5-20.1 Shop Drawings

The fabricator's detailed shop drawings and steel erection drawings, after review by WisDOT, become a part of the project plans and are used in conjunction with the general plans. During steel erection, the engineer should make sure all members are placed in the proper position in the structure by checking match marks or identification marks on the members with the location shown on the erection drawing.

WisDOT does not approve shop drawings. WisDOT's review of the drawing means only an acceptance of the character and sufficiency of the details. The contractor is responsible for any errors and omissions.

5-20.2 Shop Inspection

Structural steel is normally delivered to the job some time before receipt of shop inspection reports; there may be a question on the part of field personnel as to whether the delivered material was actually inspected in the shop.

The engineer should, in the absence of shop inspection reports, check that the structural steel has been shop inspected by contacting the Bureau of Structures (BOS).

An effort should be made to provide shop inspection reports before erection. Structural steel erection may be delayed in those instances where a shop inspection report has not been received and the structural integrity is doubted. In this instance safety could be compromised, until the BOS is contacted by the engineer and the problem is resolved. If the structural integrity is not doubted and safety is not an issue, then immediate proof of inspection is not required, although still necessary before closing the contract. This could also result in field repairs if the piece was not inspected and repairs are needed.

Occasionally, structural steel may be shop inspected by agencies other than BOS. When structural steel comprising the stress-carrying members of a structure is delivered to a job site, and the steel has been shop inspected by other than BOS (other states, consultants, inspection agencies, etc.), the region should arrange through BOS to have an on-site inspection made by an experienced shop inspector. This is not intended to be a complete inspection as to dimensions, etc., but rather to look for the type of defects that result from faulty fabrication and welding procedures.

The foregoing does not apply to minor items such as railings, anchor assemblies, signposts and other miscellaneous items that are frequently shipped from remote locations without plant inspection. The intent is that they will be inspected in the field. The engineer or designee will continue to make such field inspections and document them by field inspection reports.

5-20.3 Field Handling and Storage of Structural Steel

Standard spec 506.3.24 and standard spec 506.3.27, related to handling, storage, and erection of structural steel describes methods of storage and erection to avoid damage to steel members. Steel should be stored in areas adjacent to the work where it will not be subject to flooding or other hazards. Care should be exercised when storing steel on skids placed on frozen ground to select areas which, when thawed, will still provide a uniform support, without uneven settlement of the skids resulting in twisting of the members. Two or more pickup points must be used when handling girders 50 ft in length or longer. When picked up with a single crane, a suitable balance beam or spreader bar must be used when 2 or more pickup points are utilized. When epoxy-painted girders are moved, pads must be provided so paint is not damaged by cables or slings.

5-20.4 Bearings

5-20.4.1 General

Before erection of structural steel is started, the centerline of bearing should be laid out on all substructure units by precise methods, and the roadway centerline (or reference line) and centerline of bearings scratched or otherwise marked on the concrete. Bearing areas should be checked to see that a plane surface is provided at the correct elevation. If the concrete surface that will be in contact with the bearing pad is rough or irregular, it should be ground to provide full and uniform bearing. If a bearing area is low with respect to other areas on the unit or in relation to other units of the structure, shims of the same size as the masonry plate and of the required thickness should be ordered. Avoid using a number of thin shims if a single shim of the required thicknesses can be made from standard thicknesses of plates. The shims should be made from the same type of steel as specified for the bearings.

5-20.4.2 Bearing Pads

Bearing pads intended for use on bridge seats may consist of sheet lead or preformed fabric pads, or when so

designated, may be of the elastomeric type. Only one type of bearing pad should be used throughout any one structure unless otherwise provided on the plans or in the contract. Preformed fabric pads shall be Class A.

5-20.5 High Strength Bolts, Nuts, and Washers

5-20.5.1 Bolts

Updated guidance for bolting.

High strength bolts used on steel connections must conform to ASTM F3125 grade A325 or grade A490. ASTM A307 bolts, often referred to as "common" bolts, are not considered "high strength" bolts and should not be used unless otherwise provided on the plans or in the contract. High strength bolts shall be of the heavy hexagon head style and either Type 1 (used on steels other than weathering steel) or Type 3 (used on weathering steel). Plain bolts are typically only allowed for Type 3 bolts. Coated bolts shall be hot-dip galvanized.

A325 bolts are a medium carbon steel with a minimum yield of 120 ksi. A490 bolts are a heat treated steel with a minimum yield of 150 ksi. A325 bolts may be galvanized, however A490 bolts are not to be galvanized due to hydrogen embrittlement. Since most bolts require a hot-dip galvanized coating A490 bolts are usually not allowed.

