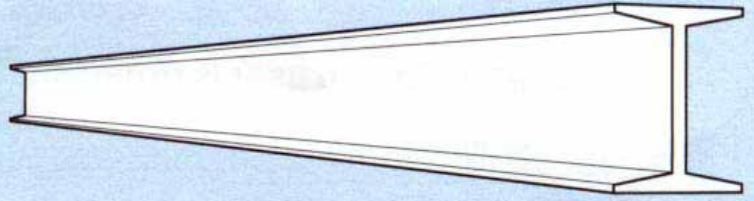


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December 2003

Steel Construction in the New Millennium

By

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This report rewrites a Steel TIPS, “Steel in the 90’s”, that was written by Robert Preece, and Alvaro L. Collin, which was very popular with structural engineers, fabricators, erectors, and inspectors. The SSEC wished to update this publication to include a discussion of new steels, new welding issues, some lessons learned from the Northridge earthquake, and pertinent developments derived from the SAC investigations. Some sections are unchanged, others are new, while others have been updated, amended, or completely revised.

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STRUCTURAL STEEL CONSTRUCTION IN THE NEW MILLENNIUM

By:

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Preface

Previous publications on this subject for the Structural Steel Educational Council (SSEC), “Structural Steel in the 80’s – Materials, Fastening and Testing,” by F. Robert Preece, and “Structural Steel Construction in the 90’s,” by F. Robert Preece, and Alvaro L. Collin, were very popular with structural engineers, fabricators, erectors, and inspectors. The SSEC wished to update this publication to include a discussion of new steels, new welding issues, some lessons learned from the Northridge earthquake, and pertinent developments derived from the SAC investigations. Some sections are unchanged, others are new, while others have been updated, amended, or completely revised.

This publication is intended to be a quick reference guideline. See the reference list for more detailed information.

Introduction

Today’s engineer has available a wide variety of structural shapes, grades of materials and joining techniques with which to design steel-frame buildings and similar structures. This proliferation of choices has come about through rapid technological advances in steelmaking and related industries over the past four decades.

The development of today’s high-strength, low alloy steels has achieved major material and energy savings in the steel fabricating process, while making possible a variety of structural shapes available to the designer. Care should be given to specifying high strength steels, especially with regard to deflection and vibration, as shall be explained herein.

This paper will review the trends in structural steel design, from massive all-riveted and bolted structures of the 40’s and 50’s to today’s simpler, lighter and more cost-efficient welded designs. As an aid to the architect and engineer, this paper will also describe the most commonly available construction steels, their features and important applications. Of particular importance will be the special considerations for “matching” electrodes to base metals, the influence the design has on joint performance, and the quality control procedures used to assure structurally sound joints.

Also of interest to the young and experienced structural engineer is an historical perspective of

the structural steel building industry and the important developments in the art and science of steel welding and the methods of minimizing weld defects.

Early Building Design

The use of steel in building construction has its roots at the end of the last century with the construction of such towering edifices as the 14-story Tacoma Building in Chicago and New York’s Tower Building, rising to a height of 129 feet.

As early designers recognized the greater strength and ductility of mild carbon steel over cast and wrought iron, office buildings grew taller and taller. However, it remained until the late 1920s before high-rise building construction became truly economical. This economy was due largely to technological advances made in the rolling process that permitted the forming of large structural shapes, coincident with the development of high-speed elevators.

Until then, steel mills rolled so-called “standard” shapes: angles, channels, I-beams and plates, which were then riveted together to make the larger columns and girders required for high-rise construction. Built-up shapes such as these are inordinately expensive by today’s standards due to the tremendous cost of detailing, drilling, riveting and assembling. Over the years, some loosely defined terms have been used to describe carbon steels. Generally, the agreed terms used in reference manuals are as follows:

Low Carbon Steel	$C \leq .20\%$
Medium Carbon Steel	$0.20 < C < .050\%$
High Carbon Steel	$C \geq .50\%$

The built-up shapes used in early building construction were necessary due to the inherent nature of the rolling process. At the time, steel mills used a single set of rolls for each structural shape. The single-roll process prevented increasing the thickness of a structural shape in one direction (e.g., the flange) without correspondingly increasing its thickness in another (web). This represented a very inefficient way of producing large shapes.

By the late 1920’s, however, the (Gray) wide flange mill had arrived on the scene. These mills used several sets of rolls arranged in tandem. Operators could now roll relatively wide-flange

shapes to 36 inches deep, while keeping the web sections relatively thin.

These wide flange shapes revolutionized the steel building industry and made possible the construction of the Empire State Building and the 30 or so other skyscrapers that dotted New York's skyline in the late 1930's.

Meanwhile, during this same period, some far-sighted designers were advocating welding, and fabricators recognized the advantage of using fewer materials with a method that would gain 100 percent elastic continuity. This, coupled with the development of the Structural Welding Code by AWS, led to the demise of riveting and the development of the all-welded joint.

Riveting, A Lost Art

The earliest methods of joining steel were by rivets and bolts. Rivets were used before 1850 in boiler work and shipbuilding. Mild steel bolts became popular around the same period for joining heavy machine parts.

During the early building industry, before the development of today's high-strength bolts, rivets were considered stronger than bolts because they filled the holes and prevented any slippage of the joint. Consequently, they enjoyed almost exclusive use in the first steel-framed buildings. The exception generally occurred in the connection of beams to girders, where the loads were relatively small and the set-up time to drive a few rivets per connection was too expensive.

Existing riveted bridges have been retrofitted by the addition of plates and the replacement of rivets with high strength bolts. This process requires a rigorous survey locating the center of each existing rivet. These rivet locations are used in the detailing and fabrication of new plates. Rivets are then removed and holes are generally reamed for new bolt installation. Reaming is necessary because rivets were very forgiving to hole tolerances, and the existing holes on all plies are not necessarily aligned.

High Strength Bolting

By 1950, high strength bolts were being given strong consideration as a replacement for rivets in high-rise buildings. A development of the railroad industry and used in heavy machinery, these bolts have a tensile strength on the order of

approximately twice their mild steel counterparts. For high strength bolts, yield or proof strength is important. A325 is 105 min ksi tensile, and A490 is 150 ksi min tensile. The proof value of A325 is 2x that of carbon steel A307 bolts, and the proof value of A490 is more than 3x that of the A307 bolts. High strength bolts are torqued so tightly that a tremendous clamping force is produced. Load is initially transferred between members by friction between the two mating surfaces, rather than by bolt bearing. This friction produces a stiffer and more rigid joint, and hence there is less distortion of the frame. If the slip load is reached, the bolts go into bearing. For seismic frames, this bolt slippage is considered a benefit as it dissipates energy. The Structural Engineer is cautioned, however, to consider the global distortion of the entire frame in the event that slippage occurs. Currently, the AISC Seismic provisions allow for use of bearing values to develop seismic forces, but bolts shall be tightened to slip critical specifications.

Today, high strength bolts are again becoming popular for field connecting moment frame connections and lateral bracing of heavy and tall buildings. Prior to the Northridge earthquake, high welding deposition rates allowed the welding on brace frames and moment frames to keep up with the erection of buildings. The lessons learned from the Northridge earthquake taught us that better joint details, high notch-toughness electrodes, along with more rigorous visual inspection, would be required to ensure seismic connection performance. These higher notch toughness electrodes have relatively slower deposition rates. Hence, the field welding process is a more time consuming operation due to these developments. Welding can often be the critical path task on completion of the construction of a steel frame. Due to these new developments, and depending on the regional demand and supply of qualified welders, some erectors would prefer bolted connections on a moment frame or braced frame building.

The AISC Seismic Provision currently requires "SC," or slip critical tightening on seismic joints, but allows engineers to use bearing values for design. Some erectors prefer to tighten by the "Turn of the Nut Method" and will fully impact tighten all bolts regardless of the SC specification. Other erectors prefer the use of tension control (TC or twist off) bolts or Direct Tension Indicating (DTI Washers). Regardless

of the method, field bolt pretension inspection should be limited to those connections that are specified to be slip critical. Other appropriate applications for slip critical bolting include bridge connections, wind resisting connections, and connections for mechanical equipment or any connections where loading can be cyclic, dynamic or vibratory.

Bearing bolts should be used wherever “SC” is not required. The most efficient use of bearing bolts is the “X” specification, which requires the threads excluded from the shear plane. The AISC Manual of Steel Construction, LRFD Third Edition, has a Table 7-2 providing parameters for threads excluded from the shear plane, based on grip and ply thickness. The erector should be able to describe their quality control procedure in writing, to ensure the use of the proper length bolt for a given grip.

Alloying – The Key to Strength and Weldability

All structural steels derive their strength characteristics from the addition of various alloys, especially carbon, and in the case of A514, by heat treatment, in addition to alloying.

Carbon, in the form of iron carbide, or in solution, is the basic alloying agent for hardening steel and increasing its tensile strength. This is done at some sacrifice to its ductility (its ability to stretch without failure), and its weldability.

Manganese is also a powerful hardening agent, serving as a ferrite strengthener. Molybdenum, chromium, vanadium and columbium are also added in small amounts, and they increase strength and toughness.

The hardness of structural steels is important as it relates not only to strength, but also weldability. Too hard a steel leads to difficulties in welding and to weld defects, such as brittle fracture and shrinkage cracking. Hardness does not cause weld defects but higher strength decreases the tolerance for flaws and increases the level of restraint. Thus, more precaution is needed for higher strength steel. To ensure good weldability, ASTM standards limit the amount of alloying elements added. “Carbon Equivalent” discussed below, is a quantitative representation of the alloying elements in a particular steel, useful for judging weldability.

Following is a quick reference of various alloying elements and their roles in structural steel:

Carbon, Manganese:

Basic Hardenability, Strengtheners

Vanadium, Columbium (also known as Niobium):

Carbide & Carbo-Nitride Stabilization, Strengthening, Grain size control

Nitrogen:

Carbide & Carbo-Nitride Stabilization, Strengthening,

Chromium Molybdenum, Nickel, Boron:

Hardenability Agents, Tempering Control

Nickel:

Improves Notch Toughness

Silicon:

Deoxidizer

Aluminum:

Deoxidizer, and grain refiner

Copper: *Helps resist atmospheric corrosion and is a precipitation strengthener*

Carbon Equivalent and Preheat

Dr. Winterton of the British Welding Institute, while studying the effects of various elements in the chemical composition of steels on their mechanical properties, noticed their effect on hardness. Knowing that hardness is related to weldability and susceptibility to cracking, he developed the relationship of the chemical elements in steel to its hardness through a Carbon Equivalent formula, basically because carbon has the greatest effect on hardness, strength and weldability in steels. Others in the USA during WWII and immediately after also developed carbon equivalent formulas.

Many different Carbon Equivalent (CE) formulas are used as a guide for pre-heat requirements and in welding procedures. The AWS D1.1-2002 Appendix XI5.1 Formula is used for structural steels:

$$CE = C + \frac{(Mn + Si)}{6} + \frac{(Cr + Mo + V)}{5} + \frac{(Ni + Cu)}{15}$$

In % to determine the Zone requirements for pre-heat.

Given the Carbon Content, one can refer to Figure 1, to determine the zone classification as a guide for the selection of either the hardness control or the hydrogen control method of

determining preheat. See the AWS D1.1 for more details.

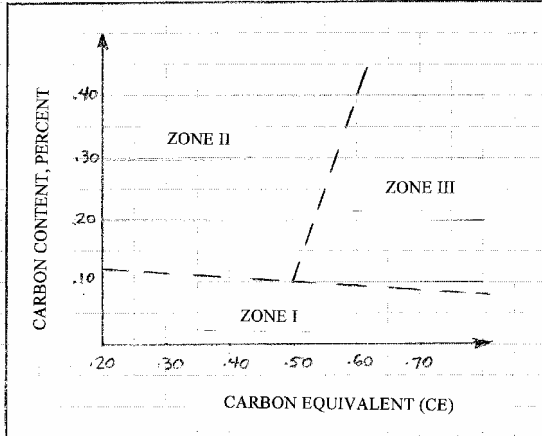


Fig. 1 Zone Classifications of Steel

Note: Chart based on AWS D1.1 2002 Annex XI Fig. XI-1. This method does not take thickness into effect.

Zone I Cracking is unlikely, but may occur with high hydrogen or high restraint. Use hydrogen control method to determine preheat.

Zone II The hardness control method and selected hardness shall be used to determine minimum energy input for single pass fillet welds without preheat.

For groove welds, the hydrogen control method shall be used to determine preheat.

For steels with high carbon, a minimum energy to control hardness and preheat to control hydrogen may be required for both types of welds.

Zone III The hydrogen control method shall be used.

Quenching and Tempering for High Strength

To obtain a high-strength steel other than by alloying it, it is possible to heat-treat certain steel formulations by quenching and tempering. Quenching hardens or strengthens steel. Tempering increases ductility but lowers tensile strength. Temperature and time are important. This procedure requires the steel to be heated initially to an elevated temperature above the upper critical to form a crystalline structure known as austenite. Rapid cooling in water to produce martensite or a partially martensitic microstructure follows this. When tempered to precipitate a fine dispersion of carbides, this structure has good ductility and fracture

toughness, along with high strength.

Of the many grades of structural steel, only A514, A852, A709 and A-913 are quenched and tempered. Yield strengths are on the order of 90 to 100 ksi, almost three times that of A36 steel and twice that of other grades. Of course, the cost of this special treatment is reflected in a significantly higher price.

Today's Structural Steels

At the turn of the century, there were only two grades of steel being produced: A-7 for bridges and A-9 for buildings (See Table A). These materials were so similar in chemical composition and mechanical properties that in 1939 they were consolidated by ASTM into one standard, A-7, which was used for both types of applications until 1960. When a higher strength was needed, primarily for bridges, silicon steel was specified. It was very difficult to weld so it was primarily used in riveted and bolted construction.

By the 1950's, the strength and economics of welding were becoming apparent. Despite earlier bridge failures in the railroad industry, which were more related to poor workmanship and technique than to materials, welding was being tried in some high-rise structures on the Gulf Coast. Here, structural engineers applied the experience gained in the welding of refineries and oil pipelines to high-rise buildings. Designers soon noticed that while most of the A-7 steel being used showed good welding characteristics, there were instances in which some A-7 steel presented welding problems because of the limited chemistry requirements. Thus, extensive research began into the metallurgy of steel to develop structural materials that were uniformly weldable.

By 1964, the American Institute of Steel Construction (AISC) had adopted five grades of steel for structural application. Table B shows the chemical composition and some of the mechanical properties of these high strength, low alloy steels. The high elongation property of today's steel (up to 25 percent) permits large overstress, due to welding and deformation during construction, without losing its ultimate strength. Figure 2 compares typical tensile stress-strain curves for these steels.

Today, the ASTM Specifications for Structural Steel cover many carbon steels, high-strength, low-alloy steels and some quenched and tempered construction alloys. Structural steels include plates, bars, shapes, pipes, and structural tubing. There are many acceptable processes for welding these steels.

Materials Specification

The following tables and sections discuss various structural materials, their properties, their availability in shapes and plates, and applicability for use. The reader should also refer to the AISC Manual of Steel Construction, LRFD Third Edition, page 2-20 through 2-29 for a complete listing and discussion of applicable and available materials.

