Mechanical Properties of Bridge Steels

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Steel Bridge Design Handbook

Bridge Steels and Their Mechanical Properties

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16. Abstract

This module presents an overview of structural steel products that are used for steel bridge construction. It is intended to serve as a reference and educational tool for structural engineers involved with bridge design and evaluation. The primary focus is on steel plate and rolled shape products that are available under the ASTM A709 Specification. This includes both a general introduction to steel making practices and a detailed discussion of mechanical properties. It also includes a brief introduction to other steel products such as bolts, castings, cables, and stainless steels that are often used for steel bridge connections and components. References are provided to the relevant AASHTO and ASTM standards for additional information.

The mechanical properties of bridge steels are presented based on the A 709 specification. The stress-strain behavior of the various steel grades is presented to provide an understanding of strength and ductility. Fracture toughness is discussed to relate how the Charpy vee-notch test relates to fracture resistance in structures. Finally, the methodology for determining atmospheric corrosion resistance is presented along with the requirements for classification as "weathering steels" for use in un-coated applications.

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Steel Bridge Design Handbook: Bridge Steels and Their Mechanical Properties

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FOREWORD

It took an act of Congress to provide funding for the development of this comprehensive handbook in steel bridge design. This handbook covers a full range of topics and design examples to provide bridge engineers with the information needed to make knowledgeable decisions regarding the selection, design, fabrication, and construction of steel bridges. The handbook is based on the Fifth Edition, including the 2010 Interims, of the AASHTO LRFD Bridge Design Specifications. The hard work of the National Steel Bridge Alliance (NSBA) and prime consultant, HDR Engineering and their sub-consultants in producing this handbook is gratefully acknowledged. This is the culmination of seven years of effort beginning in 2005.

The new *Steel Bridge Design Handbook* is divided into several topics and design examples as follows:

- Bridge Steels and Their Properties
- Bridge Fabrication
- Steel Bridge Shop Drawings
- Structural Behavior
- Selecting the Right Bridge Type
- Stringer Bridges
- Loads and Combinations
- Structural Analysis
- Redundancy
- Limit States
- Design for Constructibility
- Design for Fatigue
- Bracing System Design
- Splice Design
- Bearings
- Substructure Design
- Deck Design
- Load Rating
- Corrosion Protection of Bridges
- Design Example: Three-span Continuous Straight I-Girder Bridge
- Design Example: Two-span Continuous Straight I-Girder Bridge
- Design Example: Two-span Continuous Straight Wide-Flange Beam Bridge
- Design Example: Three-span Continuous Straight Tub-Girder Bridge
- Design Example: Three-span Continuous Curved I-Girder Beam Bridge
- Design Example: Three-span Continuous Curved Tub-Girder Bridge

These topics and design examples are published separately for ease of use, and available for free download at the NSBA and FHWA websites: http://www.steelbridges.org, and http://www.fhwa.dot.gov/bridge, respectively.

The contributions and constructive review comments during the preparation of the handbook from many engineering processionals are very much appreciated. The readers are encouraged to submit ideas and suggestions for enhancements of future edition of the handbook to Myint Lwin at the following address: Federal Highway Administration, 1200 New Jersey Avenue, S.E., Washington, DC 20590.

M. Myint Lwin, Director
Office of Bridge Technology

1.0 INTRODUCTION

Structural steels for use in bridges generally have more stringent performance requirements compared to steels used in buildings and many other structural applications. Bridge steels have to perform in an outdoor environment with relatively large temperature changes, are subjected to millions of cycles of live loading, and are often exposed to corrosive environments containing chlorides. Steels are required to meet strength and ductility requirements for all structural applications. However, bridge steels have to provide adequate service with respect to the additional Fatigue and Fracture limit state. They also have to provide enhanced atmospheric corrosion resistance in many applications where they are used without expensive protective coatings. For these reasons, structural steels for bridges are required to have fracture toughness and often corrosion resistance that exceed general structural requirements.

This module is written from a structural engineer's perspective and focuses on performance aspects of structural steel. A general overview of steel making practice is provided for information, stressing factors that may be relevant to the structural engineer and the structural performance of the product. The primary focus is on steel plate and rolled shape products that are available under the ASTM A 709 Specification. This includes both a general introduction to steel making practices and a detailed discussion of mechanical properties. It also includes a brief introduction to other steel products such as bolts, castings, cables, and stainless steels that are often used for steel bridge connections and components. References are provided to the relevant AASHTO and ASTM standards for additional information.

The mechanical properties of bridge steels are presented based on the A 709 specification. The stress-strain behavior of the various steel grades is presented to provide an understanding of strength and ductility. Fracture toughness is discussed to relate how the Charpy vee-notch test relates to fracture resistance in structures. Finally, the methodology for determining atmospheric corrosion resistance is presented along with the requirements for classification as "weathering steels" for use in un-coated applications.

2.0 PRODUCT SPECIFICATIONS

There are two organizations that publish standards for structural steel in the U.S. The American Society for Testing and Materials (ASTM) is a non-profit voluntary standards organization that develops consensus standards for steel products. Committee A-1 and subcommittee A01.02 have the primary responsibility for structural steel standards, including bridge steels (1). Membership is comprised of experts from industry, end users, government, and academia to provide a balance of perspectives. The American Association of State Highway Transportation Officials (AASHTO) publishes a separate volume of standards (2) that also include structural steel standards for bridge applications. These standards are developed by committees comprised solely of government officials responsible for construction and maintenance of the highway system. In most cases, the AASHTO standards are very similar or identical to the corresponding ASTM standards. This is particularly true for bridge steel products. The question arises, why do we need two identical standards? By keeping independent standards, AASHTO maintains the right to modify the ASTM requirements if it is determined to be in the public's interest.

Most bridge owners specify adherence to the AASHTO material specifications in their construction documents. Some specify ASTM specifications. In most cases, the two are identical for steel products. Table 1 shows the applicable AASHTO and ASTM standards for steel product categories. Some of the ASTM standards do not have an AASHTO counterpart. The following sections provide an overview of the specification provisions.

Table 1 Cross reference between AASHTO and ASTM standards for bridge steel products.

Product	AASHTO Specifications	ASTM Specifications
Structural Steel for Bridges	M 270/M 270M	A 709/A 709M
Structural Stainless Steel		A 1010
Cold-Formed Welded or Seamless Tubing		A 500 Grade B
Hot-Formed Welded or Seamless Tubing		A 501
Pins, Rollers, and Rockers	M 169 M102/M 102M	A 108 A 668/A 668M
Bolts	 M 164 M 253	A 307 Grade A or B A 325 A 490 F 1852
Galvanized Structural Bolts	M 232/M 232M Class C M 298 Class 50	A 153/A 153M B 695
Anchor Bolts	M 314-90	A 307 Grade C F 1554
Nuts	M 291	A 563
Washers	M 293	F 436 F 959
Shear Studs	M 169	A 108
Cast Steel	M 103/M 103M M 163/M 163M	A 27/ A 27M A 743/A 743M
Ductile Iron		A 536
Malleable Castings		A 47 Grade 35018
Cast Iron	M 105 Class 30	A 48 Class 30
Stainless Steel		A 176 A 240 A 276 A 666
Cables		A 510
Galvanized Wire		A 641
Epoxy Coated Wire		A 99
Bridge Strand / Bridge Rope		A 586 A 603
Wire Rope	M 277	
Seven-Wire Strand	M 203/M 203M	A 416/A 416M
High Strength Steel Bar	M 275/M 275M	A 722/A 722M

2.1 Structural Plate and Rolled Shapes

The ASTM A 709 Standard Specification for Structural Steel for Bridges (3) was established in 1974 as a separate specification covering all structural grades approved for use in main members of bridge structures. Many of the A 709 provisions are identical to those in the individual structural steel specifications applicable for more general use. Table 2 provides an overview of the various steel grades covered by the specification. The number in the grade designation indicates the nominal yield strength in ksi. The A 709M specification is the metric version of A 709.

Table 2 Overview of bridge steels available in the A 709 specification.

M 270 A 709 GRADE	ASTM Specification	Description	Atmospheric Corrosion Resistance	Product Categories			
				Plates	Shapes	Bars	Sheet Piles
36	A 36	Carbon Steel	No	X	X	X	
50	A 572	HSLA Steel	No	X	X	X	X
50S	A 992	Structural Steel	No		X		
50W	A 588	HSLA Steel	Yes	X	X	X	
HPS 50W	A 709	HSLA Steel(*)	Yes	X			
HPS 70W	A 709	Heat Treated(*) HSLA Steel	Yes	X			
HPS 100W	A 709	Q&T Cu-Ni Steel(*)	Yes	X			

(*) High Performance Steel (HPS) grades with enhanced weldability and toughness
HSLA High Strength Low-Alloy
Q&T Cu-Ni Quenched & Tempered Copper-Nickel Steel

2.1.1 Grade 36

The ASTM A 36 specification was originally adopted in 1960 as the final evolution of weldable carbon-manganese structural steel. Of all the steels in the A 709 specification, this is the easiest and cheapest to produce in steel mills that produce steel by melting iron ore in a blast furnace. Much of the steel making practice in the U.S. has now switched to electric furnace production where a large percentage of scrap is used to produce structural steel. Since scrap typically has more alloy elements than required by the A 36 specification, the resulting steel strength is typically much higher. The steels being delivered today as Grade 36 typically have strengths closer to 50 ksi than 36 ksi.

2.1.2 Grade 50

Grade 50 is the most common grade of structural steel available today. The A 572 specification was originally adopted in 1966 to introduce this higher strength grade of weldable structural steel. The strength was obtained by adding small amounts of columbium, vanadium, and sometimes titanium to the basic carbon-manganese chemistry of A 36 steel. This resulted in a 39% increase in yield strength compared to A 36 steel. The resulting increase in structural efficiency provided by the higher strength more than offset the increased cost of adding alloy to the steel. Grade 50 rapidly became the material of choice for primary bridge members that are to be painted or galvanized in service.

2.1.3 Grade 50W

Grade 50W is a special version of 50 ksi steel that was developed to have enhanced atmospheric corrosion resistance. This is commonly called "weathering" steel and is capable of performing well without paint or other coatings in many bridge applications. Different steel companies initially developed competing proprietary grades that were included in the A 588 specification in 1968. The added corrosion resistance was achieved by adding different combinations of copper, chromium, and nickel to the grade 50 chemistry to provide enhanced corrosion resistance. There is an added cost for grade 50W compared to grade 50, but this cost is often offset by the savings realized by eliminating the need to paint bridge structures.

2.1.4 Grade 50S

The A 992 specification was introduced in 1998 to keep pace with changes in rolled shape production practices in the U.S. As was previously discussed for Grade 36, the shift to scrap-based production made Grade 36 materials somewhat obsolete. Steels under the A 992 specification are dual certified to qualify for Grade 36 or Grade 50. It is more difficult to precisely control the chemical composition of scrap-based steel production since many alloys may be present in scrap steel. Therefore, the A 992 specification allows a wide range of steel chemistry. However, too much alloying can adversely affect the performance of structural steel and maximum percentages are set for C, Si, V, Co, P, S, Cu, Ni, Cr, and Mo. As long as the alloying stays below these maximum levels, the specification is largely performance-based upon meeting the required strength and ductility requirements.

2.1.5 Grade 100 and 100W

A 514 steel is a high strength (100 ksi), quenched and tempered product that was originally introduced in 1964. The specification has different grades with different chemical composition requirements corresponding to products from different steel producers. All grades have the same mechanical property requirements and can be considered equivalent for structural applications.

While the A 514 steels are weldable and are included in the D1.5 Bridge Welding Code, there have been a number of reported problems in fabrication. In some cases, delayed hydrogen cracking has been discovered both in the fabrication plant and through in-service inspection of bridges. The history of weldability problems for this grade was one of the catalysts for development of the new HPS grades discussed in the following section. In 2010, the A 709 Specification was revised to delete grades 100 and 100W. HPS 100W is now the only grade permitted for structural bridge members.