All bolts shall be marked on the top of the bolt head (e.g. Type 1 bolts should be marked "A325" and Type 3 bolts should be marked "A325").

Note: A325 and A490 bolts are now covered by ASTM F3125 under grades A325 and A490 respectfully. Refer to ASTM F3125 for additional information.

5-20.5.2 Nuts

Nuts used on high strength bolts must conform to ASTM A563 and provide a similar protective coating as the bolt (e.g. a coated Type 1 bolt must use zinc-coated DH nuts and a plain Type 3 bolt must use plain C3 or DH3 nuts).

5-20.5.3 Washers

Washers used on high strength bolts must conform to ASTM F436 and provide a similar protective coating as the bolt (e.g. a coated Type 1 bolt shall use zinc-coated Type 1 washers and a plain Type 3 bolt must use plain Type 3 washers). Flat washers are most commonly used. Tapered washers may be required for sloped bolting surfaces, such as attaching to sloped girder flanges.

5-20.6 Assembly

5-20.6.1 General

Bearing surfaces and all surfaces of metal that will be in contact must be free of rust, loose mill scale, dirt, oil, and grease. In addition, contact surfaces of beams and girder splices or main truss connections must be free of paint or lacquer, except that inorganic zinc rich primer is allowed on these surfaces. At this time there are no other exceptions to the paint or lacquer-free requirement.

The steel should fit together with very little strain or distortion. If bolt holes are only slightly out of alignment, it is usually possible to bring the pieces into their proper position with drift pins. However, if the holes fail to line up properly, to the extent that forcing the drift pin through would result in damage to the metal surrounding the hole, the hole should be drilled or reamed instead. Any fabrication error that cannot be corrected by a small amount of drifting, drilling, or reaming is cause for rejection of the material. Heavy sledging of the parts to bring them into alignment or making any cuts or adjustments with a flame-cutting torch should not be allowed. The entire structure, or as a minimum for continuous spans, the span and immediately adjacent continuous spans, should be assembled, drift pinned, bolted and adjusted to the proper grade and alignment in accordance with the plan blocking diagrams or the top flange elevations when shown on the plans, before permanent connections are made. If high strength bolts are to be used for the permanent connections, they may also be used for this "fitting up".

When deflections due to mass of steel are appreciable, the blocking diagrams should be adjusted to compensate for such deflection. Assembly and erection of continuous girders should be accomplished so the top flanges of the girders, as finally erected, are within 1/2 inch of the top flange or blocking diagram elevations, as shown on the plans or as adjusted in the field to compensate for appreciable dead load deflection.

Zinc coated A325 bolts shall not be reused. Other A325 bolts may be reused when approved by the engineer. Retightening previously tightened bolts, which may have been loosened by the tightening of adjacent bolts, shall not be considered for reuse.

High strength bolts are used for either "slip-critical" or "bearing" type connections. All high strength bolted connections on bridges are designed as slip-critical, unless otherwise provided on the plans or in the contract. Slip-critical type connections are used on bridges since connections are subject to stress reversals and slippage must not occur between the joined parts at service load levels. To prevent slippage, high-strength bolts are

installed to a specified initial tension, which causes an initial precompression (or clamping force) between the joining parts. The joining surfaces are referred to as the faying surfaces and require special surface preparations to ensure frictional resistances can be achieved. Steel connections are typically painted prior to assembly and will affect the slip resistance of the bolted connection. If the faying surface is painted, it will then require a blast cleaning and provide a Class A or B coating based on the design. Paints on the department's approved products list for Structure Painting Systems are qualified as Class A and B coatings.

Bearing type connections transfers loads by bearing the bolt shank against the side of the hole. This connection typically can be installed only to snug-tight condition since slippage is not critical. Bearing type connections may only be allowed when stated on the plans or in the contract, otherwise they are not allowed.

Connections shall be made using 3/4" high strength bolts, unless otherwise specified, and the same type should be used throughout the structure.

High-strength bolts may be tensioned using either the turn-of-nut method or the direct tension indicating (DTI)washer method.

5-20.6.2 Turn-of-Nut Method

This bolt tensioning method produces additional rotation of the nut beyond "snug tight". Snug tight is defined as the point where the power wrench ceases to rotate freely and starts to impact, and is also the approximate degree of tightness that can be obtained with an ordinary spud wrench. The entire connection should be snug tight and the faying surfaces should be in intimate contact before additional rotation is applied.

The engineer should be present for the QC testing and may verify bolt installations using a calibrated torque wrench. Torque tension determination should be documented in the field by using form DT2114 and kept in the permanent project file.

