8. METALLURGY & WELDING

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 8.1.1-1. Refer to the applicable ASTM reference standard for detailed information.

Table 8.1.1-1 - 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)
ASTM A992[1]	Steel for Structural Shapes for Use in Building Framing
Notes: 1- See Commentary	

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 8.1.1-2. 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 8.1.1-2 - 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.~~ ASTM formally adopted the new specification for structural shapes, with a yield strength of 50 ksi, under designation A992 in 1998 and ~~It is anticipated that~~ domestic mills ~~will begin~~ have begun producing ~~structural~~ wide flange 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.~~

Since the formal approval of the A992 specification by ASTM occurred after publication of the 1997 editions of the building codes and the AISC Seismic Specification, it is not listed in any of these documents as a prequalified material for use in lateral force resisting systems. Neither is it listed as prequalified in AWS D1.1-98. However, all steel that complies with the ASTM-992 specification will also meet the requirements of ASTM A572, Grade 50 and should therefore be permissible for any application for which the A572 material is approved. See also, the commentary to Section 8.2.2.

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.

Although the 1994 editions of the Uniform Building Code and the NEHRP Provisions do not prequalify the use of ASTM A913 steel in lateral force resisting systems, the ~~pending~~ 1997 edition of the UBC does prequalify its use. Both the 1997 NEHRP Provisions and the AISC Seismic Provisions prequalify the use of this steel in elements that do not undergo significant yielding, for example, the columns of moment-resisting frames designed to meet strong column - weak beam criteria. Consequently, special approval of the Building Official should no longer be required as a pre-condition of the use of material conforming to this specification, at least for columns.

8.1.2 Chemistry

There are no modifications to the Guidelines of Section 8.1.2 at this time.

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~~ the new A992specification 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~~ Table 8.1.3-1 gives the basic mechanical requirements for commonly used structural steels. Properties specified, and controlled by the mills, in current practice include minimum yield strength or yield point, 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~~ Table 8.1.3-2 summarizes these results as well as data provided by a single producer for ASTM A913 material.

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.

~~Table 8-3~~ Table 8.1.3-1 - Typical Tensile Requirements for Structural Shapes

ASTM	~~Minimum~~ Yield Strength or Yield Point, Ksi	Ultimate Tensile Strength, Ksi	Minimum Elongation % in 2 inches	Minimum Elongation % in 8 inches
A36	36 Min.	58-80[1]	21[2]	20
A242	42[4] Min.	63 MIN.	21[3]	18
A572, Gr. 42	42 Min.	60 Min.	24	20
A572, GR50	50 Min.	65 MIN.	21[2]	18
A588	50 Min.	70 MIN.	21[3]	18
A709, GR36	36 Min.	58-80	21[2]	20
A709, GR50	50 Min.	65 MIN.	21	18
A913, GR50	50 Min.	65 MIN.	21	18
A913, GR65	65 Min.	80 MIN.	17	15
A992	50 Min. – 65 Max.	65 MIN	21	18

Notes:
1- No maximum for shapes greater than 426 lb./ft.
2- Minimum is 19% for shapes greater than 426 lb. /ft.
3- No limit for Shape Groups 1, 2 and 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, and 42 ksi for Shape Groups 4 and 5.

~~Table 8-4~~ Table 8.1.3-2 - Statistics for Structural Shapes[1,2]

Statistic	A 36	Dual 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.

2. Statistical Data on the distribution of strength properties for material meeting ASTM A992 are not presently available. Pending the development of such statistics, it should be assumed that A992 material will have similar properties to ASTM A572, Gr. 50 material.

Commentary: The data given in ~~Table 8-4~~ Table 8.1.3-2 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 approximately 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.

Until recently, ~~In wide flange sections the~~ tensile test coupons in wide flange sections ~~are currently~~ were 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. In 1998 ASTM A6 was revised to specify that coupons be taken from the flange of wide flange shapes.*

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~~ Table 8.1.3-2 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~~Table 8.1.3-2 - 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.

In the period since the initial publication of the Interim Guidelines, several researchers and engineers engaged in connection assembly prototype testing have reported that tensile tests on coupons extracted from steel members used in the prototype tests resulted in lower yield strength than reported on the mill test report furnished with the material, and in a few cases lower yield strength than would be permitted by the applicable ASTM specification. This led to some confusion and concern, as to how mill test reports should be interpreted.

