8. METALLURGY & WELDING

Standard industry specifications for construction materials and processes permit wide variation in strength, toughness and other properties that can be critical to structural performance. This Chapter provides basic information on the variations in properties that occur, practical steps an engineer can take to control critical properties to acceptable levels of tolerance, and the specific instances when such measures may be appropriate.

8.1 Parent Materials

8.1.1 Steels

Designers should specify materials which are readily available for building construction and which will provide suitable ductility and weldability for seismic applications. Structural steels which may be used in the lateral-force-resisting systems for structures designed for seismic resistance without special qualification include those contained in Table 8-1. Refer to the applicable ASTM reference standard for detailed information.

Table 8-1 - Structural Steel Prequalified for Use in Seismic Lateral-Force-Resisting Systems

ASTM Specification	Description
ASTM A36	Carbon Structural Steel
ASTM A283 Grade D	Low and Intermediate Tensile Strength Carbon Steel Plates
ASTM A500 (Grades B & C)	Cold-Formed Welded & Seamless Carbon Steel Structural Tubing in Rounds & Shapes
ASTM A501	Hot-Formed Welded & Seamless Carbon Steel Structural Tubing
ASTM A572 (Grades 42 & 50)	High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality
ASTM A588	High-Strength Low-Alloy Structural Steel (weathering steel)

Structural steels which may be used in the lateral-force-resisting systems of structures designed for seismic resistance with special permission of the building official are those listed in Table 8-2. Steel meeting these specifications has not been demonstrated to have adequate weldability or ductility for general purpose application in seismic-force-resisting systems, although it may well possess such characteristics. In order to demonstrate the acceptability of these materials for such use in WSMF construction it is recommended that connections be qualified by test, in accordance with the guidelines of Chapter 7. The test specimens should be fabricated out of the steel using those welding procedures proposed for use in the actual work.

Table 8-2 - Non-prequalified Structural Steel

ASTM Specification	Description
ASTM A242	High-Strength Low-Alloy Structural Steel
ASTM A709	Structural Steel for Bridges
ASTM A913	High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching & Self-Tempering Process

Commentary: Many WSMF structures designed in the last 10 years incorporated ASTM A36 steel for the beams and ASTM A572 grade 50 steel for the columns. This provided an economical way to design structures for the strong column - weak beam provisions contained in the building code. Recent studies conducted by the Structural Shape Producers Council (SSPC), however, indicate that material produced to the A36 specification has wide variation in strength properties with actual yield strengths that often exceed 50 ksi. This wide variation makes prediction of connection and frame behavior difficult. Some have postulated that one of the contributing causes to damage experienced in the Northridge Earthquake was inadvertent pairing of overly strong beams with average strength columns.

The AISC and SSPC have been working for several years to develop a new specification for structural steel that would have both minimum and maximum yield values defined and provide for a margin between maximum yield and minimum ultimate tensile stress. AISC recently submitted such a specification, for a material with 50 ksi specified yield strength, to ASTM for development into a standard specification. It is anticipated that domestic mills will begin producing structural shapes to this specification within a few years and that eventually, this new material will replace A36 as the standard structural material for incorporation into lateral-force-resisting systems.

Under certain circumstances it may be desirable to specify steels that are not recognized under the UBC for use in lateral-force-resisting systems. For instance, ASTM A709 might be specified if the designer wanted to place limits on toughness for fracture-critical applications. In addition, designers may wish to begin incorporating ASTM A913, Grade 65 steel, as well as other higher strength materials, into projects, in order to again be able to economically design for strong column - weak beam conditions. Designers should be aware, however, that these alternative steel materials may not be readily available. It is also important when using such non-prequalified steel materials, that precautions be taken to ensure adequate weldability of the material and that it has sufficient ductility to perform under the severe loadings produced by earthquakes. The cyclic test program recommended by these Interim Guidelines for qualification of connection designs, by test, is believed to be an adequate approach to qualify alternative steel material for such use as well.

Note that ASTM A709 steel, although not listed in the building code as prequalified for use in lateral-force-resisting systems, actually meets all of the requirements for ASTM A36 and ASTM A572. Consequently, special qualification of the use of this steel should not be required.

8.1.2 Chemistry

ASTM specifications define chemical requirements for each steel. A chemical analysis is performed by the producer on each heat of steel. End product analyses can also be specified on certain products. A certified mill test report is furnished to the customer with the material. The designer should specify that copies of the mill test reports be submitted for his/her conformance review. In general, ASTM specifications for structural steels include maximum limits on carbon, manganese, silicon, phosphorous and sulfur. Ranges and minimums are also limited on other elements in certain steels. Chromium, columbium, copper, molybdenum, nickel and vanadium may be added to enhance strength, toughness, weldability and corrosion resistance. These chemical requirements may vary with the specific product and shape within any given specification.