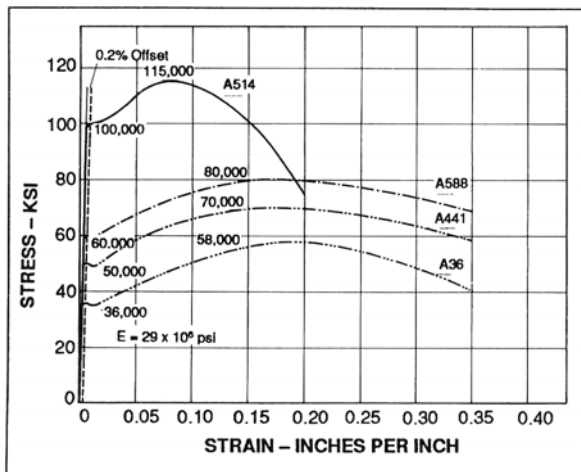


Fig. 2 Engineering stress-strain tensile curves for ASTM structural steel grades A-514, A-572 GR.60, A-441, A-588, A-572 GR.50 and A-36.

Special Material Considerations, Supplementary Requirements

In certain circumstances, Charpy V notch toughness testing of base material is required. It is required to be specified in the contract documents when applicable, per the AISC Third Edition LRFD Specification Section A3.1c.

The general requirement currently applies when ASTM A6 Group 4 and 5 rolled shapes (see Table 2-4 page 2-27 of AISC Manual of Steel Construction, LRFD Third Edition for “Group” definitions), or plates exceeding 2 in. thick, are used as members subject to primary tensile stresses due to tension or flexure, and are spliced using complete-joint-penetration groove welds. The impact test shall meet a minimum average value of 20 ft-lbs. absorbed energy at 70 degrees F. See the AISC specification Section A3.1c and corresponding commentary for a more complete discussion.

For Seismic applications, the 2002 AISC Seismic Provisions for Structural Steel Buildings, Section 6.3 specifies that for members of the Seismic Load Resisting System, ASTM A6 Groups 3,4 and 5 shapes with flanges 1 ½ in. thick and thicker, and plates that are 2 in. thick or thicker shall have a minimum Charpy V-Notch toughness of 20 ft-lbs. at 70 degrees F, determined as specified in LRFD Specification Section A3.1c. An important consideration is the frequency of testing. AISC specifies P, Piece frequency for thick plate. Alternatively, heat testing frequency of thick shapes substantially reduces testing costs and merits consideration.

The toughness requirement is intended to ensure a reasonable toughness of the base metal to preclude cracking. The steel must be able to withstand the tremendous tensile strains due to weld shrinkage generated during complete penetration welding of these thick members. The web to flange intersection of these heavy hot-rolled shapes, as well as interior portions of heavy plates may contain a coarser grain structure and/or a lower toughness due to a slower cooling rate, as well as other factors.

Table A
Historical Background of Structural Steels

ASTM REQUIREMENTS

Dates	Specification	Definition	Tensile Strength ksi	Yield Point ksi Minimum	Elongation & Chemistry
1900-1904	ASTM-A7	Soft to medium steels	52-70	32-35	No req. but usually listed
	ASTM-A9 Buildings	Medium Steel	60-70	35	No req. but usually listed
NOTE- Basic Unit stresses recommended by mgf: bldgs.-16.0 ksi, bridges, 12.5 ksi					
1905-1913	ASTM-A7 Bridges	Structural steel	60 desired	No req. But Reported	Not req. but usually reported
	ASTM-A9 Buildings	Medium & Structural steel	55-65	1/2 x T.S.	Not req. but usually reported
1914-1934	ASTM-A7 Bridges	Structural steel	55-65	1/2 x T.S.	No req. but usually reported
	ASTM-A9 Buildings	Structural steel	55-65	1/2 x T.S. 30 min.	No req. but usually reported
NOTE- AISC 1923 Basic Unit Stress = 18.0 ksi					
1934-1938	ASTM-A7 Bridges	Structural steel	60-72	1/2 x T.S. 33 min.	Usually reported
	ASTM-A9 Buildings	Structural steel	60-72	1/2 x T.S. 33 min.	
NOTE- AISC 1936 Basic Stress Unit = 20.0 ksi					
1938-1949	ASTM -A7 Buildings & Bridges	Structural steel	60 -72	1/2 x T.S. 33 min.	
NOTE- ASTM-A7 & A9 consolidated into one spec (ASTM A7)					
1954	ASTM-A373	Structural steel	58- 72	32	El.=24% (8" Ga.)
NOTE:Revised 1958; phased out when A-36 was issued (1960)					
1957	ASTM -A572	Structural steel	60-80	42-65	15-20% (8"Ga.) 17-24% (2" Ga.)
NOTE- Structural steel for riveted, bolted or welded construction					
1960	ASTM-A36 Buildings & Bridges	Structural steel	60-80	36 ksi min.	20% (8" Ga.) 23% (2"Ga.)
NOTE- Issued 1960, revised 1961 (called out as weldable) C = .28, Mn = .80-1.20, P = .04, S =.05, Si= .15-.30, Cu =.20 if specified 1961-A7: Tensile ksi - 60-75, Fy = 33 ksi, Elong (8") = 21.0% C = Not specified, Mn = N.S., Phos = .04, S= .05, Cu = 20 min. when specified					
1988	ASTM-A36 Buildings & Bridges	Structural steel	58-80	36 ksi min.	20% (8" Ga.) 23% (2" Ga.)
NOTE- Structural steel for riveted, bolted or welded construction Nominal chemistry % (Refer to ASTM specs for detail) C = 0.26-0.29, Mn = .80-1.20, Phos =0.04, S = 0.05, Si =0.15-0.40, Cu (when specified) = 0.20 min.					
1991	ASTM-A913	Structural steel	65-90	50-70	14-18% (8" Ga.) 16-21% (2" Ga.)
NOTE- High strength quenched and self tempered Structural Steel					
1999	ASTM- A992	Structural Steel	65	50-65	18% (8" Ga) 21% (2" Ga)
NOTE- Structural steel developed for seismic applications; killed, max carbon equiv = 0.45 to .47% Max yield point = 65, max yield to tensile ratio=0.85					

NOTE- based on data from AISC: Iron & Steel Beams- 1873 to 1952; past issues of ASTM Specs, AWS Codes (including First Edition - 1928), AISC Specs and other publications on steel.

Table B
ASTM Specifications

<u>ASTM SPEC.#</u>	<u>Mechanical Properties</u>			<u>Chemical Requirements %</u>						<u>Other & Notes</u>
	<u>Tensile</u> ksi	<u>Yield Str.</u> ksi	<u>Elong % 8"uno</u> Ksi	<u>C</u>	<u>Mn</u>	<u>Phos.</u>	<u>Sulf.</u>	<u>Si</u>	<u>Cu</u>	
<u>Plates and Shapes</u>										
A36	58-80	36 min.	20	0.26	0.80-1.20	0.04	0.05	0.15-0.40	0.20 min	Plate to 8", channels and angles
A913	65-90	50-70	14-18	0.12-0.16	1.60	0.03-0.04	0.03	0.40	0.35-0.45	Special Order- Quenched and self tempered
A992	65	50-65	18	0.23	0.5-1.5	0.035	0.045	0.40	0.6	Wide Flange- common specificaton
A572	60-80	42, 65	15- 20	0.21-0.26	1.35	0.04	0.05	0.15-0.40		Wide flange, Plate Gr. 42 To 6", Channel and angle in large special orders only
A588	63-70	42,46,50	18	0.10-0.19	0.50-1.35	0.04	0.05	0.15-0.90	0.20-0.50	Plate to 8" and all Shapes- Corrosion Resistant - special order
A709	58- 130	36-100	18-20	Large variations by grade, see			ASTM	spec		Bridge steels to 4" PL- special order Weathering steel in Grade 50, 70 & 100
A759	varies	varies	--	0.67-0.84	0.7-1.1	0.04	0.05	0.10-0.50	--	Crane rails- design by manufacturers
A852	90-110	70	19	0.19	0.8-1.35	0.035	0.04	0.20-0.65	0.20-0.40	Corrosion resistant Bridge Steel -special order
<u>"HSS" Hollow Sections</u>										
		<u>Round</u>	<u>Rect</u>							<u>Round and Rectangular tubes:</u>
A500 Grade A	45	33	39	(2") 25	0.26	--	0.035	0.035	--	0.20
A500 Grade B	58	42	46	(2") 23	0.26	--	0.035	0.035	--	0.20
A500 Grade C	62	46	50	(2") 21	0.23	1.35	0.035	0.035	--	0.20
A500 Grade D	58	36	36	(2") 23	0.26	--	0.035	0.035	--	0.20
A501	58	36		(2") 23	0.26	--	0.04	0.05		0.20
A618	65-70	50		(2") 22	0.15	1.0	0.07-0.15	0.025	--	0.20
A847	70	50		(2") 19	0.20	1.35	0.15	0.05	--	0.20
										High strength and corrosion resistant
<u>Steel Pipe</u>										
A53 Grade B	60	35		See ASTM	0.25	0.95	0.05	0.045	--	0.4
										Most available and commonly used structural steel pipe

NOTE - This data is selected information intended for a guideline reference; refer to ASTM for more information.

Thicker members have a greater chance that inclusions and discontinuities will be larger and more prevalent. This is due to the fact that the steel is produced with less working of the hot metal during rolling. Also, thick sections cool more slowly and this also adversely affects strength and toughness. Therefore, steel ordered from the mill with specified notch toughness requirements will most likely be “fine grain-killed” steel, as discussed briefly under ASTM A992.

Other Supplementary Requirements are listed in ASTM A6 and are for use at the option of the purchaser. These requirements must be specified in the contract documents. Those supplementary requirements that are considered suitable for each different material are listed in the ASTM under each specification, and have been included in this publication for the convenience of the reader. It should be noted that a cost might be associated with each supplementary requirement. Thus, they should be used sparingly and with a clear understanding of the costs and benefits of their use.

A major change in steel production that has occurred during the last decade or so, has been the use of continuous cast preformed slabs and shapes rather than casting of ingots. Mills continuously cast wide slabs to be rolled to plate. The slabs are several times thicker than the final plate for thinner plates. For thick plates the slabs may be only a few times thicker than the final

plate. Similarly, WF shapes are rolled from a preformed cast shape similar to an ‘H’.

The continuous casting process is subject to center shrinkage problems, as are other castings and ingots, if the continuous process is not carefully controlled. The center or mid-thickness shrinkage defect can persist to the finished plate or shape as a plane of little or no strength. This mid-thickness problem appears to be more likely if the steel is thick and not severely hot worked. Supplementary requirement S8, Ultrasonic Testing, using standard testing procedures and acceptance criteria such as A435 and A898 will not cause rejection of steel with mid thickness defects, even those that cause the steel to have almost no strength and ductility in the through thickness direction. The mid-thickness defects are similar to laminations. UT testing procedures may identify the mid-thickness reflectors as flaws but they may not be rejectable.

Better assurance of through thickness properties can be achieved by specifying that the steel shall have through thickness tensile tests in accordance with A770. This is a costly item and should be used only for conditions with through thickness loads and in areas of greatest concern. Schedule impacts need also to be considered.

ASTM A36 (as rolled)

Once the most commonly used steel in building construction, A36 is a material that has seen its use change quite drastically in the past 10 years. Due to the use of recycled steel in the production of new wide flange shapes in the modern mini-mills, most A36 can also qualify under ASTM as A575 Grade 50. Since A36 has no upper limit on yield strength, the term “dual grade” was termed and used to represent this structural steel. A36 was once the most economical steel to be used in floor systems, but soon engineers would take advantage of the inherent over-strength and specify A572 Grade 50 for the same material cost. When used in the design of moment frames, this dual grade material posed a problem in that yielding would occur at a higher force level than anticipated in the design. Furthermore, the Northridge earthquake brought to light other physical and chemical properties that were unfavorable to welded connections that were expected to go past their elastic limit. These developments brought about A992 (see section following). Plate, angle, and channel, however, are still produced. typically as A36, and their use as such is still the most economical.

Of the standardized supplementary requirements (that may be specified in the contract documents) listed in ASTM A6, the following is considered suitable for use with ASTM A36.

S5. Charpy V-Notch Impact Test

ASTM A992 (as rolled)

A992 is relatively new steel developed from the lessons learned after the Northridge earthquake. This specification provides improved properties for wide flange shapes in welded moment frame and brace frame construction, by giving engineers a more reliable limited range of force levels for design, with no significant additional cost. Furthermore, its chemical properties provide for excellent weldability. Derived from A572 Gr. 50 with special requirements as per AISC Technical Bulletin #3, March 1997, this steel is specified to provide a minimum yield strength of 50 ksi, a maximum yield strength of 65 ksi, and a minimum tensile strength of 65 ksi. It has a maximum yield to tensile ratio of 0.85, and a maximum carbon equivalent of 0.47% (not required of A572 Gr. 50). The carbon equivalent is defined and discussed in a subsequent section of this publication.

This steel is specified to be killed. Killed steel is steel that is deoxidized, either by the addition of strong deoxidizing agents, or by vacuum treatment, to reduce the oxygen content to such a level that no reaction occurs between carbon and oxygen during solidification. As such, the steel shall be affirmed in the test report by a statement of killed steel, a value of 0.10% or more for the silicon content, or a value of 0.015% or more for the total aluminum content.

A992 is also specified to contain no greater than 0.012% nitrogen, or it shall be made to a practice producing nitrogen no greater than 0.015% and contain one or more nitrogen binding elements.

Of the standardized supplementary requirements (that may be specified in the contract documents) listed in ASTM A6, the following are considered suitable for use with ASTM A992:

S1. Vacuum Treatment

S2. Product Analysis

S5. Charpy V-Notch Impact Test

S8. Ultrasonic Examination

ASTM A572 (as rolled)

A572 is a carbon manganese steel, augmented for strengthening by columbium, vanadium and nitrogen (optional) additions. A572 covers five grades ranging from 42-to 65-ksi minimum yield strengths, depending on plate thickness and product size, Grade 50 being the most common in building construction.

It is available in rolled shapes, plates, sheet piling, and bars. Grades 42, 50, and 55 are intended for bolted, or welded structures. Grades 60 and 65 are intended for bolted construction of bridges, or for bolted, or welded construction in other applications. A572 has a constant minimum yield strength within any one grade. For example, A572 grade 42 has 42-ksi minimum yield strength for all plate thicknesses to 6". Increases of minimum yield strengths for A572 grades to 65 ksi are accomplished by increasing maximum carbon content from .21% (grade 42) to .26% (grade 65), plus other chemistry adjustments within the specifications. Limits for carbon and manganese, plus grain size control, provide good weldability for this grade. Note, the maximum thicknesses for A572 grades are: Grade 42: 6 in, Grade 50: 4 in, Grade 55: 2 in, Grade 60: 1-1/4 in; and Grade 65: 1-1/4 in or less, subject to composition.

Of the standardized supplementary requirements (that may be specified in the contract documents) listed in ASTM A6, the following are considered suitable for use with ASTM A572:

S5. Charpy V-Notch Impact Test

S18. Maximum Tensile Strength

ASTM A529

A529 is a carbon manganese, killed steel available in 50 and 55 ksi yield strengths in shapes, plates, and bars for use in bolted or welded construction of buildings and general structural purposes.