If no nut or bolt head is turned by this application of the inspection torque, the connection shall be accepted as properly tightened. If any nut or bolt head is turned by the application of the inspection torque, this torque should be applied to all bolts in the connection. All bolts whose nut or head are turned by the inspection torque must be tightened and re-inspected or, optionally, the fabricator or erector may retighten all bolts in the connection and then resubmit the connection for the specified inspection.

5-20.6.3 Load-Indicator-Washer Method

This bolt tensioning method uses a specially designed washer with protrusions on one side. This load-indicator-washer is placed, in most conditions, under the head of the bolt with the protrusions facing away from the surface of the steel member being connected. The entire connection should be snug tight before final tightening. As the nut is turned, the protrusions are partially flattened and the gap between the load-indicator-washer and the bolt head is reduced to a gap specified by the manufacturer. Testing is done by measuring the gap with a metal feeler gauge.

The inspector should inspect 10% or a minimum of 2 bolts chosen randomly from each connection on the structure. If any of the gaps tested exceed the minimum gap spacing, all the bolts in that connection must be retightened by the fabricator or erector as needed, and resubmitted for inspection.

5-20.6.4 Rotational Capacity Test

Rotational Capacity (RC) testing is required on all projects using A325 or A490 structural bolts in steel-to-steel connections unless the design engineer specifies otherwise.

On jobs requiring high strength bolts to have an RC test, those bolts should be received with the RC test performed at the supplier and in the field. RC lots need to be established with the lots and combination of lots on site that will be used in erection. Each RC lot includes the nut lot, bolt lot, and washer lot. Combinations that are not going to be used in these lots should be segregated to eliminate the possibility of mixing untested lots for field use. Tension and rotational capacity testing should be documented on form DT2113 and kept in the permanent project file.

5-20.6.4.1 Long Bolts

5-20.6.4.1.1 Testing Procedure

These tests are in addition to the Rotational Capacity test and are required to determine the effective installation of the bolt to a satisfactory tension. DT2114 contains the procedure necessary to perform these tests. If DTI washers are being used, the bottom portion of the DT2114 form may be excluded, and a feeler gauge used in lieu of torque testing.

5-20.6.5 Welding

5-20.6.5.1 Restrictions

The standard specifications require welding of steel structures to conform to AASHTO/AWS D 1.5, Bridge

Welding Code. Welding pile splices should be performed according to AWS D1.1. The contractor is required to designate a quality control inspector to inspect and certify that welding conforms to AWS D1.5 by completing form DT2320 and submitting it to the engineer for project record.

Section 6, Paragraph 6.5 of AASHTO/AWS D 1.5 provides that unspecified welds are not to be added without the approval of the engineer. Unspecified welding is welding not called for on the plans or in the specifications and is done by the contractor for the contractor's convenience in performing construction operations. An example is the welding to beams of fixtures such as form hangers, bar chairs and screed holders used in the placing of the deck concrete.

Generally, where field welding is provided for in the erection of a structure, it is limited to welding of secondary members. Any welding by the contractor on structural members to facilitate the construction operations should first be approved by the engineer. To assist the field inspector with the inspection of welds on primary members, notification is to be given to the Quality Management Section in advance of any welding of primary members. The Quality Management Section will then make arrangements to have a shop inspector present when such inspection is performed to assist project personnel in assessing the adequacy of welding procedures and quality of weld.

Due to the variable stresses set up in the steel where welding is performed and the consequent effect on the fatigue strength of the steel, unspecified welding will not be allowed in the tension areas of beams or other main stress carrying members. Generally, the tension portion of the top flange of a beam of a bridge can be identified as the area from which shear lugs are omitted. The plans generally contain a note prohibiting welding in this area. An example of allowed field welding is shown in Figure 1.

All welding of steel bridges shall be performed by certified welders or certified welder operators.

All welding of steel bridges shall be performed according to written welding procedures, which must be approved by the engineer. PQR's may be required for non standard base materials or filler metals.

Equipment, weld joint preparation and fit up, welding procedure, and qualification of welders should all be checked by a qualified inspector before welding.

Welding electrodes and filler materials must be properly stored. Low-hydrogen E7018 electrodes, once their sealed container is broken, should be stored in holding ovens if they will not be used within 4 hours. Fluxes and flux-cored electrodes need protection from moisture, rain, and high humidity. Welding rods that have cracked from over-drying, or those that have been rained on or have been thoroughly wetted in any manner, should be discarded and not used. Electrode containers that have been opened or received in a non-hermetically sealed cans or boxes are considered wet and should not be used.

Preparation for welding includes beveling ends of pieces which are to be butt welded, smoothing rough edges and thoroughly cleaning all surfaces where a weld is to be made, for a distance of not less than 25 mm beyond the edges of the weld on all sides. Surfaces should be perfectly clean and free from rust or oil. An approved weld-able primer may be used to maintain this condition. A thin coat of clean, pure linseed oil need not be removed.