Commentary: Some concern has been expressed with respect to the movement in the steel producing industry of utilizing more recycled steel in its processes. This results in added trace elements not limited by current specifications. Although these have not been shown quantitatively to be detrimental to the performance of welding on the above steels, a new specification for structural steel proposed by AISC does place more control on these trace elements. Mill test reports now include elements not limited in some or all of the specifications. They include copper, columbium, chromium, nickel, molybdenum, silicon and vanadium. The analysis and reporting of an expanded set of elements should be possible, and could be beneficial in the preparation of welding procedure specifications (WPSs) by the welding engineer if critical welding parameters are required. Modern spectrographs used by the mills are capable of automated analyses. When required by the engineer, a request for special supplemental requests should be noted in the contract documents.

8.1.3 Tensile/Elongation Properties

Mechanical property test specimens are taken from rolled shapes or plates at the rolling mill in the manner and location prescribed by ASTM A6 and ASTM A370. Table 8-3 gives the basic mechanical requirements for commonly used structural steels. Properties specified, and controlled by the mills, in current practice include minimum yield strength, ultimate tensile strength and minimum elongation. However, there can be considerable variability in the actual properties of steel meeting these specifications.

SSPC, in cooperation with SEAOC, has collected statistical data on the strength characteristics of two grades (ASTM A36 and ASTM A572 Grade 50) of structural steels, based on mill test reports from selected domestic producers for the 1992 production year. Data were also collected for "Dual

Grade" material that was certified by the producers as complying with both ASTM A36 and ASTM A572 Grade 50. Table 8-4 summarizes these results as well as data provided by a single producer for ASTM A913 material.

Table 8-3 - Typical Tensile Requirements for Structural Shapes

ASTM	Minimum Yield Strength, Ksi	Ultimate Tensile Strength, Ksi	Minimum Elongation % in 2 inches	Minimum Elongation % in 8 inches
A36	36	58-80[1]	21[2]	20
A242	42[4]	63 MIN.	21[3]	18
A572, GR50	50	65 MIN.	21[2]	18
A588	50	70 MIN.	21[3]	18
A709, GR36	36	58-80	21[2]	20
A709, GR50	50	65 MIN.	21	18
A913, GR50	50	65 MIN.	21	18
A913, GR65	65	80 MIN.	17	15

Notes: 1- No maximum for shapes greater than 426 lb./ft.
2- Minimum is 19% for shapes greater than 426 lb. /ft.
3- Minimum is 18% for shapes greater than 426 lb./ft.
4. Minimum is 50 ksi for Shape Groups 1 and 2, 46 ksi for Shape Group 3

Unless special precautions are taken to limit the actual strength of material incorporated into the work to defined levels, new material specified as ASTM A36 should be assumed to be the dual grade for connection demand calculations, whenever the assumption of a higher strength will result in a more conservative design condition.

Commentary: The data given in Table 8-4 for A36 and A572 Grade 50 is somewhat weighted by the lighter, Group 1 shapes that will not ordinarily be used in WSMF applications. Excluding Group 1 shapes and combining the Dual Grade and A572 Grade 50 data results in a mean yield strength of 48 ksi for A36 and 57 ksi for A572 Grade 50 steel. It should also be noted that 50% of the material actually incorporated in a project will have yield strengths that exceed these mean values. For the design of facilities with stringent requirements for limiting post-earthquake damage, consideration of more conservative estimates of the actual yield strength may be warranted.

In wide flange sections the tensile test coupons are currently taken from the web. The amount of reduction rolling, finish rolling temperatures and cooling conditions affect the tensile and impact properties in different areas of the member. Typically, the web exhibits about five percent higher strength than the flanges due to faster cooling.

Table 8-4 - Statistics for Structural Shapes[1]

Statistic	A 36	Dual Grade GRADE	A572 GR50	A913 GR65

Statistic	A 36	Dual Grade GRADE	A572 GR50	A913 GR65
Yield Point (ksi)				
Mean	49.2	55.2	57.6	75.3
Minimum	36.0	50.0	50.0	68.2
Maximum	72.4	71.1	79.5	84.1
Standard Deviation [s]	4.9	3.7	5.1	4.0
Mean + 1 s	54.1	58.9	62.7	79.3
Tensile Strength (ksi)				
Mean	68.5	73.2	75.6	89.7
Minimum	58.0	65.0	65.0	83.4
Maximum	88.5	80.0	104.0	99.6
Standard Deviation [s]	4.6	3.3	6.2	3.5
Mean + 1 s	73.1	76.5	81.8	93.2
Yield/Tensile Ratio				
Mean	0.72	0.75	0.76	0.84
Minimum	0.51	0.65	0.62	0.75
Maximum	0.93	0.92	0.95	0.90
Standard Deviation [s]	0.06	0.04	0.05	0.03
Mean + 1 s	0.78	0.79	0.81	0.87
Mean - 1 s	0.66	0.71	0.71	0.81

1: The data presented for ASTM A36, "Dual Grade" and ASTM A572 Grade 50 were included as part of the SSPC study (SSPC-1994). The data for ASTM A913 were derived from a single producer and may not be available from all producers.