Engineers may still specify A 514 Grades 100 and 100W for bearing components and other secondary components in bridges. Since A 514 steels are used in other industries, there may be better availability in small quantities.

2.1.6 HPS Grades

The high performance steel (HPS) grades were developed through a cooperative agreement between the Federal Highway Administration, the U.S. Navy, and the American Iron and Steel Institute. The goal was to enhance weldability and toughness compared to previous versions of grade 70 and 100 steel (4). Prior to HPS, steels with yield strength greater than 50 ksi (A 852 and A 514) were very sensitive to welding conditions and fabricators often encountered welding problems. The HPS grades have essentially eliminated base metal weldability concerns. In addition, HPS grades provide enhanced fracture toughness compared to non-HPS grades. Because of the greatly enhanced properties, the original grade 70W steel (A 852) has been replaced in the A 709 specification and HPS 70W is the only 70 ksi option for bridge use. For similar reasons, the HPS 100W grade has now replaced grades 100 and 100W for fabrication of structural bridge members where 100 ksi strength is desired.

The properties of HPS are largely achieved by dramatically lowering the percentage of carbon in the steel chemistry. Since carbon is traditionally one of the primary strengthening elements in steel, the composition of other alloying elements must be more precisely controlled to meet the required strength and compensate for the reduced carbon content. There are also stricter controls on steel making practice and requirements for thermal and/or mechanical processing to meet the required strength. These refinements in steel making practice result in a very high quality product. However, this also limits the number of steel mills that have the capability of producing HPS steels in the US. As of this writing, HPS steels come with a cost premium and additional lead-time is required in ordering versus non-HPS grades. However, experience is showing that HPS steels, due to their higher strength, can result in more efficient bridges with lower first cost. This benefit generally is greater as the size and span length of bridges increase. Because it costs more than conventional steel, use of HPS should be carefully considered by the designer to insure the benefits outweigh the additional cost of the product.

HPS 50W is an as-rolled steel produced to the same chemical composition requirements as grade HPS 70W. Similar to the higher strength HPS grades, HPS 50W has enhanced weldability and toughness compared to grades 50, 50W, and 50S. However, the need for enhanced weldability is questionable at this strength level since few weldability problems are reported for the non-HPS grades. The primary advantage of HPS 50W is that it can be delivered with high toughness that exceeds the current AASHTO specification requirements for grades 50 and 50W. Enhanced toughness may be beneficial for certain fracture critical members with low redundancy such as the tension ties in tied arch bridges. Research is underway to integrate the benefits of higher toughness into the AASHTO Fracture Control Plan discussed in Section 4.6. Since HPS 50W is a higher cost material compared to grade 50W, engineers should carefully consider the need for higher toughness before specifying HPS 50W.

2.2 Stainless Steels

Stainless steels are occasionally used to fabricate bearings and other parts for bridges where high corrosion resistance is required. Traditionally, the relative high cost of stainless steel has limited its use in primary bridge members. Recently, the FHWA funded research to develop more cost effective grades of structural steel with higher corrosion resistance compared to conventional weathering steel grades. Unfortunately, the goal of developing low cost structural steels with enhanced corrosion resistance remains elusive and there is currently a substantial cost premium associated with high corrosion resistance. However, given the expanding trend toward life-cycle cost analysis, stainless steels merit consideration for some structural applications.

The most promising product for structural bridge use is ASTM A 1010 Grade 50, a dual phase stainless steel with a 12% chromium content (5). This product meets the mechanical property requirements for A 709 Grade 50 and can meet the supplemental CVN requirements for grade HPS 50W material. The product has been shown to have greatly enhanced corrosion resistance compared to weathering steel grades (6) and can provide adequate performance without paint in higher chloride bridge environments. The grade is currently available in thicknesses up to 2 in. Currently, some special provisions are required to utilize this grade within the existing bridge specifications. A 1010 steel is weldable using all processes currently employed for bridge fabrication. However, this product is not currently included in the D1.5 Bridge Welding Code, therefore supplemental provisions need to be invoked based on recommendations by the manufacturer. The grade can be processed using standard fabrication practices including cold bending, heat curving, and machining. One exception is that the material is not suitable for cutting using oxy-fuel processes. Plasma or laser cutting is required. Another exception is that blast cleaning needs to be performed with non-metallic media to avoid staining of the surface in service.

Stainless steels are subject to increased corrosion if they are placed in contact with regular carbon steel. This requires the use of either stainless steel or galvanized fasteners. In addition, special care is needed to avoid contact with or connections to regular carbon steel components.

Since there is currently limited experience with the use of A 1010 steel in bridges (7) and this grade has not yet been included in the AASHTO specifications, projects will require special provisions and may require supplemental testing at the discretion of the engineer.

2.3 HSS Tubular Members

Hollow structural sections (HSS) are commonly used in building construction and they can be considered as an option for some bridge members. Increased lateral bending and torsional resistance can make them an attractive option for cross bracing and other secondary members subjected to compression. HSS have also been used to fabricate trusses used for pedestrian bridges that are subject to lower fatigue loading. HSS commonly refers to cold-formed welded or seamless structural steel tubing produced under the A 500 specification (8). Grade C has minimum specified yield and tensile strengths of 50 ksi and 62 ksi, respectively. The shapes are usually formed by cold bending carbon steel plate into the required shape and making a longitudinal seam weld along the length. Both round and rectangular shapes are available with various cross sections and wall thicknesses.

The suitability of HSS for bridge members subject to the fatigue and fracture limit states has not been established. Cold bending of the corners of rectangular shapes can lead to reduced notch toughness in the corner regions. Testing procedures have not yet been established to perform CVN or other toughness tests in the curved wall regions. In addition, HSS requires different connection details for which limited fatigue data currently exists. Another possible concern for bridge use is the need to control internal corrosion within the tubes, since the interior of the tube cannot be accessed for visual inspection. Sealing of the tube ends or galvanizing are possible options to control internal corrosion. Designers specifying HSS should consider connection design procedures from the AISC Manual of Steel Construction and the AWS D1.1 Structural Welding Code.

2.4 Bolts and Rivets

Structural bolts for members requiring slip critical connections in bridges are required to comply with either the ASTM A 325 or A 490 specifications. The A 307 specification provides a lower cost option for anchor bolts and non-slip critical connections. Compatible nuts are required to be used with all bolts meeting provisions for the appropriate grade in the A 563 specification. Hardened steel washers meeting the F 436 specification are required underneath all parts of the bolt assembly that are turned during installation. The surface condition and presence of lubrication is important for proper installation of the bolt-nut assemblies. The A 325 and A 490 specifications require bolt lots to be subjected to tensile testing and hardness testing to ensure that the minimum specified tensile strength shown in Table 3 is met.

Table 3 Tensile strength of structural bolts for bridge use.

Grade	Diameter (in)	Tensile Strength (ksi)
A 307 (Grade A or B)	All	60
A 325	0.5 to 1.0	120
	1.125 to 1.5	105
A 490	All	150

The A 325 and A 490 specifications have two different chemistry requirements for bolts. Type 1 bolts are basic carbon-manganese steel with silicon additions and possibly boron. Type 1 bolts are suitable for use with painted and galvanized coatings. Type 3 bolts have additional requirements for copper, nickel, and chromium to be compatible with the chemistry of weathering steel grades. Type 3 bolts are required for use in un-painted applications where both the bolts and base metal develop a compatible protective rust patina in service.

Galvanized bolts are available conforming to the A 325 and A 307 specifications. Both hot-dip and mechanically galvanized bolts are permitted under the AASHTO specifications. Bolts are required to be tested after galvanizing to ensure the strength and ductility is not degraded by the process. Galvanized A 490 bolts are not allowed by AASHTO for bridge use. Because of their higher strength, A 490 bolts are susceptible to possible stress corrosion cracking and embrittlement during galvanizing.

Rivets are rarely used today for new construction, however a significant number of bridges still exist with riveted construction. The ASTM A 502-03 specification provides three rivet grades with different chemistry requirements. The Grade 1, 2, and 3 chemistries correspond to basic carbon steel, HSLA steel, and weathering steel chemistries, respectively. Many bridge structures were built prior to this specification and the exact rivet grade and strength may be unknown.

Anchor bolts used to connect steel components to concrete foundations with diameters up to 4 in. are required to comply with the ASTM F 1554 specification. Three grades are available (36, 55, and 105) corresponding to the yield strength of the bolt in ksi. Similar to structural bolts, anchor bolts are required to be used with compatible nuts and washers. Both galvanized and non galvanized options are available. The F 1554 specification has supplemental provisions for notch toughness that can be invoked by the engineer for anchor bolts loaded in tension, if needed. The A 307 Grade C specification, although still allowed in the AASHTO design code, has been replaced by the F 1554 specification in ASTM.

2.5 Wires and Cables

Cables used in bridge construction are generally referred to as bridge strand (ASTM A 586) or bridge rope (ASTM A 603). They are constructed from individual cold-drawn wires that are spirally wound around a wire core. The nominal diameter can be specified between 1/2 in and 4 in. depending on the intended application. Strands and cables are almost always galvanized for use in bridges where internal corrosion between the wires is a possibility. Because cables are an assemblage of wires, it is difficult to define a yield strength for the assembly. Therefore, the capacity is defined as the minimum breaking strength that depends on the nominal diameter of the cables.

Since cables are axial tension members, the axial stiffness needs to be accurately known for most bridge applications. Because relative deformation between the individual wires will affect elongation, bridge strand and rope is pre-loaded to about 55% of the breaking strength after manufacturing to "seat" the wires and stabilize the deformation response. Following pre-loading, the axial deformation becomes linear and predictable based on an effective modulus for the wire bundles. Bridge rope has an elastic modulus of 20,000 ksi. The elastic modulus of bridge strand is 24,000 ksi (23,000 ksi for diameters greater than 2 9/16 in.).

Seven-wire steel strand is used in some structural steel applications although its primary use is for prestressed concrete. Possible uses include cable stays, hangers, and post-tensioning of steel components. Seven-wire strands consist of seven individual cold drawn round wires spirally wound to form a strand with nominal diameters between 0.25 and 0.60 in. Two grades are available (250 and 270) where the grade indicates the tensile strength of the wires (f_{pu}). Because of the voids between wires the cross sectional area of the strand will be less than that calculated based on the nominal diameter. The standard strand type is classified as low-relaxation. When a strand is stretched to a given length during tensioning, relaxation is an undesirable property that causes a drop in strand force over time. Strands are usually loaded by installing wedge-type chucks at the ends to grip the strand.

Mechanical properties for seven-wire strands are measured based on testing the strand, not the individual wires. The tensile strength is calculated by dividing the breaking load by the cross-sectional area of the strand wires. Compared to structural steels, strands do not exhibit a yield plateau and there is a gradual rounding of the stress-strain curve beyond the proportional limit. The yield strength (f_{py}) is determined by the 1% extension under load method where the strand elongates 1% during testing. Strands loaded to the yield stress will therefore experience increased permanent elongation compared to other structural steel products. AASHTO defines the yield strength as $f_{py} = 0.90\ f_{pu}$ for low relaxation strands. The elastic modulus of strands (E = 28,500 ksi) is lower than the modulus for the individual wires due to the bundling effect.