It is available in both grades for rolled shapes of Groups 1 and 2, and plates 1 inch thick and 12 inches wide. Bars are available in grade 50 to 2 ½ inches thick, and in grade 55 to 1 ½ inches thick. A common use for A529 in building construction are bars (standard width flat bars) used for shear plates and stiffeners.

Of the standardized supplementary requirements (that may be specified in the contract documents)

listed in ASTM A6, the following are considered suitable for use with ASTM A529:
S5. Charpy V-Notch Impact Test
S78. Maximum Carbon Equivalent
S79. Maximum Tensile Strength

ASTM A913 (quenched and self-tempered)

This specification covers high strength, low alloy steel shapes in Grades 50,60,65, and 70 produced by the quenching and self-tempering (QST) process. The QST process, evolved from the “thermo-mechanical control processes” (TMCP), produces fine-grained steel by a combination of chemical composition and integrated controls of manufacturing processes from ingot or bloom reheating to in-line interrupted quenching and self-tempering.

The members may not be heated to temperatures exceeding 1100 deg. F. during or after welding or other fabrication processes. This steel is ideal for use in large axially loaded columns or truss chords. Its weldability is excellent due to relatively low carbon equivalent of between 0.38% to 0.45%, depending on the Grade. Preheat is not required by AWS, however good practice would dictate enough preheat to remove the moisture from the steel prior to welding.

Of the standardized supplementary requirements (that may be specified in the contract documents) listed in ASTM A6, the following are considered suitable for use with ASTM A913:

- S1. Vacuum Treatment
- S2. Product Analysis
- S3. Simulated Post-Weld Heat Treatment of Mechanical Test Coupons
- S5. Charpy V-Notch Impact Test
- S18. Maximum Tensile Strength

ASTM A242 (as rolled)

This steel has essentially been superseded by ASTM A709, Grade 50W and by A588.

A high strength, low-alloy steel with enhanced atmospheric corrosion resistance of approximately two times that of carbon structural steel with copper, the same as A588 (or four times carbon structural steel without copper). It has three grades of yield strengths; 42,46 and 50

ksi, depending on thickness, and is weldable with proper welding procedures, but is limited to material up to 4 inches in thickness. It is available in shapes, plates and bars for welded or bolted construction intended primarily for use as structural members where savings in weight or added durability are important.

ASTM A514 (quenched and tempered)

This steel has essentially been superseded by ASTM A709 Grade 100 for bridges.

A514 covers a number of low-alloy grades, (basically manufacturers’ “recipes”), with a variety of alloying elements. Since A514 is a quenched and tempered steel with 90-100 ksi minimum yield strength, depending on plate thickness. Alloying elements are balanced to provide hardenability, tempering and notch toughness controls:

Tempering at the mill is performed at rather high temperatures (1200-1250 degrees F) for steels included in this grade. However, any temperatures in excess of the tempering temperature will reduce the strength. Therefore, exposures to heating such as in welding cycles must be controlled in order not to soften the hardened structure, thereby lowering the strength. Similarly, overheating may transform the steel structure and result in a structure that is too hard and brittle. Controlled welding techniques can be expected to produce consistently good results for this steel.

A-514 is quite suitable as a structural material, but is intended mainly for welded bridges. This steel is not available in wide-flange or hot-rolled shapes, and therefore is not commonly used in building construction. Aside from cost considerations, the resulting lightweight structures may produce higher deflections in long span members with consequent undesirable vibration characteristics.

ASTM A709 (as rolled and quenched and tempered)

These structural steel specifications cover carbon and high-strength, low-alloy and quenched and tempered steels in seven grades with four yield strengths, available in plates and shapes for use in bridges. Grade names are 36,50,50W,HPS,

50W, HPS 70W, 100, 100W. These steels are basically A36, A572, A588, or A514, but with minimum impact test requirements for non-fracture critical and fracture critical applications in Zones 1, 2 & 3 as set by the American Association of State Highway and Transportation Officials. Grades 36 and 50 are semi-killed or killed. Grades 50W, HPS 50W and HPS 70W are made to fine grain practice. Grades HPS 50W and HPS 70W are made using a low hydrogen practice. Grade 100 and 100W meet the requirements for fine austenitic grain size per A6. Grades HPS 50W, HPS 70W, may be furnished as rolled, controlled rolled, thermo-mechanical control processed, or quenched and tempered. Grades, 100 and 100W are heat treated and quenched and tempered.

Of the standardized supplementary requirements (that may be specified in the contract documents) listed in ASTM A6, the following are considered suitable for use with ASTM A709:

- S8. Ultrasonic Examination
- S60. Frequency of Tension Tests
- S83. Non-Fracture-Critical, T, Material; Toughness Tests and Marking
- S84. Fracture-Critical, F, Material; Toughness Testing and Marking
- S92. Atmospheric Corrosion Resistance
- S93. Limitation on Weld Repair (Fracture Critical Material Only)

ASTM A759 (controlled cooled)

This specification covers carbon steel crane rails of special designs only for crane runway use. Design details for the special crane rails are given in the crane rail catalogs of individual manufacturers and are referred to in the AISC manual as well as in other publications. This steel is not listed in AWS D1.1. The high carbon content indicates that careful consideration be given to alternatives and potential problems before specifying that these rails be arc welded.

Of the standardized supplementary requirements (that may be specified in the contract documents) listed in ASTM A6, the following are considered suitable for use with ASTM A759:

- S1. End Hardening
- S2. High strength rails (heat treated, head hardened, or alloy rails)
- S3. Chamfering (ends)
- S4. Ends prepared for electric arc welding

ASTM A852 (quenched and tempered) *This is A588 steel with Q&T and is similar to A709 70W*

This specification covers quenched and tempered high strength, low-alloy structural steel plates for welded, riveted, or bolted construction. It is intended primarily for use in welded bridges and buildings where savings in weight, added durability, and good notch toughness are important. The atmospheric corrosion resistance of this steel in most environments is substantially better than most carbon steels. Other properties include impact toughness meeting 20 ft-lbs at 50 degrees F, and a fine austenitic grain size produced by fine grain practice and heat treatment.

Of the standardized supplementary requirements (that may be specified in the contract documents) listed in ASTM A6, the following are considered suitable for use with ASTM A852:

- S1. Vacuum Treatment
- S2. Product Analysis
- S3. Simulated Post-Weld Heat Treatment of Mechanical Test Coupons
- S6. Drop-Weight Test
- S8. Ultrasonic Examination

ASTM A53 (as rolled)

This specification covers seamless and welded black and hot-dipped galvanized carbon steel pipe in nominal pipe size (NPS) 1/8" to 26" diameter with nominal wall thickness from .068" to 2.344" depending on diameter. Pipe weight class is Standard (STD), Extra Strong (XS), and Double Extra Strong (XXS). This pipe is very popular for structural use as its chemistry and mechanical properties for Types E and S, Grade B, are very similar to ASTM A36. Grade B, the most common structural pipe used, has a minimum yield strength of 35 ksi, and a tensile strength of 60 ksi.

ASTM A500 (cold-formed structural tubing)

This specification covers cold-formed welded and seamless carbon steel structural tubing in round, square and rectangular shapes. It is available up to a maximum periphery of 64 inches (20-3/8" diameter and 16" x 16" square)

with a maximum wall thickness of 0.625", and yield strength grades A, B and C of from 33 to 50 ksi, depending on shape and grade. Its yield strengths and chemistry make it compatible with A36 and HSLA, high strength low alloy steels. Normally stocked in local steel service centers. ASTM A500 Grade B, the most common specification for rectangular tubing has a minimum yield strength of 46 ksi and a tensile strength of 58 ksi. The use of ASTM A500 Grade C would most likely be a special order.

ASTM A501 (hot-formed structural tubing)

This specification covers hot-formed welded and seamless carbon steel structural tubing in round, square, rectangular, or special shapes for general structural purposes. Round tubing is furnished in nominal pipe-size (NPS) ½" to 24" diameters, in wall thicknesses of 0.109" to 1.000" depending on size. Square and rectangular tubing is available in sizes 1" to 10" across the flat sides, in wall thicknesses of 0.095" to 1.000" and may be furnished with hot-dipped galvanized coating. It has one yield strength, 36 ksi, and chemistry comparable to A36. Check for availability.

ASTM A618 (hot-formed structural tubing)

This specification covers hot-formed welded and seamless high-strength low alloy steel structural tubing in round, square, rectangular, or special shapes for general structural purposes. It has yield strengths of 46 to 50 ksi. Check for availability.

ASTM A847 (cold formed structural tubing)

This specification covers cold-formed, welded and seamless high strength, low-alloy round, square, rectangular, or special shaped structural tubing for welded, or bolted construction of bridges or buildings and for general structural purposes where high strength and enhanced atmospheric corrosion resistance are required. Generally, tubing is available in welded sizes with a maximum periphery of 64 inches, a maximum wall thickness of 0.625 inches, and in seamless with a maximum periphery of 32 inches and a wall thickness of 0.5 inches. Tubing in other dimensions and special shapes

may be available by inquiry and negotiation with the manufacturer. Yield strength is 50 ksi, and the tensile strength is 70 ksi.

ASTM A27 (steel for castings)

This specification covers carbon steel castings for general applications that require up to 70 ksi minimum tensile strength. Castings can be an effective solution to highly restrained welded joints when the amount of weld or complexity of the joint becomes extreme. Castings allow the designer to tailor the geometry of the node and thereby directly design the stress state. The following paragraphs are excerpts from a paper on Steel Castings presented at the 2003 North American Steel Construction Conference (please see reference list):

Many believe that cast steel is brittle because the cast iron that is commonly used in automotive and household goods, like cookware, easily cracks. However, the properties of steel are very different from iron. Steel castings can meet or exceed the ductility, toughness, or weldability of rolled steels. Technically, all steel is cast. Designers generally think of design requirements in terms of strength, but the design is commonly constrained by modulus of elasticity, fatigue, toughness or ductility. Increasing the strength of steel normally reduces the ductility, toughness, and weldability. It is often more desirable in steel casting design to use a lower strength grade and increase the section size or modify the shape. The design freedom makes castings an attractive way to obtain the best fabrication material performance and the needed component stiffness and strength.

Rolled sections of steel have their structure elongated in the direction of rolling. The strength and ductility is improved in that direction but they are reduced across the rolling direction. The lack of a rolling direction in steel castings gives them uniform properties in all directions. Rolling steel cold can also strengthen the steel but reduces ductility and toughness. Cast steel grades achieve the same trade off by alloying and heat treatment.

Steel castings are used in demanding applications that are safety critical, highly specified, and performance demanding. A railroad coupler is a good example of a common application that is critical. Castings are used in

high-pressure service in nuclear power plants. The use of steel castings in pressure containing systems is common and specified in the ASME Boiler and Pressure Vessel Code. One aspect of the ASME code is the requirement that suppliers develop and demonstrate a weld procedure including welded properties for the components and materials they supply. The cast carbon steels that would be used in building construction are already well known and established in the Code, including their design requirements and welded properties.

The biggest advantage in quality that forged or rolled shapes have over steel castings is their ability to begin with a simple optimal casting. The ingot or bar can be easily inspected prior to rolling or forging. The use of casting processes to make uniquely designed shapes requires inspection that is correlated to the casting process, part design, and performance requirements. Often the purchaser of steel castings uses nondestructive examination, mechanical testing, and engineering analysis to ensure the desired reliability.

Steel casting producers routinely test each heat of steel to make sure it meets the mechanical properties required in the material specification. The heat is also analyzed chemically to certify that it meets the standard. Other specialized tests can be required like low temperature impact testing when service performance requirements dictate. The dominant material used in building construction is carbon steel because of its reliable properties, low cost, and ease of fabrication. One common grade used for building construction in rolled sections is ASTM Specifications A36. The use of steel castings is permitted in building construction, using material from either ASTM A27 Grade 65-35 or ASTM A148 Grade 80-50 (AISC, 1998). The properties of common steel depend on the composition and heat treatment. Because designers use yield strength as a basic property in design, often material is ordered to higher strength without considering the advantage in castings of using a lower strength material with optimum ductility and weldability. Since the load-carrying cross-section can be increased to accommodate lower strengths, the casting can be supplied in the highest ductility with strength levels that are compatible with the rolled structural shapes. This use of cast carbon steel in its optimal condition makes sure that the casting will perform safely and reliably and that

excessive loads will cause failure to occur first in the rolled section familiar to the designer. The use of ASTM A27 Grade 65-35 in the normalized and tempered condition will give a strong, ductile, weldable steel.

Traditionally, nondestructive testing has been used to certify casting quality. Soundness is verified through the use of radiographic inspection. Surface quality is evaluated using magnetic particle inspection. More recently, the use of computer simulation of solidification of the casting integrated with finite element analysis of its performance has been used to design optimal casting configurations. The development of these tools allows the designer to ensure that critical areas of the part meet requirements while ensuring the most economical means of manufacturing the whole part. Additional information covering steel castings is available from the Steel Founders' Society of America at <http://www.sfsa.org>.

The Welding Processes

Along with the development of better structural materials came improved methods of joining steel. Of the methods described below, some are restricted to shop use only; others find widespread use in both the shop and the field. There are four pre-qualified welding processes:

- Shielded Metal Arc Welding (SMAW)
- Flux Cored Arc Welding (FCAW)
- Gas Metal Arc Welding (GMAW in the spray mode)
- Submerged Arc Welding (SAW)

Other welding processes such as Electro Slag Welding (ESW), Electro Gas Welding and Gas Metal Arc Welding—short circuit mode can be qualified-by-the test.

Electrodes

There are many different types and brands of electrodes used by the industry. The structural engineer needs only to specify the design strength and the notch toughness requirement for his or her design. The design strength is based on matching filler metals to the base metals, (see Table C), and the notch toughness demands depend on the use of the structure. For seismic design, Section 7.3a of the AISC Seismic Provisions requires electrodes to meet 20 ft-lbs @ minus 20 degrees F. Given these specifications, it is then the responsibility of the steel fabricator and erector, to choose the electrode that best suits the skills of their

welders, accommodates their equipment, and is appropriate for the weld type and position in which it is used. Electrode storage and exposure limitations vary by electrode. FEMA 353 Appendix D recommends a testing protocol, to be performed by the manufacturer, to establish acceptable limitations on electrode exposure to the atmosphere. The fabricator or erector must then work within the limitations of these test results after packaging has been broken. The engineer should request electrode product data sheets for review. This data provides recommended welding parameters along with the electrode exposure limitations. Welding procedures should be reviewed to work within the parameters of the product data sheets.

Shielded Metal Arc Welding (SMAW)

Commonly referred to as “stick” or manual welding, this is the oldest and was the most popular method of structural welding and involves the use of flux-coated electrode (stick), which is consumed in the process, see Fig. 3. SMAW has a long and successful history. As compared to other processes, it is ideal for weld joint repairs and light field applications. The equipment is less costly; it is more portable; electrodes may be purchased “off the shelf” in most locations. SMAW is particularly advantageous when the job involves repetitive starting and stopping, as in short fillet welds. Stick welding, however is slower and more costly than other methods of welding, and is more dependent on operator skill for high quality welds.

