED PLATES

The AISC LRFD *Specification for Structural Steel Buildings* and various ASTM material standards cover requirements for the use of anchor rods and base plates. This commentary includes a discussion of portions of these provisions and subsequent recommendations. Additional information is available from AISC Design Guide #1 *Column Base Plates* (Dewolf and Ricker, 1990) and AISC Design Guide #10 *Erection Bracing of Low-Rise Structural Steel Frames* (Fisher and West, 1997).

#### 7.1. Anchor Rods

##### 7.1.1. *Why has AISC initiated a change in nomenclature with the term anchor rod?*

AISC has changed its terminology to anchor “rod” to eliminate confusion between structural bolting applications, such as those covered by the AISC and RCSC Specifications, and anchorage applications between steel members and concrete elements. This includes such issues as installed tension, slip resistance, and hole sizes, which do not apply in anchor-rod applications, but which may be required for all-steel bolting applications.

Anchor rods may be configured to provide anchorage into concrete by means of a head, threading with a nut on the end, a hook, or by swaging. The term anchor bolt, when used with ASTM A307 grade C, A325, or A490 material, however, only describes the first option for the following reasons:

1. These specifications include heading requirements.
2. ASTM A325 and A490 include defined threaded lengths.
3. Bolts meeting these specifications are generally only available in lengths up to about 8 in., except by special order.

##### 7.1.2. *To what material specifications are anchor rods ordered?*

There are three basic alternatives:

1. ASTM F1554 covers anchor rods (though the term anchor bolt is still used) in headed, threaded and nutted, and hooked configurations with three yield-strength levels: 36, 55, and 105 ksi. It is intended that this umbrella specification cover the full range of material needs for anchor rods, including galvanized applications. While this is a relatively new specification, it is expected that it will gradually become the industry standard for anchor rods.
2. Headed anchor rods can also be obtained in ASTM A449 and A354 material
3. Threaded and nutted or hooked rods can be obtained to meet the following material specifications: ASTM A36, A572, A449, A354, A588, and A687.

##### 7.1.3. *Are rolled and cut threads equally acceptable for anchor rods?*

Yes. The use of either rolled or cut threads is permitted in ASTM F1554 Section 6.2. Rolled threads are formed by pressing threading dies into the shank to displace the surplus of the metal outward. The original rod diameter must be

slightly less than the nominal diameter, although the root area will still be critical (see 6.11.1), unless the rod end is upset. The steel is cold-worked, compressing its grain and increasing the yield and tensile strength, generally from 10 to 30 percent. Cut threads are made with a thread-cutting die or by lathe cutting. The original rod diameter is approximately equal to the nominal diameter; again, the root area will be critical as is normal in design.

7.1.4. *Can the same nut be used on both cut and rolled threads?*

Yes. Both rolled and cut threads are produced to meet the same threading specification.

7.1.5. *How can short anchor rods be extended above base plates when the nut threads will not be fully engaged?*

There are two common methods to extend misplaced anchor rods that are too short to fully engage the nut threads. With either method, it may be necessary to enlarge the base-plate holes, which can be done by flame-cutting.

In the first method, a thin-walled threaded coupler with adequate strength for the application is used to attach a threaded extension. It may be necessary to remove concrete near the top of the foundation to permit the installation of the coupler.

In the second method, a threaded extension is welded to the top of the existing rod. The threaded extension is prepared for welding by beveling the contact end to a chisel point as illustrated in Figure 7.1.5-1 and is subsequently welded using suitable electrode material. The surface of this welded transition is typically non-uniform and may necessitate the use of plate washers of sufficient quantity to allow free rotation of the nut. This method, which requires welding, may be unsuitable for heat-treated anchor rod material, such as ASTM A449.

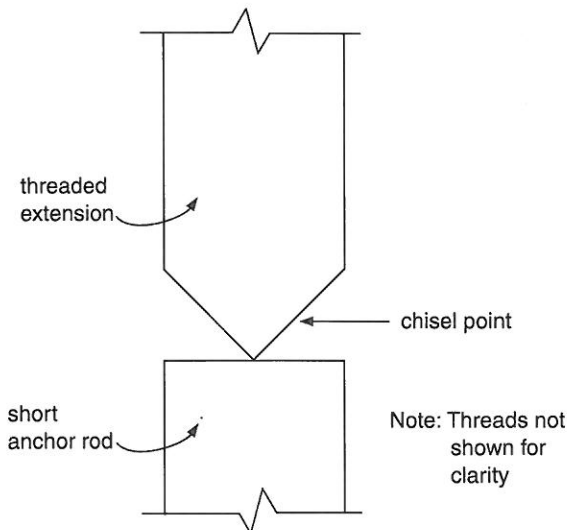


Fig. 7.1.5-1. *Beveled end of threaded extension to be welded to short anchor rod.*

Note that plug-welding the partially engaged nut to the anchor rod is not considered to be an effective means of attachment.

7.1.6. *Can anchor rods be welded to a base plate?*

Yes, if the rod material is weldable. Anchor rods are used primarily to provide a pre-positioned location upon which to erect the column and to provide stability during erection. They are also used in conjunction with the dead load of the structure to resist uplift forces. Subsequent welding of anchor rods to the base plate will not serve the first two purposes, but can be helpful in providing uplift resistance. Because the base-plate holes are oversized and the anchor rod is rarely centered in the hole, a heavy plate washer is required as illustrated in Figure 7.1.6-1(a); see also 7.2.4. The welding of rod to washer involves a fillet weld profile with a weld length that is equal to  $\pi$  times the rod diameter, which develops relatively little strength. Welding to the threaded portion of a rod is permissible. If larger uplift forces are present, an alternative column base detail, such as the boot in Figure 7.1.6-1(b) should be considered.

7.2. **Base Plates**

7.2.1. *How can one account for base-plate distortion due to welding?*

When thin base plates are welded to a column shaft, or when large welds are used, base plates tend to curl or distort. When base plates over 2½-in. thick are shop welded to the column shaft, provisions should be made for grouting the base plates to proper elevation. When base plates less than 2½-in. thick are welded to the column shaft, the flatness tolerances of AISC LRFD Specification Section M4.4 should be maintained. Adequate contact bearing will be achieved in either case.

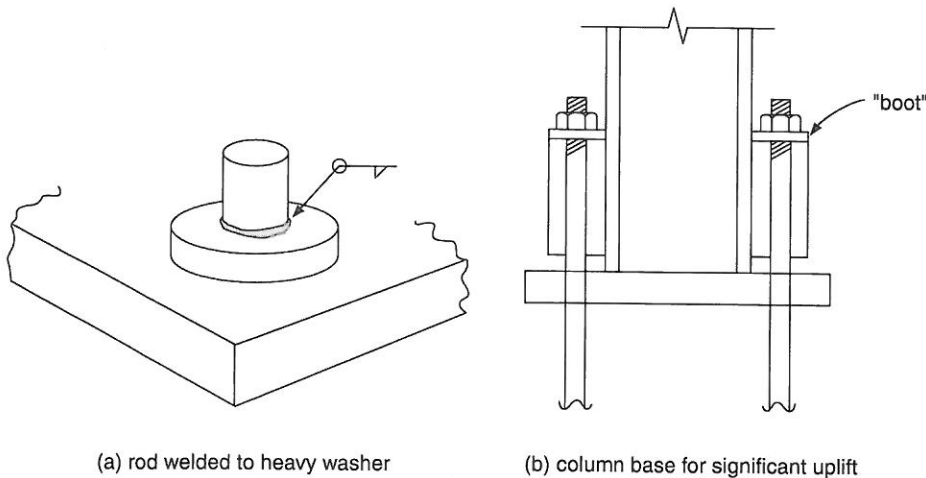


Figure 7.1.6-1.

7.2.2. *When a steel base plate bears on less than the full area of concrete, the design bearing strength  $\phi(0.85f'_cA)$  is multiplied by the lesser of 2 or the square root of the ratio of geometrically similar concrete area to base-plate area. Why?*

AISC LRFD Specification Section J9 is consistent with ACI 318 provisions, which recognize the increase in bearing strength that results from the confinement that is provided by the concrete surrounding that providing direct bearing resistance.

7.2.3. *What are the preferred hole diameters in base plates?*

The recommended maximum hole sizes for anchor rods in base plates are given in LRFD Manual Table 11-3. It is noted that these hole sizes permit a reasonable tolerance for misalignment in setting the bolts and more precision in the adjustment of the base plate or column to the correct centerlines. Note that these hole sizes are such that flame-cutting will often be required. An adequate washer (see Section 7.2.4) should be provided for each anchor rod. Because these hole sizes are recommended as maximum sizes, the use of smaller hole sizes is often justified if anchor-rod groups are set accurately.