High strength steel bars are another product has applications for steel construction although their primary use is in prestressed concrete. Although they do not meet the definition of a wire or cable, high strength bars are included in this section since they are used for the same purposes as seven-wire strand. The bars are available in diameters ranging from 5/8 to 1-3/8 in. and can either be undeformed (Type 1) or have spiral deformations (Type 2) along their length that serve as a coarse thread for installing anchorage and coupling nuts. Unlike bolts, the bars cannot be tensioned by turning the nuts, the nuts act like the wedge anchors used for prestressing strand. Similar to seven-wire strands, high strength steel bars are specified based on their tensile strength (commonly f_{pu} = 150 ksi). AASHTO defines the yield strength as $f_{py} = 0.80f_{pu}$ for deformed bars and the modulus is E = 30,000 ksi.

2.6 Castings

Cast Iron is primarily made from pig iron with carbon and silicon as the main alloying elements. It can provide strength similar to mild structural steel and can be poured into molds to produce parts with complex geometries. The disadvantage is that the material tends to be brittle with little ductility. In bridges, the use of cast iron is generally limited to bearings, machine parts for movable bridges, and other parts that are primarily loaded in compression. Historically in the 19th century, wrought iron, which has better ductility than cast iron, was used to fabricate bridges. However, its use was discontinued after the introduction of steel. Cast irons and wrought iron are generally considered to be non-weldable although some materials can be welded using special techniques.

Ductile cast iron is a relatively new product that has more applicability for use in bridges. Unlike cast iron, ductile cast iron can be welded to structural steel members to form composite sections. Ductile iron has been used as a joint to connect structural steel tubes to form truss or frame systems. Recently, there has been some research to develop ductile iron end caps for HSS tubes that can simplify their connection details for use as bridge cross-frame elements. The cost of producing custom ductile iron parts may be prohibitive at the current time but mass production may eventually make them cost effective.

3.0 STEEL MANUFACTURING

3.1 Overview

Structural steels are produced by combining iron, carbon, and other alloying elements in a molten state, casting the steel into solidified ingots or blocks, and processing the ingots or blocks through rollers to form finished plates or structural shapes. This basic process for steel making has been in existence for hundreds of years, but modern refinements have steadily improved the quality of modern structural steels. The chemical composition of steel along with the casting, rolling, and possible post-rolling heat treatment operations will determine the mechanical properties, uniformity, and quality of the final product.

3.1.1 Chemistry

The chemical composition of steel is the starting point for steel production. Modern structural steels are primarily a combination of iron (Fe), carbon (C), and manganese (Mn). Many grades specify additional alloying elements to improve strength, toughness, and ductility. Alloy elements may also be added for quality control purposes or to enhance corrosion resistance. There is considerable interaction between the effects of the various alloying elements and the chemical composition of steel must be tightly controlled to obtain the required properties. The ASTM A 709 and other steel specifications provide tables indicating the allowable range of elemental composition for each grade. The limits may be expressed as minimums, maximums, or a range between a minimum and maximum depending on the effect of the individual elements.

Carbon is the principal hardening element in steel and is a relatively low-cost alloy for this purpose. However, carbon has a moderate tendency to segregate in casting resulting in non-uniformity. It can also degrade ductility, toughness, and weldability in high concentrations. For these reasons, the new HPS grades were developed with carbon levels significantly lower than conventional structural steels. Manganese is also a hardening element in steel though it has a lesser impact than carbon. It tends to combine with sulfur to form manganese sulfides, thereby reducing the harmful effects of sulfur. Aluminum and silicon are the primary deoxidizing elements in the traditional manufacture of carbon and alloy structural steels. The need for deoxidization will be discussed under quality control measures.

Table 4 Effect of alloying elements on steel.

Element	Symbol	Advantages	Disadvantages
Carbon	C	Increases strength and hardness Low cost	Decreases ductility and toughness Decreases weldability Moderate tendency to segregate
Manganese	Mn	Increases strength Controls harmful effects of sulfur	
Phosphorous	P	Increases strength and hardness Can increase atmospheric corrosion resistance	Decreases ductility and toughness Can be considered an impurity Strong tendency to segregate
Sulfur	S	Increases machinability	Generally considered undesirable Decreases ductility and toughness Decreases weldability Strong tendency to segregate
Silicon	Si	Used to deoxidize (kill) molten steel	
Aluminum	Al	Used to deoxidize (kill) molten steel Refines grain size, thereby increasing strength and toughness	
Vanadium	V	Small additions increase strength	
Columbium	Nb	Small additions increase strength	
Nickel	Ni	Increases strength and toughness	
Chromium	Cr	Increases strength Increases atmospheric corrosion resistance	
Copper	Cu	Increases atmospheric corrosion resistance	
Nitrogen	N	Increases strength and hardness	Decreases ductility and toughness
Boron	B	Small amounts increase hardenability in low carbon, heat treated steels	
Titanium	Ti	Reduces carbon inclusions	

3.1.2 Steel Casting

The traditional method of steel making was to pour molten steel into molds and cast it into ingots. These ingots are removed from their molds, reheated, and rolled into rectangular cross sections called slabs or blooms as the first step in processing them into the final product shapes.

The chemical composition of steel ingots tends to vary due to segregation of the elements during solidification. The cooling rate is higher where the ingot is in contact with the cooler mold and decreases toward the center. The solidification of the various elements in steel is dependent on the cooling rate. At high cooling rates, around the mold edge, segregation does not have time to occur and the composition is relatively uniform. In the center, the slower cooling rate allows iron to solidify first and some elements migrate into the still molten regions of the ingot. The final portions to solidify therefore have higher concentrations of sulfur, phosphorous, carbon, and other elements with a higher tendency to segregate than iron. As a result, steel products produced from ingots have an inherent variability in chemical composition at different locations. Ingot variability has been historically controlled by changing the ingot size and shape and cropping off portions of the ingot prior to rolling.

Most steel making today is done by the process of continuous casting. Continuous casting machines were developed in the 20th century to directly cast molten steel into slabs thereby bypassing the ingot casting stage. The molten steel is poured into an oscillating, water-cooled mold at a controlled rate and a continuous slab emerges from the mold. The continuous slab is water cooled and cut to the required lengths for product rolling operations. Continuous casting creates a higher cooling rate and minimizes segregation compared to the ingot process. Another advantage is that the intermediate step of rolling slabs from the ingots is eliminated. The end result is that continuous casting results in more uniform steel products and improves the cost-effectiveness of steel making.

3.1.3 Rolling

The cast slabs must be reheated and passed back and forth through a series of rollers to form the slabs into the final sizes of structural plates or shapes. Traditional hot-rolled products are heated, rolled to shape, and allowed to air cool. The relative temperature versus time history of hot rolling is illustrated in Figure 1. It shows that the slabs are heated to about 2,350°F, passed back and forth under rollers at relatively high temperature to plastically deform the plate to final dimensions, and allowed to air cool. The zig-zag portion of the line indicates where the rolling occurs in the temperature cycle.

Hot rolling is the conventional method of steel processing and is still widely utilized in steel making. If enhanced properties are needed, post-rolling heat treatments can be applied to alter the strength, ductility, and fracture toughness of the steel. More precise control of temperature during the rolling process can lead to property enhancement without the need for additional heat treatment. Many modern steel mills employ thermo-mechanical controlled processing (TMCP) methods as a more cost effective alternative to conventional post-rolling heat treatment. TMCP introduces more precise control of temperature during the rolling process. This can be achieved by introducing hold times, water spray cooling, or possible reheating to optimize temperature at various stages of the rolling process. The various processing methods will be discussed further in this section.

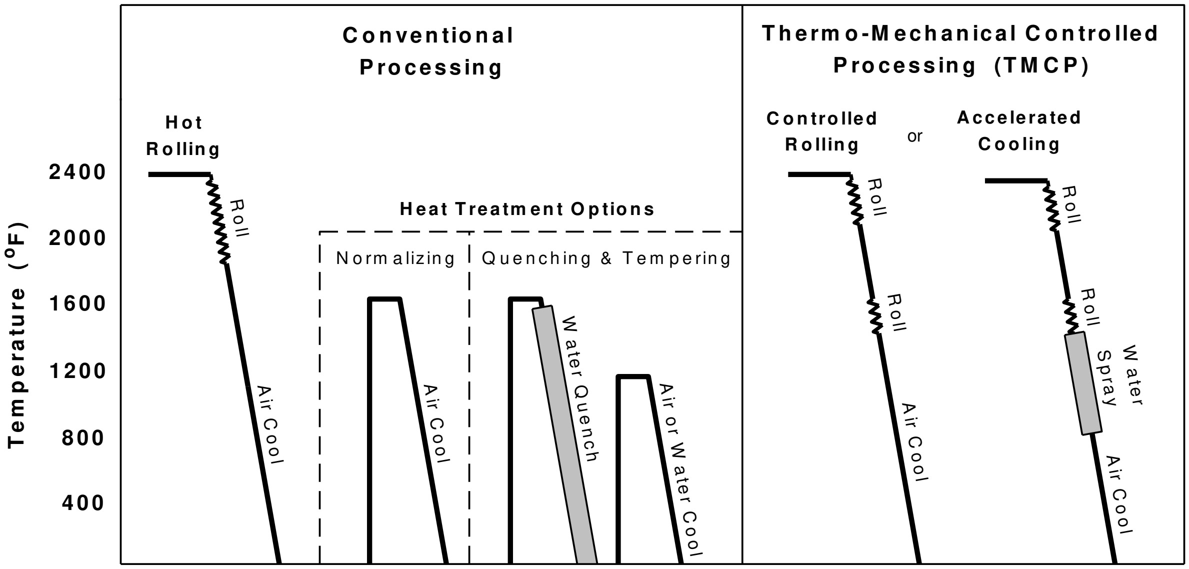


Figure 1 Relative temperature-time history for plate rolling and heat treating processes.

3.2 Quality Control Measures

The ASTM A 6 specification specifies that structural products shall be free of injurious defects and shall have a workmanlike finish (9). Visual inspection is the usual requirement for inspection of the surface of plates, although the definition of injurious defects is vague. Crack-like defects are generally considered injurious for bridge applications. Surface roughness and dimples due to rolling in mill scale may or may not be acceptable based on aesthetics. The specification allows plates to be conditioned by welding and/or surface grinding repairs to remove defects prior to delivery. The specification also acknowledges that some defects may be hidden by mill scale and not apparent until the mill scale is removed in fabrication.

Many different quality control measures are employed in the production of bridge steels to minimize defects and promote uniformity of the final products. It is important to minimize the presence of trapped gasses in the molten steel and to minimize segregation of alloy elements during solidification and rolling of the steel products. Trapped gasses can lead to crack-like defects in plates. It is particularly important to control these defects in bridge steels to insure adequate performance with respect to the fatigue and fracture limit states. Segregation can lead to variability in mechanical properties.

3.2.1 Degassing

Dissolved oxygen combines with carbon to form carbon dioxide gas in the molten steel. During solidification, the solubility of carbon monoxide and other gasses decreases and they come out of solution causing non-uniformity and porosity in the solidified ingots. This leads to undesirable defects and strength variability in the final rolled products. Aluminum and Silicon additions reduce the amount of oxygen available for formation of carbon dioxide, thereby reducing or eliminating gas evolution while the ingots are solidifying. Such steels are called "Killed" since they lie quietly in the mold without gas evolution during cooling. Grades 36, 50, and 50S are required to be killed or semi-killed in the A709 specification.

Grades 50W, HPS 50W, HPS 70W, and HPS 100W are required to be produced to fine grain practice. This is defined as achieving a fine austenitic grain size as specified in ASTM A 6. The methods to achieve fine grain size, such as aluminum additions, also have the effect of binding oxygen and nitrogen.

Grades HPS 50W, HPS 70W, and HPS 100W are required to be produced using a low hydrogen practice such as vacuum degassing. Hydrogen trapped during steel solidification tends to eventually migrate to the surface by breaking the bonds between grains, thereby creating crack-like defects in the steel. This is also a concern for welding where moisture control is important to prevent hydrogen cracking in weld metal. Hydrogen control is particularly important for bridge steels since crack-like defects can reduce the fatigue and fracture resistance. For the non-HPS grades, the need for low hydrogen practice is determined by the individual mills to avoid rejectable defects in their products.