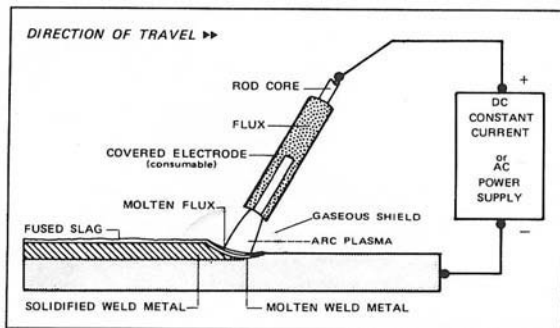


Fig. 3 Schematic Illustration of Shielded Metal-Arc Welding (SMAW). Reverse polarity is shown (D.C. electrode positive)

Developments through the years have been directed at minimizing the formation of hydrogen gas by removing water from the electrodes (low hydrogen). Therefore common field practice calls for the electrodes to be dry before use. The low hydrogen electrodes are

normally shipped in hermetically sealed containers. Low hydrogen electrodes have these designations: EXX15, 16, 18, 28 and 48. The XX indicates the tensile strength in ksi. After a container is opened, and electrodes are exposed to the atmosphere beyond the manufacturer or AWS specified time limit, electrodes are dried with the use of on-site ovens, or discarded. Low hydrogen electrodes that are shipped in other than hermetically sealed containers are required to be baked in special high temperature ovens prior to use. The important exceptions to the electrode drying/ baking are cellulose-coated electrodes (for example, E6010 and E6011 that give good penetration but are not low hydrogen electrodes), which are compounded to contain 3 to 7 percent moisture. Redrying can actually impair the quality of these electrodes. It is imperative that AWS D1.1-2002 requirements for storage handling of low hydrogen electrodes and the electrode manufacturer’s recommendations for all electrodes be followed. D1.1 does not address electrodes other than low hydrogen. Improper storage can lead to hydrogen intrusion creating underbead cracking, transverse cracking, and or porosity. SMAW is normally used with a constant current, drooping characteristic power source.

Flux-Cored Arc Welding (FCAW)

This process, shown in Figure 4, employs a tubular electrode with the flux contained within the core of the tube.

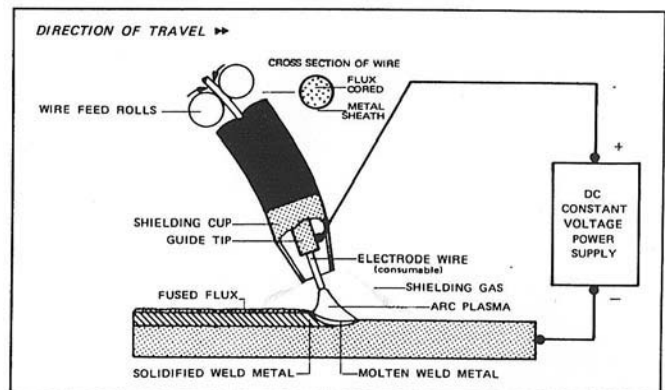


Fig. 4 Schematic illustration of Flux-Cored Arc Welding (FCAW). Reverse polarity is shown (D.C. electrode positive).

TABLE C**FILLER METAL REQUIREMENTS**

AWS Group	ASTM Steel Spec. & Grade	Electrode Specification & Classification	
I	A36 <=3/4" A53 Grade B A500 Grades A, B A501 A709 Grade 36 < =3/4"	<u>SMAW</u> AWS A5.1 A5.5*	E60XX, E70XX E70XX-X
		<u>SAW</u> AWS A5.17 A5.23*	F6XX-EXXX, F6XX-ECXXX F7XX-EXXX, F7XX-ECXXX F7XX-EXXX-XX, F7XX-ECXXX-XX
		<u>GMAW</u> AWS A5.18 A5.28*	ER70S-X, E70C-XC, E70C-XM* E70S-XXX, E70C-XXX
		<u>FCAW</u> AWS A5.20*	E6XT-X, E6XT-XM E7XT-X, E7XT-XM
		A5.29*	E6XTX-X, E6XTX-XM E7XTX-X, E7XTX-XM
		II	A36 >3/4" A572 Grades 42,50,55 A709 Grade 36 >3/4" A709 Grades 50, 50W A913 Grade 50 A992
<u>SAW</u> AWS A5.17 A5.23*	F7XX-EXXX, F7XX-ECXXX F7XX-EXXX-XX, F7XX-ECXXX-XX		
<u>GMAW</u> AWS A5.18 A5.28*	ER70S-X, E70C-XC, E70C-XM* ER70S-XXX, E70C-XXX		
<u>FCAW</u> AWS A5.20 A5.29*	E7XT-X *, E7XT-XM* E7XTX-X, E7XTX-XM		
III	A572 Grades 60,65 A913 Grades 60,65		
		<u>SAW</u> AWS A5.23*	F8XX-EXXX-XX, F8XX-ECXXX-XX
		<u>GMAW</u> AWS A5.28*	ER80S-XXX, E80C-XXX
		<u>FCAW</u> AWS A5.29*	E8XTX-X, E8XTX-XM
IV	A709 Grade 70W A852	<u>SMAW</u> AWS A5.5*	E9015-X, E9016-X, E9018-X, E9018-M
		<u>SAW</u> AWS A5.23*	F9XX-EXXX-XX, F9XX-ECXXX-XX
		<u>GMAW</u> AWS A5.28*	ER90S-XXX, E90C-XXX
		<u>FCAW</u> AWS A5.29*	E9XTX-X, E9XTX-XM
		<p><i>This is an abbreviated table for selected steels based on AWS D1.1: 2002, Table 3.1. See the AWS table for more detail.</i></p> <p><i>* Some exclusions apply, see AWS D1.1: 2002, Table 3.1</i></p>	

There are two versions: The self-shielded type uses flux compounds alone to protect the weld from oxidation during cooling. The gas-shielded type uses flux compounds, plus an auxiliary shielding gas (usually carbon dioxide) for weld protection. To minimize spatter, a mixture of 75% CO₂+25%A is becoming popular. In general, only the self-shielded type is used for field application due to the effect of wind. Both are semi-automatic, high production methods. Although equipment is bulky, FCAW is the method of choice for high production, deep penetration welding on low-rise and high-rise structures. Typical deposition rates (about 8 to 12 pounds per hour) are about twice that obtained from normal stick welding. Figure 5 compares the depth of penetration of a fillet weld produced by SMAW and FCAW. The power source should be a constant voltage type.

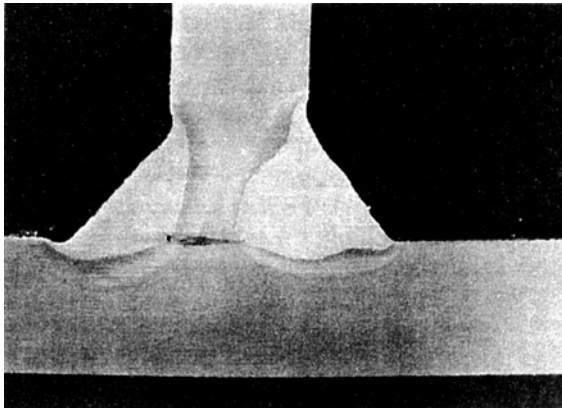


Fig. 5 Fillet welds by SMAW, left, and FCAW, right, in A36 steel. Note the increased penetration of FCAW.

Gas Metal Arc Welding (GMAW)

This welding process is pre-qualified when used in the spray transfer mode; it must be qualified by test when used in the short-circuiting transfer mode. The electrode is a solid welding wire or metal cored wire, and the shielding gas is Argon, Helium, or CO₂ or a combination of these gases, and is semi-automatic (see Figure 6). The power source should be a constant voltage type, direct current, and is normally used in reverse polarity (DCEP). It produces a very clean weld and deposit rates are very good, comparable to FCAW. Though developed primarily for the aircraft industry, it is now very popular in structural steel fabrication shops, but is not practical for outside or field welding due to wind effects.

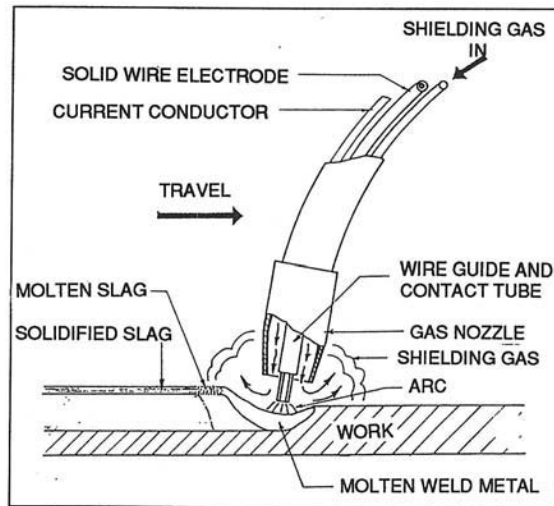


Fig. 6 Gas Metal Arc Welding- spray arc mode.

Submerged Arc Welding (SAW)

Most structural sections for buildings and bridges, welded in prefabrication plants or temporary fabrication plants, use the SAW process with a fully automatic setup. This process, as shown in Figure 7, deposits a flux powder in advance of the electrode, so that the resulting arc produced is submerged in the flux and is not visible to the operator. It is the workhorse of the structural shop for built up members. SAW is particularly well suited to long welding runs of thirty feet or more. It can be used on thin or thick sections of metal and is capable of producing high quality fillet, partial, and complete joint penetration welds at typically high deposition rates, but is restricted to flat or horizontal welding positions.

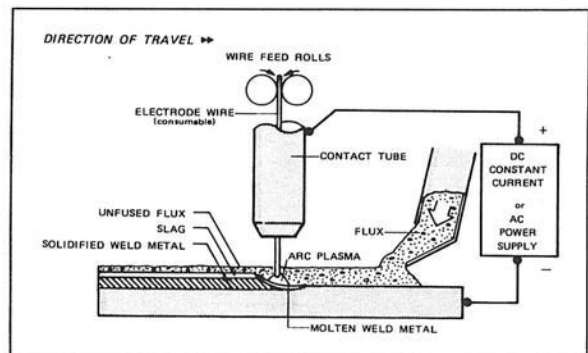


Fig. 7 Schematic illustration of Submerged Arc Welding (SAW. Reverse polarity is shown (DCEP).

Electroslag Welding (ESW)

Introduced to construction use in the 1960's, electroslag welding is the newest production welding process. Its chief feature is its unsurpassed production capacity, depositing filler metal at 35 to 50 pounds per hour, while producing a clean, high quality joint on a continuous casting basis. This is not a pre-qualified process, but must be qualified by test, and the test results submitted. As illustrated in Figure 8, there is no arc. The slag, heated by electrical resistance, melts the filler electrode and parent metal. Melting of the electrode in the slag cleans the metal by providing excellent slag to metal contact. The parent metal, surrounding the molten pool, is heated deeply and the resultant slow cooling allows time for gas bubbles to escape, keeping porosity to a minimum. Although the process is used primarily in the shop for butt welding of plates and for the final closure welds of interior stiffeners used in box columns using the key-hole weld technique, it has been adapted for field welding of solid prismatic members of between 4 and 6 inches thick.

In spite of cleanliness, welded sections using the ESW process will often show lower fracture toughness than the parent metal, especially at temperatures below 0 degrees F. The mechanical properties are equivalent in all other respects. Special ESW techniques have been developed recently to improve weld toughness by grain size control. A process known as "Narrow Gap Improved Electroslag" (NGIE) welding has been developed intended for use on bridge welding. This process, still being perfected, utilizes a narrow gap to reduce weld consumables and corresponding heat input, as well as a Nickel-molybdenum alloy electrode wire to produce improved toughness.

In shops so equipped, the fabricator can also obtain further improvements by "normalizing" the weld zone. This involves reheating the welded area to between 1650 and 1700 degrees F to form austenite, followed by air-cooling. Normalizing produces a uniform, refined grain structure with improved fracture toughness. This process is not generally used in construction because the high cost cannot be justified.

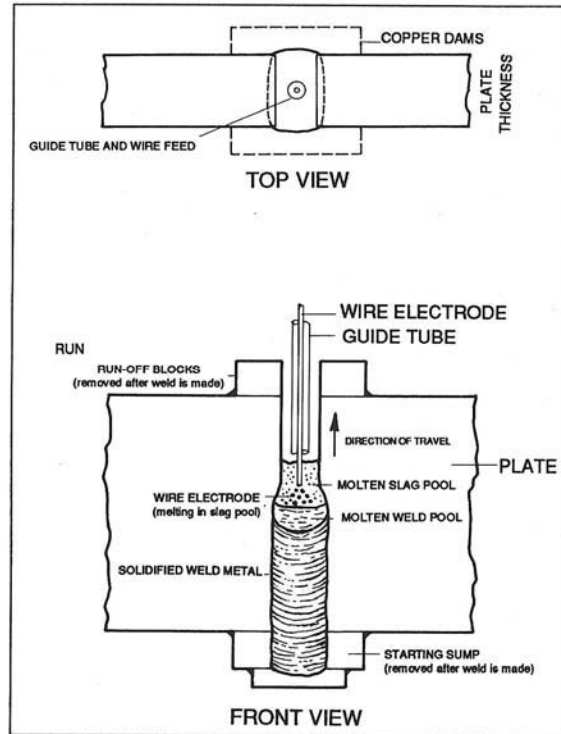


Fig. 8 Schematic of electroslag process for welding typical butt joint.

Note: Single or multiple electrodes may be used: with or without oscillation for either will depend on plate thickness. Width of gap is 1" to 1-1/2". Techniques are being developed to eliminate the starting sump.

Avoiding Weld Defects

Although welding has been with us a long time, its application to structural use was impeded by early failures in the bridge and shipbuilding industries. Early failures had many causes including lack of understanding in the engineering community, poor quality steel, inferior welding electrodes, poor details, and workmanship. WWII era steels were less weldable than current steels. Low hydrogen electrodes did not become common until after WWII. Also, there were poor designs that caused stress risers which contributed to some failures.

Much research has been done over the past five decades, not only in the development of better base materials, but also in providing electrode filler metals that better fit the metallurgical properties of today's structural steels. Table C shows a variety of filler materials available for the type of steel and welding processes being used. Many of the weld defects found in structural applications are caused less by the

quality of the base and filler metals than by poor joint design in combination with improper welding practice. These factors lead to a variety of weld defects including those described in the following sections.

Weld Cracking

Figure 9 illustrates the metallurgical features of a typical weld. The weld area is divided into a fusion zone and a heat affected zone (HAZ).

