A Guide to

Engineering and Quality Criteria for Steel Structures

Common Questions Answered

بخش دوم

Fourth Edition

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 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 ⁵⁄₁₆-in. thick; two 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 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 ⁵⁄₁₆-in.-thick ASTM F436 washers must be used; two 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 ¾-in. diameter to 0.79 for 1⅛-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.[4] 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 ¾-in. and ⅞-in. diameter bolts if the ply thickness closest to the nut is not less than ⅜-in.; the same is true for 1-in. and 1⅛-in. diameter bolts if the ply thickness closest to the nut is not less than ½-in. With one washer under the nut, these values may be reduced by ⅛-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.

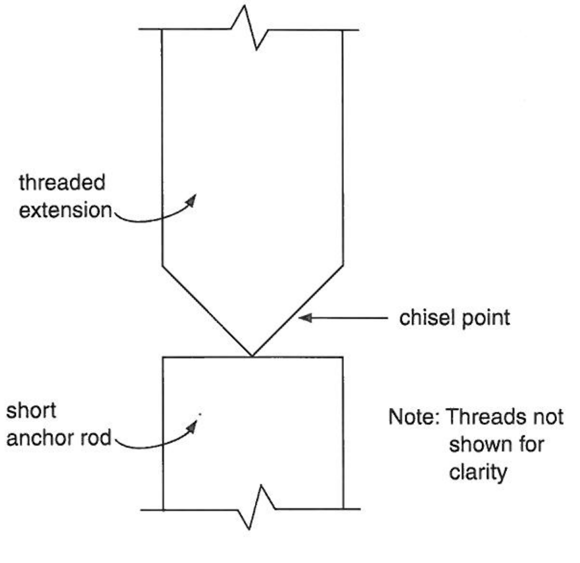


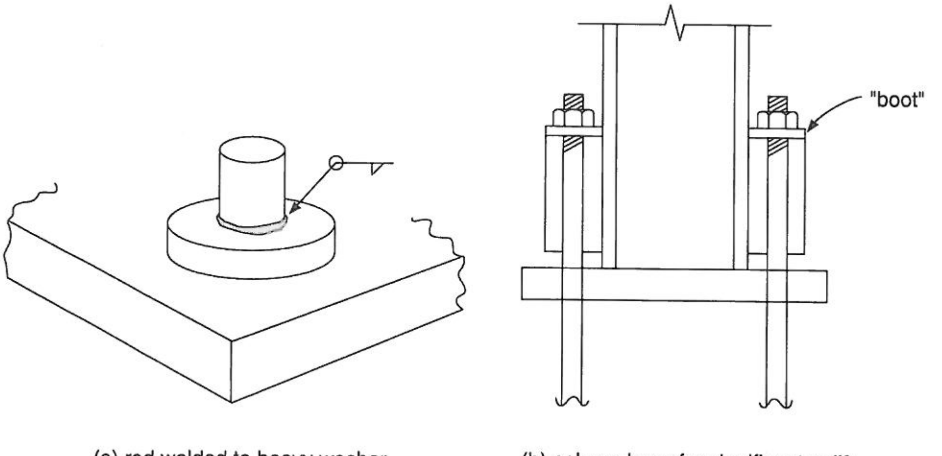
Fig. 7.1.5-1. Beveled end of threaded extension to be welded to short anchor rod.

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 π 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.



(a) rod welded to heavy washer

(b) column base for significant uplift

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 $\phi(0.85f_c'A)$ 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 π 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 ½-in. fillet weld uses four times as much weld metal as a ¼-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-penetration 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

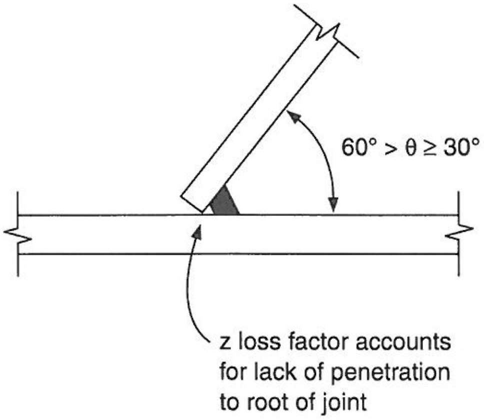


Fig. 8.2.2-1. z loss factor.

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 $1/16$-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

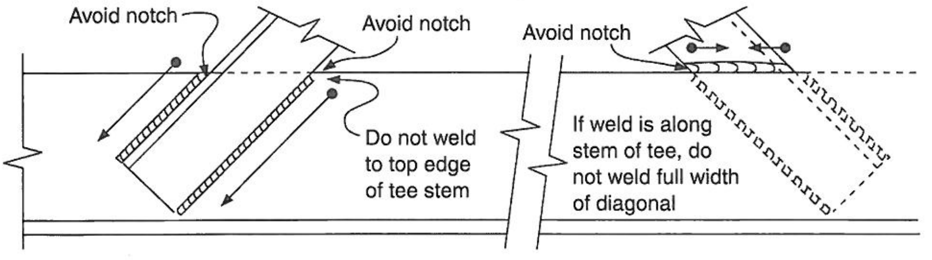


Figure 8.3.5-1.

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

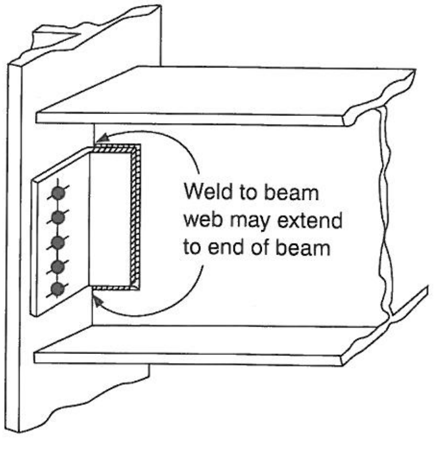


Figure 8.3.5-2.

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 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 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 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 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

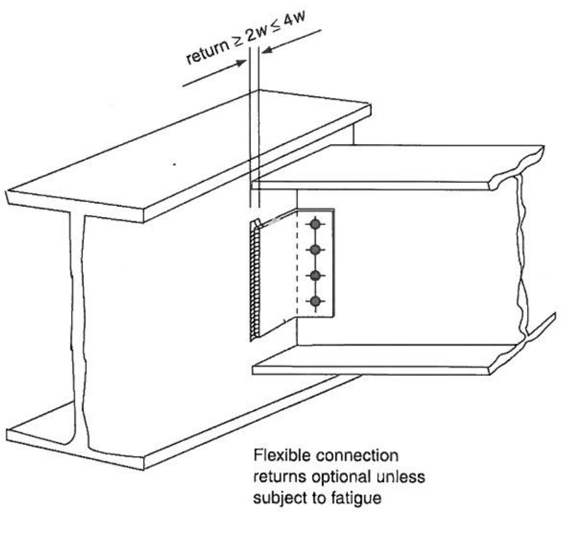


Figure 8.3.5-3.

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 $1/16$-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

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