Design professionals should be aware of the variation in actual properties permitted by the ASTM specifications. This is especially important for yield strength. Yield strengths for ASTM A36 material have consistently increased over the last 15 years so that several grades of steel may have the same properties or reversed properties, with respect to beams and columns, from those the designer intended. Investigations of structures damaged by the Northridge earthquake found some WSMF connections in which beam yield strength exceeded column yield strength despite the opposite intent of the designer.

As an example of the variations which can be found, Table 8-5 presents the variation in material properties found within a single building affected by the Northridge earthquake. Properties shown include measured yield strength (F_{ya},), measured tensile strength (F_{ua}) and Charpy V-Notch energy rating (CVN).

Table 8-5 - Sample Steel Properties from a Building Affected by the Northridge Earthquake

Shape	F_{ya}[1] *ksi*	F_{ua}, *ksi*	*CVN, ft-lb.*
W36 X 182	*38.0*	*69.3*	*18*
W36 X 230	*49.3*	*71.7*	*195*

Note 1 - ASTM A36 material was specified for both structures.

The practice of dual certification of A36 and A572, Grade 50 can result in mean yield strengths that are fifty percent higher than the specified yield of A36. Since there is no practical way to discern whether dual grade steel will be supplied, unless direct purchase of steel from specific suppliers is made, in the absence of such procurement practices, the prudent action for determining connection requirements, where higher strengths could be detrimental to the design, would be to assume the dual grade material whenever A36 or A572 Grade 50 is specified.

8.1.4 Toughness Properties

For critical connections, non-redundant components and unusual or difficult geometries involving Group 3 (with flanges 11/2 inches or thicker) 4 and 5 shapes and plates and built-up sections over two inches thick with welded connections, the designer should consider specifying toughness requirements on the parent materials. A Charpy V-Notch (CVN) value of 20 ft.-lb. at 70 degrees F. should be specified when toughness is deemed necessary for an application. Refer to Figure 8-1 for typical CVN test specimen locations. The impact test should be conducted in accordance with ASTM A673, frequency H, with the following exceptions:

a) The center longitudinal axis of the specimens should be located as near as practicable to midway between the inner flange surface and the center of the flange thickness at the intersection of the web mid-thickness. Refer to AISC LRFD specification, Section A3-1c, Heavy Shapes (American Institute of Steel Construction - 1993)

b) Tests should be conducted by the producer on material selected from a location representing the top of each ingot or part of an ingot used to produce the product represented by these tests. For the continuous casting process, the sample may be taken at random.

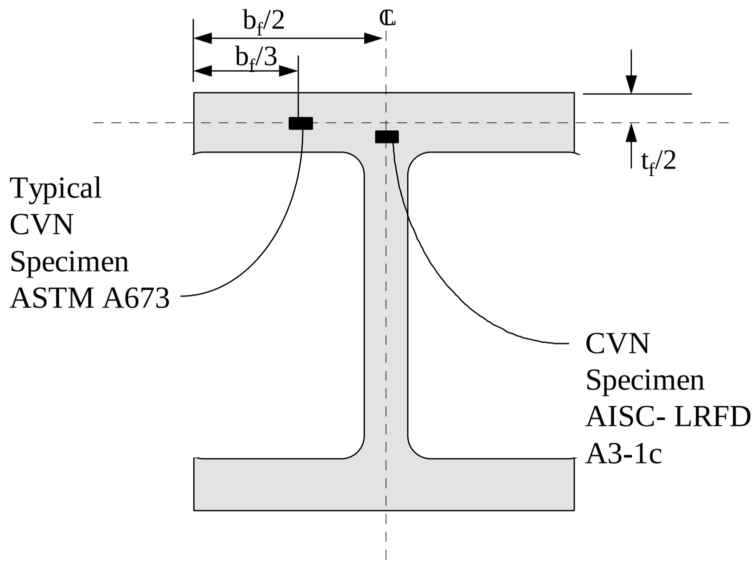

Figure 8-1 - Standard Locations for Charpy V-Notch Specimen Extraction, Longitudinal Only

Commentary: Many variables are recognized in analyzing the metallurgy of WSMF members. Until more research is available on the through-thickness properties of members thicker than two inches, a conservative approach is indicated. Specifying toughness properties in critical, unusual or non-redundant connections should be considered.