The variation of the measured yield strength of coupons reported by researchers engaged in connection prototype testing, as compared to that indicated on the mill test reports, is not unusual and should be expected. These variations are the result of a series of factors including inconsistencies between the testing procedures employed as well as normal variation in the material itself. The following paragraphs describe the basis for the strengths reported by producers on mill certificates, as well as the factors that could cause independent investigators to determine different strengths for the same material.

Mill tests of mechanical properties of steel are performed in accordance with the requirements of ASTM specifications A6 and A370. ASTM A6 had historically required that test specimens for rolled W shapes be taken from the webs of the shapes, but recently was revised to require testing from the flanges of wide flange shapes with 6 inch or wider flanges. A minimum of two tests must be made for each heat of steel, although additional tests are required if shapes of significantly different thickness are cast from the same heat. Coupon size and shape is specified based on the thickness of the material. The size of the coupon used to test material strength can effect the indicated value. Under ASTM A6, material that is between 3/4 inches thick and 4 inches thick can either be tested in full thickness "straps" or in smaller 1/2" diameter round specimens. In thick material, the yield strength will vary through the thickness, as a result of cooling rate effects. The material at the core of the section cools most slowly, has larger grain size and consequently lower strength. If full-thickness specimens are used, as is the practice in most mills, the recorded yield strength will be an average of the relatively stronger material at the edges of the thickness and the lower yield material at the center. Many independent laboratories will use the smaller 1/2" round specimens, and sometimes even sub-sized 1/4" round specimens for tensile testing, due to limitations of their testing equipment. Use of these smaller specimens for thick material will result in testing only of the lower yield strength material at the center of the thickness.

ASTM A370 specifies the actual protocol for tensile testing including the loading rate and method of reporting test data. Strain rate can affect the strength and elongation values obtained for material. High strain rates result in elevated strength and reduced ductility. Under ASTM A370, yield values may be determined using any convenient strain rate, but not more than 1/16 inch per inch, per minute which corresponds to a maximum loading rate of approximately 30 ksi per second. Once the yield value is determined, continued testing to obtain ultimate tensile values can proceed at a more rapid rate, not to exceed 1/2 inch per inch per minute.

Under ASTM A370, there are two different ways in which the yield property for structural steel can be measured and reported. These include yield point and yield strength. These are illustrated in Figure 8.1.3-1. The yield point is the peak *stress that occurs at the limit of the elastic range, while the yield strength is a somewhat lower value, typically measured at a specified offset or elongation under load. Although a number of methods are available to determine yield point, the so-called "drop of the beam" method is most commonly used for structural steel. In this method the load at which a momentary drop-off in applied loading occurs is recorded, and then converted to units of stress to obtain the yield point. Yield strength may also be determined by several methods, but is most commonly determined using the offset method. In this method, the stress - strain diagram for the test is drawn, as indicated in Figure 8.1.3-1. A specified offset, typically 0.2% strain for structural steel, is laid off on the abscissa of the curve and a line is drawn from this offset, parallel to the slope of the elastic portion of the test. The stress at the intersection of this offset line with the stress-strain curve is taken as the yield strength.*