3.2.2 Segregation

Low resistance to lamellar tearing is an adverse consequence of segregation during the rolling process. As plates and shapes are processed through the rolls they undergo higher cooling rates and plastic deformation strains at the surface. Metallic and non-metallic elements tend to segregate to the mid-thickness location of the finished product. In plates, this tends to form a planar inclusion at mid thickness, that is parallel to the rolling direction of the plate. The same effect can be seen in rolled shape flanges in the "k" line region where the flanges meet the web. The typical location of these planar inclusions is shown in Figure 2.

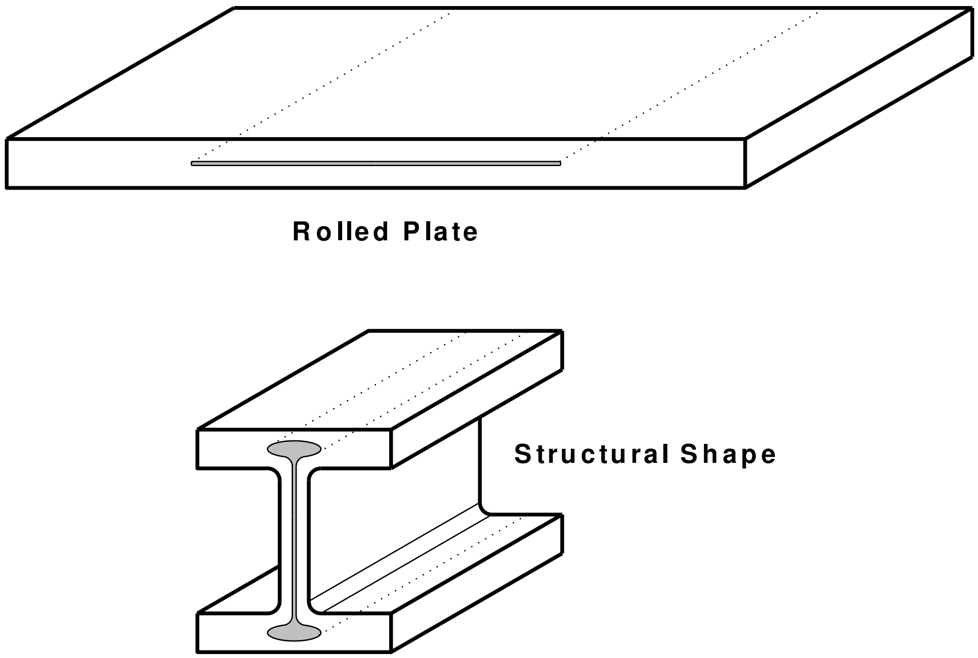


Figure 2 Segregation causes planar inclusions at the mid-thickness location of steel products.

The planar inclusions create a weak layer at the mid-thickness location of the plates or elements of shapes. Weathering steels, due to the added alloy elements such as copper, tend to have more pronounced segregation layers in some cases. This layer has no effect on mechanical properties unless the plates are loaded to create a through-thickness stress in the plate. Lamellar tearing is discussed further in Section 4.5.

3.3 Heat Treatment

Heat treatment can be applied to steel during or following the rolling process to alter mechanical properties. For a given chemical composition, the final microstructure of steel is greatly influenced by the heating and cooling history. Mechanical properties can be enhanced or degraded depending on how heat treatment is applied. The hardenability of steel is a property determined by the alloy composition that indicates the ability to increase hardness (and thereby tensile strength) through heat treatment. For structural steels in the A 709 specification, grades 36, 50, and 50S have relatively low hardenability. The weathering elements in grade 50W increase hardenability and it is possible to boost strength to 70 ksi through heat treatment. Grades 100W, HPS 70W, and HPS 100W rely on their hardenability and heat treatment to achieve their required strength properties.

Normalizing, quenching, and tempering are the conventional methods of heat treatment shown in Figure 1. These methods are performed in a furnace and are applied to steel products after rolling is completed. Controlled rolling and accelerated cooling are TMCP methods that incorporate heating and cooling directly during the rolling process.

3.3.1 Normalizing

Normalizing is a process where the plates are reheated after rolling to a temperature between 1,650°F and 1,700°F followed by slow cooling in air. This process refines grain size and improves uniformity of the microstructure, leading to improvements in ductility and toughness. Normalized plates tend to also have low variability of mechanical properties. Because normalizing requires reheating in a furnace, plate lengths are limited to the available furnace size at the mill, usually between 50 and 60 ft.

3.3.2 Quench and Tempering

The traditional method of hardening structural steel and boosting strength is quenching and tempering (Q&T). After rolling, the steel is reheated to about 1,650°F and held at this austenitizing temperature until the desired changes occur in the microstructure. The steel is then rapidly quenched by immersion in water to create a rapid cooling rate. Quenching results in steel with high hardness and strength, but the steel tends to be brittle and have low ductility. Therefore, quenching is usually followed by tempering, where the steel is reheated to between 800°F and 1,250°F, held at this temperature for a designated amount of time, and cooled under slower rate controlled conditions to obtain the desired properties. Tempering tends to reduce strength, but restores and enhances fracture toughness and ductility lost in the quenching operation. The net result of Q&T processing is a steel with elevated strength, good ductility, and good fracture toughness. The process variables for Q&T treatment are determined by the steel manufacturer and may be different for different mills and steel chemistries. Because Q&T processing requires plates to be uniformly heated in a furnace, plate lengths are limited by the furnace size (typically 50 to 60 ft.).

3.3.3 Controlled Rolling

This is a thermo-mechanical processing method that adds control of temperature and cooling rate during the rolling process. This is accomplished by introducing hold times into the rolling schedule to allow cooling to occur. The thickness reduction rate is varied depending on plate temperature during the rolling process. High reduction rates are applied when steels are over 1,800°F when the steel has higher workability. Final rolling is performed at lower temperatures between 1,500°F and 1,300°F. This can involve hold-periods during the rolling process to allow plates to cool before rolling is resumed. Controlled rolling can increase strength, refine grain size, improve fracture toughness, and may eliminate the need for normalizing. However, if plate temperatures are not uniform, controlled rolling can lead to property variability between different regions of the plate. Because high roll pressures are required for thick plates at low rolling temperatures, controlled rolling is usually limited to plates less than 2 in. thick.

3.3.4 Thermo-Mechanically Controlled Processing (TMCP)

TMCP is a more advanced process of controlled rolling that involves much more precise control of the plate temperature and reduction rates during the rolling operation. Modern TMCP facilities have the capability of accurately measuring plate temperature at multiple locations, applying localized heating, and performing accelerated cooling through water spray to precisely control the uniformity of temperature during the rolling process. The rate of accelerated cooling can be varied to provide a quenching and hardening effect to the steel as needed. TMCP processing can provide plates and shapes with a very refined and uniform grain structure leading to increases in strength, toughness, and ductility. In many cases, properties can be achieved with lower alloy chemistries helping to reduce cost. This may be offset, however, by the time delays and cost of the TMCP equipment. Currently, there are only a limited number of mills in the US that have TMCP capability for plates. Like controlled rolling, TMCP processing is usually limited to plates with 2 in. or less thickness. Because all heating and cooling occurs in the rolling operation, TMCP plates are not subject to the plate length limits of Q&T and normalized plates.

3.3.5 Stress Relieving

Welding, cold bending, normalizing, cutting, and machining can introduce internal residual stresses in steel products. Stress relieving involves heating to temperatures between 1,000 and 1,700°F, holding at that temperature for sufficient time to allow relaxation of stress, followed by very slow cooling. This process is not intended to alter microstructure or mechanical properties. Stress relieving is not usually required for structural plates and shapes in bridge applications. It may be indicated as an option to control distortions in welded fabrication or to prevent distortion of large parts due to hot-dip galvanizing.

3.3.6 Designer Concerns

The need for and specifics of heat treatment procedures should generally not be specified by the designer when ordering steel products. The need for heat treatment should be determined by the mill to meet the required mechanical properties and requirements of the applicable ASTM grade. Any products that rely on Q&T or TMCP to achieve mechanical properties will list the tempering temperature on the mill report. It is important to insure that this temperature is not exceeded during fabrication heating operations to avoid degradation of mechanical properties. This is addressed in the AASHTO/AWS D1.5 Bridge Welding Code. It is also important to consider the tempering temperature when evaluating heat treated steel products after exposure to fire. The post-fire residual strength and toughness may be significantly altered depending on the fire duration, temperature, and quenching effects of water application from emergency responders.

4.0 MECHANICAL PROPERTIES

4.1 Stress-Strain Behavior

The ASTM A 370 specification (10) defines requirements for application of the ASTM E 8 (11) tension testing procedures for determining the strength of steel products. The test method only requires determination of the yield strength, tensile strength, and percent elongation for each test. A complete engineering stress-strain curve can be measured by graphically or digitally recording the load and elongation of an extensometer during the duration of the test.

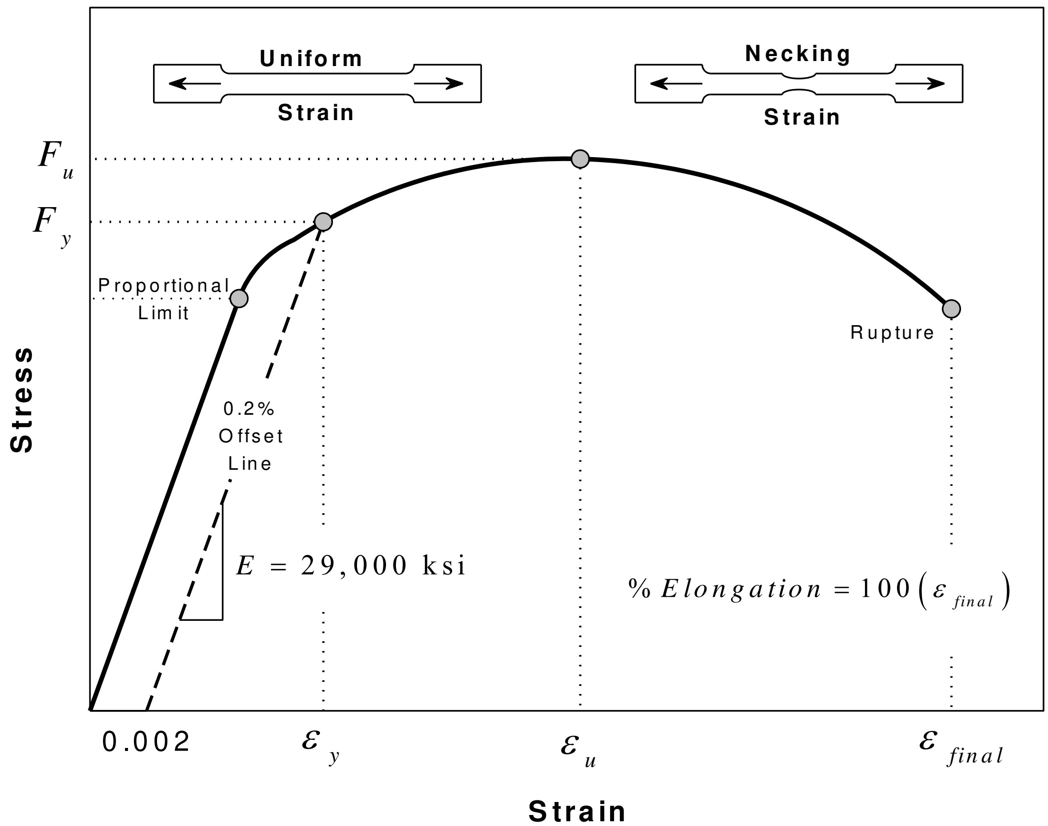


Figure 3 Engineering stress versus strain curve for structural steels without a defined yield plateau.