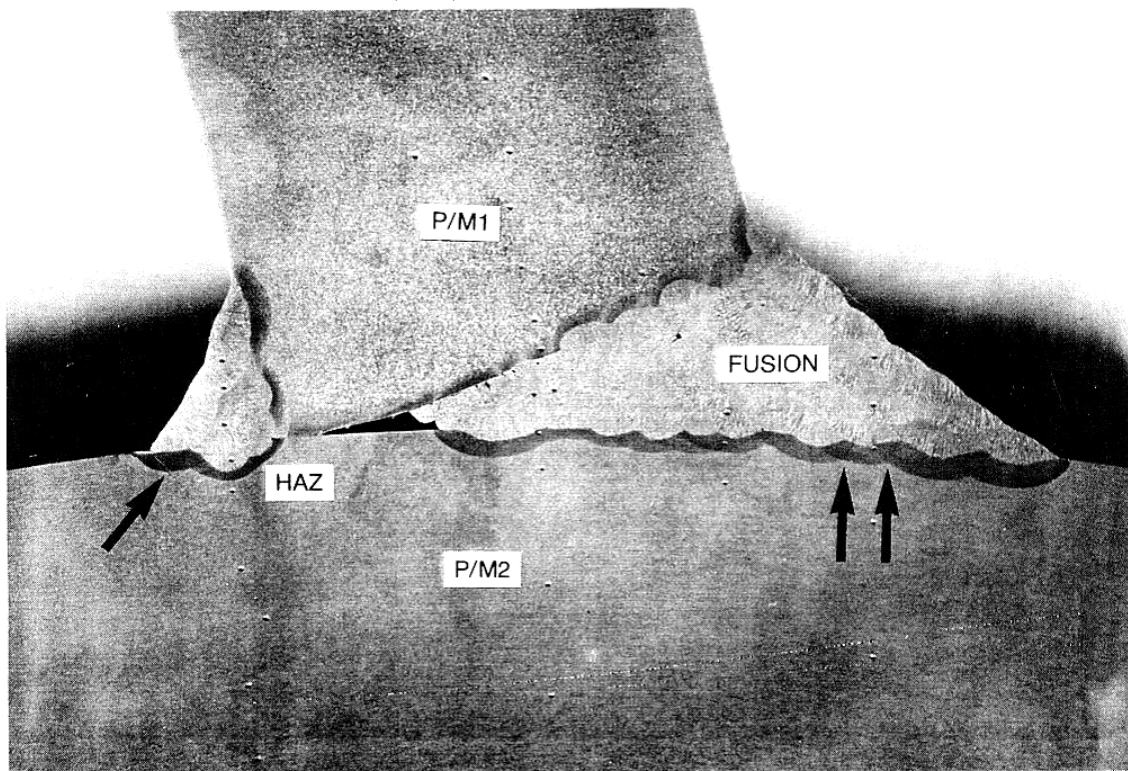


Fig. 9 Weld macrostructure (at 1.2X size), A36 steel, P.M1, joined to heat treated steel, P/M2. Good fusion is indicated, accompanied by prominent heat affected zones (HAZ), which show darkest etched structures; however, there is lack of penetration at the root of the partial penetration groove weld. Arrows designate cracks in HAZ of P/M2. The indentations are Rockwell hardness test impressions. Joint was fabricated by SMAW (“stick welding”), with E7018 electrode. (Cracking due to lack of preheat and unbalance of weld.)

The HAZ microstructure reflects changes in the grain structure of the parent metal produced by heat from the adjacent molten metal. In figure 9, good fusion is indicated in the weld itself. However, the lack of preheat allowed the joint to cool too quickly. This in turn produced hard, brittle martensite in the HAZ and led to cracking in the HAZ and the parent metal. Preheat is especially critical in thicker sections due to the fact that more heat will be lost through conduction than through convection.

All structural steels experience some degree of hardening in the HAZ due to high heat inputs during welding. The hardness increase is proportional to the carbon content and is affected by the alloying elements and weld cooling rates. Some modern structural steels have rather low carbon contents and the HAZ do not get very hard. Q&T steels require that the HAZ have a hardened martensite structure, otherwise the strength and toughness will deteriorate. Hardened HAZ are susceptible to hydrogen related cracking and under bead cracking.

Specific welding controls that consider the hydrogen potential of weld filler metal should be used. Preheat, post heat, and slow cooling such as with insulating blankets are sometimes used. AWS D1.1 has prequalified preheat requirements for non-low hydrogen covered electrodes that require higher temperatures than for other electrodes.

With proper weld procedures and good workmanship, such cracking can be eliminated. Other types of cracks can be encountered in structural welding, such as under bead cracking, cold cracks and hot cracks.

Hydrogen Embrittlement

When steel is melted during welding, hydrogen may dissolve in the molten metal and diffuse into the parent metal HAZ. Molten metal has a great affinity for atomic hydrogen, but low affinity as cooling takes place and hydrogen is rejected. Some hydrogen gas may become trapped in the weld metal and create high internal pressure which can induce micro-cracks in the steel. Hydrogen cracking is controlled or eliminated by the use of low hydrogen electrodes and proper preheat. The source of hydrogen is water, which may originate from the weld environment, the steel base material, the shielding of a SMAW or FCAW electrode or the flux of the SAW process. In the welding arc, water will break down into hydrogen and oxygen. Hydrogen in the fusion zone deposited metals or in the HAZ of the parent metal can cause embrittlement. If the HAZ has become hardened during welding, the sensitivity to hydrogen embrittlement is even greater.

Preheating of the metal by AWS standards and use of low-hydrogen electrodes are the best means of avoiding hydrogen embrittlement. See Table D. Preheating dries the steel surface, slows the cooling rate to limit HAZ hardening, and retards cooling so as to permit hydrogen to diffuse out of the steel. As mentioned earlier, proper storage, as well as heat drying of electrodes is desirable for removing excessive moisture.

Lamellar Tearing

One of the most disconcerting welding defects caused by poor joint design in combination with bad welding practice is “lamellar tearing.” This occurs in highly restrained joints because the designer may not fully understand the

anisotropic properties of the base metal, and the fabricator has not undertaken adequate preventive measures during welding. Design information through AISC is available to aid the engineer, detailer and fabricator to reduce the occurrence of these defects.

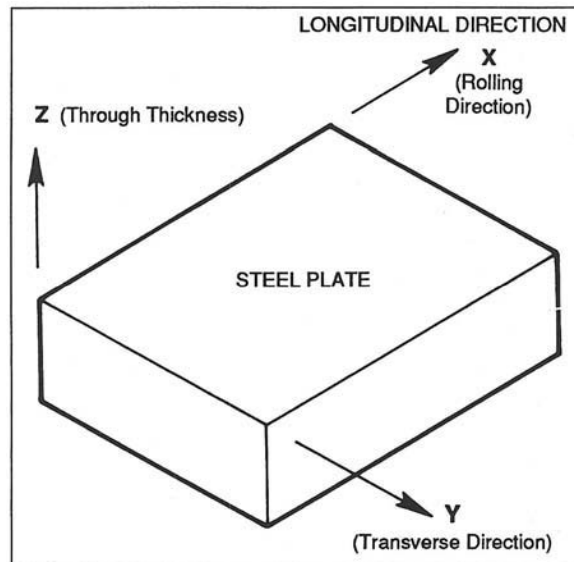


Fig. 10 Directional nomenclature for describing anisotropy in rolled plates.

In structural steels, mill test reports primarily address the longitudinal properties. Figure 10 and 11 illustrate the orientation of a steel plate relative to the direction of rolling, or its longitudinal (X) direction. Transverse (Y) direction properties, while usually lower in ductility and toughness than longitudinal properties, are nonetheless predictably good. Strength and ductility are the lowest in the through-thickness (Z) direction

Certain non-metallic compounds are formed during the alloying process and become trapped in the steel. These so-called “inclusions” are round in shape when the steel is cast into ingots, but become flattened and elongated during rolling. They are usually microscopic in size and difficult at best to detect even with ultrasonic testing.

Due to the through-thickness tensions developed during welding of a highly restrained joint, these flattened inclusions, usually silicates, aluminates, sulfides and oxides, can link up to form micro-fissures and eventually form visible cracks. Figure 12 illustrates how lamellar tearing can develop in a T-weld section. Following welding,

contraction strains are generated as the heated metal cools. When the assembly is highly restrained, i.e., when portions or all of the assembly are kept from contracting during cooling, and when the welding sequence is such that the strain demand cannot be accommodated by plastic deformation in some element, the micro-fissures grow into tearing cracks.

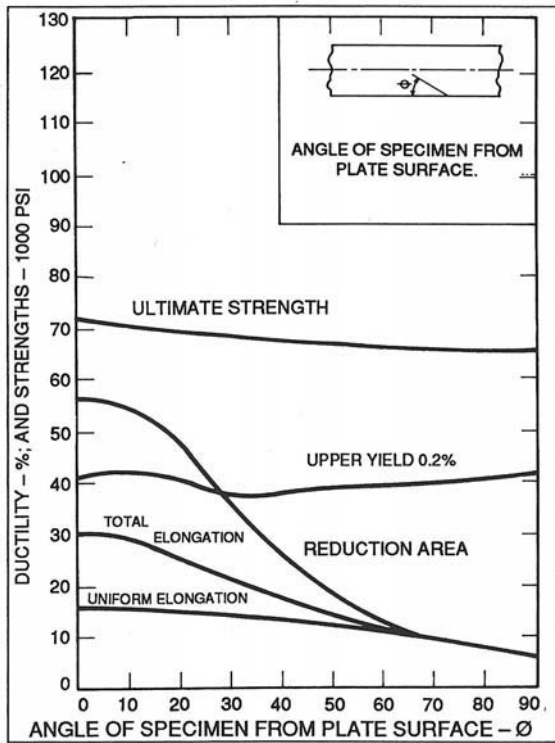


Fig. 11 Relation between tensile properties and angle of specimen from the plate surface to demonstrate loss of properties in the Z direction.

The worst case of lamellar tearing is delamination, or complete internal separation of the steel, as shown in Figure 12. This is more likely to occur in larger weldments using thicker base materials because more non-metallic inclusions collect with size and because of the greater restraint derived from rigidity.

ASTM A770

This delamination effect occurs very rarely if the steel is clean and virtually free of fine non-metallic inclusions. A mill order specifying mill testing in conformance to ASTM A770 can provide such a steel. Although costly, this specification will normally instigate a mill to use special procedures such as Calcium Argon blowing, shape control additions, and a reduced sulfur content to satisfy a through thickness

tensile test with a minimum 20% reduction in area.

Other precautionary measures would include:

- Careful detailing of weld joint preparations and weld root openings to minimize weld volume
- Proper preheat of the entire joint per AWS D1.1 requirements
- Following an approved WPS
- Careful sequencing of welds to allow for contraction during cooling
- Controlled cooling if warranted, for the specific structural restraint and environmental conditions
- Use of fine grain killed steel for members with through-thickness stresses.
- Specify through thickness mill testing of the steel in accordance with ASTM A770.

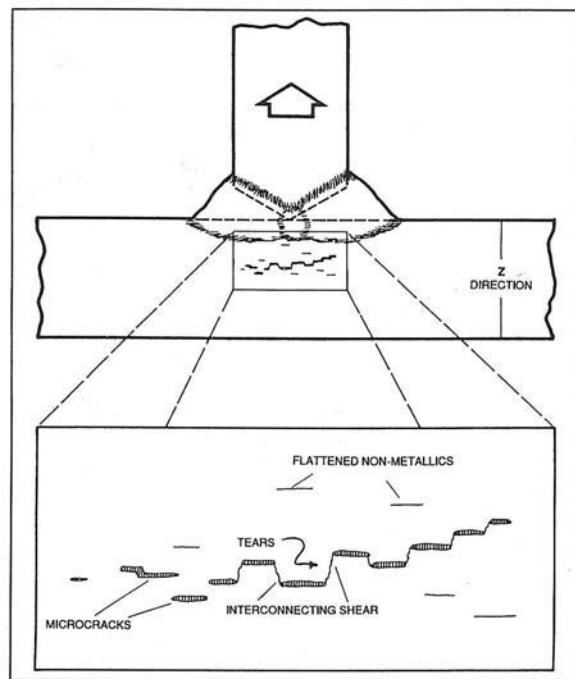


Fig. 12 Lamellar tearing crack developing under a T-Weld. Microfissures initiate at flattened non-metallic particles which grow to cracks by interconnecting tears, note stepped ruptures; can be picked up by UT.

Engineer's Role in Minimizing Weld Defects

In general, weld defects are most likely to occur when using higher-strength carbon steels, when joining thicker sections of steel over 1-1/2 inches, and when applying large amounts of weld metal.

The higher carbon steels by their chemical nature are less ductile, and therefore cannot as easily accommodate the strain demand accompanying weld shrinkage. Thus, low alloy steels are the structural steels used today.

The tendency to produce weld defects when thicker sections are being used arises from the fact that for most steels to maintain a constant minimum yield strength as the thickness is increased, there is a corresponding increase in the amount of carbon and manganese requirements. Therefore, as a section gets thicker the steel becomes less ductile. Furthermore, thicker steels receive less working of the hot metal during the rolling operations, and therefore have a greater chance that inclusions and discontinuities will be larger and more prevalent, with less chance of being forged-welded together during rolling.

The Structural Engineer is in the best position to avoid situations that lead to weld defects by designing joints that are not highly restrained. Figures 13 through 18 show some typical joints that are highly restrained, as compared with alternative joint configurations designed to minimize restraint.

Equally important in the design of weldments is not to "overweld". Often, if a joint is difficult to analyze, the Engineer specifies that every available edge is to be welded, and then the joint is considered conservatively designed. Not only is weld electrode material expensive, but also the cost to deposit it is about 20 to 30 times the cost of the base material. Therefore, the designer is obligated to minimize the amount of welding. Savings in weld metal also means less distortion, less tendency for restraint, and consequently, less cracking, less shrinkage, and less residual stress. Do not specify CJP welds when unnecessary.

In cases where the Engineer cannot avoid designing a highly restrained joint, there are compensating techniques available to the fabricator of which he should be aware. Among

the techniques are the use of preheat, post-heat, controlled cooling and the selection of more ductile electrodes. Peening is helpful when performed under knowledgeable and close supervision. The sequence of welding is also important. For most applications, welding should begin at the center of the mass of the weldment, where restraint is likely to be concentrated, and proceed outward in block steps with the electrode travel directed toward the center of the mass. Wire shims (called "softies") may be used at critical points to provide the necessary air gap within which shrinkage can occur. Weld shrinkage and related strains are directly related to weld volume and heating cycles. Many tests have shown that a few larger weld passes result in less shrinkage than many stringer beads. Stringer beads should not be specified if control of weld shrinkage, distortion and minimizing residual stress are important. AWS D1.1 has adequate controls on weld pass size. Under restrained conditions adding more heat to the weld area can increase the total shrinkage strain.

The Engineer of record should not try to dictate restraint compensating techniques or special welding sequences or procedures to the steel fabricator or erector. Rather, where warranted, the Engineer should require a special submittal by the fabricator or erector, outlining special sequences or procedures for particularly restrained elements of the structure. Most experienced fabricators and erectors have a good understanding of potential problems due to weld shrinkage. Simply requiring the fabricator/erector to create a written plan, and stimulating thought on the issue often averts potential problems. If the Engineer is not comfortable performing the review of the submittal, he or she may opt for review by a welding or erection expert.

The Engineer should also call out for the standard submittals by the fabricator of welding procedures as per the most current AWS D1.1 and of shop detail drawings as per AISC Specification for Structural Steel Buildings and Code of Standard Practice.