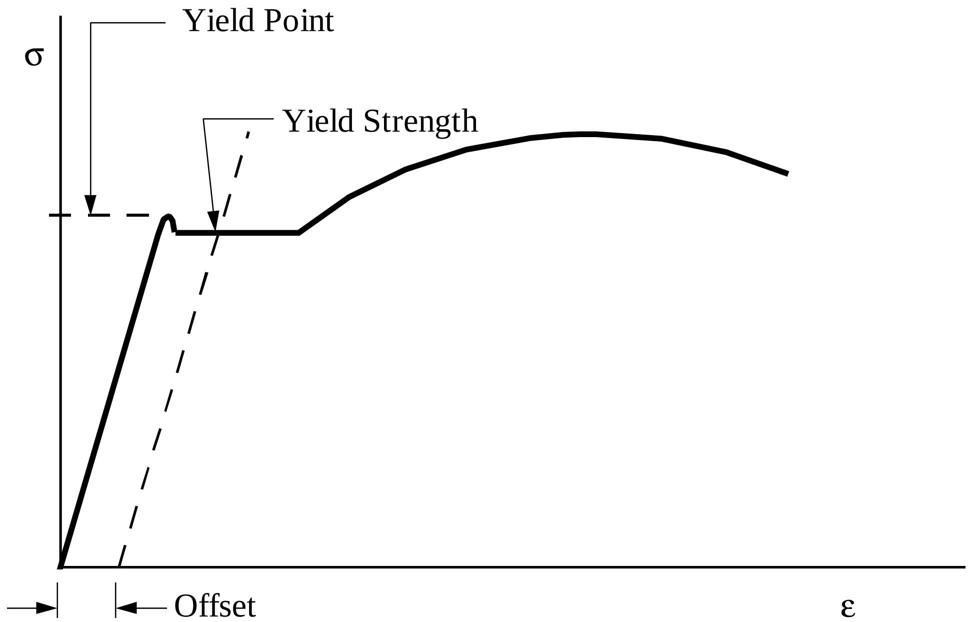

Figure 8.1.3-1 Typical Stress - Strain Curve for Structural Steel

The material specifications for structural steels typically specify minimum values for yield point but do not control yield strength. The SSPC has reported that actual practice among the mills varies, with some mills reporting yield strength and others reporting yield point. This practice is permissible as yield strength will always be a somewhat lower value than yield point, resulting in a somewhat conservative demonstration that the material meets specified requirements. However, this does mean that there is inconsistency between the values reported by the various mills on certification reports. Similarly, the procedures followed by independent testing laboratories may be different than those followed by the mill, particularly with regard to strain rate and the location at which a coupon is obtained.

Under ASTM A6, coupons for tensile tests had historically been obtained from the webs of structural shapes. However, most engineers and researchers engaged in connection testing have preferred to extract material specimens from the flanges of the shape, since this is more representative of the flexural strength of the section. Coupons removed from the web of a rolled shape tend to exhibit somewhat higher strength properties than do coupons removed from the flanges, due to the extra amount of working the thinner web material typically experiences during the rolling process and also because the thinner material cools more rapidly after rolling, resulting in finer grain size. Given these differences in testing practice, as well as the normal variation that can occur along the length of an individual member and between different members rolled from the same heat, the reported differences in strength obtained by independent laboratories, as compared to that reported on the mill test reports, should not be surprising. It is worth noting that following the recognition of these differences in testing procedure, the SSPC in coordination with AISC and ASTM developed and proposed a revision to the A6 specification to require test specimens to be taken from the flanges of rolled shapes when the flanges are 6 inches or more wide. ~~It is anticipated that mills will begin to alter practice to conform to a revised specification in early 1997~~ This has since become the standard practice.

The discovery of the somewhat varied practice for reporting material strength calls into question both the validity of statistics on the yield strength of structural steel obtained from the SSPC study, and its relevance to the determination of the expected strength of the material for use in design calculations. Although the yield point is the quantity controlled by the ASTM material specifications, it has little relevance to the plastic moment capacity of a beam section. Plastic section capacity is more closely related to the stress along the lower yield plateau of the typical stress-strain curve for structural steel. This strength may often be somewhat lower than that determined by the ~~offset~~ drop-of-the-beam method. Since the database of material test reports on which the SSPC study was based appears to contain test data based on both the offset and drop-of-the-beam methods, it is difficult to place great significance in the statistics derived from it and to draw a direct parallel between this data and the expected flexural strength of rolled shapes. It would appear that the statistics reported in the SSPC study provide estimates of the probable material strength that are somewhat high. Thus, the recommended design strengths presented in Tables 6.6.6.3-1 and 7.5.1-1 of the Interim Guidelines would appear to be conservative with regard to design of welds, panel zones and other elements with demands limited by the beam yield strength.

Under the phase II program of investigation, SAC, together with the shape producers, is engaged in additional study of the statistical distribution of yield strength of various materials produced by the mills. This study is intended to provide an improved understanding of the statistical distribution of the lower *yield plateau strength of material extracted from section flanges, measured in a consistent manner. In addition, it will provide correlation with yield strengths determined by other methods such that the data provided on mill test certificates can be properly interpreted and utilized. In addition, the possibility of revising the ASTM specifications to provide for more consistent reporting of strength data as well as the reporting of strength statistics that are directly useful in the design process will be evaluated. In the interim period, the data reported in Table 8-1.3-2, extracted from the SSPC study, remain the best currently available information.*

8.1.4 Toughness Properties

There are no modifications to the Guidelines or Commentary of Section 8.1.4 at this time.

8.1.5 Lamellar Discontinuities

There are no modifications to the Guidelines or Commentary of Section 8.1.5 at this time.