7.2.4. *What thickness and size of washer is required for the preferred hole diameters in base plates?*

A general rule of thumb is given in AISC Design Guide #10 *Erection Bracing of Low-Rise Structural Steel Frames* (Fisher and West, 1997): the minimum thickness should be one-third the diameter of the anchor rod and that the minimum diameter (or length and width for a non-circular washer) should be 1-in. larger than the hole diameter. When the anchor rod transmits tension, the washer size must be sufficient to transmit the force to the base plate. Washers of the appropriate size can generally be fabricated from plate.

7.2.5. *When should grout holes be provided for base plates?*

Grouting of base plates can be accomplished for common base-plate sizes without the need for grout holes. To assure that no air pockets are left under the plate during the grouting operation, the grout should be fed in all from one side only until it emerges from the opposite side. The use of pre-set leveling plates simplifies the grouting process. However when the smaller dimension of the base plate exceeds 24 in., the use of a grout hole(s) should be considered. Grout holes are usually about 2 in. to 3 in. in diameter and spaced approximately 18 in. apart if more than one is required. The loss of area due to grout holes and anchor bolt holes is generally ignored when determining base plate area.

### 7.3. Embedded Plates

7.3.1. *How can rod-type concrete anchors be welded to embedded plates?*

If a common shear stud connector size is suitable, the stud welding provisions of AWS D1.1 Section 7 can be used. Other rod-type anchors can be square-cut and fillet welded if strength is adequate. Because the weld length is  $\pi$  times the rod diameter, such welding provides limited strength. When a greater welded strength is required, the rod can be beveled on two sides to a chisel point as illustrated in Figure 7.1.5-1, which allows for easier deposition of weld metal than beveling to a pencil point.

## CHAPTER 8

### WELDING

The AISC LRFD *Specification for Structural Steel Buildings* and AWS D1.1-96 cover requirements for the use of welding in structural steel connections. This commentary includes a discussion of portions of these provisions and subsequent recommendations.

#### 8.1. Economical Suggestions

##### 8.1.1. *Why is welding preferably done in the flat position?*

In the flat position, the base metal provides support for the molten pool of weld metal. Therefore, this position provides for the fastest deposition rate and the most economical weld. Welding in the horizontal position is similar, but slightly less efficient. Welding in the vertical or overhead position requires slower deposition rates to maintain the integrity of the molten pool against the effects of gravity. These welding positions are illustrated in AISC LRFD Manual Figure 8-21.

##### 8.1.2. *Why is use of the least possible size fillet weld desirable?*

Because the volume of weld metal in a fillet weld is proportional to the square of the weld size, a 1/2-in. fillet weld uses four times as much weld metal as a 1/4-in. fillet weld of the same length. Because the cost of welding is essentially proportional to the volume of weld metal, the most economical fillet-welded detail will result when the least possible fillet weld size is used. Accordingly, it is common practice in welded joint design to select fitting and weld length to minimize fillet weld size, when possible. Additionally, smaller welds reduce the possibility of warping and distortion due to heat input.

##### 8.1.3. *Why are fillet welds preferred over groove welds?*

Fillet welds generally require less weld metal than groove welds. Additionally, fillet welds do not generally require beveling and similar base metal preparation and do not require the same level of operator skill as for groove welds. As a result, fillet welds are generally more economical to make than groove welds. Thus, fillet welds are preferred.

#### 8.2. Groove Welds

##### 8.2.1. *Are weld quality criteria applicable to the root area of partial-joint-penetration groove welds?*

No. Attempts are sometimes made to apply weld quality criteria to the root area of partial-joint-penetration groove welds. Evaluation of weld quality in the root area should be limited to the verification of proper joint penetration and weld area, as provided in AWS D1.1 Section 2.3, and proper welding procedures.

##### 8.2.2. *When a weld is placed between plates forming an angle that is less than 60 degrees, why is a Z loss factor applied to determine the effective throat?*

The Z loss factor is applied at angles below 60 degrees to recognize that this weld cannot reliably penetrate to the root of the joint and is thus a partial-joint-pene-

tration groove weld; see Figure 8.2.2-1. Note that, below 30 degrees, this joint is no longer prequalified.

8.2.3. *What is the difference between a flare weld and a partial-joint-penetration groove weld?*

A flare weld is a special kind of partial-joint-penetration groove weld wherein the convex surface of the connected part(s) creates the joint preparation. A flare weld is illustrated in LRFD Manual Figure 8-47.

8.2.4. *What purpose does a weld access hole serve?*

The primary purpose of a weld access hole, as the name implies, is to allow the welder access to start and stop the weld beyond the plane of a beam web or other obstruction. At the same time, the weld access hole also minimizes restraint to allow for shrinkage in the welded joint and eliminates the intersection of welds in orthogonal directions (and the associated intersection of stresses).

8.2.5. *When should backing bars and run-off tabs be removed after welding?*

To produce sound welds on many welded joint geometries, run-off tabs projecting from the finished member may be required to permit starting and stopping welds beyond the edge of the member; AWS D1.1 Sections 5.10 and 5.31 should be followed. Additionally, AISC LRFD Specification Section J1.5 addresses requirements for the removal of backing bars and weld tabs at complete-joint-penetration groove welded splices in ASTM A6 Group 4 and 5 rolled shapes and plates exceeding 2 in. thickness subject to primary tensile stresses. When such welding aids are required to be removed, the surface should be finished as indicated in 2.2.6 and 2.2.7.

Damage to welded beam-to-column-flange moment connections in the 1994 Northridge earthquake has raised several welding and seismic detailing issues and new criteria have been established. Explicit requirements for the removal of back-up bars and run-off tabs in seismic projects have been included in the AISC *Seismic Provisions* (AISC, 1997). An exception is included for tested assemblies

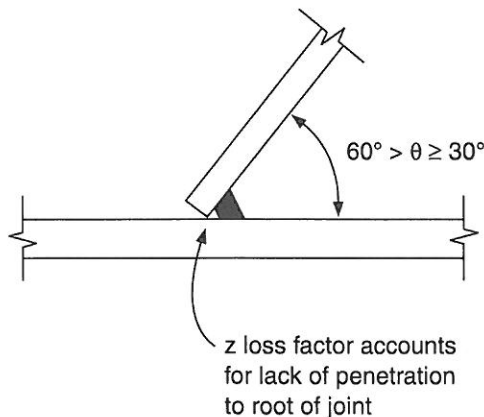


Fig. 8.2.2-1. *z* loss factor.

that can be demonstrated to have acceptable performance with alternative treatments.

### 8.3. Fillet Welds

#### 8.3.1. *Are fillet welds stronger when loaded transversely than when loaded longitudinally?*

Yes. This long known variation in strength as a function of load angle is now formally recognized in AISC LRFD Specification Appendix J2.4. The maximum strength increase permitted therein is 50 percent, which occurs for a load perpendicular to the fillet weld. When the load angle is intermediate between longitudinal and transverse, the strength increase will vary between none and 50 percent, respectively.

#### 8.3.2. *Does the fusion zone along the leg of a fillet weld need to be checked in addition to the theoretical throat to determine the strength of a fillet weld?*

No. As long as a matching electrode strength is used (see also 8.7.5) as required in AISC LRFD Specification Table J2.5, the weld throat will always be more critical than the fusion zone (base metal) at the weld leg.

#### 8.3.3. *When fillet welds are oversized, what corrective procedures are required?*

Acceptable and unacceptable weld profiles are specified in AWS D1.1 Section 5.24. Such profiles are subject to misinterpretation when a fillet weld has been inadvertently oversized. AISC recommends that either or both legs of fillet welds may be oversized without correction, provided the excess weld metal does not interfere with the satisfactory end use of the member. Attempts to remove such excess weld metal may cause shrinkage, distortion, and/or cracking. The profile of fillet welds shall be in accordance with AWS D1.1 Section 5.24.1.

#### 8.3.4. *Are corrective procedures required when fillet welds are undersized?*

From AWS D1.1 Table 6.1, "A fillet weld ... shall be permitted to underrun the nominal fillet weld size specified by  $\frac{1}{16}$ -in. without correction, provided that the undersized portion of the weld does not exceed 10 percent of the length of the weld." If this limit is exceeded, additional weld metal can be deposited on top of the deficient area to increase the size as required.

#### 8.3.5. *How should fillet welds be terminated?*

A November 21, 1995 revision to the AISC LRFD Specification set forth the following provisions for the termination of fillet welds:

##### *LRFD Specification Section J2.2b*

Fillet weld terminations may extend to the ends or sides of parts or may be stopped short or may be boxed [returned at the top and bottom] except as limited by the following cases:

- (a) In lap joints between parts subjected to calculated tensile stress in which one part extends beyond the edge or side of the part to which it is connected, fillet welds shall terminate not less than the size of the weld from the start of the extension.
- (b) For details and structural elements such as brackets, beam seats, framing angles, and simple end plates, the outstanding legs of which are subject to

cyclic (fatigue) forces and/or moments of a frequency and/or magnitude that would tend to cause a progressive failure initiating from a point of maximum stress at the weld termination, fillet welds shall be returned around the side or end for a distance not less than two times the weld size or the width of the part, whichever is less.