As temperature decreases or strain rate increases, toughness properties decrease. Charpy V-notch impact (CVN) tests, pre-cracked CVN tests and other fracture toughness tests can identify the nil ductility temperature (NDT) - the temperature below which a material loses all ductility and fractures in a brittle manner. On a microscopic level, this equates to a change in the fracture mechanism from shear to cleavage. Fracture that occurs by cleavage at a nominal tensile stress below yield is referred to as a brittle fracture. A brittle fracture can occur in structural steel when a particular combination of low temperature, tensile stress, high strain rate and a metallurgical or mechanical notch is present.

Plastic deformation can only occur through shear stress. Shear stress is generated when uniaxial or bi-axial straining occurs. In tri-axial stress states, the maximum shear stress approaches zero as the principal stresses increase. When these stresses approach equality, a cleavage failure can occur. Welding and other sources of residual stresses can set up a state of tri-axial stress leading to brittle fractures.

The necessity for minimum toughness requirements is not agreed to by all. There is also disagreement as to how much toughness should be required. The

AWS Presidential Task Group recommended toughness values of 15 ft-lb. at different temperatures, depending on the anticipated service conditions. A temperature of 70 degrees F was recommended for enclosed structures and 40 degrees F for exposed structures. The 1993 AISC LRFD Specification, Section A3-1c, Heavy Shapes, requires toughness testing [Charpy V-Notch] under the following conditions for Group 4 and 5 shapes and plates exceeding 2 inches in thickness: a) When spliced using complete joint penetration welds; b) when complete joint penetration welds through the thickness are used in connections subjected to primary tensile stress due to tension or flexure of such members." Where toughness is required, the minimum value should be 20 ft-lb. at 70°F.

Plates thicker than two inches and sections with flanges thicker than two inches can be expected to have significantly variable grain sizes across the section. The slower cooling rate of the web-flange intersection in thick sections produces a larger grain size which exhibits less ductility and notch toughness.

ANSI/ASTM A673 and A370 establish the procedure for longitudinal Charpy V-notch testing. The impact properties of steel can vary within the same heat and piece, be it as-rolled, controlled rolled, or heat treated. Normalizing or quenching and tempering will reduce the degree of variation. Three specimens are taken from a single test coupon or location. The average must exceed the specified minimum, but one value may be less than the specified minimum but must be greater than the larger of two thirds of the specified minimum or 5 ft-lb. The longitudinal axis of the specimen is parallel to the longitudinal axis of the shape or final rolling direction for plate. For shapes, the specimen is taken from the flange 1/3 the distance from the edge of the flange to the web. The frequency of testing [heat or piece], the test temperature, and the absorbed energy are specified by the user. [NOTE: heat testing (frequency H) for shapes, means one CVN test set of samples from at least each 50 tons of the same shape size, excluding length, from each heat in the as-rolled condition. Piece testing (frequency P) for shapes, means one CVN test set of specimens from at least each 15 tons or each single length of 15 tons of the same shape size, excluding length, from each heat in the as-rolled condition.] Heat testing is probably adequate in most circumstances.

The specimen location required by ASTM A673 is not at the least tough part of a W shape. For a W shape, the volume at the flange web intersection has the lowest ratio of surface area to volume and hence cools the slowest. This slow cooling causes grain growth and reduced toughness. The finer the grain, the tougher the material. Also, ASTM A673 does not specify where in the product run of an ingot to sample. Impurities tend to rise to the upper portion of the ingot during cooling from molten metal. Impurities reduce the toughness of the finished metal. Hence, shapes produced from the upper portions of an ingot can be expected to have lower toughness, and samples should be taken from shapes *produced from this portion of the ingot. In the continuous casting process, impurities tend to be more evenly distributed; hence, samples taken anywhere should suffice. The AISC LRFD specification requires testing from the upper portion of the ingot and near the web flange intersection. Even though the AISC LRFD specification does* ***not*** *require toughness testing for the typical WSMF connection, i.e., a Group 2 beam to a Group 4 column, it appears that there may be inadequate through thickness toughness in the Group 4 and 5 column flanges.*

In response to concerns raised following the Northridge Earthquake, the AISC conducted a statistical survey of the toughness of material produced in structural shapes, based on data provided by six producers for a production period of approximately one year (American Institute of Steel Construction - 1995). This survey showed a mean value of Charpy V notch toughness for all shape groups that was well in excess of 20 ft-lb. at 70 degrees F. However, not all of the samples upon which these data are based were taken from the core area, recommended by these Interim Guidelines. Consequently, this survey does not provide definitive information on the extent to which standard material produced by the mills participating in this survey will meet the recommended values.