8.1.6 K-Area Fractures

~~Recently, there have been~~In the period 1995-96 there were several reports of fractures initiating in the webs of column sections during the fabrication process, as flange continuity plates and/or doubler plates were welded into the sections. This fracturing typically initiated in the region near the fillet between the flange and web. This region has been commonly termed the “k-area” because the AISC Manual of Steel Construction indicates the dimension of the fillet between the web and flange with the symbol “k”. The k-area may be considered to extend from mid-point of the radius of the fillet into the web, approximately 1 to 1-1/2 inches beyond the point of tangency between the fillet and web. The fractures typically extended into, and sometimes across, the webs of the columns in a characteristic “half-moon” or “smiley face” pattern.

Investigations of materials extracted from fractured members have indicated that the material in this region of the shapes had elevated yield strength, high yield/tensile ratio, high hardness and very low toughness, on the order of a few foot-pounds at 70°F. Material with these properties can behave in a brittle manner. Fracture can be induced by thermal stresses from the welding process or by subsequent weld shrinkage, as apparently occurred in the reported cases. There have been no reported cases of in-service k-line fracture from externally applied loading, as in beam-column connections, although such a possibility is perceived to exist under large inelastic demand.

It appears that this local embrittling of sections can be attributed to the rotary straightening process used by some mills to bring the rolled shapes within the permissible tolerances under ASTM A6. The straightening process results in local cold working of the sections, which strain hardens the material. The amount of cold working that occurs depends on the initial straightness of the section and consequently, the extent that mechanical properties are effected is likely to vary along the length of a member. The actual process used to straighten the section can also affect the amount of local cold working that occurs.

Engineers can reduce the potential for weld-induced fracture in the k-area by avoiding welding within the k-area region. This can be accomplished by detailing doubler plates and continuity plates such that they do not contact the section in this region. The use of large corner clips on beam flange continuity plates can permit this. Selection of column sections with thicker webs, to eliminate the need for doubler plates; the use of fillet welds rather than full penetration groove welds to attach doubler plates to columns, when acceptable for stress transfer; and detailing of column web doubler plates such that they are offset from the face of the column web can also help to avoid these fabrication-induced fracture problems.

Commentary: It appears that detailing and fabrication practice can be adjusted to reduce the potential for k-area fracture during fabrication. However, the acceptability of having low-toughness material in the k-area region for service is a question that remains. It is not clear at this time what percentage of the material incorporated in projects is adversely affected, or even if a problem with regard to serviceability exists. SAC recently placed a public call, asking for reports of fabrication-induced fractures at the k-area, but only received limited response. However, in one of the projects that did report this problem, a significant number of columns were affected. This may have been contributed to by the detailing and fabrication practices applied on that project.

Other than detailing structures to minimize the use of doubler plates, and to avoid large weldments in the potentially sensitive k-area of the shape, it is not clear at this time, what approach, if any, engineers should take with regard to this issue. There are several methods available to identify possible low notch toughness in structural carbon steels, including Charpy V-Notch testing and hardness testing of samples extracted from the members. However, both of these approaches are quite costly for application as a routine measure on projects and the need for such measures has not yet been established.

Following publication of advisories on the k-line problem by AISC, and the publication of similar advisory information in FEMA-267a,reports on this problem diminished. It is not clear whether this is due to revised detailing practice on the part of engineers and fabricators, revised mill rolling practice, or a combination of both. SAC, AISC and SSPC are continuing to research this issue in order to identify if a significant problem exists, and if it does, to determine its basic causes, and to develop appropriate recommendations for mill, design, detailing, and fabrication practices to mitigate the problem.

8.2 Welding

8.2.1 Welding Process

There are no modifications to the Guidelines or Commentary of Section 8.2.1 at this time.

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: A welding procedure specification identifies all the important parameters for making a welded joint including the material specifications of the base and filler metals, joint geometry, welding process, requirements for pre- and post-weld heat treatment, welding position, electrical characteristics, voltage, amperage, and travel speed. Two types of welding procedure specifications are recognized by AWS D1.1. These are prequalified procedures and qualified-by-test procedures. Prequalified procedures are those for which the important parameters are specified within the D1.1 specification. If a prequalified procedure is to be used for a joint, all of the variables for the joint must fall within the limits indicated in the D1.1 specification for the specific procedure. If one or more variables are outside the limits specified for the prequalified procedures, then the fabricator must demonstrate the adequacy of the proposed procedure through a series of tests and submit documentation (procedure qualification records) demonstrating that acceptable properties were obtained. Regardless of whether or not a prequalified or qualified-by-test procedure is employed, the fabricator should prepare a welding procedure specification, which should be submitted to the engineer of record for review and be maintained at the work location for reference by the welders and inspectors. The following information is presented to help the engineer understand some of the issues surrounding the parameters controlled by the welding procedure specification.