- (c) For framing angles and simple end-plate connections, top angles of seated connections, and similar fittings that depend upon flexibility of the outstanding legs for connection flexibility, if end returns are used, their length shall not exceed four times the nominal weld size.
- (d) Except where the ends of stiffeners are welded to the flange, fillet welds joining transverse stiffeners to plate-girder webs shall start and terminate not less than four times nor more than six times the thickness of the web from the web-side toe of the web-to-flange welds.
- (e) Fillet welds on opposite sides of a common plane shall be interrupted at the corner common to both welds.
- (f) The length and disposition of welds, including end returns or boxing, shall be indicated on the design and shop drawings.

A corresponding revision has also been made to the Commentary as follows:

*LRFD Specification Commentary Section CJ2.2b*

Fillet weld terminations do not affect the strength or serviceability of connections in most cases. However, in certain cases, the disposition of welds affects the planned function of connections and notches may affect the static strength and/or the resistance to crack initiation if cyclic loads of sufficient magnitude and frequency occur. For these cases, limitations are specified to assure desired performance.

- (a) At lapped joints where one part extends beyond the end or edge of the part to which it is welded and if the parts are subject to calculated stress at the start of the overlap, it is important that the weld terminate a short distance from the stressed edge. For one typical example, the lap joint between the WT chord and the web members of a truss, the weld should not extend to

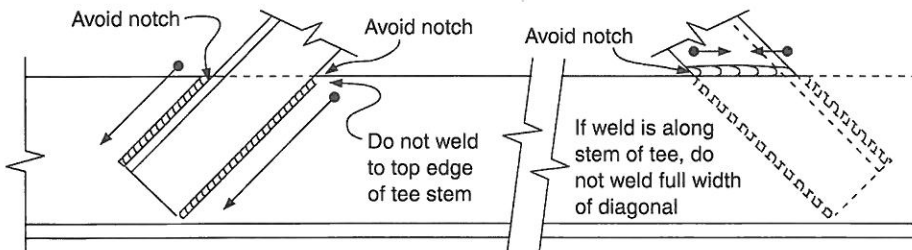


Figure 8.3.5-1.



the edge of the WT stem; see Figure [8.3.5-1]. The best technique to avoid inadvertent notches at this critical location is to strike the welding arc at a point slightly back from the edge and proceed with welding in the direction away from the edge. On the other hand, where framing angles extend beyond the end of the beam web to which they are welded, the free end of the beam web is subject to zero stress; thus, it is permissible for the fillet weld to extend continuously across the top end, along the side and along the bottom end of the angle to the extreme end of the beam; see Figure [8.3.5-2].

- (b) For connections that are subject to maximum stress at the weld termination due to cyclic forces and/or moments of sufficient magnitude and frequency to initiate cracks emanating from unfilled start or stop craters or other discontinuities, the end of the weld must be protected by boxing or returns. If the bracket is a plate projecting from the face of a support, extra care must be exercised in the deposition of the boxing weld across the thickness of the plate to assure that a fillet free of notches is provided.
- (c) For connections such as framing angles and simple end plates that are assumed in the design of the structure to be flexible connections, the top edges of the outstanding legs must be left unwelded over a substantial portion of their length to assure flexibility of the connection. Research has shown that the static strength of the connection is the same with or without end returns; therefore, the use of returns is optional. If used their length must be restricted to not more than four times the weld size; see Figure [8.3.5-3].
- (d) Experience has shown that when ends of intermediate transverse stiffeners on the webs of plate girders are not welded to the flanges (the usual practice), small torsional distortions of the flange, which occur near shipping bearing points in the normal course of shipping by rail or light truck, may cause high out-of-plane bending stress at or near the yield point and

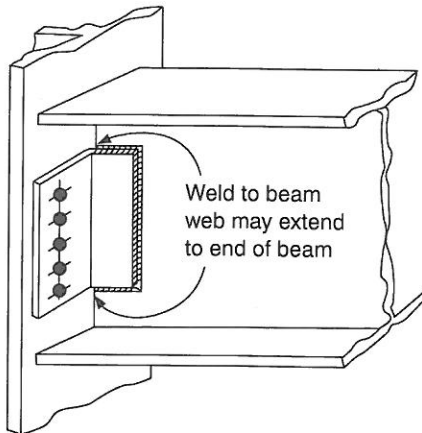


Figure 8.3.5-2.

fatigue cracking at the toe of the web-to-flange welds if the web-to-stiffener welds terminate close to the web toe of the web-to-flange welds. This has been observed even with closely fitted stiffeners. The intensity of these out-of-plane stresses may be effectively limited and cracking prevented if “breathing room” is provided by holding back the web-to-stiffener welds four times the web thickness from the toe of the web-to-flange welds. The unwelded distance should not exceed six times the web thickness to assure that column buckling of the web within the unwelded length does not occur.

- (e) For fillet welds that occur on opposite sides of a common plane, it is not possible to deposit a weld continuously around the corner from one side to another without causing a gouge in the corner of the parts joined. Therefore, the welds must be interrupted at the corner. See AISC LRFD Manual Figure 8-40 (page 8-123).

8.3.6. *Why is a fillet weld size generally limited to  $\frac{1}{16}$ -in. less than the material thickness when placed along the edge of a connected part?*

As explained in AISC LRFD Specification Commentary Section J2.2b, “For plates of  $\frac{1}{4}$ -in. or more in thickness, it is necessary that the inspector be able to identify the edge of the plate to position the weld gage.” This is illustrated in AISC LRFD Manual Figure 8-38. Note that this requirement is qualified in AISC LRFD Specification Section J2.2b: the weld toe is permitted to be less than  $\frac{1}{16}$ -in. away from the edge of the base metal, provided the weld size is clearly verifiable. Additionally, the weld size can match the thickness of the plate edge for plates that are less than  $\frac{1}{4}$ -in. thick.

8.3.7. *Is the weld-all-around symbol acceptable when a fillet weld must be continued out-of-plane?*

No. Use of the weld-all-around symbol at conditions that would require the weld to be continued out-of-plane calls for a condition that is specifically prohibited

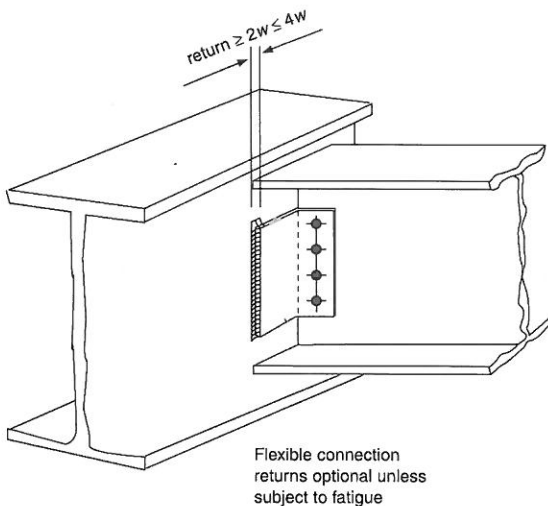


Figure 8.3.5-3.

in AISC LRFD Specification Section J2.2b and AWS D1.1 Section 2.4.7.2. Instead, when an out-of-plane transition occurs, the welds must be interrupted at the corner common to both welds.

**8.3.8** *What constitutes acceptable fit-up in fillet-welded joints?*

From AWS D1.1 Section 5.22.1, a root opening not exceeding  $\frac{1}{16}$ -in. is permitted without modification. A root opening not exceeding  $\frac{3}{16}$ -in. is generally permitted therein if the weld size is increased by the amount of the root opening or it is demonstrated that the required effective throat has been obtained. For plate thicknesses greater than or equal to 3 in., a  $\frac{5}{16}$ -in. root opening is permitted if suitable backing is used.

**8.4. Plug and Slot Welds**

**8.4.1.** *When are plug and slot welds used?*

Plug and slot welds are permitted for the transfer of shear force only. As such, they are sometimes used to transmit shear in lap joints, to join components of built-up members, or to prevent buckling of lapped parts. Their design and usage is covered in AISC LRFD Specification Section J2.3.

**8.5. Repairs**

**8.5.1.** *Is it necessary to remove temporary welds that are not incorporated into the permanent welds?*

In some cases, tack welds for temporary fitting aids are not to be incorporated into the permanent welds. Generally, such welds should be allowed to remain in Statically Loaded Structures, unless their removal is required in contract documents. In Cyclically Loaded Structures, such temporary welds should be removed. This topic is addressed in greater detail in AWS D1.1 Section 5.18.