The elastic modulus or Young's modulus for steel is the slope of the elastic portion of the stress-strain curve as shown in Figure 3. It is conservatively taken as E = 29,000 ksi for structural calculations for all structural steels used in bridge construction. The ASTM E 8 tension testing procedures are usually not capable of producing accurate measurements of Young's modulus. Modulus values are extremely sensitive to the accuracy of the extensometer used in testing. The ASTM E 111 standard (12) provides special procedures for modulus measurement involving multiple, high accuracy extensometers to counteract bending effects and multiple load cycles with a data averaging procedure. Modulus measurement by less rigorous procedures can result in considerable error. Experimental studies have reported modulus values between 29,000 and

30,000 ksi, however much of this variability can be attributed to variations in experimental techniques, not material variability.

The yield strength is typically determined by the 0.2% offset method. A line is constructed parallel to the elastic portion of the stress-strain curve below the proportional limit with an x-axis offset of 0.2% (0.002) strain. The intersection of the offset line with the stress-strain curve defines the yield strength. Figure 4 shows the 0.2% offset method applied to steels that exhibit a yield plateau. It is typical for these steels to exhibit an upper yield point that is greater than the yield strength. When yielding first occurs, there is typically a slight drop in load before the steel plastically deforms along the yield plateau. The magnitude of the upper yield point is highly dependent on loading rate, therefore the upper yield point cannot be counted on for design purposes. The 0.2% offset method effectively excludes the upper yield point effect from yield strength determination.

Following first yield, steels with $F_y \leq 70$ ksi undergo plastic deformation at a relatively constant load level defining the yield plateau. The length of this plateau varies for different steels but $\varepsilon_{st} \approx 10\varepsilon_y$ is a typical value. There is typically some small load variation along the yield plateau and it may exhibit a slight upward or downward slope. This is typically approximated by a horizontal line for structural analysis that defines perfect elastic-plastic behavior.

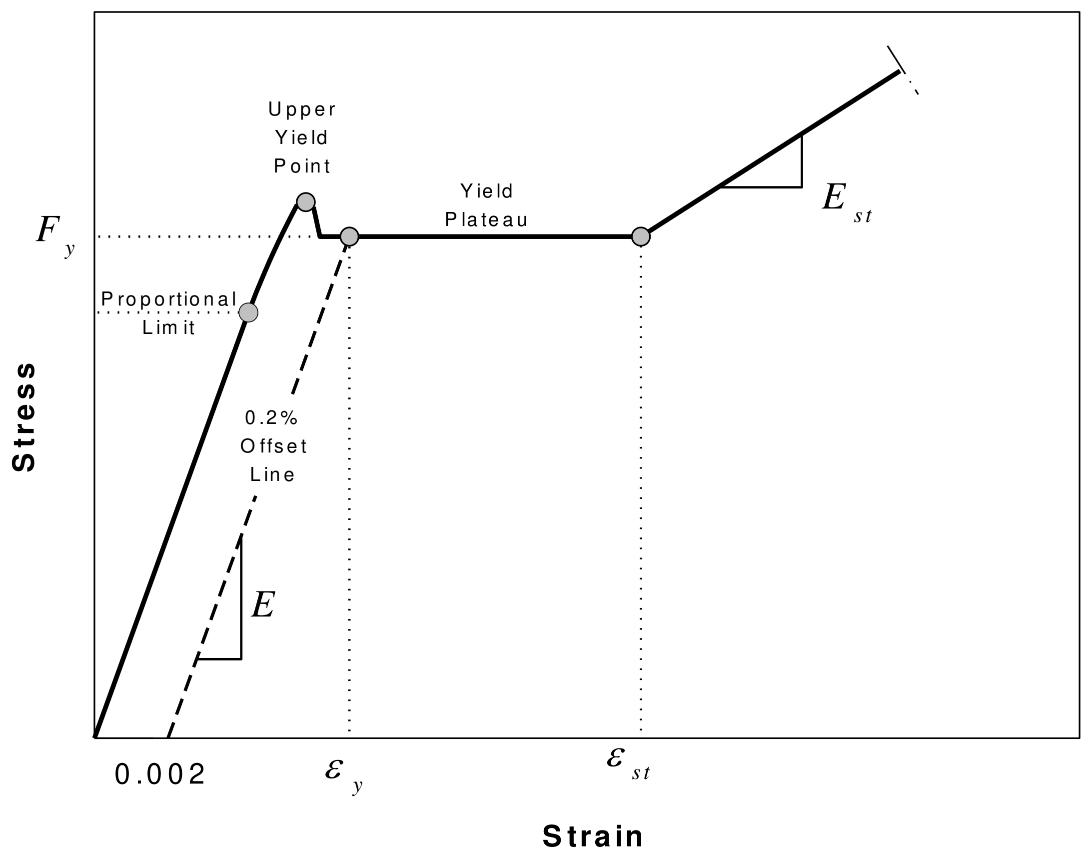


Figure 4 Calculation of parameters for steels with a yield plateau.

Strain hardening begins at the end of the plateau and continues until the maximum load is achieved corresponding to the tensile strength F_u. The slope of the stress-strain curve constantly varies during strain hardening. The tangent slope of the curve at the onset of strain hardening (E_{st}) is often used for analysis of steel behavior at high strain levels.

Tension test results are usually presented by engineering stress-strain curves where stress is calculated based on the un-deformed cross sectional area of the specimen. As the specimen is loaded, the cross sectional area is constantly being reduced by the Poisson contraction of the specimen. The true stress at any given point can be calculated with respect to the contracted area at that point in time. The area reduction can be directly measured during testing but it requires use of transverse extensometers, making it impractical except for research purposes. For some purposes, such as non-linear structural analysis, true stress-strain curves are required by the engineer. Lacking direct data, these can be calculated from the engineering stress strain curves by equations that approximate the Poisson contraction effect.

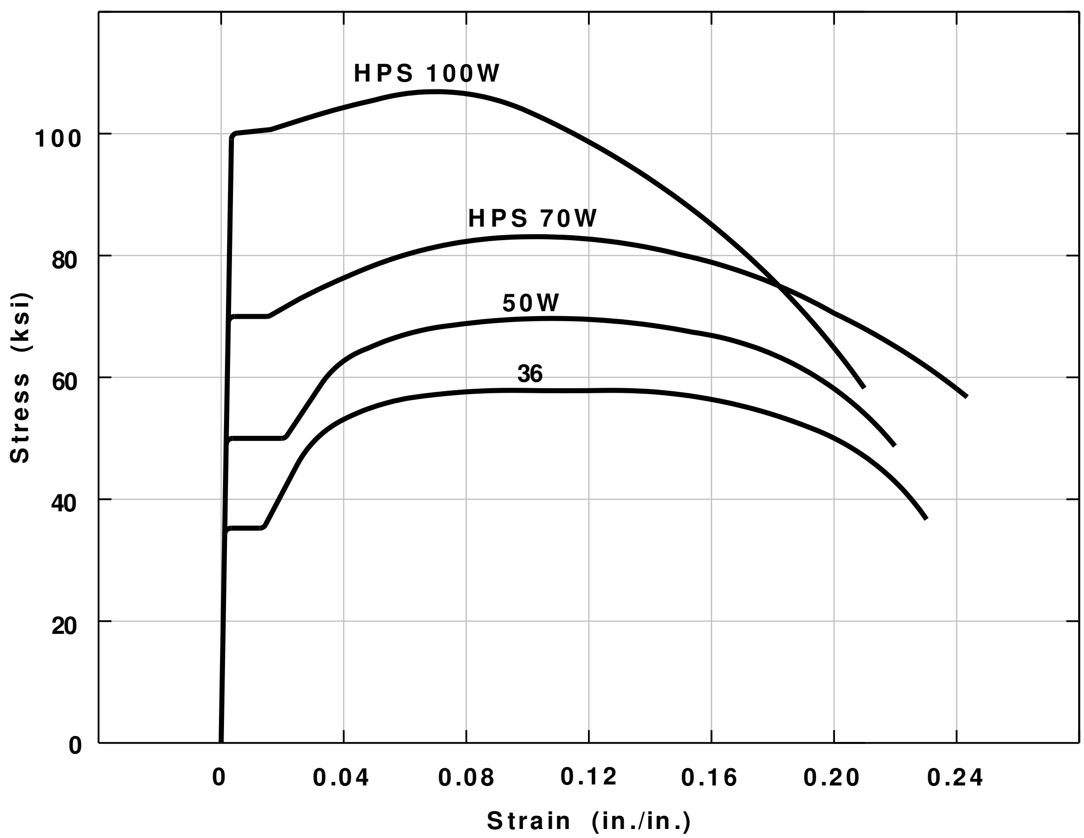


Figure 5 Typical engineering stress-strain curves for structural bridge steels.

Figure 5 shows typical stress-strain curves for steels in the A 709 Specification. Steels with $F_y \leq$ 70 ksi show definite yield plateaus with similar ductility. The HPS 100W steel does not have a clearly defined yield plateau and shows slightly lower ductility compared to the lower strength grades. The amount of strain hardening decreases with increasing yield strength.

4.2 Strength

The minimum specified yield strength (F_y) and tensile strength (F_u) is shown in Table 5 for steel grades included in the A 709 specification. Plates with thickness up to 4 in. are available in all grades (except 50S). Rolled shapes are not available in the HPS grades.

Table 5 Nominal strength of A 709 steel grades

Grade	36	50	50S	50W	HPS 50W	HPS 70W	HPS 100W	
Plate Thickness (in)	$t \leq 4.0$	$t \leq 4.0$	N/A	$t \leq 4.0$	$t \leq 4.0$	$t \leq 4.0$	$t \leq 2.5$	$2.5 < t$ $t \leq 4.0$
Shapes	All Groups	All Groups	All Groups	All Groups	N/A	N/A	N/A	N/A
F_u (ksi)	58	65	65	70	70	85	110	100
F_y (ksi)	36	50	50	50	50	70	100	90

4.3 Shear Strength

The Von Mises yield criterion is usually used to predict the onset of yielding in steel subject to multi-axial states of stress:

$$F_y = \sqrt{\frac{\left(\sigma_x - \sigma_y\right)^2 + \left(\sigma_y - \sigma_z\right)^2 + \left(\sigma_z - \sigma_x\right)^2 + 6\left(\tau_{xy}{}^2 + \tau_{yz}{}^2 + \tau_{zx}{}^2\right)}{2}}$$

For the state of pure shear in one direction, five of the six stress components reduce to zero and the shear yield strength (F_{yv}) is defined as:

$$F_{yv} = \frac{1}{\sqrt{3}} F_y \approx 0.58 F_y$$

The shear modulus (G) based on Young's modulus (E) and Poisson's ratio (ν) is given as:

$$G = \frac{E}{2(1+\upsilon)} = 11{,}200 \text{ ksi}$$

4.4 Effect of Strain Rate and Temperature

Steels loaded at higher strain rates have elevated stress-strain curves. Yielding is a time dependent process. At higher loading rates the yielding slip planes do not have sufficient time to develop and there is an apparent elevation in strength. Madison and Irwin (13) recommended the following equation for estimating the dynamic yield strength as an alternative to direct measurement:

$$\sigma_{YD} = \left[\sigma_{YS} + \frac{174,000}{\log\left(2 \times 10^{10} t\right)\left(T + 459\right)} - 27.4 \right]$$

Where σ_{YD} is the yield strength at a given rate and temperature (ksi), σ_{YS} is the room temperature 0.2% offset yield strength at static load rate (ksi), t is the load rise time from start of loading to maximum load (sec.), and T is the temperature (˚F). According to the ASTM E 399 Standard, this equation is useful only for steels with $\sigma_{YS} \leq 70$ ksi for evaluation of fracture resistance.