For guidance in the selection and use of proper electrodes for use in welding exposed unpainted applications of AASHTO M270 Grade 345W steel, refer to Table 1.

The base metal to be welded shall be preheated as shown in Table 2. Steels with a chemical composition other than the ASTM grades listed in the figure are not covered. Refer instead to the Bridge Welding Code, D1.5.

5-20.6.5.2 Inspection

When long pieces are to be welded, the welding should be started at the middle of the piece and progress toward both ends simultaneously, using the skip method. A finished dependable weld of good workmanship has the following characteristics: A reasonably uniform cross-section with a flat or slightly convex face; reasonably straight edges merging almost imperceptibly into the base metal; a fairly smooth contour without overlap, undercutting or deficiency in section and freedom from pits, oxides, slag conclusions and gas pockets. The weld metal should be bright and without color, after being cleaned with a wire brush. Discoloration indicates burnt metal.

When making a visual inspection of welds and the base metal parts for cracks or other defects, the inspector should use a strong light and a magnifying glass. A dye penetrant, if available, is a good inspection aid, but should only be used by SNT-TC-1a qualified technicians.

The size and length of each fillet weld must be compared with the dimensions shown on the plans. The size or length may be slightly oversize or longer than specified, but shall not be less. A chart showing standard welding symbols is given in Figure 3.

Before requiring any large amount of weld to be cut out, the engineer should request confirming inspection from

BHS.

If erection bolts are to be removed, the plans will indicate whether or not bolt holes should be filled and in what manner.

5-20.6.5.3 Correction of Defective Welds

Welds can be removed by chipping, grinding, oxygen cutting, oxygen gouging or air arc gouging, but shall be completed in such manner that the remaining weld metal or base metal is not be nicked or undercut. Caulking of welds is not allowed.

Defective or unsound welds may also be corrected as follows:

1. Excessive Convexity

   Reduce to size by removal of excess weld metal. Generally, this condition is caused by improper welding techniques or insufficient current.

2. Shrinkage Cracks, Cracks in Base Metal, Crater and Excessive Porosity

   Remove defective portions of base metal and weld metal down to sound metal (at least 50 mm beyond each end of crack), and deposit additional weld metal.

3. Undercutting, Undersize, and Excessive Concavity

   Clean and deposit additional weld metal. The electrode used shall preferably be smaller than the electrode used in making the initial weld. This problem is usually caused by using excessive welding current or arc lengths.

4. Overlapping and Lack of Fusion

   Remove and replace the defective length of weld. Overlap generally occurs on fillet welding and is caused by incorrect welding technique or electrical conditions.

5-20.6.5.4 Field Welding

Whenever it is necessary for field welding on steel bridges, only electrodes of the proper type are to be used.

For guidance in the selection and use of proper electrodes for welding steel to be painted, reference should be made to Table of Matching Filler Metal Requirements shown as Table 1. The table incorporates the pertinent requirements of the Bridge Welding Code.

5-20.6.5.5 Certifications

The AASHTO amendments to the AWS Code require the contractor furnish certified copies of test reports for electrodes used on the project. The reports must also include the manufacturer's certification that the process and materials requirements were the same for the tested electrodes as for the furnished electrodes. The reports shall include the results of all pertinent required tests of AWS A5.1 or A5.5 as applicable. An appropriate form for reporting electrode tests is shown in Figure 3. Certified test reports are also required for all combinations of gasses used with gas-metal arc and flux-cored arc welding.

5-20.6.6 Stud Shear Connectors

Stud shear connectors are end welded to the upper flanges of steel beams, girders or plates after such members have been erected and floor beams placed. The use of stud shear connectors, rather than shop attached angular connectors, allows easier and safer erection and placement of forms and reinforcing steel; however, the studs are to be used only when and where designated on the plans. Requirements for welding and inspection of attached studs are covered in standard spec 506.3.20.

5-20.6.7 Intermediate Diaphragms for Prestressed Girders

For steel diaphragm to prestressed concrete girder web connections, the contractor must tighten connections to a snug-tight plus 1/4 turn condition at interior girders with through-bolts and to 80 foot-pounds at exterior connections when connecting to a ferrule loop insert with cap screws, unless the plan details show otherwise. Interior girder connections require bolt lengths that are typically not in stock for A325 bolts. Use of ASTM A449 bolts may be used instead in lieu of A325 bolts at interior girder bolted web connections.