**8.5.2.** *Is it necessary to remove arc strikes?*

In Statically Loaded Structures, arc strikes need not be removed, unless such removal is required in the contract documents. However, in Cyclically Loaded Structures, arc strikes may result in stress concentrations that would be detrimental to the serviceability of such structures and should be ground smooth and visually inspected for cracks.

**8.5.3.** *What corrective procedures are required when distortion occurs from weld shrinkage?*

Correction of distortion causes additional stresses. If the end use of the weldment does not justify such corrective action, these additional stresses can often do more harm than good. Correction of out-of-tolerance conditions should be made if required for structural adequacy and erection requirements. When required, the tolerances in AWS D1.1 Section 5.23 are reasonable and workable and should be followed.

## 8.6. Welding Procedure Specification (WPS)

### 8.6.1. *What elements are essential for proper workmanship in welding?*

Proper selection of the weld type and profile by the designer are essential. In addition, proper filler metal selection and workmanship in joint preparation, fit-up, cleaning, preheat, technique, position, process, and procedure should be properly described in a written WPS. The essential elements of quality are adequately described in AWS D1.1. While each is important when considered individually, they are more important when considered collectively, because deviations from good practice in any one element, when combined with deviations in other elements, can reduce the probability that suitable welds will be attained. All requirements should be consistent with the end use of the member.

### 8.6.2. *Why is adherence to an approved WPS important?*

Strict adherence to an approved WPS, when combined with monitoring of the essential elements described in 8.6.1 during the welding operation provides a greater degree of quality assurance than mere cosmetic inspection after welding. The end use of the product (static or cyclic loading, seismic loading, tensile or compressive loading, relative level of stress) should be considered in evaluating any deviations.

### 8.6.3. *What constitutes sufficient evidence of qualification of welding procedures and personnel?*

AWS D1.1 Section 4 covers two types of welding procedures, prequalified and qualified; as well as the qualification of welders, welding operators, and tackers. With prequalified procedures, as described in AWS D1.1 Section 3, project-specific qualification by weld procedure testing is not required. However, procedures that deviate from tolerances described therein must be qualified by weld procedure testing as indicated in AWS D1.1 Section 4.1.1. Such testing is time-consuming and costly, and may be repetitious if similar joints have already been tested for previous projects. Likewise, arbitrary re-qualification of personnel, as sometimes specified in contract documents, will unjustifiably increase the cost of welded construction.

As recommended in AWS D1.1 Section 4.1.1, properly documented evidence of previous qualification of joint welding procedures should be accepted without re-qualification. Additionally, properly documented evidence of previous qualification of welders, welding operators, and tackers should be accepted without re-qualification, provided that the period of effectiveness has been maintained as described in AWS D1.1, Section 4.1.3.1.

## 8.7. Other General Information

### 8.7.1. *When a box of welding electrodes is opened, what precautions are required for their protection from contamination?*

From AWS D1.1 Sections 5.3.1.4 and 5.3.1.5, "Welding consumables that have been removed from the original package shall be protected and stored so that the welding properties are not affected. Electrodes shall be dry and in suitable condition for use." In addition, AWS D1.1 Section 5.3.2 contain provisions for

storage and rebaking for low-hydrogen electrodes, which are more susceptible to moisture absorption.

8.7.2. *When dual-certified material (i.e., A36/A572 Grade 50) is specified, should welding be performed in accordance with AWS Group I or Group II requirements?* ASTM A36 steel is classified as AWS Group I material and, as such, may be welded with non-low-hydrogen processes. In contrast, ASTM A572 Grade 50 steel is classified as AWS Group II material, which, because of its higher yield strength, must be welded using low-hydrogen processes. Because dual-certified steel, by definition, meets the chemistry and strength requirements of ASTM A572 Grade 50 steel, welding should be performed using low-hydrogen processes, unless the suitability of an appropriate weld procedure specification using a non-low-hydrogen process can be demonstrated through qualification testing.

8.7.3. *How are seal welds sized and made?*

Seal welds are sometimes made to provide a water- or air-tight joint that otherwise would not be. In building construction, seal welded joints are rarely required to withstand internal pressures as would be common in steel tanks and piping circuits. Consequently, they can be sized for any load transfer requirements or from minimum size requirements in AWS D1.1.

In most cases, seal welds commonly assume a fillet weld profile. Any aesthetic requirements for seal welds should be specified in the contract documents.

8.7.4. *Is steel in older existing structures weldable?*

Possibly. If the chemical properties of steel to be welded are known, either by valid mill certification or by laboratory sample testing, its weldability can be judged by computing the carbon equivalent value. A more obvious approach would be to examine the existing structure for evidence of original welding. Alternatively, an on-site investigation could be performed to address weld ductility and base-metal hardening. Other factors should also be considered, such as past history of the structure, the nature of the loads, weather conditions, and whether the members to receive welds are loaded; refer to Ricker (1988).

8.7.5. *The term matching weld metal is used in AISC LRFD Specification Section J2. To what are these weld metals matched and in what document are the matching weld metals defined?*

Weld metals are matched to the steel grade being welded. Matching weld metals are specified in AWS D1.1 Table 3.1.



## CHAPTER 9

# WELDING INSPECTION AND NON-DESTRUCTIVE EXAMINATION (NDE)

The AISC LRFD *Specification for Structural Steel Buildings* and AWS D1.1 cover requirements for the inspection of welding in structural steel connections. This commentary includes a discussion of portions of these provisions and subsequent recommendations.

### 9.1. NDE Methods

#### 9.1.1. *What are the commonly used methods of non-destructive examination?*

The most commonly used NDE method in structural steel fabrication is visual (VT). Other examination methods are also used: dye penetrant (DT), magnetic particle (MT), radiographic (RT), and ultrasonic (UT). The method to be used is established after consideration of the importance of the weld as well as the defect identification capability and relative cost of each method. When NDE is required, the process, extent, techniques and standards of acceptance must be clearly defined in the contract documents.

#### 9.1.2. *What NDE inspection beyond visual should be specified? What acceptance criteria should apply?*

The SER should identify members and connections that must be inspected and specify how they should be inspected. Inspection requirements can be specified, if desired, by the SER as some percentage, with subsequent testing requirements identified if a significant defect rate is discovered. For example, 15 percent initial inspection might be deemed acceptable for an AISC Quality Certified fabricator, with no further testing required if all inspected joints are found to be compliant; if a significant defect rate were found, the inspection of an additional 15 percent might be required.

#### 9.1.3. *What level of quality assurance is implied by each NDE method?*

When specified by the SER, VT, MT and DT inspection imply that internal soundness adequate for the service conditions will be provided by adherence to the requirements of AWS D1.1. Rework required to correct profile, size, undercut or overlap, and/or excessive pin holes or cracks is considered to be part of the contract requirements. However, because these are essentially surface or near-surface inspection methods that do not describe the internal condition, rework required by the owner to correct internal discontinuities, if found by other means, is considered to be a change in contract requirements.

When specified by the SER, RT and UT inspection imply that the total internal soundness of the weld is important to the structural integrity and must meet the established standard of acceptance. Any rework required to meet this standard of acceptance is considered to be part of the contract requirement. It should be understood that there are practical limitations to the effectiveness of RT and UT, such as geometry and thickness of the joined pieces.

*9.1.4. When non-destructive inspection is specified for base metal, what acceptance criteria are appropriate?*

Occasionally, severe service conditions may necessitate NDE to verify a high degree of soundness of the parent material. While UT techniques are suitable for such investigation, standard acceptance criteria do not exist. From AISC LRFD Specification Section M5.3, both a clear set of acceptance criteria and a definition of the areas to be tested should be specified in the contract. The acceptance criteria in ASTM A435/A435M for plates or ASTM A898/A898M for shapes may be appropriate.

*9.1.5. How are parent-metal discontinuities that prohibit UT examination of the weld zone handled?*

Parent metal sometimes contains discontinuities that are within the acceptance criteria, but prevent a full examination of a weld under UT inspection. In such cases, the alternate scanning procedures of AWS D1.1 Section 6.26.5.2 should be used. When such procedures still do not allow full examination of the weld, the condition should be reported to the SER for resolution.

## **9.2. Other General Information**

*9.2.1. When multiple inspection agencies are involved on the same project, how is their work coordinated?*

When shop work is subjected to inspection by two or more inspectors or inspection agencies, interpretations and evaluations often conflict because acceptance criteria vary dramatically from inspector to inspector. Therefore, work performed by two or more inspectors or inspection agencies should be coordinated and standardized.