Table D

Prequalified Minimum Preheat and Interpass Temperature

This table is intended as a convenient reference for selected steels based on Table 3.2 AWS D1.1:2002.

See the AWS Table for additional materials, cross references and footnotes.

Steel Specifications		Welding Process	Thickness (thickest part) (inches)	Minimum Preheat & Int. Temp. (deg F)
			1/8 to 3/4 incl.	32*
A	ASTM A 36 <= 3/4 in.	SMAW with other than low hydrogen electrodes	Over 3/4 thru 1-1/2 incl.	150
	ASTM A 53 Grade B			
	ASTM A 139 Grade B			
	ASTM A 500 Grade A,B		Over 1-1/2 thru 2-1/2 incl.	225
	ASTM A 501			
ASTM A 709 Grades 36 <=3/4 in.	Over 2- 1/2	300		
			1/8 to 3/4 incl.	32
			Over 3/4 thru 1-1/2 incl.	50
B	ASTM A 36 (>3/4 in.)	SMAW with low-hydrogen electrode SAW, GMAW, FCAW	Over 1- 1/2 thru 2-1/2 incl.	150
	ASTM A 53 Grade B			
	ASTM A500 Grade A,B			
	ASTM A501			
	ASTM A529 Grade 50,55			
	ASTM A 572 Grades 42, 50, 55		Over 2-1/2 thru	225
ASTM A 588 Grade 36 (> 3/4in.)				
ASTM A 709 Grade 50				
ASTM A 913 Grade 50				
ASTM A 992				
			1/8 to 3/4 incl.	50
			Over 3/4 thru 1-1/2 incl.	150
C	ASTM A 572 Grades 60,65	SMAW with low-hydrogen electrode SAW, GMAW, GTAW	Over 1- 1/2 thru 2-1/2 incl.	225
	ASTM A 709 Grade 70W			
	ASTM A 913 Grade 60,65		Over 2-1/2 thru	300
D	ASTM A913 Grade 50,60,65	SMAW,SAW GMAW, and FCAW**	All thicknesses >= 1/8 in.	32*

*When the base metal temperature is below 32 deg. F, the base metal shall be preheated to 70 deg. F

**With electrode/flux system capable of depositing weld metal with max diffusible hydrogen content of 8ml/100g when tested according to AWS A4.3.

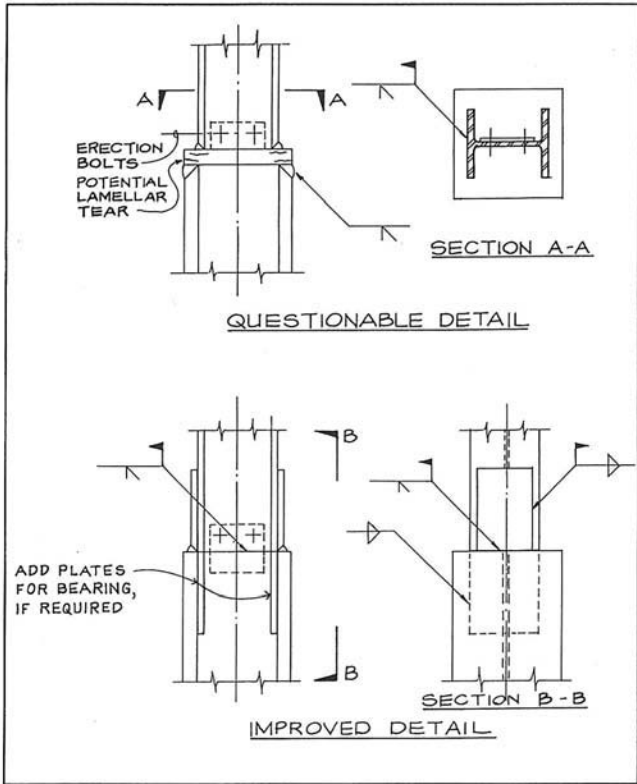


Fig. 13 Column Splices

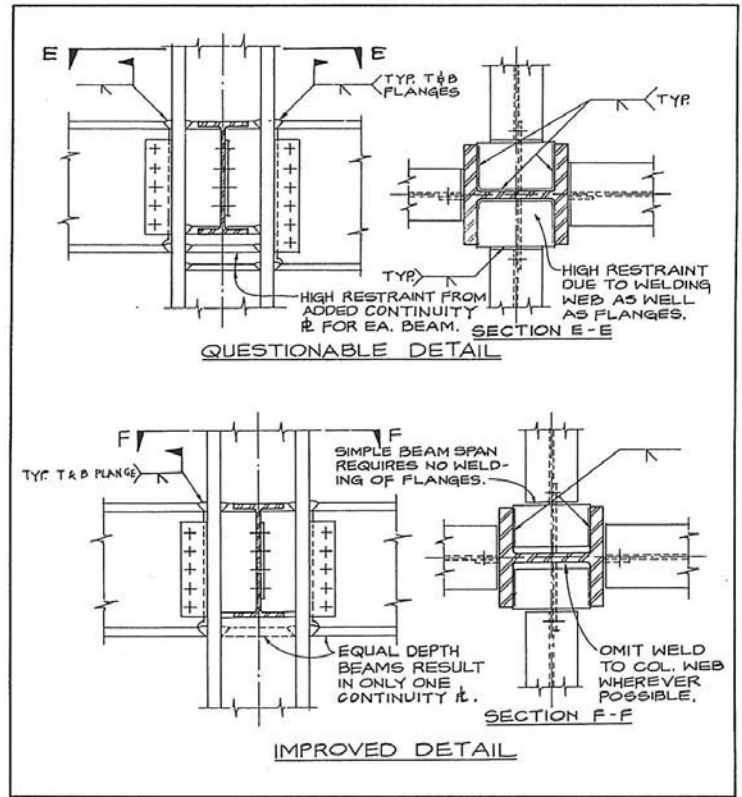


Fig. 15 Beam-Column Details- (note also that bottom continuity plates could be sloped at unequal beam depths)

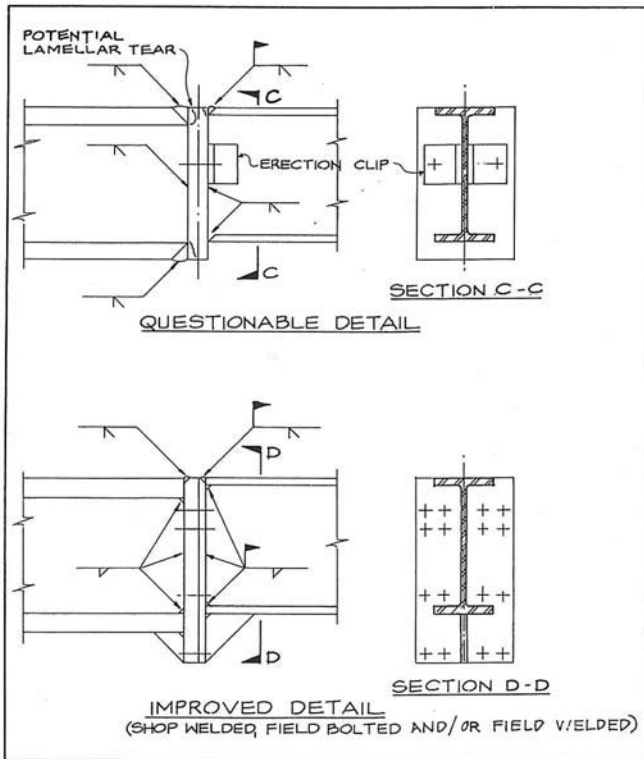


Fig. 14 Beam Splices

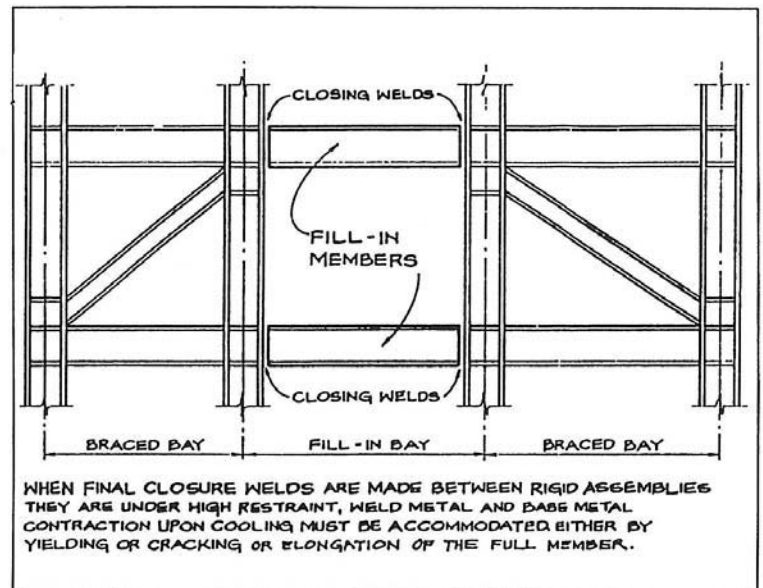


Fig. 16 Full Member Restraint

Correcting Weld Defects

Welding codes in general prohibit cracks of any type in the completed weldments. When cracks are detected, the AWS D1.1 requires that they be repaired by removal and rewelding. The repair of such defects is a normally encountered process during welding operations, and AWS procedures are applicable for repair by the fabricator and approval by the knowledgeable Engineer. There are occasions, however, when repeated attempts at repair are met with repeated failure, and the Structural Engineer is brought in for consultation either by the inspector in support of his rejection or by the fabricator because he may believe the design is contributing to the problem. It is in this type of adversary situation that the Engineer must prove his worth as a diplomat and mediator as well as a good engineering technician.

It is important that the Structural Engineer retains his composure and makes every effort to determine the facts without letting the “people problems” outshine the welding problems. Keep in mind that repeated repairs are costly to the fabricator who wants to produce an acceptable product, preferably without flaws. Try to determine whether there is an actual rejection of AWS D1.1:2002 requirements and if the inspector is being fair and reasonable in his demands or whether it is a case of punitive reprisal for past, real or imagined, grievances.

It is wise to resist taking over and directing the fabricator how to perform the repairs. However, the designer can be helpful by asking for review of the proposed repair procedure and by following a formalized checklist to determine that all possible sources for trouble have been considered. Among some of the basic questions to consider are the following:

1. Is the exact chemistry as well as mechanical properties of the base metal known? A rough check of carbon equivalent would be helpful in checking weldability.
2. Do the electrodes and other joining materials comply with AWS and ASTM standards? Ask for certification or, if in doubt, have them tested and check storage conditions.
3. Are the electrodes and base metal compatible as called for under AWS “matching” standards (see Table B)?
4. Has the extent of the crack or defect been determined? Where is it located with respect to the weld?
5. Do the welder, his supervisor and the inspector all agree as to location and extent of the indicated defect?

Perhaps a third party may be helpful to settle arguments of this nature.

6. Can the design be revised to minimize restraint?
7. Is the fabricator using more than minimum required preheat to help slow the cooling down after weld completion? Sometimes post heating and/or insulation blankets will help prevent cracking.
8. Are the welder and inspector using a heat indicator to determine preheat and interpass temperature? Guessing is not sufficient.
9. Prior to beginning the replacement weld, was magnetic particle testing (MT) used to make sure the entire defect was removed?
10. The entire repair procedure for important weldments should be written out and reviewed prior to starting repairs and should include:
 - Size, type and AWS designation of electrode material.
 - ASTM designation of base metal.
 - Sketch of defect showing size, extent, and location in weldment.
 - Procedures followed for detection (NDT).
 - Preheat and interpass temperatures to be used.
 - If necessary, post heat treatment or methods to slow the cooling rate, such as asbestos insulating blankets. Post heating is rarely required and electric blankets are a major issue.
 - Procedures for re-inspection after completion of repairs.

Keep in mind that once the welding repair has started it is mandatory to complete the repair without interruption. Repeated heating and cooling invites repeated cracking due to increased potential of contamination.

If the fabricator has not already done so, it may be helpful to suggest that a welding engineer or a metallurgist knowledgeable in practical welding problems be called in for consultation. Having faced such problems many times previously, he may be able to point out the technical cause of the problem immediately.

Usual practice calls for the inspector to make a daily inspection report and the fabricator is given a copy with the original to the engineer. If the fabricator disagrees with the inspector’s report, the inspector sends a non-conformance report (NCR) to the Engineer, copy to the fabricator, for resolution of the dispute.

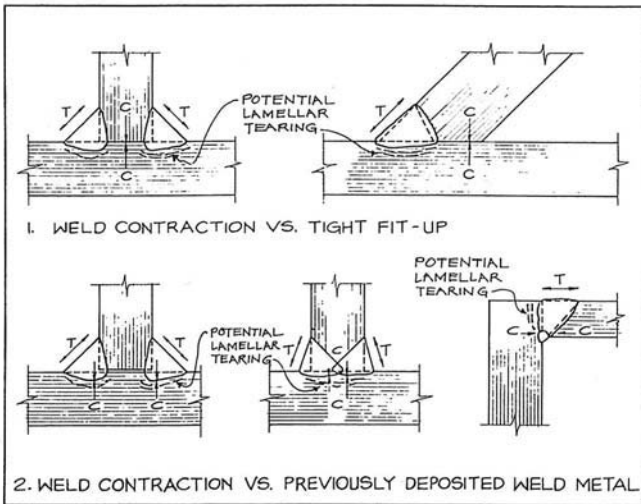


Fig. 17 Internal Restraint in Weldments

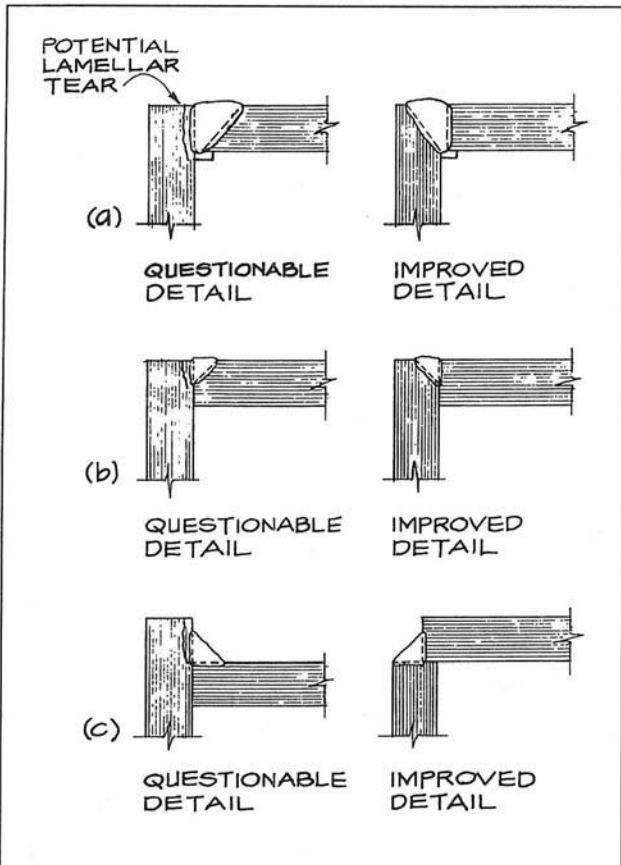


Fig. 18 Welded Corner Joints

A fair, open-minded approach with a desire to work cooperatively with the fabricator can have the best chance of successfully correcting the problem and keep it in its place, out of court.