For steel-to-steel connections within the diaphragms, the contractor must tighten according to standard spec 506.3.12.3 using 7/8" high strength bolts, unless the contract says otherwise. Either the turn-of-nut method or the direct tension indicating (DTI) washer method may be used.

Field rotational capacity testing is not required for intermediate diaphragm bolts. The contractor must submit a certificate of compliance certifying that those bolts conform to the specifications.

5-20.7 Final Alignment of Bearings

Due to fabrication tolerances and inaccuracies in laying out the bearing locations, it is sometimes necessary to make slight adjustments in the position of the bearings, after the erection is complete, to allow the proper

clearance between units or at abutments and to provide the correct opening at expansion devices. If the expansion bearings are of the rocker type, the rockers are adjusted by the erector according to the prevailing temperature so they will be vertical at the standard temperature shown on the plans.

When the final position of the bearings has been established, and the steel has been fastened, holes for the anchor bolts are drilled and the bolts set in a premixed non-shrink commercial grout.

5-20.8 Miscellaneous

5-20.8.1 Masses of Structural Steel and Other Metals

Refer to standard spec 506.4.

5-20.8.2 Inventory Inspection

Refer to CMM 5-15.

5-20.8.3 Bridge Deck Removal, Partial and Complete

Significant damage to upper flanges of steel and reinforced concrete girders has occurred as a consequence of the use of excessive and sometimes careless efforts to remove structure decks. The use of hydro-rams, whip hammers, heavy jackhammers, and cutting saws must be monitored by the inspector to ensure that excessive vibrations or saw cuts are not imparted to already completed work or units that are to remain in place. The inspection and monitoring of deck removal is equally as important as the placement of new work. Contractor indifference towards protecting the integrity of the supporting members, whether stringers, floor beams, girders or fracture-critical members, cannot be tolerated.

In the matter of partial deck removal, there have been instances where jackhammers have penetrated through the deck and shattered portions of the deck without any effort made to restore the integrity of the damaged area. Again, careful topside and bottom side monitoring and inspection must be exercised.

The requirements of standard spec 203.3.2 give the engineer full authority to order the contractor to discontinue or modify the use of any equipment or methods which damage any part of the structure to remain in place.

Obvious damage such as distortion of or damage to reinforcing steel and girders or the over breaking of concrete on work to remain should be cause for eliminating high energy removal equipment.

When damage will or might be imparted to any elements scheduled to remain, heavy hydraulic hammers should be confined to working outside an area 610 mm from any retained girder or from the line of neat removal. Additionally, the use of these hammers should be avoided on slab spans where the shock of impact might be detrimental to future performance of the remaining deck.

The contractor can be advised to avoid damage by bringing in lighter hammers or by utilizing hand-held pneumatic hammers. Proper use of equipment can minimize damage; however, the greater the energy capability of the hammer the more apt the operator is to work it to its maximum at the expense of the structural integrity of the structure.

A further ramification of partial removals is the increased dead load imparted to the first line of girders adjacent to a bay in which deck removal is taking place. The resultant cantilevered deck loads this girder with dead load that had originally been shared with the exterior or adjacent girder. If this dead load originally borne by the outside girder is not returned to the outside girder by shoring before placement of the additional deck, the load will remain on the interior girder. Furthermore, as the fresh concrete is placed, its mass may be assumed by the exterior or previously unloaded girder with little or no load transfer to the interior girder.

Careful and continuing inspection of the removal operation is necessary to avoid damage or overloading of girders, and we believe that discussions with the contractor about equipment and removal methods before removal operations commence can eliminate many subsequent problems.

If you have any question about the possibility of damage occurring which is not discernable, contact the Geotechnical section. They will dispatch monitoring equipment to determine if intolerable wave energy is being imparted to adjoining members or newly completed work. Based on guides and charts at their disposal, they can provide guidance as to whether corrective measures are necessary.