The contract documents must include all requirements, in detail or by reference, to appropriate standards and codes that are applicable to the satisfactory end use of the structure. Additional requirements cannot be imposed on the work by inspection personnel under the generic heading of workmanship. When subsequent requirements to those specified in the contract documents are deemed appropriate for end use of the structure, they should be appended to the contract documents through contract changes.

All involved parties should cooperate with the fabricator's inspection department and agree on interpretations of acceptance criteria before work is completed and shipped. Rejection of members subjected to re-inspection activities should be limited to structurally significant conditions. Minor conditions that do not affect the serviceability of the structure should not be cause for rejection. A pre-fabrication conference can facilitate production in a timely and economic manner. Timeliness of inspection is important to the efficiency of fabrication and inspection and the avoidance of rework and delays.

*9.2.2. What quality assurance procedures must fabricators follow?*

The fabrication shop should maintain a quality control program to assure that all work is performed in accordance with the codes and specifications applicable to the contract. AISC recommends that owners use the AISC Quality Certification Program to evaluate the quality program of fabricators for specific structures. The AISC Quality Certification Program assures that fabricators have the expertise,



equipment, procedures, and ability to produce steel structures consistent with their level of certification. See also Appendix A. If the owner requires a more extensive quality program or independent inspection, this should be clearly stated in the contract documents, including the definition of the scope of such inspection.



## CHAPTER 10

### PAINTING AND SURFACE PREPARATION

The AISC LRFD *Specification for Structural Steel Buildings* and various Steel Structures Painting Council (SSPC) documents cover requirements for the painting of structural steel. This commentary includes a discussion of portions of these provisions and subsequent recommendations.

The *Steel Structures Painting Manual, Volume I, Good Painting Practice* (SSPC, 1982) and *Volume II, Systems & Specifications* (SSPC, 1991) provides a knowledgeable framework for the selection of suitable paint systems and establishes appropriate means of achieving the desired result in both the shop and field. The proper design of a total paint system suitable for the end use of the product is clearly identified as a fundamental design prerogative of the owner, architect and/or engineer.

The *Steel Structures Painting Manual* and SSPC surface preparation standards serve as generally workable and practical guides for the surface preparation and painting of fabricated structural steel. Although they have removed a great deal of the misunderstandings that once occurred in this area, there are still varying interpretations that may arise. This commentary provides AISC recommendations for clarification and resolution of several problem areas.

#### 10.1. Painting Requirements

##### 10.1.1. *When must structural steel be painted?*

As stated in AISC LRFD Specification Section M3.1, “shop paint is not required unless specified by the contract documents.” Therefore, fabricated structural steel is left unpainted unless painting requirements are outlined in the contract documents.

In building structures, steel need not be primed or painted if it will be enclosed by building finish, coated with a contact-type fireproofing, or in contact with concrete. When enclosed, the steel is trapped in a controlled environment and the products required for corrosion are quickly exhausted. As indicated in AISC LRFD Specification Commentary Section M3, “The surface condition of steel framing disclosed by the demolition of long-standing buildings has been found to be unchanged from the time of its erection, except at isolated spots where leakage may have occurred. Even in the presence of leakage, the shop [primer] coat is of minor influence (Bigos et al., 1954).” A similar situation exists when steel is fireproofed or in contact with concrete; in fact, paint is best omitted when steel is to be fireproofed because primer may decrease its adhesion.

In exterior exposed applications, steel must be protected from corrosion by painting or other means. Likewise, steel must be protected from corrosion in special applications such as the corrosive environment of a paper processing plant or a structure with oceanfront exposure.

### 10.1.2. *When a paint system is required, how should it be selected?*

When paint is required, SSPC emphasizes the importance of the development of a total paint system. Among the primary considerations for this design decision by the owner, architect, or engineer are:

1. The end use of the member.
2. A realistic estimate of time and severity of exposure of each coat of paint.
3. An economic evaluation of the initial cost as compared to future maintenance cost.
4. A practical determination of the division between shop and field work and responsibilities.

### 10.1.3. *What should be included in contract documents when steel is to be painted?*

The following information should be specified when steel is to be painted:

1. The type and manufacturer of the specified paint (one alternative is the fabricator's standard shop primer)
2. The required level of surface preparation (expressed as an SSPC designation, i.e., SP2)
3. The desired dry film thickness

All technical data and directions for application of the specified paint, including required curing time, will be obtained by the fabricator from the paint manufacturer and need not be repeated in the contract documents, other than by reference.

### 10.1.4. *What paint system is implied by the general requirement of a "shop coat" or "paint"?*

When contract documents call for a "shop coat" or "paint" without specific identification of a paint system, this is interpreted as the fabricator's standard primer applied to a minimum thickness of 1 mil on steel that has been prepared in accordance with SSPC-SP2, with no conditional performance implied.

## 10.2. **Paint Film Thickness**

### 10.2.1. *How is paint film thickness determined?*

The most commonly used paint-film-thickness measuring devices are wet-film thickness gauges and magnetic instruments for dry-film thickness measurement. When properly used during paint application, a wet film gauge is a direct-reading instrument that furnishes an immediate indication of thickness at a time when inadequacies can be corrected, usually without the need for a full subsequent coat. The residual dry-film thickness can be determined from the wet-film thickness because the percent volume of solids in most paints is known. Alternatively, the correlation can be determined from actual dry-film thickness measurements taken at several areas. The readings of magnetic instruments for measurement of dry film thickness are often misinterpreted because they depend upon a number of variables such as initial calibration, type of cleaning, blast pattern profile, amount of mill scale remaining, and relative hardness of the paint film. However, when properly used, both wet-film and dry-film measurements provide an indication of the thickness of the paint over the peaks of the surface profile.

The primary measuring device for most types of paint should be the wet-film thickness gauge used during actual painting, with proper correlation to the percent volume of solids in the paint being applied. When magnetic instruments are used as a check on the dry film, SSPC-PA2 should be used for the dry-film thickness measurement.

*10.2.2. What frequency of paint film thickness inspection is appropriate?*

A sampling plan is defined in SSPC-A2 on the basis of the square footage of the structure being painted, which is useful for field painting applications. For sampling in shop painting applications, AISC recommends that 2 members be tested in every 25 tons or each shop layout of pieces to be painted. Any deficiencies in paint thickness or other specification requirements must be called to the attention of the fabricator by the owner/inspector at the time of completion of painting.

*10.2.3. Is a thicker paint film thickness than required acceptable?*

Yes. Because the specified paint thickness is usually a minimum requirement, greater thickness is permitted if it does not cause excessive mud cracking, runs, sags, or other defects of appearance or function.

### **10.3. Surface Preparation Requirements**

*10.3.1. What surface preparation should be specified for steel that is to remain unpainted?*

Steel that is to remain unpainted need only be cleaned of heavy deposits of oil and grease by appropriate means after fabrication. If other considerations dictate more stringent cleaning requirements, an SSPC-SP2 or other appropriate grade of cleaning should be specified in the contract documents.

*10.3.2. What level of surface preparation is specified for painted surfaces in the AISC Code of Standard Practice?*

As indicated in AISC *Code of Standard Practice* Section 6.5.2, in the absence of other requirements in the contract documents, the fabricator hand cleans the steel of loose rust, loose mill scale, dirt, and other foreign matter, prior to painting, by means of wire brushing or by other methods elected by the fabricator, to meet the requirements of SSPC-SP2 (hand tool cleaning).

*10.3.3. Is it permissible for a fabricator to perform surface preparation beyond that called for in the contract documents?*

Yes, unless prohibited in the contract documents.

*10.3.4. What degree of cleaning is implied when surfaces are indicated to be "blast cleaned"?*

When blast-cleaned surfaces are specified in contract documents without identification of the desired degree of cleaning, SSPC-SP6 (commercial blast cleaning) is assumed.

*10.3.5. Where are surface cleaning requirements defined?*

The acceptance criteria for the degree of preparation are specified in SSPC-VIS-1, *The Pictorial Surface Preparation Standards for Painting Steel Surfaces*, for all SSPC surface preparation levels (SP1 through SP10).

*10.3.6. How is the blast profile inspected?*

When blast profile limits are specified, a Keane-Tator profile comparator, or equivalent, is acceptable for spot checking representative production blasting. Note that the specified profile range must be evaluated relative to the profile of the steel prior to blasting. Therefore, the total profile range will usually be greater than the range specified.

*10.3.7. When inspection of surface preparation is required, when should such inspection be made?*

When inspection is required in the contract documents, it should be made as soon as practical after the surface has been prepared. Inspection should be scheduled to avoid delays in the fabrication shop. Additionally, because the adequacy of surface preparation cannot be readily verified after painting, it should be inspected prior to application of the primer coat.