Correcting Weld Distortion

Weld metal shrinks as it changes from liquid to solid in contrast to water which expands as ice forms. The solidification shrinkage is in addition to the thermal contraction that occurs on cooling from approximately 2,800F. The volume change must be absorbed as internal elastic and plastic strains, as movement in some element, or as a rupture. A lack of fabrication skill in coping with these movements is evidenced by distortion of the finished structure or by cracking.

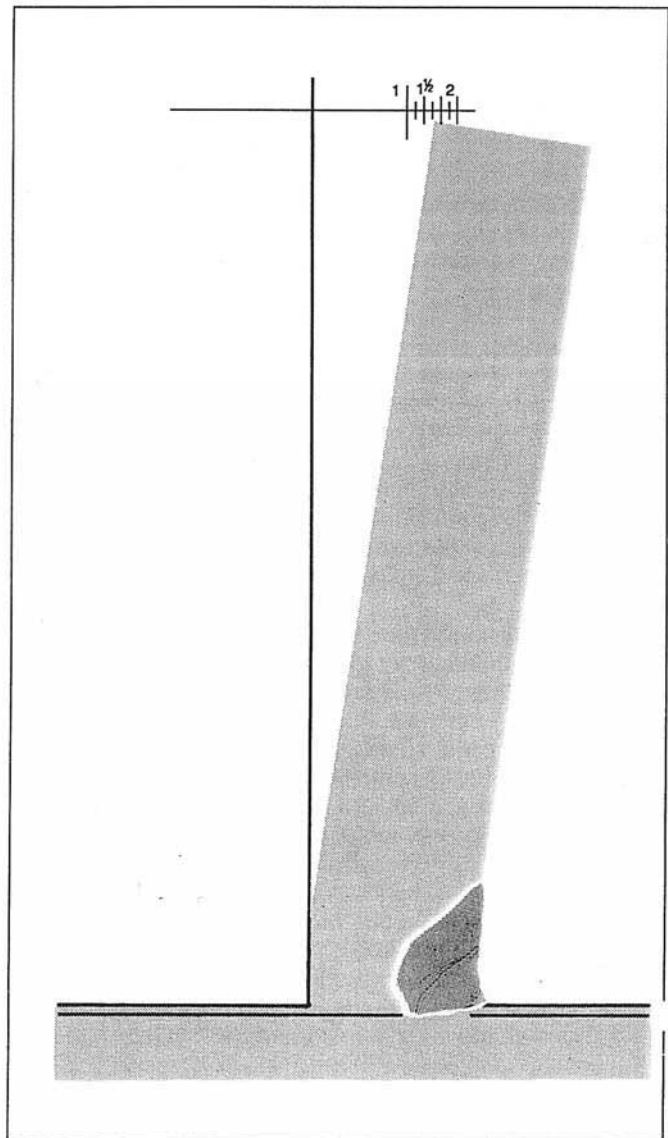


Fig. 19 Angular Distortion has resulted from weld shrinkage. Compensating tilt of vertical member and/or use of strong-back arc methods to control vertical alignment to vertical position when welded.

Two thick plates fit up, tacked at 90 degrees and welded together without fixtures can create angular distortion, as shown in Figure 19. The accumulative angular distortion of three weld passes is shown schematically in Figure 20.

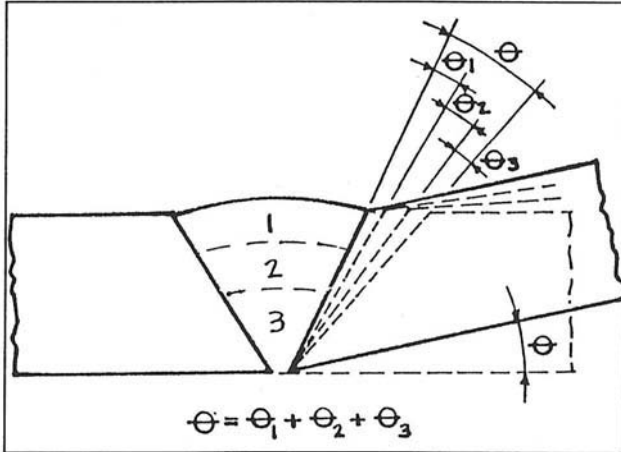


Fig. 20 Rotation in a butt weld. The rotation equals the cumulative shrinkage from each weld layer. Techniques have been developed which will minimize this effort.

Distortion from transverse weld strain demand with the welds shown balanced and the plate flat is shown in Figure 21, below. The fabrication should position the plates to account for the changes caused by the first and second welds.

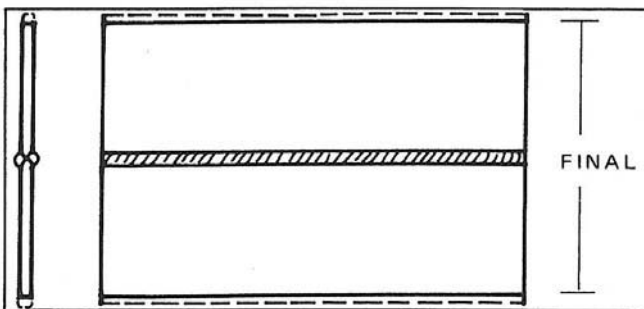


Fig. 21 Transverse shrinkage in a butt weld

Distortion from longitudinal weld strain demand is shown in Figure 22. The position of the weld relative to the center of gravity of the cross section produces the bow. Supplementary weld beads are sometimes used to achieve the desired camber. Sequence, technique, and peening will minimize distortion.

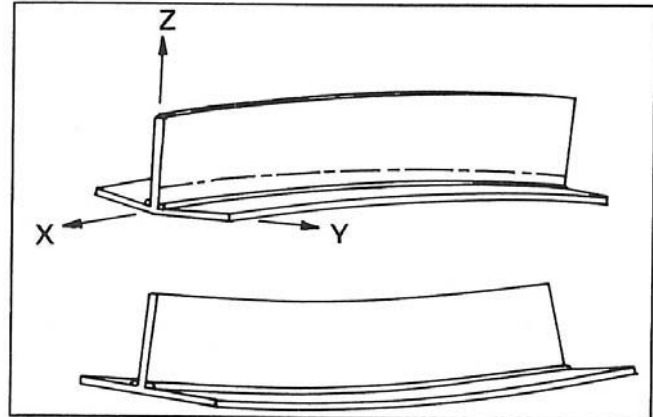


Fig. 22 Longitudinal bowing in a welded beam may produce either positive or negative camber (in X-Z plane). Lateral bow in X-Y plane can occur.

A method of straightening or curving steel, known as “Heat Straightening” or “Flame Bending,” uses thermal upsetting to plastically deform the material. Localized heating of steel causes thermal expansion and a reduction of yield strength in the heated section, while the material is still hot. The expansion is inhibited by the cold, stronger, surrounding metal, forcing the heated portion to yield plastically to accept its own demand for increased volume. (See Figure 23 for yield strength at elevated temperatures and Figure 24 for variations in modulus of elasticity.) After cooling, the shape of the steel piece is changed, and the heated zone recovers its strength. Several heating and cooling cycles may be required to complete an operation.

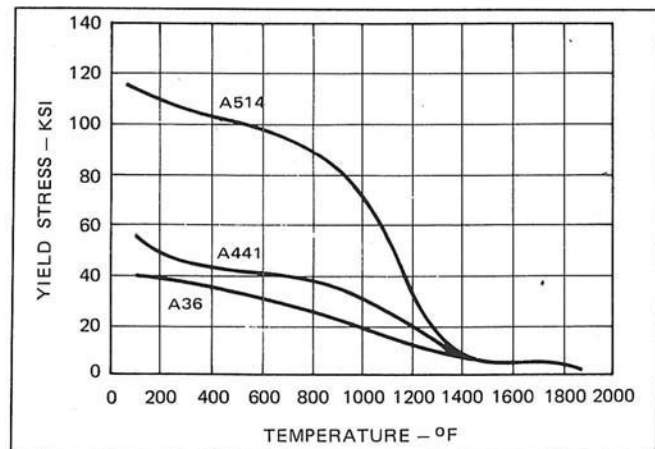


Fig. 23 Variation in yield strength with temperature.

The “flame bending” technique is used in the shop to flatten web plates, to camber beams, or to straighten work distorted by welding. The maximum temperature recommended for this operation is 1,200 degrees Fahrenheit for as rolled structural steels, and

1,100 degrees Fahrenheit for quenched and tempered steels such as those of A709 but not higher than the tempering temperature. Cooling may be in air, or by water spray or wet rags for more rapid cooling.

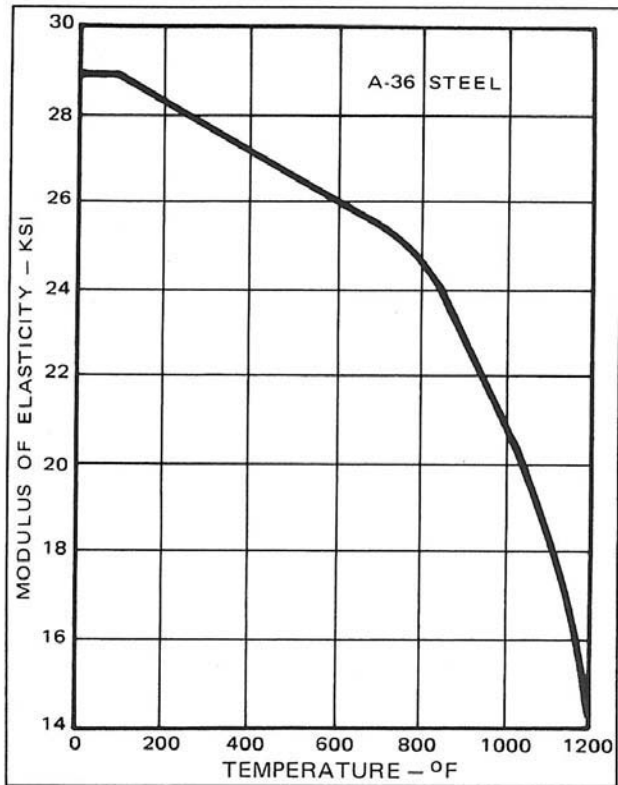


Fig. 24 Variation in modulus of elasticity

Localized heating should not exceed the critical temperature of 1,333 degrees Fahrenheit, or undesirable changes in mechanical properties may result. Heat-treated steels such as those of A709 must not be heated above the tempering temperature, which should be obtained from technical information furnished by the steel manufacturer.

Nondestructive Testing (NDT)

One of the main reasons for the success of all welded structures in the building industry has been the development of fast and accurate methods of examining welded joints without destroying or impairing their actual usefulness. Currently, there are a variety of techniques being employed by the fabricator and independent inspection agencies to assess the reliability of weldments. Used properly, these methods can reveal practically all of the common surface and internal defects that normally occur with improper welding procedures and practices, and will result in a quality level consistent with Project Specification requirements. As in all inspection methods, the experience and skill of the technician and an inspection procedure developed by a

Quality Control Engineer are very important criteria for reliable nondestructive testing. Users should become familiar with all the limitations of NDT methods.

The engineer may also request written documentation as to the type of quality assurance program established by the fabricator. Many fabricators have their own quality assurance program meeting the nondestructive testing specifications established by both local building codes and the American Welding Society. AISC has also developed a set of standards for quality certification and has designated member firms meeting these standards under the following categories: Conventional Buildings, Complex Buildings, Simple Bridges, Major Bridges. In addition, there are several supplemental endorsements to up-grade the basic categories.

The engineer who realizes the high degree of protection afforded by these various organizations through their time-tested standards and specifications can do much to simplify his own design specifications related to the welding of structural steel. Streamlined specifications referring to accepted industry-wide standards and avoiding unnecessary abstruse verbiage would do more to assure that the specification will be read and followed by the fabricator and contractor.

Visual Inspection (VI)

This is a requirement of the AWS D1.1-90 Structural Welding Code wherein the duties are detailed. In-progress visual (edge preparation, fit-up, root pass and fill-in-passes) by a qualified and experienced inspector is considered the most reliable method and most cost effective. By far, an alert welder or inspector detects most cracks in weldments visually. Sometimes detection is made hours or days after completion of the weld. This has been termed "delayed cracking" when, in most cases, the cracks were probably there at the completion of welding, but merely opened up wide enough to see when the entire weldments cooled. Proper visual inspection requires careful examination in the areas outside the weldments, particularly along plate edges and parallel to the weld where cracking and lamellar tearing can occur. AWS D1.1 requires NDT of high strength Q&T steels A514, A709 grades 100 and 100W to be performed 48 hours after completion of welding. Delayed inspection and delayed NDT is generally not necessary for the more common structural steel.

Magnetic Particle Testing (MT)

This method is primarily for detecting surface cracks or defects on or just below the surface of the metal. MT is currently being used per FEMA 353

recommendations on some fillet welds and some groove welds of seismic connections. MT is particularly applicable to crack-sensitive material and especially useful in detecting fatigue cracking. During the test, a very strong magnetic field is applied to the weld area, and the surface covered with a suspension of ferro-magnetic particles. Defects such as cracks, inclusions, etc., interrupt lines of force, causing the particles to concentrate around these areas. Often, the residual magnetic properties created by welding are sufficient to allow the use of magnetic particles without the application of a magnetic field. Because this method is simple, easy to read, and the equipment is portable, it is preferred for examination of welds and adjacent areas for surface cracking caused by weld shrinkage. The magnetic particle method is also very useful during repairs to see if the defect has been completely removed and to examine individual weld passes and layers for hot cracking.

Fundamentally MT is an enhanced surface examination and as such, the cost and schedule impacts need to be balanced with the perceived benefits.

Ultrasonic Testing (UT)

Since the development in the 1960's, ultrasonic testing has grown to become the most important tool in nondestructive testing of structural welded joints. In this method, high-frequency sound waves are used to locate and measure discontinuities in welded joints and base metals prior to welding. This method is very sensitive in detecting both surface and subsurface discontinuities. During testing, a sound wave is directed towards the weld joint and reflected back from the discontinuity and shown on a calibrated screen of an oscilloscope as shown in Figure 25 and 26. This method is highly sensitive in detecting planar defects, such as incomplete weld fusion, delamination, or cracks; however, orientation is very important. As the wave strikes the defect, the time-distance relationship will locate this interception. This is shown on the oscilloscope and indicates the location of the defect in the weld joint.