Structural steel strength also varies as a function of temperature. At low temperatures, the yield and tensile strengths both increase. The above equation can be used to predict the yield strength increase in steels below room temperature. The increase in yield strength at low temperatures and high strain rates can be either beneficial or detrimental to structural performance depending on fracture toughness. If toughness is sufficient to prevent fracture, the strength increase can provide increased reserve capacity to prevent yielding under momentary dynamic overloads. However, the fact that stresses can reach higher values before yield decreases the resistance to brittle fracture. For practical bridge loading rates and temperatures, the effects of any yield strength elevation can be conservatively ignored by designers.

ASTM A 370 specifies that the loading rate of tension test specimens must be between 10 and 100 ksi/min until the specimens have yielded. After yield, the strain rate must be maintained between 0.05 and 0.5 in/in/min. The resulting measured yield strength is typically a few percent higher at the upper bound loading rate versus the lower bound. The difference can be even greater between the upper bound ASTM rate and quasi-static tests. The load rate effect must be considered when comparing test results reported in mill reports, that are presumably performed close to the ASTM upper bound loading rate, to supplemental product tests.

Structural steels undergo a dramatic decrease in strength at high temperatures, such as during fires or other extreme heating events. Both the yield strength and tensile strength start to significantly decrease when temperatures exceed about 400°F. This loss of strength reduces the factor of safety for structures at high temperatures and can cause yielding and permanent deflections in structures under load. Young's modulus also decreases at higher temperatures leading to an increase in elastic deflections. Additionally, creep can also occur at high temperatures leading to a time dependent increase in deflections. More information on high temperature structural properties can be found in publications by the ASCE and in the Eurocode (14 and 15).

In general, structural steels can be expected to have about a 50% reduction in yield strength at temperatures of 1,100°F. There is also a corresponding reduction in tensile strength and about a 30% reduction in Young's Modulus. Bridge structures exposed to temperatures exceeding about 1,100°F can be expected to experience possible large deformations or possible collapse as shown in Figure 6.

Figure 6 Fuel truck crash causes severe fire under the I-65 South over I-65 North overpass in Birmingham, Alabama, January 2002.

Post fire evaluation of damage involves assessment of the residual strength of structural steel after cooling. Work is underway in project NCHRP 12-85 to develop methodologies for post-fire inspection of bridge structures. For most fire heating situations, carbon and HSLA steels retain most of their original strength properties after cooling. Special consideration is required for heat treated steels when the fire temperature exceeds the heat treatment temperatures.

4.5 Lamellar Tearing

Lamellar tearing is a possible failure mode when steel plates are loaded in the transverse, through thickness direction. Steel is generally considered to be an isotropic material with identical properties with respect to all directions of loading. However, as shown in Figure 2, plates and shapes can sometimes have planar inclusions along the centerline as a byproduct of steel making practices. This does not present a problem for mechanically fastened or welded plates loaded in plane. The introduction of welded construction makes it possible to load plates in the transverse direction as shown in Figure 7. If significant inclusions are present they create a plane of weakness that can cause the plates to fail along the lamellar plane. Thicker plates are more susceptible to this phenomenon compared to thinner plates. Fortunately, modern steel making practices have greatly reduced mid-plane segregation for grades 36, 50, and 50S compared to older vintage steels. Grade 50W weathering steel has shown some increased propensity for segregation due to the additions of copper and other alloying elements added for corrosion resistance. The lamellar tearing strength of the new HPS grades has not been specifically investigated but no special problems are anticipated since the tightly controlled alloy content and processing promotes through thickness uniformity. The use of low sulfur with calcium treatment for inclusion shape control, can be a benefit as the low sulfur and low inclusion contents have been found to improve the toughness, ductility, and fatigue properties of steel (16).

Lamellar tearing is generally not a concern for steel bridges since most plates are loaded in the planar direction. Most lamellar tearing problems have occurred during fabrication of highly constrained connections with thick plates due to weld shrinkage. Problems have also occurred in welded beam-column moment connections in building structures exposed to high forces and strains during seismic events. These high constraint, high through-thickness loading conditions rarely occur in bridge structures but the possibility of lamellar tearing should be considered when designing certain non-typical connections.

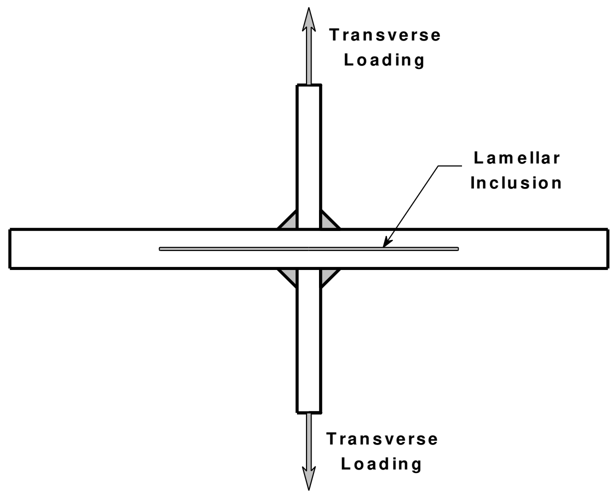


Figure 7 Lamellar tearing potential in a plate loaded in the through-thickness direction.

Lamellar tearing resistance is not addressed in the A 6 and A 709 specifications for bridge steels. The reduction in area (necking) that occurs in a round tension test specimen can provide some measure of lamellar tearing resistance. If a designer has special concerns for steel to be used in highly constrained connections, this should be discussed with the fabricator and steel producer.

4.6 Hardness

Hardness is the property of steel to resist indentation in the presence of a localized concentrated force. There are a number of different hardness testing methods, including the Brinell, Vickers, and Rockwell methods. The most accurate methods employ a laboratory testing apparatus but portable techniques have been developed for measuring hardness on large components. In general, all of the methods involve pressing an indenter ball or pin into the material surface under a known force and measuring the resulting indentation. Hardness is not a directly useful property for structural engineers, but hardness can be used as an indirect measure to help approximate the tensile strength, ductility, and wear resistance of steels. Higher hardness generally indicates higher tensile strength and reduced ductility. Hardness is often used as a measure of the strength increase following heat treatments.

Hardness is too inaccurate to use as a quality control test for steel mechanical properties. It is most commonly used to assess the heat treated condition of high strength steels when the heating history is not precisely known. For example, Grade 100 (A 514) steel has relatively high hardenability and the tensile strength can rise as high as 180 ksi if heating is followed by rapid quenching. Tempering is required to reduce the tensile strength back to the specification limits and restore ductility to the steel. In an un-tempered condition, A 514 steel can be vulnerable to stress corrosion cracking and fatigue. Hardness testing can therefore be useful as a screening tool to estimate the properties of steels that have been exposed to different heating conditions in service or in fabrication. Examples in fabrication include evaluation of thermally cut edges, weld heat affected zones, and plates that have been heat curved. Hardness testing is commonly used to assess the residual properties of structural steel that has been exposed to fire. Hardness measurement is also useful to assess the heat treated condition of high strength fasteners.

4.7 Ductility

Ductility is a required mechanical property that is not directly used in structural steel design. However, it is an important property to assure that steel members and connections can perform as required in structural systems. For steel products, relative ductility is measured as the percent elongation that occurs before rupture in a standard tension test. This is an indicator of the maximum strain capacity of steel members without holes, notches, or other stress concentrating effects. The percent elongation is somewhat dependent on the test specimen geometry and the gage length used to measure elongation during testing. For the same material, tension specimens with a 2 in. gage length will exhibit a lower percent elongation compared to those with an 8 in gage length. From a designers perspective, the ASTM A 709 specification assures that structural steel for bridges has an adequate level of material ductility to perform well in structural applications.

Material ductility does not automatically translate into structural ductility. The designer makes many decisions about connections, section transitions, and bracing that can make steel members fail in a relatively brittle mode relative to the overall structure. Any time a hole or other notch is placed in a structural member it creates a reduced net section where localized yielding is expected to occur first under increasing loads. Without strain hardening, the localized material at the net section will yield and reach the rupture strain before the gross section of the member yields. Since the only plastic strain occurs at the localized net section, the overall elongation of the structural member is very small at rupture and the member fails in a brittle manner from a structural perspective. To provide structural ductility, the steel must have sufficient strain hardening capability to increase the local net section strength sufficiently to allow the gross section to reach yield before rupture occurs at the net section. The most significant parameter to insure structural ductility is the yield-tensile ratio (YT ratio) defined as: $YT = F_y / F_u$.

Considerable research has been performed to determine what YT ratio is required for structural steel (17). In general, the rotational capacity of flexural members decreases with increasing YT ratios. Similarly, higher YT ratios tend to increase the likelihood of the rupture limit state controlling bolted connection behavior. In general, Brockenbrough concludes that the strength equations in AASHTO are valid to predict behavior for steels with $YT \leq 0.93$. He also reports that steels with $YT \approx 1.0$ have been used successfully for some structural applications. Since there is no clear consensus, there are no requirements for YT ratio in the A 709 specification. A recent study shows that Grade 50 and 50W structural plates produced in North American mills have YT rations varying between 0.63 and 0.81 (18). At higher strengths, the YT ratio typically increases, approaching $YT \approx 0.93$ for grade HPS 100W. The current AASHTO design codes do not allow use of an inelastic strength basis for steel with F_y > 70 ksi. For steels specified in the A 709 specification, there is no need for special consideration of the YT ratio for most bridge structural applications. Steels not covered by A 709 should be appropriately evaluated by the engineer for their intended use. In addition, there may be special applications where limits may be required on the YT ratio. As an example, steel can be ordered under the A 992 specification with a supplemental provision limiting YT ≤ 0.80 for seismic applications where enhanced structural ductility is required.

4.8 Fracture Toughness

Steels for use in primary bridge members are required to have sufficient fracture toughness to reduce the probability of brittle failure in the presence of a fatigue crack or other notch-like defect. AASHTO introduced a fracture control plan in 1978 (19) in the aftermath of the Silver Bridge collapse in 1967 due to brittle fracture. All primary bridge members are now required to have a specified minimum level of fracture toughness. Primary members are divided into two classifications; fracture critical and non-fracture critical. Fracture critical members are defined as tension members or portions of members whose failure may be expected to cause collapse of the structure. These members are required to have higher levels of fracture toughness compared to non-fracture critical members. Secondary members are not required to have any specified fracture toughness. Examples of secondary members are bearing plates, utility conduit hangers, and other members that are not part of the main structural system.

Linear-elastic fracture mechanics (LEFM) analysis is the basis for predicting brittle fracture in structural steels. Conventional stress analysis cannot be applied to crack-like defects since the theoretical stress concentration factor is infinite. This led to development of the stress intensity factor (K_I) as a means to characterize the crack tip singularity. For a given plate geometry, the stress intensity present at a crack tip is a function of the crack size and the applied stress. The basic functional relationship is $K_I = \sigma(\pi a)^{0.5}$, however modifiers must be added to account for plate geometry, the crack shape, and residual stress state before this can be practically applied to engineering problems. The material fracture resistance is characterized by the critical stress intensity factor (K_{Ic}) that can be sustained without fracture. When the applied stress intensity K_I equals or exceeds the material fracture resistance K_{Ic}, fracture is predicted. This relationship is schematically shown in Figure 8. For a given material toughness, a fracture prediction curve can be constructed to represent the possible combinations of stress and crack size that are expected to cause fracture. Fracture is predicted for any combination of stress and crack size that plots above the curve.