*10.3.8. What edge preparation is required for painting?*

Generally none, however, because a wet paint film is drawn by surface tension to a lesser thickness over sharp edges, some paint system specifications for severe exposures call for special edge treatments, such as grinding a light chamfer on sharp edges, striping corners or edges with shop paint to increase film thickness, or grinding corners to a minimum  $\frac{1}{16}$ -in. radius. It should be noted that the term radius has precise meaning and an attempt is sometimes needlessly made to check corners with a radius template and require repairs at corners that do not conform closely to the specified radius. Because there is no significant difference in paint film thickness or life between a beveled corner and a corner that is ground to a small radius such treatment of edges is discouraged unless specified in the bid documents or in the paint manufacturer's directions. When required, edge treatment requirements should be limited to "breaking" the corner (eliminate the sharp 90 degree edge) with no reference to a specific dimension.

#### **10.4. SSPC Surface Preparation Levels**

*10.4.1. What is the appropriate acceptance criteria for surface preparation in accordance with either SSPC-SP2 or SSPC-SP3?*

While AISC *Code of Standard Practice* Section 6.5.2 calls for the removal of loose rust, loose mill scale, etc., the lack of specific definition (especially as to what constitutes "loose" mill scale) leaves the acceptance criteria subject to varying interpretation for both SSPC-SP2 (hand tool cleaning) and SSPC-SP3 (power tool cleaning). A mutually acceptable standard should be agreed upon by the owner so that the architect or engineer may knowledgeably design the paint system and the fabricator may realistically furnish the degree of surface preparation required.

*10.4.2. When SSPC-SP6 surface preparation is specified, what acceptance criteria should be applied?*

As stated in SSPC-SP6 (commercial blast cleaning) Section 2.2, "staining shall be limited to no more than 33 percent of each square inch of surface area and may consist of light shadows, slight streaks, or minor discolorations caused by stains of rust, stains of mill scale or stains of previously applied paint. Slight residues

of rust and paint may also be left in the bottoms of pits if the original surface is pitted.” Because specifying this requirement for *each square inch* is impractically restrictive, AISC recommends that this requirement be applied instead to the total surface area.

*10.4.3. When SSPC-SP10 surface preparation is specified, what acceptance criteria should be applied?*

As stated in SSPC-SP10 (near-white blast cleaning) Section 2.2, “staining shall be limited to no more than 5 percent of each square inch of surface area and may consist of light shadows, slight streaks, or minor discolorations caused by stains of rust, stains of mill scale or stains of previously applied paint.” Because specifying this requirement for *each square inch* is impractically restrictive, AISC recommends that this requirement be applied instead to the total surface area.

## **10.5. Field Touch-up and Repair**

*10.5.1. How should contract documents address the problem of job-site mill-scale flaking?*

When SSPC-SP2 (hand tool cleaning) or SSPC-SP3 (power tool cleaning) surface preparation is specified and a short-exposure-life prime coat is subsequently applied, tight mill scale generally remains on the surface prior to shop painting. Likewise, tight mill scale may remain with SSPC-SP7 (brush-off blast cleaning) surface preparation. Depending upon the time of exposure, job-site conditions, and type of prime coat, some of this tight mill scale may loosen, resulting in mill-scale flaking. When required, provision should be made in the contract documents for an appropriate field touch-up and repair program. Traditionally, this work has been delegated to a painting contractor.

*10.5.2. Is the fabricator/erector responsible to clean steel after it has been erected?*

No. Shop-painted steel that is stored in the field pending erection should be kept free of the ground and so positioned as to minimize water-holding pockets, dust, mud, and other contamination of the paint film. However, because site conditions are frequently muddy, sandy, dusty, or a combination of all three, it may be impossible to store and handle the steel in such a way as to completely avoid accumulation of mud, dirt, or sand on the surface of the steel. When required, provision should be made in the contract documents for an appropriate cleaning program.

*10.5.3. Is the fabricator/erector responsible for field touch-up to the repair of blemishes and abrasions that result during handling and storage after painting?*

No. During storage, loading, transport, unloading, and erection, blemishes and abrasions caused by slings, chains, blocking, tie-downs, etc. occur in varying degrees and should be expected. Responsibility for field touch-up should be assigned in the contract documents. Traditionally, this work has been delegated to a painting contractor.

## 10.6. Other General Information

### 10.6.1. *When welded surfaces are to be painted, what considerations are required?*

Some by-products of welding may be detrimental to paint performance and should be removed or neutralized prior to painting. Slag, chemical residue, and spatter compounds other than weld metal that are determined to be incompatible with the coating system should be removed or neutralized. Compatible residue, spatter compounds, and spattered weld metal that cannot be removed by hand scraping need not be removed provided that it is not detrimental to the performance of the structure or paint system.



# CHAPTER 11

## FIRE PROTECTION

### 11.1. Fire Protection Systems

- 11.1.1. What surface preparation should be specified for steel that is to be fireproofed?*
- Steel that is designated to receive a field-applied contact-type fireproof coating should be shop cleaned of dirt, oil, grease, and loose mill scale by appropriate means. Rust, dirt, and other materials that might impair bond that accumulates between the time of fabrication and the time of application of the fireproof coating is not the responsibility of the fabricator/erector; such responsibility should be assigned in the contract documents.

### 11.2. Fire Exposure

- 11.2.1. What procedures should be followed when assessing steel that has been exposed to a fire?*

Dill (1960) concludes that, while exposure to fire will almost certainly cause warping and twisting of members, it does not inevitably follow that the strength of the steel is reduced. It is almost certain that any steel that has been heated hot enough to undergo damaging grain coarsening or that has been cooled rapidly enough to harden it will be so badly distorted that it would have no consideration for re-use anyway. This leads to the general statement that steel that has been through a fire but that can be made dimensionally re-usable by straightening with the methods that are available may be continued in use with full expectation of performance in accordance with its original specified mechanical properties. Essentially then, the question is one of economics: if the steel can be straightened for less money than fabricating and installing a new piece, then that should be done.

Two possible exceptions to the above include quenched and tempered structural steels and high-strength fasteners. The mechanical properties of such heat-treated items may be affected by prolonged fire exposure and should be tested to determine the effects of the fire, if any.

Another reference is Council on Tall Buildings and Urban Habitat (1980).



## APPENDIX A

### AISC QUALITY CERTIFICATION

The AISC Quality Certification Program confirms that an AISC-certified structural steel fabricating plant has the personnel, organization, experience, procedures, knowledge, equipment, capability and commitment to produce fabricated structural steel of the required quality for a given category of structural steelwork. The following commentary covers some of the common questions about the AISC Quality Certification Program. A complete description is available in the AISC LRFD Manual beginning on page 6-477.

*A1. How long does it take for a fabricator to become certified?*

The most difficult part of the certification process is assembling the information that forms the quality assurance (QA) manual portion of the application. This information should reflect a true picture of what is happening in the facility as well as meet the minimum requirements of the AISC Certification Program. After AISC has received the completed application, it takes approximately eight weeks to review and conduct the audit. Under normal circumstances, the fabricator will know the result of the audit when it is complete. The paperwork after the audit can take six weeks or more, but an intermediate letter showing successful completion of the certification process can be provided upon request, usually within one week after the audit. Note that the certification process must be completed prior to the bidding of work as an AISC Quality Certified fabricator.

*A2. What does certification require that a fabricator doesn't normally do?*

The philosophy of the program is to require only those procedures and practices that are common to well run fabricator organizations and required by the common specifications. Most requirements are based upon common fabrication specifications with which the appropriate people in a fabricator's organization should be familiar. While the following is not an all inclusive list, AISC recommends particular attention to the *AISC LRFD Specification for Structural Steel Buildings* Chapter M, *AISC Code of Standard Practice*, AWS D1.1 Sections 2, 3, 4, 5, and 6, and the ordering requirements in ASTM specifications including ancillary items like bolts and galvanizing.

Other requirements that are not based upon common specifications, such as design and detail logs, are considered a necessity for control of a quality oriented organization. These logs must uniquely identify contract documents, detail drawings, and instructions to the shop. Dates and revision numbers must be tracked. Material including *drops* or *crops* (pieces returned to stock from processing) must be identified with a grade of material. If the fabricator's procedures call for a purchase order or heat number to be shown, it will be looked for during the review.

A procedure for traceability may be required by the checklist; if it is, it must be available, though this procedure need not be implemented except on jobs specifying it. If a current or past job required traceability, it is the prerogative of the auditor to both review the records of that job to see that the procedure was followed and look through the stock yard to see material marked with the required information.

Procedures indicated in the checklist as written must be on paper. Procedures not specifically noted to be written need not be, but the auditor will ask what those procedures are and will expect to see evidence that they are followed. Because it is most common that procedures affect more than one person or department, all parties affected should be familiar with their part of the procedure. Note that the two main parts to quality systems are planning and communication. The plan is no good if those using it do not know it.