8.1.5 Lamellar Discontinuities

For critical joints (beam to column CJP welds or other tension applications where Z-axis or tri-axial stress states exist), ultrasonic testing (UT) should be specified for the member loaded in the Z axis direction, in the area of the connection. A distance 3 inches above and below the location to be welded to the girder flange is recommended. The test procedure and acceptance criteria given in ASTM A898-91, Standard Specification for Straight Beam Ultrasonic Examination of Rolled Steel Structural Shapes, Level I, should be applied. This testing should be done in the mill or fabrication shop for new construction. For repair welding, the same procedure should be applied in the field, as access permits.

Commentary: Very little test data exist on the through thickness properties of structural shapes nor are there any standard test methods for determining these properties. Nevertheless, the typical beam-column joints in WSMFs rely heavily on the through-thickness properties of column flanges. Some of the proposed strengthening and reinforcing solutions will transmit even more forces into the Z axis of the column flanges. Laminations (pre-existing planes of weakness) and lamellar tearing (cracks parallel to the surface) will impair the Z axis strength and toughness properties. These defects are mainly caused by non-metallic sulfides and oxides which begin as almost spherical in shape, and become elongated in the rolling process. When Z axis loading occurs from weld shrinkage strains or external loading, microscopic cracks may form between the discrete, elongated nonmetallic inclusions. As they link up, lamellar tearing occurs.

Longitudinal wave ultrasonic testing is very effective in mapping serious lamellar discontinuities. Improved quality steel does not eliminate weld shrinkage and, by itself, will not necessarily avoid lamellar tearing in highly restrained joints. Ultrasonic testing should not be specified without due regard for design and fabrication considerations.

In cases where lamellar defects or tearing are discovered in erection or on existing buildings, the designer must consider the consequences of making repairs to these areas. Gouging and repair welding will add additional cycles of weld shrinkage to the connection and may promote crack extensions or new lamellar tearing. When secondary cracking is discovered, a welding engineer should be consulted to generate a special WPS for the repair.

8.2 Welding

8.2.1 Welding Process

The welding process to be used to execute the joint weld [e.g. shielded metal (SMAW), flux cored (FCAW), submerged (SAW), gas metal arc weld (GMAW), or electroslag (requires qualification of the welding procedure specification)] should be specified in the Contract Documents for weld repairs. Contract documents for new construction should state any restrictions on weld parameters or processes. Most pre-Northridge production welding was executed using FCAW using a self-shielding process (FCAW-SS). Shielded metal arc welding (stick welding) is often used for damage repairs, in tight conditions and in some shop applications.

Commentary: At this time there is no clear evidence that one method can produce uniformly superior welds although poor welds can be produced with any of the methods.

8.2.2 Welding Procedures

Welding should be performed within the parameters established by the electrode manufacturer and the Welding Procedure Specification (WPS), required under AWS D1.1.

Commentary: For example, the position (if applicable), electrode diameter, amperage or wire feed speed range, voltage range, travel speed range and electrode stickout (e.g. all passes, 0.072 in. diameter, 248 to 302 amps, 19 to 23 volts, 6 to 10 inches/minute travel speed, 170 to 245 inches/minute wire feed speed, 1/2" to 1" electrode stickout) should be established. This information is generally submitted by the fabricator as part of the Welding Procedure Specification. Its importance in producing a high quality weld is essential. The following information is presented to help the engineer understand some of the issues surrounding these parameters.

The amperage, voltage, travel speed, electrical stickout and wire feed speed are functions of each electrode. If prequalified WPSs are utilized, these parameters must be in compliance with the AWS D1.1 requirements. For FCAW and SMAW, the parameters required for an individual electrode vary from manufacturer to manufacturer. Therefore, for these processes, it is essential that the fabricator/erector utilize parameters that are within the range of recommended operation published by the filler metal manufacturer. Alternately, the fabricator/erector could qualify the welding procedure by test in accordance with the provisions of AWS D1.1 and base the WPS parameters on the test results. For submerged arc welding, the AWS D1.1 code provides specific amperage limitations since the solid steel electrodes used by this process operate essentially the same regardless of manufacture. The filler metal manufacturer's guideline should supply data on amperage or wire feed speed, voltage, polarity, and electrical stickout. The guidelines will not, however, include information on travel speed which is a function of the joint detail. The contractor should select a balanced combination of parameters, including travel speed, that will ensure that the code mandated weld-bead sizes (width and height) are not exceeded.

8.2.3 Welding Filler Metals

The current AWS D1.1 requirements should be incorporated as written in the Code. The welding parameters should be clearly specified using a combination of the Project Specifications, the Project Drawings, the Shop Drawings and the welding procedure specifications, as required by AWS D1.1. For welding on ASTM A572 steel, the AWS D1.1 code requires the use of low-hydrogen electrodes. With SMAW welding, a variety of non-low hydrogen electrodes are commercially available. These electrodes are not appropriate for welding on the higher strength steels used in building construction today, although they were popular in the past when lower strength steels were employed. All of the electrodes that are employed for flux cored arc welding (both gas shielded and self-shielded), as well as submerged arc welding, are considered low hydrogen.