Figure 1 Example of Allowed Field Welding

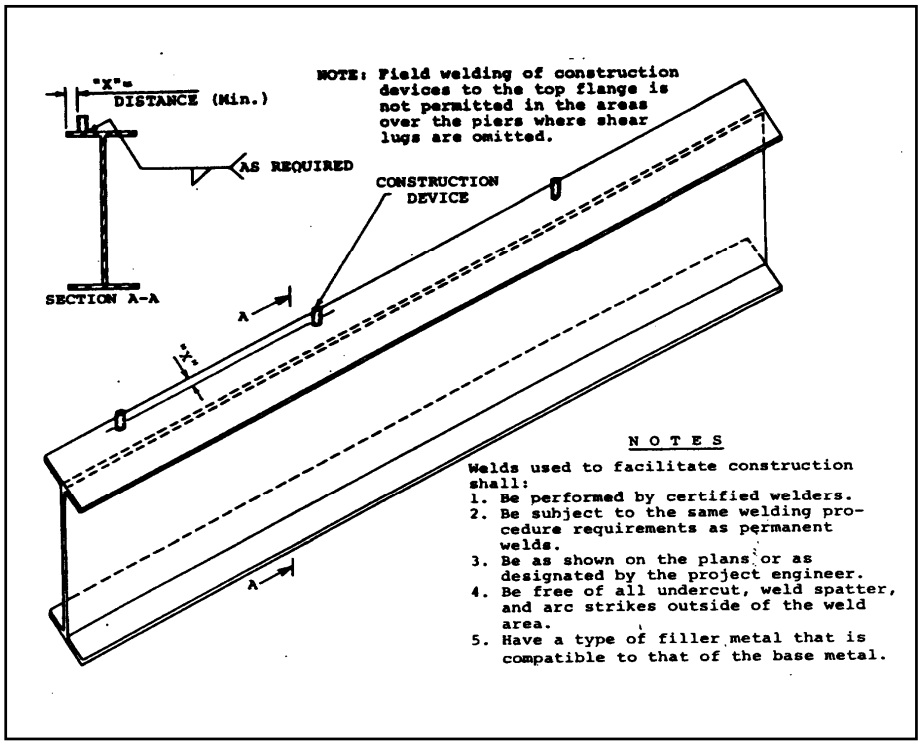

Table 1 Matching Filler Metal Requirements

Base Metal[1]	Welding Process[2], [3]	
AASHTO/ASTM Designation	**Shielded Metal-Arc[2]**	**Flux Cored Arc[3]**
M 183/A 36	AWS A 5.1 or A 5.5[5] E 7016, 18, 28 or 48	AWS A 5.20[4] E 6XT-1,5,6 or 8 E 7XT-1,5,6 or 8
M 223/A 572 GR 345, Type 1,2 or 3 M 222/A 588	AWS A 5.1 or A 5.5[5] E 7016, 18, 28 or 48[7]	AWS A 5.20[4] E 7XT-1,5,6 or 8
M 244/A 514, A 517 over 63mm thick	AWS A 5.5 E 10018-M	Grade[6] E 100T
M 244/A 514, A 517 63mm thick or less	AWS A 5.5 E 11018-M	Grade[6] E 110T

[1] In joints involving base metals of 2 different yield points or strengths, filler metal electrodes applicable to the lower strength base metal may be used.

[2] Only low hydrogen electrodes shall be used.

[3] See 4.16 of AWS D 1.1 for electrogas and electroslag weld metal requirements.

[4] Electrodes of other classifications in the specification will be acceptable for use if the manufacturer's certificate of test shows that the mechanical properties (tensile strength, yield strength and elongation) and the impact properties meet the requirements of the classification or specification listed.

[5] Special welding materials and procedures (e.g. E 80XX low alloy electrodes) may be required to match the notch toughness of base metal (for applications involving impact loading or low temperature) or for atmospheric corrosion and weathering characteristics (see 4.1.4).

[6] Deposited weld metal shall have a minimum impact strength of 27 Nm at -18C when Charpy V-Notch specimens are

used.

[7] E 8016 may be substituted.