A3. *What constitutes separation of quality from production?*

To some degree this may depend on the level of certification, the size and sophistication of the fabricator's operation, and the type of work it does. In general, acceptance criteria and disposition of non-conforming material must be established by management outside the production department. Furthermore, a high-level general manager outside the production department should be responsible for reviewing the effectiveness of quality control functions and establishing the level and performance of quality control functions.

It is acceptable and expected to have inspectors receive lists of the pieces subject to inspection from production supervisors. It is desirable to have inspectors report problems to production supervisors as well as those responsible for disposition of those problems.

Again, the need for a distinct quality assurance manager depends upon the size and sophistication of the operation. Quality assurance functions can be performed by an engineering manager, general manager or project manager.

A4. *If a fabricator does not do certain types of work that are included in a specific category, does that fabricator still have to demonstrate a capability to do those types of work, despite having no intent to sell that kind of product?*

In general, yes. For example, if certification is desired for AISC QC Category Major Steel Bridges, such capability must be demonstrated, even if the intent is to fabricate only buildings or components; owners specifying such a category in a contract assume such capability exists.

There are specific exceptions that can be determined from the checklist. For example, while it is not necessary to maintain an in-house drafting room, a person capable of reviewing and directing outside drafters is necessary. Similarly, it is not necessary to maintain an in-house painting facility, but in some levels of certification, a qualified outside painter and practices for controlling this outside painter are required. Conversely, if a fabricator tells an auditor he or she intends to fabricate continuous bridges, when asked where the assemblies are done, the fabricator will be expected to show an area big enough to assemble three typical spans and the crane capacity to lift them. Such facility requirements are dependent upon the type of work the fabricator intends to do.

A5. *How can a fabricator demonstrate the capability to do and get certified for work that he or she has not done before?*

Where it is possible, the fabricator should run a job as if it were the type for which he or she is trying to get certified. For example, run a job as if it were a fracture critical (FC) job: make up purchase orders for FC material, work up the repair procedures, weld procedures, PQRs (without the qualification test), etc. Documentation of such training demonstrates capability.

As a rule, the auditor will demand to see evidence that systems function. It is recognized that companies that are new to the program or entering a new market may develop a system in preparation for certification. AISC expects to see records of most systems going back in time but will accept one or two systems with records going back one month. AISC will not accept an untested procedure as evidence of capability where it was possible or appropriate for that procedure to have been implemented.

*A6. How quickly can a facility be re-evaluated after an unsuccessful audit?*

Once the commitment has been made to become certified, the initial review points out deficiencies that were found. Many companies are not certified in their first evaluation, but address the findings and become certified soon after the initial review. That is not to say that all problems are discovered in one review; it is common to find system defects as the program progresses. Usually the findings in subsequent audits can be addressed quickly or are not essential and certification can continue.

After an unsuccessful review, the time to return depends upon the nature and magnitude of the deficiencies and is determined by AISC and the auditor. Systematic deficiencies must be corrected and put into operation for at least one month to show evidence of a functioning system.

*A7. Can one get certified in a lower category without another review?*

Auditors of the AISC Quality Certification program occasionally find that a facility that fails the level of certification requested could pass a lower level of certification. AISC reserves the right to grant certification for a level that is lower than that requested.

*A8. Can one get help in preparing for the Certification?*

Although AISC does offer *A Guide to Becoming Certified*, a publication that is helpful in many respects, AISC is not equipped to train companies in quality systems or fabrication practices for the purpose of becoming certified. There are, however, consultants in the quality systems business, some of which are familiar to a varying degree with the AISC QC Program. AISC, however, is not familiar with their work nor aware of the cost of their services. Furthermore, AISC makes no recommendation regarding any consultant. Though it may be of assistance, the use of a consultant will not guarantee success.

AISC cautions that the use of an outside source for creation of a quality-related procedures manual can be detrimental. Because the auditor will review against the manual submitted by the fabricator, lack of compliance with that manual will result in failure. While it may be advantageous to use outside sources for ideas it is most effective to have those performing the procedures be an integral part of the writing of the procedures. Note that AISC does not distribute sample QC manuals.



## APPENDIX B

### OTHER ORGANIZATIONS AND USEFUL DOCUMENTS

While limited information from the following sources has already been incorporated into the text of this document, more detailed and other useful information can also be found. See also Referenced Specifications, Codes, and Standards, and Bibliography.

#### AASHTO

444 N. Capitol Street, N.W., Suite 249, Washington, DC 20001  
202/624-5800 voice  
202/624-5806 fax

#### ACI

22400 West Seven Mile Road, P.O. Box 19150, Detroit, MI 48219-0150  
313/532-2600 voice  
313/538-0655 fax

#### AISC

One East Wacker Drive, Suite 3100, Chicago, IL 60601-2001  
312/670-5414 voice  
312/670-5403 fax

The *Engineering Journal* is the quarterly technical journal published by AISC. Peer-reviewed articles on topics of interest may be found in the *Engineering Journal 30-Year Annual Index (1964-1993)*. An annual index is found at the end of each 4th Quarter issue for all years since 1993.

The *Proceedings* of the AISC National Steel Construction Conference contains many papers of interest. While these papers are written by competent individuals, in most cases they have not been peer reviewed. Topics and authors of interest can be found in the *AISC Conference Proceedings 48-year Index (1949-1963, 1980-1996)*. Note that from 1964 to 1979, Conference papers were printed in the *AISC Engineering Journal*.

The AISC Design Guide series provides a synthesis of available information on specific topics, such as:

1. Column Base Plates (DeWolf and Ricker, 1990)
2. Steel and Composite Beams with Web Openings (Darwin, 1990)
3. Serviceability Design Considerations for Low-rise Buildings (Fisher and West, 1990)
4. Extended End-Plate Moment Connections (Murray, 1990)
5. Design of Low- and Medium-Rise Steel Buildings (Allison, 1991)
6. Load and Resistance Factor Design of W-shapes Encased in Concrete (Griffis, 1992)
7. Industrial Buildings from Roofs to Column Anchorage (Fisher, 1993)
8. Partially Restrained Composite Connections (Leon et al., 1996)

9. Torsional Analysis of Structural Steel Members (Seaburg and Carter, 1997)
10. Erection Bracing of Low-Rise Structural Steel Frames (Fisher and West, 1997)
11. Floor Vibrations Due to Human Activity (Murray et al., 1997)

The development of AISC Design Guides is ongoing.

While primarily non-technical in nature, AISC's *Modern Steel Construction* Magazine regularly contains articles of interest in engineering decision-making. Articles on topics of interest may be found in the *Modern Steel Construction 15-Year Index (1980–1995)*.

#### AISI

1101 17th Street, N.W., Suite 1300, Washington, DC 20036-4700  
202/452-7100 voice  
202/463-6573 fax

#### ANSI

11 West 42nd Street, New York, NY 10036  
212/642-4973 voice  
212/398-0023 fax

#### API

1220 L Street, N.W., Washington, DC 20005  
202/682-8000 voice  
202/682-8115 fax

#### ASCE

1801 Alexander Bell Drive, Reston, VA 20191-4400  
800/548-ASCE or 703/295-6000 voice  
703/295-6222 fax

The ASCE Committee on Design of Steel Buildings Structures has undertaken the task of resolving questions of long-standing interest in the design office. The results of their efforts have been published in an ongoing series of papers in the *ASCE Journal of Structural Engineering* including the following:

- “Wind Drift Design of Steel-Framed Buildings: State-of-the-Art Report,” September, 1988.
- “Compendium of Design Office Problems,” December, 1992.
- “Compendium of Design Office Problems (Volume II),” February, 1996.

In addition to these specific papers, articles on other topics of interest may be found in the *ASCE Structures Journal*.