For critical joints (beam to column CJP welds or other tension applications where Z-axis loading or tri-axial stress states exist), toughness requirements for the filler metals should be specified. A minimum CVN value of 20 ft.-lb. at a temperature of 0 degrees F. should be required, unless more stringent requirements are indicated by the service conditions and/or the Contract Documents. The filler metal should be tested in accordance with the AWS A5 filler metal specification to ensure it is capable of achieving this level of notch toughness. The filler metal manufacturers Typical Certificate of Conformance, or a suitably documented test performed by the contractor, should be used to document the suitability of the electrode used. These tests should be performed for each filler metal by AWS classification, filler metal manufacturer and filler metal manufacturer's trade name. The sizes as specified by the AWS A5 document should be tested, although the exact diameter used in production need not be specifically tested. This requirement should not be construed to imply lot or heat testing of filler metals.

Electrode specification sheets should be provided by the Fabricator/Erector prior to commencing fabrication/erection.

Commentary: Currently, there are no notch toughness requirements for weld metal used in welding ASTM A 36 or A 572, Grade 50, steel in AWS D1.1. This topic has been extensively discussed by the Welding Group at the Joint SAC/AISC/AISI/NIST Invitational Workshop on September 8 and 9, 1994, and by all participants of the SAC Invitational Workshop on October 28 and 29, 1994. The topic was also considered by the AWS Presidential Task Group, which decided that additional research was required to determine the need for toughness in weld metal. There is general agreement that adding a toughness requirement for filler metal would be desirable and easily achievable. Most filler metals are fairly tough, but some will not achieve even a modest requirement such as 5 ft-lb. at + 70°F. What is not in unanimous agreement is what level of toughness should be required. The recommendation from the Joint Workshop was 20 ft-lb. at -20°F per Charpy V-Notch [CVN] testing. The recommendation from the SAC Workshop was 20 ft-lb. at 30°F lower than the Lowest Ambient Service Temperature (LAST) and not above 0°F. The AWS Presidential Task Group provided an interim recommendation for different toughness values depending on the climatic zone, referenced to ASTM A709. Specifically, the recommendation was for 20 ft-lb. at temperatures of 70 degrees F for Zone 1, 40 degrees F for Zone 2, and 10 degrees F for Zone 3. The AWS also suggested toughness values for base metals used in these applications.

Some fractured surfaces in the Northridge and Kobe Earthquakes revealed evidence of improper use of electrodes and welding procedures. Prominent among the misuses were high production deposition rates. Pass widths of up to 1-1/2 inches and pass heights of 1/2 inch were common. The kind of heat input associated with such large passes promotes grain growth in the HAZ and attendant low notch toughness. Root gap, access capability, electrode diameter, stick-out, pass thickness, pass width, travel speed, wire feed rate, current and voltage were found to be the significant problems in evaluation of welds in buildings affected by the Northridge earthquake.

Welding electrodes for common welding processes include:

AWS A5.20: Carbon Steel Electrodes for FCAW
AWS A5.29: Low Alloy Steel Electrodes for FCAW
AWS A5.1: Carbon Steel Electrodes for SMAW
AWS A5.5: Low Alloy Steel Covered Arc Welding Electrodes (for SMAW)
AWS A5.17: Carbon Steel Electrodes and Fluxes for SAW
AWS A5.23: Low Alloy Steel Electrodes and Fluxes for SAW
AWS A5.25: Carbon and Low Alloy Steel Electrodes and Fluxes for Electroslag Welding

In flux cored arc welding, one would expect the use of electrodes that meet either AWS A5.20 or AWS A5.29 provided they meet the toughness requirements specified below.

Except to the extent that one requires Charpy V-Notch toughness and minimum yield strength, the filler metal classification is typically selected by the Fabricator. Compatibility between different filler metals must be confirmed by the Fabricator, particularly when SMAW and FCAW-SS processes are mixed. Generally speaking, SMAW-type filler metals may not be applied to FCAW-SS type filler metals (e.g. when a weld has been partially removed) while FCAW-type filler metals may be applied to SMAW-type filler metals. This recommendation considers the use of aluminum as a killing agent in FCAW-SS electrodes that can be incorporated into the SMAW filler metal with a reduction in impact toughness properties.