~~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.*

Recently, ASTM approved a new material specification for structural steel shape, ASTM A992. This specification is very similar to the ASTM A572, Grade 50 specification except that it includes additional limitations on yield and tensile strengths and chemical composition. Although material conforming to A992 is expected to have very similar welding characteristics to A572 material, it was adopted too late to be included as a prequalified base material in AWS D1.1-98. Although the D1 committee has evaluated A992 and has taken measures to incorporate it as a prequalified material in AWS D1.1-2000, technically, under AWS D1.1-98, welded joints made with this material should follow qualified-by-test procedures.

In reality, structural steel conforming to ASTM A992 may actually have somewhat better weldability than material conforming to the A572 specification. This is because A992 includes limits on carbon equivalent, precluding the delivery of steels where all alloys simultaneously approach the maximum specified limits. Therefore, it should be permissible to utilize prequalified procedures for joint with base metal conforming to this specification.

8.2.3 Welding Filler Metals

There are no modifications to the Guidelines of Section 8.2.3 at this time.

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. In evaluation of welds in buildings affected by the Northridge earthquake, the parameters found to be most likely to result in damage-susceptible welds included 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.g. E71T1)*
$E^1X^2X^3T^4X^5X^6$	*For electrodes specified under AWS A5.29 (e.g. E70TGK2)*
$E^1XX^7X^8X^9X^{10}$	*For electrodes specified under AWS A5.1 or AWS A5.5. (e.g. E7018)*

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.

***Low-hydrogen electrodes**. Low hydrogen electrodes should be used to minimize the risk of hydrogen assisted cracking (HAC) when conditions of high restraint and the potential for high hardness microstructures exist. Hydrogen assisted cracking can occur in the heat affected zone or weld metal whenever sufficient concentrations of diffusible hydrogen and sufficient stresses are present along with a hard microstructure at a temperature between 100 C and –100 C. Hydrogen is soluble in steel at high temperatures and is introduced into the weld pool from a variety of sources including but not limited to: moisture from coating or core ingredients, drawing lubricants, hydrogenous compounds on the base material, and moisture from the atmosphere.*

At the present time, the term "low hydrogen" is not well defined by AWS. The degree of hydrogen control required to reduce the risk of hydrogen assisted cracking will depend on the material being welded, level of restraint, preheat/interpass temperature, and heat input level. When a controlled level of diffusible hydrogen is required, electrodes can be purchased with a supplemental designator that indicates a diffusible hydrogen concentration below 16, 8, or 4 ml

H_2/100g in the weld metal can be maintained (H16, H8, and H4 respectively) under most welding conditions .

The diffusible hydrogen potential (measured in ml/100g deposited weld metal) will depend on the type of consumable, welding process, plate/joint cleanliness, and atmospheric conditions in the area of welding. Some consumables may absorb moisture after exposure to the atmosphere. Depending on the type of consumable, this may result in a significant increase in the weld metal diffusible hydrogen concentration. In situations where control of diffusible hydrogen concentrations is important, the manufacturer should be consulted for advice on proper storage and handling conditions required to limit moisture absorption.

Hydrogen assisted cracking may be avoided through the selection and maintenance of an adequate preheat /interpass temperature and/or minimum heat input. Depending on the type of steel and restraint level, a trade-off between an economic preheat/interpass temperature and the diffusible hydrogen potential of a given process exists. There have been several empirical approaches developed to determine safe preheat levels for a given application that include consideration of carbon equivalent, restraint level, electrode type, and preheat. When followed, the guidelines for preheat that have been established in AWS D1.1 and D1.5 are generally sufficient to reduce the risk of hydrogen assisted cracking in most mild steel weldments.

Hydrogen assisted cracking will typically occur up to 72 hours after completion of welding. For the strength of materials currently used in moment frame construction, inspection of completed welds should be conducted no sooner than 24 hours following weld completion.

8.2.4 Preheat and Interpass Temperatures

There are no modifications to the Guidelines or Commentary of Section 8.2.4 at this time.

8.2.5 Postheat

There are no modifications to the Guidelines or Commentary of Section 8.2.5 at this time.

8.2.6 Controlled Cooling

There are no modifications to the Guidelines or Commentary of Section 8.2.6 at this time.

8.2.7 Metallurgical Stress Risers

There are no modifications to the Guidelines or Commentary of Section 8.2.7 at this time.

8.2.8 Welding Preparation & Fit-up

There are no modifications to the Guidelines or Commentary of Section 8.2.8 at this time.