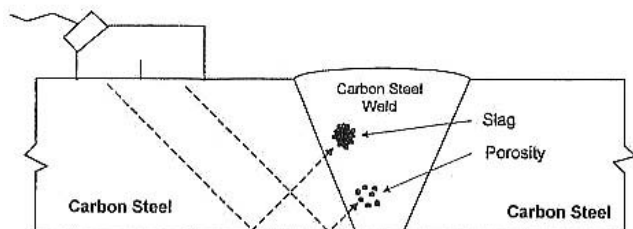


Fig 25. Ultrasonic testing of weld

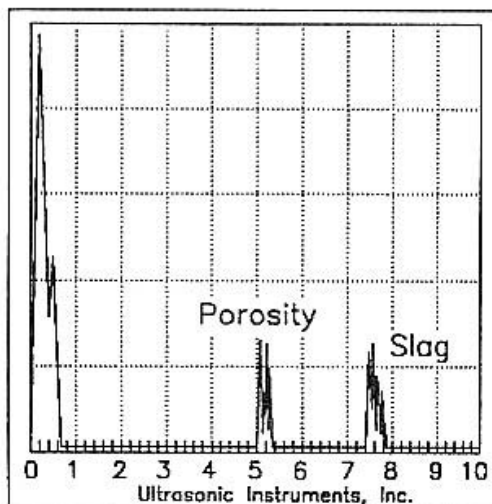


Fig. 26 Typical CRT (cathode ray tube) display of ultrasonically tested weld with indications.

This ultrasonic method can detect internal planar defects in sections of practically unlimited thickness. AWS D1.1, sets procedures for steel thickness from 5/16" to 8", but other thicknesses may be tested by qualified procedures. It is portable and relatively fast. Most importantly, it requires access to only one side of a test section.

There are some limitations to ultrasonic testing. Rough surfaces reduce its sensitivity and reliability. Also, the method does not produce a permanent record of the tested weld joint. In addition, globular defects, such as gas bubbles and other porosities are not easily detected. Because of the spherical nature of these defects, ultrasonic waves tend to pass around them rather than reflect back as with planar defects. However, this deficiency of the ultrasonic methods is not considered serious. Ultrasonic techniques, as practiced, are normally limited to joints with plate thicknesses above 5/16 inch, and are very sensitive to flaw orientation and geometry. Most building codes (and FEMA 353 recommendations) require ultrasonic testing of complete joint penetration groove welds.

The ultrasonic method is highly dependent on the skill and integrity of the operating technician for proper interpretation of the results, and therein arises a major weakness. An operator can quickly lose credibility if he calls for a joint to be completely gouged out for a defect that cannot be found. Consequently, it is easier for the operator, unless technically competent, to say nothing rather than risk being found wrong and then subsequently challenged repeatedly by the fabricator or contractor on the project.

On all special inspections calling for ultrasonic testing of welded joints by an independent testing agency, it

is important for the Engineer to seek evidence as to the qualifications of the ultrasonic technicians involved. UT technicians are usually qualified in accordance with the American Society for Nondestructive Testing Recommended Practice SNT-TC-1A.

In order to improve the reliability of UT in seismic and other selected connections, future training and qualification of UT operators will utilize some of the techniques and provisions outlined in Annex K of AWS D1.1. Annex K outlines a more rigorous procedure for UT. It was originally developed by use on off-shore structures that undergo extreme fatigue loading. There is still some debate within the industry as to the supply of qualified personnel capable of using these procedures, and therefore, it is intended to adopt the requirements gradually.

Radiographic Testing (RT)

Radiography relies on the use of electromagnetic radiation to determine the soundness of a weld. X-rays and Gamma rays are the two types of waves used to penetrate solid materials such as a welded joint. A permanent record of the weld structure is obtained by placing a sensitized film at the back of the weldment. As the rays pass through the weld material, they fall on the sensitized film and produce a negative of varying intensity. If the rays pass through gas bubbles, slag inclusions or cracks, more rays will pass through these less dense areas and will register on the film as dark areas. Orientation of the discontinuity is very important, especially for planar discontinuity.

Although radiography is a superior method of detecting porosity defects and slag inclusions, for most steel building construction, RT is impractical. This is because the film must be placed opposite the source of radiation to graphically record the defects, and the actual geometry of completed joints, particularly T-joints, generally prohibits proper placement of the film. RT is not commonly applied to structural fabrication because of the cost and schedule impacts associated with the need to shield personnel from radiation.

Liquid Dye Penetrant Testing (PT)

This method relies upon surface tension and capillary action of certain dye-carrying liquids to penetrate small surface flaws such as cracks. Subsequent application of suitable developer brings out the dye and outlines the defect. During the test, the surface weld is cleaned and dried, then coated with a thin film of the penetrant. After waiting a short time for the dye to flow into the flaw, the surface is wiped clean and the developer applied. The liquid penetrant will then

bleed out onto the surface to react with the developer and sharply outline the flaw so it can be seen or photographed. The use of dye penetrants in multi-pass welding has been limited to investigative use only because of the interruptions to welding process and consequent cost to fabricator.

Project Specifications

It has been the writers' experience in reviewing hundreds of project specifications through the years that there has been a needless waste of effort on the part of the designers in writing and rewriting portions of all of the standard AISC and AWS specifications. Sometimes, needless litigation has ensued as a direct result of rewriting nationally accepted standards to include the personal bias of the designer, albeit arising from previous bitter experience.

For the most part, fabricators, welders and inspectors are well aware of the national standards and keep up with them. The designers are well advised to do likewise. They will get a better product with less confusion and discord if they adopt them by reference and omit any attempt to elaborate, clarify or otherwise tamper with the nationally accepted standards unless there is specific conflict with the project specifications. There have been many large, successful projects completed with a one line specification limit that merely states, "All materials and workmanship shall be in accordance with the latest revised edition of the AISC manual of Steel Construction, which includes the specification for Structural Steel Buildings, the code of Standard Practice and the AWS Structural Welding Code."

However, for those who feel a project specification is only sufficient when it has a few pages under each section, a recommended list of items is included as a checklist and reference in the following section, "Project Specifications Check List."

Normally, the nondestructive testing section of a project specification is more detailed than other sections devoted to structural steel. The AISC code of Standard Practices requires that, "When nondestructive testing is required, the process, locations, extent, technique and standards of acceptance are clearly defined in the contract documents." This is also in the AWS D1.1:2002 Structural Welding Code-Steel.

It is advised to keep in mind that inspection requirements may vary between local, state and federal building regulatory jurisdictions. Standard inspection requirements should satisfy most jurisdictions because normal practice requires

continuous inspection by a qualified inspection agency paid for by the owner, for whose benefit the inspection is being performed. However, the designer is cautioned to determine for himself what differences, if any, are required by the governing agency for each project.

It is suggested that the structural steel designer obtains a copy of the AISC publication “Quality Criteria and Inspection Standards” and studies it carefully, particularly as it relates to dimensional tolerances. If the structure being designed requires closer construction tolerances than allowed, either change the design to accommodate them or put a large sign on your drawings to the effect that care must be exercised by the fabricator and erector to meet closer than normal tolerances, and then spell them out so there is no room for misunderstanding.

If the structure is tied into or otherwise supported on a masonry or concrete structure built prior to erection of the steelwork, don’t expect the anchor bolts to be in exact position. Make provisions in your design of the connections for misalignment vertically and horizontally of such anchors and/or make a field check mandatory. A review of normal construction tolerances for such construction will be enlightening, to say the least.

If the building structure will not resist wind or earthquake forces until materials other than structural steel are in place, it is recommended practice to notify the contractor with a note on the drawings or in the specifications which clearly state that fact. Such a requirement is contained in the AISC Code of Practice; Section 7.10 entitled “Temporary Support of Structural Steel Frames.”

FEMA 353 Quality Recommendations

Although there were no collapses of steel structures, serious concern grew out of the Northridge Earthquake’s moment frame beam flange weld fractures (refer to FEMA 350 for background information). In general, the typical project’s steel specifications have changed significantly since research and recommendations were released by the SAC Joint Venture, which was the most comprehensive steel research program ever undertaken. Out of this project, among other documents, came FEMA 353, which recommends various quality programs that could be applied to a steel construction project. The document was intended to be used as a guide that would be used judiciously. Engineers should not broadly specify “FEMA 353” in job specifications. Rather, an

engineering firm should review the document and decide which provisions are prudent and necessary for their project. The cost and benefit of each requirement should be evaluated. Engineering firms with experience in steel structures have merely updated their specification using FEMA 353 just as a guide. Other engineers may not have the knowledge or the expertise to do this on their own. Fortunately, the next issue of the AISC Seismic Provisions will be a consensus document which contains quality guidelines for steel construction. These will outline performance and documentation of everything from visual inspection to non destructive testing, submittals of contractor quality control programs, and owner quality assurance requirements for seismic resisting systems.

Project Specifications Check List

The following list of items is recommended as a reference and checklist to help develop the project specifications:

- Scope of work
- References to National Standards (AISC, AWS, AST, UBC, RCSC, SSPC, etc.)
- Shop Detail Drawings submittals
- Welding Procedure Specifications submittals
- Materials (List ASTM Specifications, Structural Steel, Pipe, Structural Tubing, High-Strength Bolts, Std. Bolts, Nuts and Washers)
- Welding Processes: Shop and Field: Prequalified and qualified-by-test.
- Filler Metal Specifications and Classifications
- Quality Control and Quality Assurance
- Fabrication
- Erection
- Galvanizing, Painting: Shop and Field
- Inspection: Shop and Field, including verification of welder certifications
- Specific requirements for the Seismic Force Resisting Frame.

Welding Procedure Specifications (WPSs)

Confusion may exist among some structural engineers and fabricators regarding written welding procedures. The AWS D1.1, clearly states in section 3.6 that “All pre-qualified WPSs to be used shall be prepared by the manufacturer, fabricator or Contractor as written pre-qualified WPSs, and shall be available to those authorized to use or examine them.” AWS D1.1 requires any WPS qualified by test to be approved by

the Engineer of record. In spite of this mandatory requirement, some Engineers do not require their submittal. Some fabricators always submit them for review while others neglect to do so. Some engineers review the WPS submittal in house, while others hire a consulting engineer that is familiar with the welding issues of steel fabrication and erection.

The Engineer reviewer can compare the submittals with the sample forms shown in Annex E and checklists in Annex H. These Annexes cover the mandatory code requirements of a (WPS). Where welds are not pre-qualified, they must be qualified by test. For these welds that are qualified by test, a Procedure Qualification Record (PQR) is required to accompany the WPS. WPSs should be submitted with the electrode manufacturer's product data sheets that outline the recommended parameters for voltage, amperage, electrical stick-out, polarity, and other pertinent variables. WPSs should show that the procedure falls within the manufacturer's recommended parameters.

Shop Detail Drawings

Shop drawings have been the subject of much debate for many years, and yet there still remains an absence of a uniform understanding within the design professions, legal professions, and the construction industry. There has been a wide variation in the manner in which shop drawings have been used, leading to a great deal of confusion.

Shop drawings are necessary to facilitate steel fabrication, erection and installation of various elements of the work. Their very nature is such that they are required to comply with the contract documents. Review of shop drawings is simply to confirm compliance and to facilitate progress of the work. It is the position of the author of this paper that much of the confusion that exists has come about because of the use of shop drawings as design documents. The misconception that shop drawings are part of the design process must be eliminated.

Shop drawings are not part of the contract documents and must not be used as such. If changes are proposed or made by the contractor or engineer, they must be done through a change order process. When shop drawings are used as an instrument of change, they can only lead to confusion. Clearly, changes proposed by the contractor, or the submittal of details or systems based upon performance-type specifications, must be reviewed and approved by the engineer of record. This process must take place through

documents other than shop drawings, such as change order or "supplemental design details."

Electronic Data Interchange (EDI)

The steel industry has been working to produce a set of standards for sharing electronic data. CIMsteel, or CIS/2 Integrated Standard, has been incorporated into many design, detailing, and fabrication packages. The standard allows a single electronic model to be carried through the entire project, from design, to material orders, detailing, fabrication, and erection drawings. Engineers may have concern over liability of releasing their models, but with the appropriate agreements and understandings with contractors on the accuracy of models, and the methods of handling revisions, all parties can benefit. The industry has yet to scratch the surface of the capabilities and future application of EDI. Numerous articles on EDI may be found at AISC's website, www.aisc.org, upon searching for "EDI."

Conclusions

Steel has been and will continue to be a very versatile structural material. From the manufacture of steel plate and shapes, through structural design, detailing, fabrication and erection, the steel industry continues to evolve along with technology to meet the demands of modern structures. It is the privilege and the responsibility of engineers to combine the latest technology with the state of the art and the state of the practice, along with lessons learned from earthquakes and research, to produce a sound and practical design.

Design engineers and plan check engineers can benefit from a reminder that no material or process is perfect, and that it is the engineer's job to manage those imperfections. Quoting the first paragraph from Salmon and Johnson's "STEEL STRUCTURES Design and Behavior", 2nd edition: "*Structural design may be defined as a mixture of art and science, combining the experienced engineer's intuitive feeling for the behavior of a structure with a sound knowledge of the principals of statics, dynamics, mechanics of materials, and structural analysis, to produce a safe economical structure which will serve its intended purpose.... Computations involving scientific principles should serve as a guide to decision making and not be followed blindly. The art or intuitive ability of the experienced engineer is utilized to make the decisions, guided by the computational results.*"

References

AISC (2001), *Manual of Steel Construction, Load and Resistance Factor Design*, American Institute of Steel Construction, Chicago, Illinois.

AISC (2002), *Seismic Provisions for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, Illinois.

ASTM (2001), *Standards for Structural Steel Fabrication*, West Conshohocken, Pennsylvania..

American Welding Society (2002), *Structural Welding Code*, Miami, Florida.

Blodgett, Omer W., P.E., Sc.D., R. Scott Funderburk, Duane Miller, P.E., Sc.D., and Marie A. Quintana, P.E., (1997), *Fabricators' and Erectors' Guide to Welded Steel Construction*, The James F. Lincoln Arc Welding Foundation, Miami, Florida.

FEMA-350 (2001), "Seismic Design Criteria for Steel Moment-Frame Structures", *Report*, Federal Emergency Management Agency, Maryland.

FEMA-353 (2001), "Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications", *Report*, Federal Emergency Management Agency, Maryland.

Monroe, Raymond and David Poweleit, (2003), *Opportunities for the Use of Steel Castings in Building Construction*, NASCC Proceedings, Baltimore, Maryland.

Preece, Robert F. and Alvaro L Collin, (1991), *Structural Steel Construction in the 90's, Technical Information and Product Service*, Structural Steel Educational Council, Walnut Creek, California.

Salmon, Charles G. and John E. Johnson, (1980), "Steel Structures Design and Behavior" Second Edition Harper & Row Publishers, New York

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Patrick M. Hassett completed his Bachelor's degree in Civil Engineering in 1983 and his Master's degree in Structural Engineering and Structural Mechanics in 1985, from The University of California at Berkeley. He is a licensed Civil and Structural Engineer in California and is also licensed in Illinois and Missouri. He has fifteen years experience in the fabrication and erection of major structural steel construction projects. He is a member of SEAONC, AWS, and ASCE, is past chairman of the Structural Steel Educational Council, has served on the SAC Joint Venture Project, and currently serves on the AISC Seismic Design and the AISC CPRP committees.

He is currently running his own consulting firm in Castro Valley, California, primarily serving the engineering needs of steel fabricators and erectors. Among his recent projects are the erection engineering for the Walt Disney Concert Hall in Los Angeles, erection engineering of the San Francisco International Airport, and the connection design and erection engineering on the 54-story Torre Mayor in Mexico City.

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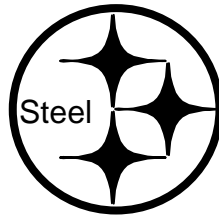
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