As an aid to the engineer, the following interpretation of filler metal classifications is provided below:

$E^1X^2X^3T^4X^5$ *For electrodes specified under AWS A5.20*
$E^1X^2X^3T^4X^5X^6$ *For electrodes specified under AWS A5.29*
$E^1XX^7X^8X^9X^{10}$ *For electrodes specified under AWS A5.1 or AWS A5.5.*

NOTES:

1. *Indicates an electrode.*

2. *Indicates minimum tensile strength of deposited weld metal (in tens of ksi, e.g., 7 = 70 ksi).*

3. *Indicates primary welding position for which the electrode is designed (0 = flat and horizontal and 1 = all positions).*

4. *Indicates a flux cored electrode. Absence of a letter indicates a "stick" electrode for SMAW.*

5. *Describes usability and performance capabilities. For our purposes, it conveys whether or not Charpy V-Notch toughness is required (1, 5, 6 and 8 have impact strength requirements while 2, 4, 7, 10 and 11 do not). A "G" signifies that the properties are not defined by AWS and are to be agreed upon between the manufacturer and the specifier. Impact strength is specified in terms of the number of foot-pounds at a given temperature (e.g., 20 ft-lb. at 0 degrees F). Note that for electrodes specified under AWS A5.20, the format for usage is "T-X".*

6. *Designates the chemical composition of deposited metal for electrodes specified under AWS A5.29. Note that there is no equivalent format for chemical composition for electrodes specified under AWS A5.20.*

7. *The first two digits (or three digits in a five digit number) designate the minimum tensile strength in ksi.*

8. *The third digit (or fourth digit in a five digit number) indicates the primary welding position for which the electrode is designed (1 = all positions, 2 = flat position and fillet welds in the horizontal position, 4 = vertical welding with downward progression and for other positions.)*

9. *The last two digits, taken together, indicate the type of current with which the electrode can be used and the type of covering on the electrode.*

10. *Indicates a suffix (e.g., A1, A2, B1, etc.) designating the chemical composition of the deposited metal.*

***Electrode Diameter**: (See AWS D1.1 Section 4.14.1.2) The issue of maximum electrode diameter has not been studied sufficiently to determine whether or not electrode diameter is a critical variable. Recent tests have produced modified frame joints with acceptable test results using the previous standard-of-practice 0.120 in. diameter wire. The use of smaller diameter electrodes will slow the rate of deposition (as measured by volume) but will not, in and of itself, produce an acceptable weld. The following lists the maximum allowable electrode diameters for prequalified FCAW WPS's according to D1.1:*

- *Horizontal, complete or partial penetration welds: 1/8 inch (0.125")**
- *Vertical, complete or partial penetration welds: 5/64 inch (0.078")*
- *Horizontal, fillet welds: 1/8 inch (0.125")*
- *Vertical, fillet welds: 5/64 inch (0.078")*
- *Overhead, reinforcing fillet welds: 5/64 inch (0.078")*
  - * *This value is not part of D1.1-94, but will be part of D1.1-96.*

For a given electrode diameter, there is an optimum range of weld bead sizes that may be deposited. Weld bead sizes that are outside the acceptable size range (either too large or too small) may result in unacceptable weld quality. The D1.1 code controls both maximum electrode diameters and maximum bead sizes (width and thickness). Prequalified WPS's are required to meet these code requirements. Further restrictions on suitable electrode diameters are not recommended.

8.2.4 Preheat and Interpass Temperatures

The preheat temperatures and conditions given in AWS D1.1, Chapter 4 should be strictly observed with special attention given to Section 4.2, for the thickness of metal to be welded. For repair welding of earthquake damage, the *AASHTO/AWS D1.5 Bridge Welding Code* preheat requirements for fracture-critical, non-redundant applications should be considered.

Cracking of welds and heat affected zones should be avoided. One type of weld cracking is hydrogen induced cracking (HIC). For a given steel, variables that reduce HIC tendencies are prioritized as follows:

1. Lower levels of hydrogen.
2. Higher preheat and interpass temperatures.
3. Postheat.
4. Retarded cooling (insulating blankets).

Only low hydrogen electrodes should be used for fabrication and/or repair of seismically loaded structures. Proper preheat and interpass temperatures should be maintained. AWS D1.1 requirements are generally adequate for new construction. Highly restrained repair welds may require higher preheat levels.

Control of hydrogen and proper preheat and interpass temperature is much more powerful for overcoming HIC than postheat or retarded cooling methods. Retarded cooling has limited benefit if the entire piece is not preheated - obviously impractical for structural applications.

The engineer is encouraged to emphasize proper preheat and the use of low hydrogen electrodes and practice. If these measures are insufficient to prevent cracking, a welding engineer should be consulted to determine appropriate measures that should be incorporated to eliminate cracking. These measures may or may not call for additional preheat, postheat, or retarded cooling.

While low hydrogen electrodes and proper preheat is essential, postheat and retarded cooing is not generally required and should not be used for routine construction or repair.