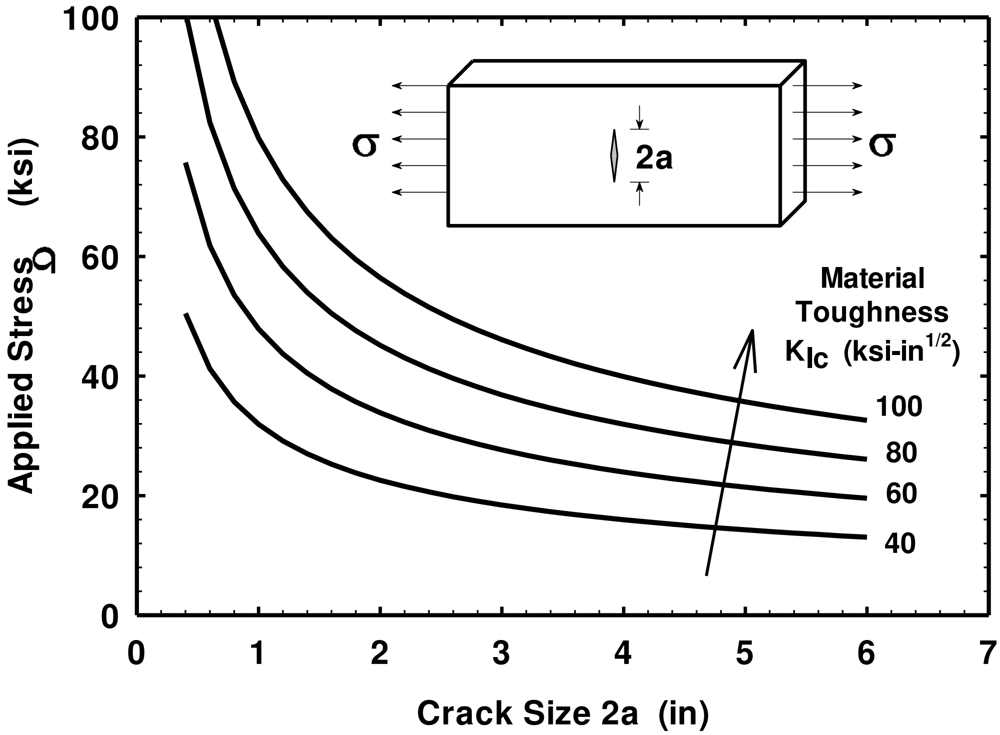


Figure 8 Basic relationship between applied stress, crack size, and material fracture toughness based on LEFM. Increasing material toughness raises the fracture prediction curve.

Similar to yield strength, the fracture toughness of steel is dependent on the temperature and loading rate. However, the relationship is quite different. Figure 9 shows that the basic relationship can be defined by a sigmoid curve. At high and low temperatures the fracture toughness can be characterized by the relatively constant "upper shelf" and "lower shelf" toughness levels. The metallurgical fracture mode transitions from brittle cleavage on the lower shelf to ductile tearing on the upper shelf. Mixed mode fracture is expected in the transition region.

The Charpy V-Notch (CVN) test is commonly utilized to measure the fracture toughness for structural steel (20) A small 10 x 10 mm bending specimen with a machined notch is impacted by a hammer and the energy required to initiate fracture is measured. This provides a relative measure of toughness but it cannot be directly used to predict the K_{Ic} fracture toughness. The solid curve in Figure 9 represents the CVN transition curve developed from testing multiple CVN specimens at different temperatures. The CVN test is performed at dynamic "impact" loading rates that are much higher than the loading rate experienced by bridges due to live load.

The CVN test cannot directly predict the K_{Ic} fracture toughness of steel. More elaborate fracture mechanics tests are required using fatigue cracked specimens with measurement of the load and displacement during testing (21 and 22). These tests are too expensive to use for quality control in steel production. However, correlations have been developed to predict the K_{Ic} fracture toughness from CVN test data.

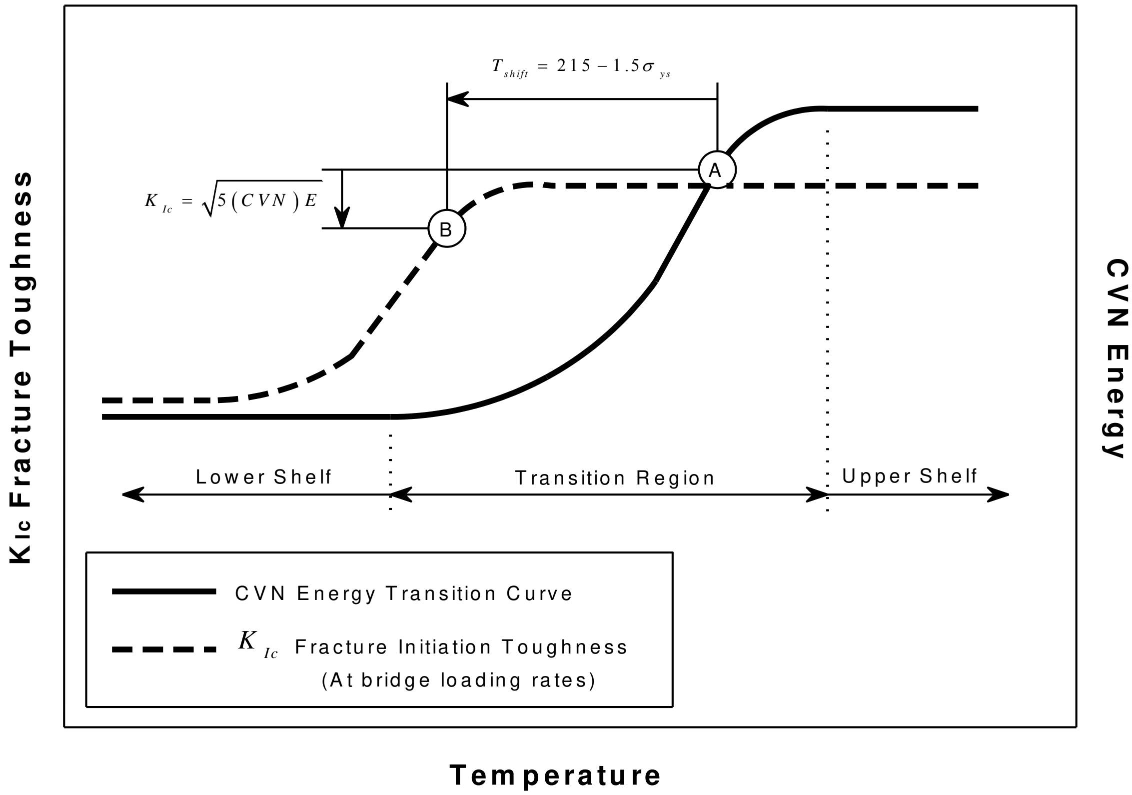


Figure 9 Effect of temperature and loading rate on the fracture toughness of structural steels.

Barsom and Rolfe developed a two-step procedure to calculate the K_{Ic} toughness from CVN data (23). The first step is to calculate the dynamic toughness K_{Id} using the following equation to scale the CVN data.

$$K_{Id} = \sqrt{5(CVN)E}$$

The second step is to calculate a temperature shift between the static and impact transition curves:

$$T_{shift} = 215 - 1.5\sigma_{YS}$$

K_{Ic} is equal to K_{Id} at the shifted temperature. Both of the above equations are unit sensitive, K_{Id} is in psi-in$^{1/2}$, E is in psi, CVN is in ft-lb, and T is in °F.

The dashed line in Figure 9 represents the K_{Ic} fracture initiation toughness as a function of temperature under the intermediate (1 sec.) loading rate typically caused by live load on bridges. The figure illustrates how point B on the K_{Ic} curve can be calculated from point A on the CVN curve using the two step correlation procedure. Although the two curves are shown including the upper shelf behavior, the two-step correlation procedure is only valid for lower shelf and transition behavior. The K_{Ic} toughness at the temperature of interest can be used as shown in Figure 8 to predict when fracture will initiate from a structural flaw.

The CVN testing requirements in the AASHTO Bridge Design Specifications were originally derived using the Barsom & Rolfe correlation procedure in the original AASHTO Fracture Control Plan (23 and 24). The requirements for non-fracture critical members were set to keep the K_{Ic} fracture toughness above the lower shelf at bridge service temperatures. The requirements for fracture critical members were set higher in the transition region to provide added resistance to brittle fracture. The use of the temperature shift concept results in CVN test temperatures that are higher than the actual service temperatures in bridges.

The AASHTO LRFD Bridge Design Specifications divide the U.S. into three temperature zones for specifying fracture toughness of bridge steels. The zones are delineated by the lowest anticipated service temperature as shown in Table 6.

Table 6 AASHTO temperature zones for specifying CVN toughness.

Lowest Anticipated Service Temperature	Temperature Zone
0°F and above	1
-1°F to -30°F	2
-31°F to -60°F	3

The CVN toughness requirements for bridge steels were originally set forth in the AASHTO Fracture Control Plan (19). This document has been discontinued and the CVN toughness requirements for bridge steels are now maintained in the AASHTO LRFD Bridge Design Specifications. The welding and fabrication quality control provisions of the Fracture Control Plan can now be found in the AASHTO/AWS D1.5 Bridge Welding Code.

Table 7 shows the CVN toughness requirements for bridge steels from the 2010 5th Edition of the AASHTO LRFD Bridge Design Specifications. These requirements are subject to periodic change and the current specification edition should always be consulted before using these values. There are two basic categories of primary bridge members, fracture critical and non-fracture critical. Fracture critical members are defined as members whose failure may be reasonably expected to cause collapse of the bridge. Members and portions of members that are deemed to be fracture critical are required to be designated by the engineer on the design drawings. The bridge fabricator is then required to purchase plate that meets the applicable requirements shown in Table 7.

Experience has shown that thick plates are more vulnerable to brittle fracture, hence the CVN toughness requirements are increased for thicker plates for some steel grades. Prior to 2010 different CVN toughness requirements were specified for mechanically fastened versus welded members. This distinction is no longer required by the specification. Note that the CVN test temperatures do not correspond with the lowest anticipated service temperatures shown in Table 6. This difference generally reflects the temperature shift defined in Figure 9, although adjustments have been made based on experience. Another feature of the requirements is that higher CVN toughness is specified for higher strength steels. The permissible design stress and possible residual stress in higher strength steel members will both increase in relative proportion to the yield strength. Therefore, referring back to Figure 8, higher K_{Ic} material toughness is required at higher stress levels to maintain the same critical crack size tolerance in structural members. We currently have not established a crack size that must be tolerated in bridge members without risk of fracture. However, research is currently underway to establish a link between tolerable crack size and inspectability.

Table 7 AASHTO Table 6.6.2-2 fracture toughness requirements for bridge steels (2010).

GRADE (Y.P./Y.S.)	THICKNESS (in)	FRACTURE CRITICAL				NON-FRACTURE CRITICAL		
		MIN. TEST VALUE ENERGY (ft-lb)	ZONE 1 (ft-lb @ °F)	ZONE 2 (ft-lb @ °F)	ZONE 3 (ft-lb @ °F)	ZONE 1 (ft-lb @ °F)	ZONE 2 (ft-lb @ °F)	ZONE 3 (ft-lb @ °F)
36	t ≤ 4	20	25 @ 70	25 @ 40	25 @ 10	15 @ 70	15 @ 40	15 @ 10
50/50S/50W	t ≤ 2	20	25 @ 70	25 @ 40	25 @ 10	15 @ 70	15 @ 40	15 @ 10
	2 ≤ t ≤ 4	24	30 @ 70	30 @ 40	30 @ 10	20 @ 70	20 @ 40	20 @ 10
HPS 50W	t ≤ 4	24	30 @ 10	30 @ 10	30 @ 10	20 @ 10	20 @ 10	20 @ 10
HPS 70W	t ≤ 4	28	35 @ -10	35 @ -10	35 @ -10	25 @ -10	25 @ -10	25 @ -10
HPS 100W	t ≤ 2-1/2	28	35 @ -30	35 @ -30	35 @ -30	25 @ -30	25 @ -30	25 @ -30
	2-1/2 ≤ t ≤ 4	36	Not permitted	Not permitted	Not permitted	35 @ -30	35 @ -30	35 @ -30

The new HPS steels have inherently higher toughness compared to the traditional non-HPS grades. Table 7 shows that the CVN toughness values for the HPS grades are identical for all three temperature zones. At a minimum, all HPS grades meet the requirements for use in zone 3. The actual toughness of HPS typically exceeds the specification requirements by a large margin. Research is currently underway to determine how higher toughness can reduce the fracture vulnerability of bridges.