Figure 2 American Welding Society Standard Welding Symbols

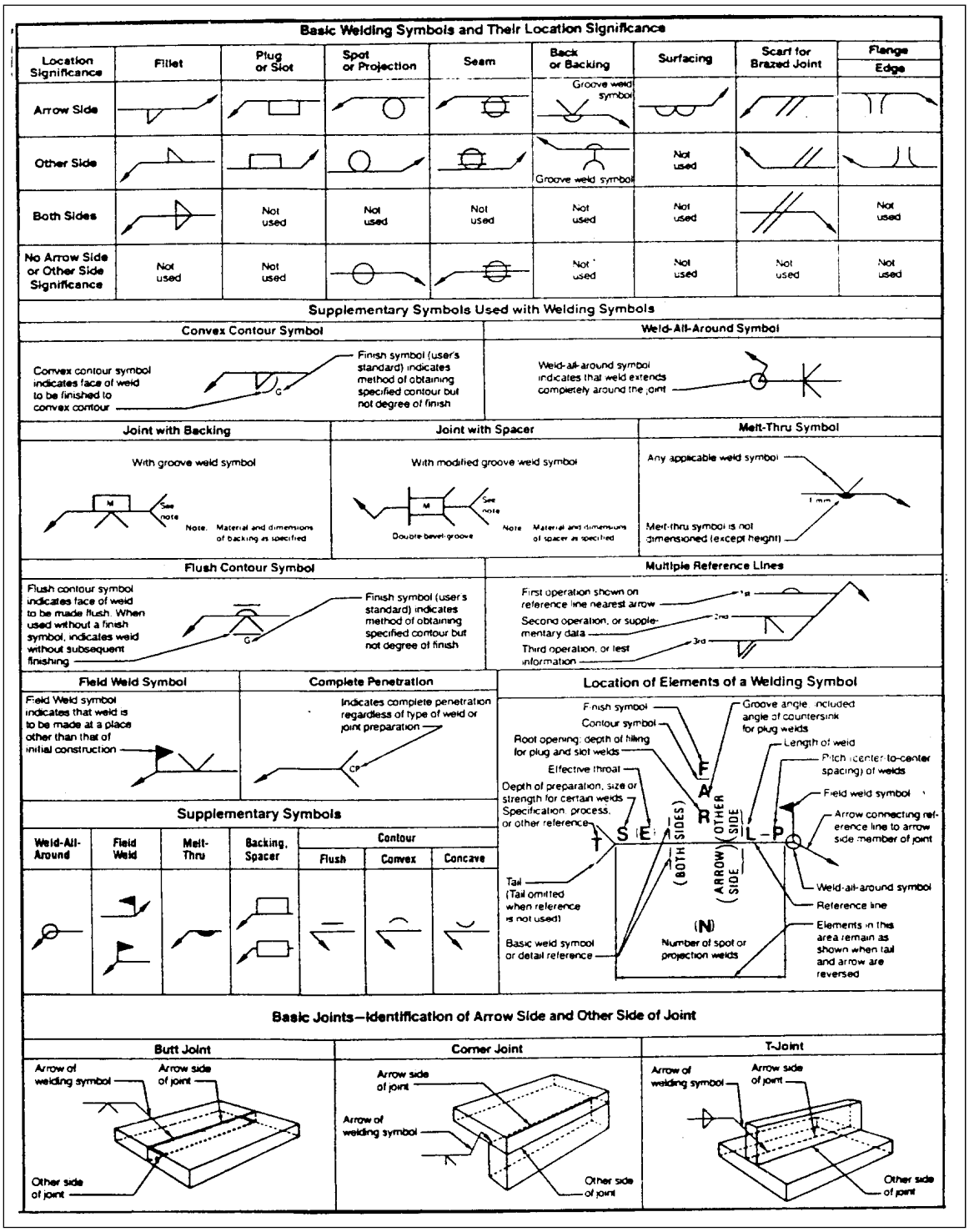

Figure 2 American Welding Society Standard Welding Symbols (cont'd)