Commentary: There are two primary purposes for preheating and interpass temperature requirements:

(1) To drive off any surface moisture or condensation which may be present on the steel so as to lessen the possibility of hydrogen being introduced into the weld metal and HAZ, and

(2) To prevent the steel mass surrounding the weld from quenching the HAZ as cooling occurs after welding.

Virtually all weld repairs are made under conditions of high restraint. Consequently, higher preheat/interpass temperatures may be required for repair applications. As steel is cooled from the austenitic range (above about 1330 degrees F), it goes through a critical transition temperature. If it goes through that temperature range too fast, a hard, brittle phase called martensite forms (quenching). If it passes through that temperature range at a slower rate, ductile, tougher phases called bainite or ferrite/pearlite form. Preheating of the surrounding mass provides a slower cooling rate for the weld metal and HAZ.

The American Association of State Highway and Transportation Officials (AASHTO) recognizes repair welding as more critical in its guidelines for the repair of fracture-critical bridge members. The purpose, in part, is to allow more plastic flow and yielding, at welding temperatures, in the area near the weld. The requirements are given in Table 8-6:

Table 8-6 - AASHTO Preheat Requirements for Fracture Critical Repairs[1]

Steel	*Thickness, in.*	*Minimum Preheat/Interpass Temp., °F*
A36/A572	*to 1-1/2*	*325*
A36/A572	*>1-1/2*	*375*

1- Reference AASHTO/AWS D1.5-95 Bridge Welding Code

Preheat temperatures should be measured at a distance from the weld equal to the thickness of the part being welded, but not less than three inches, in any direction including the through thickness of the piece. Where plates are of different thicknesses, the pre-heat requirement for the thicker plate should govern. Maintenance of these temperatures through the execution of the weld (i.e. the interpass temperature) is essential. Maximum interpass temperatures should be limited to 550 degrees F for prequalified WPSs, for fracture-critical applications. Higher interpass temperatures could be employed if those higher temperature limits are qualified by test.

8.2.5 Postheat

Postheat is the application of heat in the 400 degrees F to 600 degrees F range after completion of welding. It may be helpful in mitigating some cracking tendencies.

Commentary: A postheat specification might require that complete joint penetration groove welds in existing buildings be postheated at 450 degrees F for two hours. The purpose of this postheat is to accelerate the removal of hydrogen from the weld metal and HAZ and reduce the probability of cracking due to hydrogen embrittlement. Hydrogen will migrate within the weld metal at approximately 1 inch per hour at 450 degrees F, and at about 1 inch per month at 70 degrees F. To the extent that hydrogen embrittlement is of concern, postheat *is one method of mitigating cracking. The use of low hydrogen electrodes, proper welding procedures, and uniformly applied and maintained preheat may represent a cost-effective method of addressing the problem of hydrogen embrittlement in lieu of postheat.*

When postheat is required, AASHTO/AWS D1.5-95 specifications require this to be done immediately upon completion of welding. The postheat is between 400 to 500 degrees F for one hour minimum, for each inch of the thickest member or for two hours, whichever is less.

8.2.6 Controlled Cooling

Most of the weldment cooling is effected by conductance within the steel rather than radiation. Retarded cooling should only be specified in cases where large weldments subject to significant residual stresses due to restraint (e.g. multiple members framing into one connection with Z axis loading) or ambient temperatures that would result in rapid cooling of large weldments. The length of time to cool down the weld and the level of insulation required are a function of weldment temperature, thickness of base metal and ambient temperature.

Commentary: Active systems of ramp-down cooling are generally not required; however, in highly restrained conditions they may offer an added advantage.

8.2.7 Metallurgical Stress Risers

Metallurgical discontinuities such as tack welds, air-arc gouging and flame cutting without preheating or incorporation into the final weld should not be permitted. Inadvertent damage of this type should be repaired by methods approved by the engineer, following the AWS D1.1 criteria and a specific WPS covering repairs of this type.

Commentary: Metallurgical stress risers may result from tack welds, air-arc gouging and flame cutting performed without adequate preheat. However, preheating is not necessarily required for air arc gouging or flame cutting used in the preparation of a surface to receive later welding. The subsequent heat input during the welding process should adequately anneal the affected area. The AWS D1.1 code requires the same preheating for tack welding operations as normal welding, with the exception of tack welds that are incorporated into subsequent submerged arc weld deposits.

Arc strikes can also be a source of metallurgical stress risers and should not be indiscriminately made. AWS D1.1 Section 3.10 indicates that "arc strikes outside the area of permanent welds should be avoided on any base metal. Cracks or blemishes caused by arc strikes should be ground to a smooth contour and checked to ensure soundness."

8.2.8 Welding Preparation & Fit-up

Any cracked columns, welds, or beam flanges should be prepared to receive the welding contemplated by the engineer. AWS D1.1 provides guidance on the precise nature of the fit-up requirements and tolerances.

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