#### ASTM

1916 Race Street, Philadelphia, PA 19103  
215/299-5400 voice  
215/977-9679 fax



AWS

550 N.W. LeJeune Road, P.O. Box 351040, Miami, FL 33135  
800/443-9353 or 305/443-9353 voice  
305/443-7559 fax

NAAMM

Association Headquarters, 600 South Federal, Suite 400, Chicago, IL 60605  
312/201-0101 voice  
312/922-2734 fax

SSPC

40 24th Street, Pittsburgh, PA 15222  
412/281-2331 voice  
412/281-9992 fax



## REFERENCED SPECIFICATIONS, CODES, AND STANDARDS

### AISC

LRFD *Specification for Structural Steel Buildings*, December 1, 1993  
 Specification for LRFD of Single-Angle Members, December 1, 1993  
*Specification for the Design of Steel Hollow Structural Sections*, April 15, 1997  
*Seismic Provisions for Structural Steel Buildings*, April 15, 1997  
*Code of Standard Practice for Steel Buildings and Bridges*, June 15, 1992

### ACI

ACI 318-95

### ANSI

A1264.1-89

### ASCE

ANSI/ASCE 7-95

### ASTM

|               |                |               |
|---------------|----------------|---------------|
| A6/A6M-96b    | A449-93        | A847-93       |
| A36/A36M-96   | A490-93        | A898/A898M-96 |
| A53-96        | A490M-93       | F436-93       |
| A194/A194M-96 | A500-93        | F436M-93      |
| A307-94       | A563-94        | F959-96       |
| A325-96       | A563M-93       | F959M-94      |
| A325M-93      | A572/A572M-94c | F1554-94      |
| A354-95       | A588/A588M-94  |               |
| A435/A435M-96 | A770/A770M-96  |               |

### AWS

D1.1-96 *Structural Welding Code—Steel*

### RCSC

LRFD *Specification for Structural Joints Using ASTM A325 or A490 Bolts*,  
 June 3, 1994



## BIBLIOGRAPHY

- Allison, H., 1991, *Design of Low- and Medium-Rise Steel Buildings*, AISC, Chicago, IL.
- American Institute of Steel Construction, 1997a, *Specification for the Design of Steel Hollow Structural Sections*, AISC, Chicago, IL.
- American Institute of Steel Construction, 1997b, *Seismic Provisions for Structural Steel Buildings*, AISC, Chicago, IL.
- American Institute of Steel Construction, 1994, *LRFD Manual of Steel Construction*, 2<sup>nd</sup> Edition, AISC, Chicago, IL.
- American Institute of Steel Construction, 1993a, *LRFD Specification for Structural Steel Buildings*, AISC, Chicago, IL.
- American Institute of Steel Construction, 1993b, *Specification for LRFD of Single-Angle Members*, AISC, Chicago, IL.
- American Institute of Steel Construction, 1992, *Code of Standard Practice for Steel Buildings and Bridges*, AISC, Chicago, IL.
- American Institute of Steel Construction, 1988, *Quality Criteria and Inspection Standards*, AISC, Chicago, IL.
- American Institute of Steel Construction, 1973, "Commentary on Highly Restrained Welded Connections," *Engineering Journal*, Vol. 10, No. 3, (3rd Qtr.), AISC, Chicago, IL.
- American National Standards Institute, 1981, *Specifications for Underhung Cranes and Monorail Systems*, MH27.1, ANSI, New York, NY.
- American Society of Civil Engineers, 1995, *ASCE 7 Minimum Design Loads for Buildings and Other Structures*, ASCE, Reston, VA.
- American Welding Society, 1996, *Structural Welding Code—Steel, D1.1*, AWS, Miami, FL.
- Anderson, J. P. and Woodward, J. H., 1972, "Calculation of Effective Lengths and Effective Slenderness Ratios of Stepped Columns," *Engineering Journal*, Vol.9, No. 4, (October), AISC, Chicago, IL.
- Association of Iron and Steel Engineers, 1991, *Technical Report No. 13: Specifications for the Design and Construction of Mill Buildings*, AISE, Pittsburgh, PA.
- Bigos, J., Smith, G. W., Ball, E. F., and Foehl, P. J., 1954, "Shop Paint and Painting Practice," *Proceedings of the 1954 AISC National Engineering Conference*, AISC, Chicago, IL.
- Brockenbrough, R. L. and Johnston, B. G., 1981, *USS Steel Design Manual*, United States Steel Corporation, Pittsburgh, PA.

- Carter, C. J., 1996, "Specifying Bolt Length for High-Strength Bolts," *Engineering Journal*, Vol. 33, No. 2, (2nd Qtr.), pp. 43–54, AISC, Chicago, IL.
- Council on Tall Buildings and Urban Habitat, 1980, *Monograph on Planning and Design of Tall Buildings, Volume CL, Tall Building Criteria and Loading*, ASCE, Reston, VA.
- Darwin, D., 1990, *Steel and Composite Beams with Web Openings*, AISC, Chicago, IL.
- DeWolf, J. T. and Ricker, D. T., 1990, *Column Base Plates*, AISC, Chicago, IL.
- Dill, F. H., 1960, "The Effects of Fire on Structural Steel," *Proceedings of the 1960 AISC National Engineering Conference*, AISC, Chicago, IL.
- Easterling, W. S., Gibbings, D. R., and Murray, T. M., 1993, "Strength of Shear Studs in Steel Deck on Composite Beams and Joists," *Engineering Journal*, Vol. 25, No. 1, (1st Qtr.), pp. 1–16, AISC, Chicago, IL.
- Fisher, J. M. and West, M. A., 1997, *Erection Bracing of Low-Rise Structural Steel Frames*, AISC, Chicago, IL.
- Fisher, J. M., 1993, *Industrial Buildings—Roofs to Column Anchorage*, AISC, Chicago, IL.
- Fisher, J. M. and West, M. A., 1990, *Serviceability Design Considerations for Low-Rise Buildings*, AISC, Chicago, IL.
- Galambos, T. V., 1988, *Guide to Stability Design Criteria for Metal Structures*, 4th Edition, p. 168, John Wiley & Sons, New York, NY.
- Griffis, L. G., 1992, *Load and Resistance Factor Design of W-shapes Encased in Concrete*, AISC, Chicago, IL.
- Iwankiw, N. R. and Schlafly, T. J., 1982, "Effect of Hole-Making on the Strength of Double Lap Joints," *Engineering Journal*, Vol. 19, No. 3, (3rd Qtr.), AISC, Chicago, IL.
- Kulak, G. L., Fisher, J. W., and Struik, J. H. A., 1987, *Guide to Design Criteria for Bolted and Riveted Joints*, John Wiley and Sons, New York, NY.
- Leon, R. T., Hoffman, J. J., and Staeger, T., 1996, *Partially Restrained Composite Connections*, AISC, Chicago, IL.
- Murray, T. M., 1997, *Floor Vibrations Due to Human Activity*, AISC, Chicago, IL.
- Murray, T. M., 1990, *Extended End-Plate Moment Connections*, AISC, Chicago, IL.
- Research Council on Structural Connections, 1994, *LRFD Specification for Structural Joints Using ASTM A325 or A490 Bolts*, AISC, Chicago, IL.
- Ricker, D. T., 1988, "Field Welding to Existing Structures," *Engineering Journal*, Vol. 30, No. 2, (2nd Qtr.), pp. 44–55, AISC, Chicago, IL.
- Ricker, D. T., 1982, "Tips for Avoiding Crane Runway Problems," *Engineering Journal*, Vol. 19, No. 4, (4th Qtr.), pp. 181–205, AISC, Chicago, IL.

- Schilling, C. G. and Kleppstein, K. H., 1978, "New Method for Fatigue Design of Bridges," *Journal of the Structural Division*, Vol. 104, No. ST3, (March), pp. 425-438, ASCE, New York, NY.
- Seaburg, P. A. and Carter, C. J., 1997, *Torsional Analysis of Structural Steel Members*, AISC, Chicago, IL.
- Sherman, D.R. and Ales, J. M., 1991, "The Design of Shear Tabs with Tubular Columns," *Proceedings of the 1991 AISC National Steel Construction Conference*, AISC, Chicago, IL.
- Steel Structures Painting Council, 1991, *Steel Structures Painting Manual, Volume II, Systems & Specifications*, Sixth Edition, SSPC, Pittsburgh, PA.
- Steel Structures Painting Council, 1982, *Steel Structures Painting Manual, Volume I, Good Painting Practice*, Second Edition, SSPC, Pittsburgh, PA.
- Tanner, N. S., 1985, "Allowable Bending Stress for Overhanging Monorails," *Engineering Journal*, Vol. 22, No. 3, (3rd Qtr.), pp. 133-138, AISC, Chicago, IL.
- Thornton, C. H., 1973, "Quality Control in Design and Supervision Can Eliminate Lamellar Tearing," *Engineering Journal*, Vol. 10, No. 4, (4th Qtr.), pp. 112-116, AISC, Chicago, IL.
- Thornton, W. A., 1996, "A Rational Approach to the Design of Tee Shear Connections," *Engineering Journal*, Vol. 33, No. 1, (1st Qtr.), pp. 34-37, AISC, Chicago, IL.
- Thornton, W. A., 1995, "Treatment of Simple Shear Connections Subject to Combined Shear and Axial Forces," *Modern Steel Construction*, September, pp. 9-10, AISC, Chicago, IL.
- Tide, R. H. R. and Krogstad, N. V., 1993, "Economical Design of Shelf Angles," *Proceedings of the Symposium on Masonry: Design and Construction, Problems and Repair*, STP 1180, American Society for Testing and Materials, Philadelphia, PA.
- Yura, J. A. and Helwig, T. A., 1995, Lecture notes for SSRC/AISC seminar "Bracing for Stability", AISC, Chicago, IL.





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