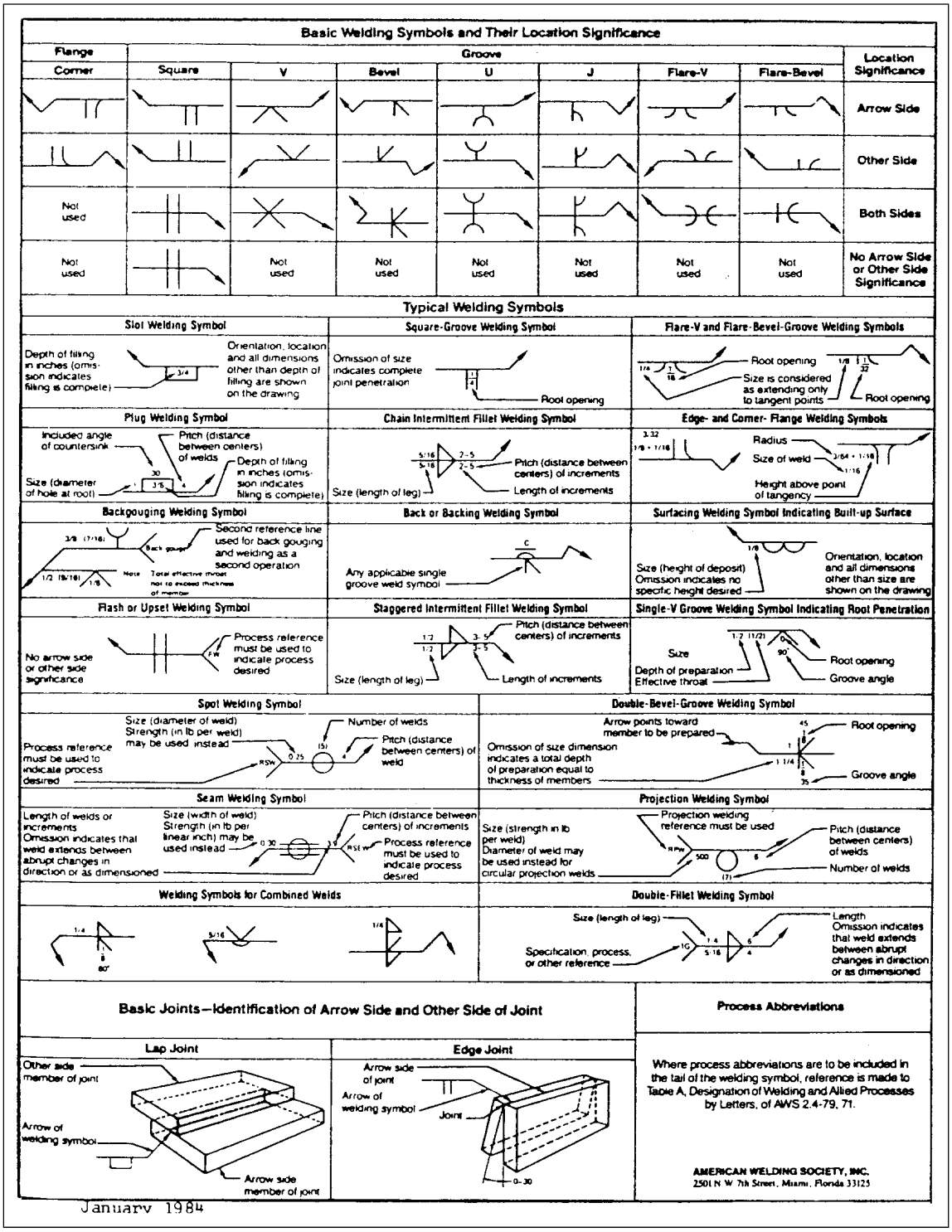

Table 2 Minimum Preheat and Interpass Welding Temperatures

		Thickness of Thickest Part at Point of Welding - mm		
Welding Process (Base Metal)	**To 19mm Incl.**	**Over 19mm to 38mm Incl.**	**Over 38mm to 63mm Incl.**	**Over 63mm**
Shielded Metal-Arc Welding; Submerged Arc Welding; Gas Metal-Arc Welding; of Flux Cored Arc Welding (ASTM A36, A572 Grade 50; A588)	10C	21C	65C	107C
Shielded Metal-Arc Welding; Submerged Arc Welding; Gas Metal-Arc Welding; or Flux Cored Arc Welding (ASTM A514/A517)	10C	52C	80C	107C

Welding shall not be done when the ambient temperature is lower than zero F. When the base metal is below the temperature listed for the welding process being used and the thickness of material being welded, it shall be preheated (except as otherwise provided) in such manner that the surfaces of the parts on which weld metal is being deposited are at or above the specified minimum temperature for a distance equal to the thickness of the part being welded, but not less than 75mm both laterally and in advance of the welding. Preheat and interpass temperatures must be sufficient to prevent crack formation. Temperature above the minimum shown may be required for highly restrained welds. For ASTM A514/A517 steel the maximum preheat and interpass temperature shall not exceed 205C for thicknesses up to 38mm inclusive, and 230C for greater thicknesses. Heat input when welding ASTM A514/A517 steel shall not exceed the steel producer's recommendation. Welding shall be carried continuously to completion or to a point that will insure freedom from cracking before the joint is allowed to cool below the minimum specified preheat and interpass temperature.

In joints involving combinations of base metals, preheat shall be as specified for the higher strength steel being welded.

Figure 3 Typical Certificate of Conformance for Welding Electrodes

(Manufacturer's name and address)

CERTIFICATE OF CONFORMANCE TO REQUIREMENTS
FOR WELDING ELECTRODES

Supplied to:_____

Date Quantity Order No. Heat or Lot No.

This is to certify that __(Trade name or No.)__ AWS classification __(EXXXX)__, as supplied under the above order number, is of the same classification, manufacturing process, and material requirements, as the electrodes tested on _____ 19___.

All tests required by specification AWS A5.1 or AWS A5.5, were performed in conformance with the specification, and the above electrode met all of the requirements. The electrodes are marked in conformance with AWS A5.1 or AWS A5.5.

The chemistry and mechanical properties of the deposited weld metal were as follows:

	4mm		5mm		6mm	
	DC+	AC	DC+	AC	DC+	AC

Tensile Strength MPa _____

Yield Strength Mpa_____

Elongation % in 50mm_____

Charpy V Notch

Nm at C. _____

Manganese % _____

Silicon %_____

Nickel % _____

Chronium % _____

Molybdenum % _____

Vanadium % _____

Fillet Tests Position

as required_____

Radiographic test_____

Fillet Test, Radiograph, chemistry and mechanical properties are not required for the following size:

_____ _____ _____

Operations Supervised by Chief Engineer Director