4.9 Fatigue Resistance

For bridge structures, all structural steel grades are considered to have equivalent fatigue resistance corresponding to AASHTO Category A. This category is set for smooth base metal without any geometric stress concentrations from notches, welds, or holes. Fatigue specifications in other industries recognize that different steel grades have slightly different base metal fatigue resistance. This has no practical significance for bridge structures where almost all members are governed by fatigue Categories B through E'. Fatigue data generated on bridge members shows that there is no significant difference in fatigue resistance between grade 36 and grade 100 steels (25 and 26). The stress concentration effects created by welded and bolted details overshadow any small differences in the base metal fatigue resistance. Therefore, fatigue design is governed by the allowable stress range for the applicable fatigue category, irrespective of steel grade. All bridge steel grades are therefore considered to have equivalent fatigue resistance.

4.10 Strength Property Variability

Like any material, steel properties are not always uniform at all locations within a steel plate, nor are they always uniform between different plates. The AASHTO design specifications are based on the nominal yield and tensile strength "minimum" requirements. Most steel products are delivered with strength that exceeds the nominal minimums since steel makers target higher strengths in production to account for variability. Data from six different North American mills have been collected for over 3,000 tests on Grade 50 and Grade 50W plates with varying thickness (18). Results show the measured yield strength averaged about 58 ksi.

The variability of properties measured at different locations within the same plate has been statistically evaluated by ASTM Subcommittee A01.02 (9). Based on the data, one standard deviation from the mean corresponds to about 4% variation in tensile strength, about 8% variation in yield strength, and about 3% variation in the percent elongation. Based on this variability and the fact that the measured strength typically exceeds the nominal specification value, there is a slight possibility that testing at some plate locations will produce results below the nominal strength. This fact should be considered if supplemental product testing is performed on a given steel plate in addition to the mill certification report. The ASTM A 6 specification allows for a retest if any tensile test falls slightly below the nominal specification value (1 ksi below F_y, 2 ksi below F_u). Plate variability is an inherent consequence of steel manufacturing and it has been considered when calibrating the resistance factors in the LRFD Bridge Design Specifications.

4.11 Residual Stresses

The processes of rolling steel products naturally introduce internal residual stresses due to plastic deformation and differential cooling effects during their production. The resulting residual stress distribution has both tensile(+) and compressive(-) stresses that are always in static equilibrium. Figure 10 shows some typical residual stress distributions for plates and rolled W sections. Welding, flame cutting, and hole drilling will alter the residual stress pattern for fabricated members. Figure 10 also shows typical residual stress distributions for built-up boxes and I-sections. Determining the exact distribution and magnitude of residual stress in fabricated members is a very complicated subject that depends on the shape geometry, processing, and the sequence of fabrication operations. It is possible to measure residual stresses through destructive sectioning and hole drilling techniques and through non-destructive X-ray diffraction and neutron diffraction techniques. However, these techniques are impractical except in a research environment.

One consequence of residual stress is to induce distortion during fabrication. The plate flatness, twist, and straightness of steel products is influenced and must be compatible with the internal residual stress distribution. Fabrication operations that alter the residual stress pattern will also alter the shape of the steel members. Experienced fabricators have learned to compensate for distortional changes in many cases to insure the proper tolerances are met for fabricated steel members. Steel producers often subject plates to leveling and straightening operations to meet the required dimensional tolerances as specified in ASTM A 6. In some cases, fabricators must straighten fabricated members after welding using heat straightening and mechanical bending techniques to meet the required tolerances. The use of such techniques should be subject to agreement between the bridge owners and fabricators. From the designer's perspective, residual stresses do not need to be known when calculating the strength of bridge members. There has been extensive research studying the effect of residual stress on strength, particularly for compression members. The buckling equations in the AASHTO codes for flexural and compression members all consider residual stresses in their formulation. Likewise, residual stresses are inherently imbedded in the data used to establish the S-N curves used to define fatigue resistance. There are, however, some situations where designers require knowledge of residual stresses to evaluate localized issues and details.

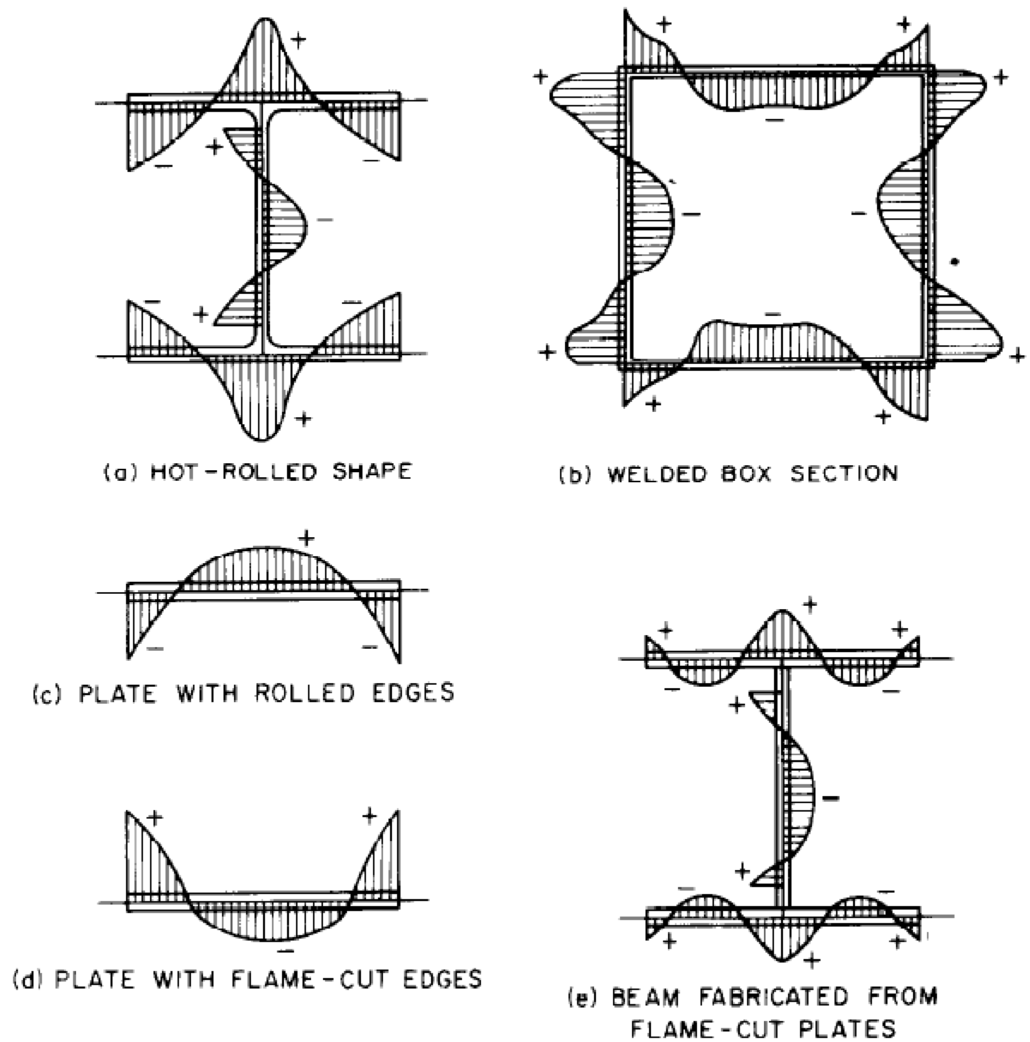


Figure 10 Typical residual stress distributions in rolled shapes, plates, and built-up members.

4.12 Plastic Deformation and Strain Aging

The mechanical properties of steel change when the material is subjected to high levels of plastic strain. Normally bridge structures are designed to prevent large inelastic deformation of material under the strength and service loading conditions. However, it is possible that some members will experience large plastic strain under extreme event loading. It is also possible that high plastic strains can be introduced through cold bending in fabrication. The residual properties of steel that has experienced plastic deformation will be somewhat different compared to elastic material. When the maximum strain is below the strain where strain hardening begins (ε_{st}) there will be a reduction in ductility (percent elongation) under future loadings. If the maximum strain exceeds ε_{st}, the steel will have a residual increase in both the yield and ultimate strength under future loadings. There will also be a greater decrease in ductility. The strength increase is time dependent and may take a period of several months to completely stabilize.

The effects of plastic strain are conceptually illustrated in Figure 11. A steel loaded to failure will follow the path along the original stress-strain curve, A1-B-C-D-E and the failure strain is A5 - A1. Now consider what happens if the material is loaded to point F on the yield plateau and unloaded following path A1-B-F-A2. The steel will have permanent plastic strain, A2 - A1. In addition, the stress-strain curve for future loadings to failure will be altered, following path A2-F-C-D-E and the failure strain will be reduced to A5 - A2. The material will have the same yield strength with a reduced length yield plateau, the same tensile strength, and slightly reduced ductility.

Now consider what happens when the material is loaded to produce strain beyond the onset of strain hardening and unloaded following path A1-B-C-D-A3. If the material is immediately reloaded to failure, it will follow path A3-D-E and the failure strain is A5 - A3. The material returns to the original stress-strain curve and continues on the original path to failure at point E. However, if there is a delay before reloading (months), the material strain ages and reloading to failure will follow path A3-G-H-I. Strain aging permanently changes the material properties resulting in an elevation of both the yield and tensile strength along with restoration of a yield plateau on the stress-strain curve. However, failure strain will be reduced to A4 - A3 indicating a notable loss of ductility.

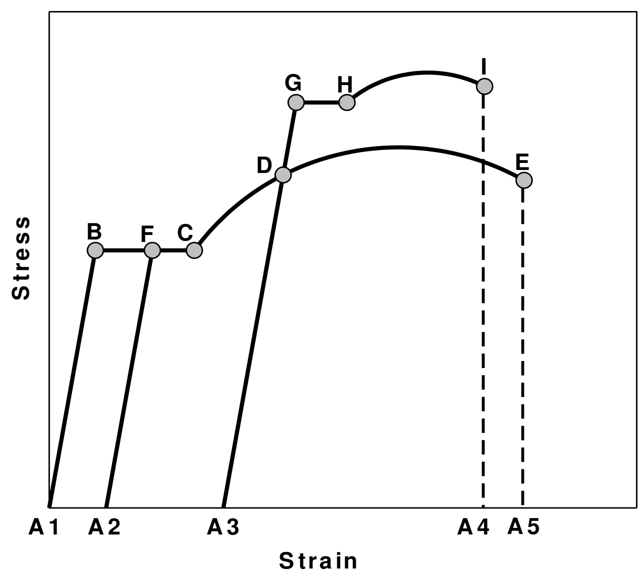


Figure 11 Stress-strain behavior showing the effects of strain hardening and strain aging.

From a strength perspective, strain aging is beneficial and increases the elastic capacity of the member to resist future loadings. However, the material ductility is greatly reduced compared to the original material. If needed, the original properties can be restored by applying a heat treatment to the steel.

4.13 Testing Requirements

Steel plates and shapes used for bridges are required to have Mill Certificates documenting the test procedures performed according to the ASTM A 6 specification (9). The default testing requirements are performed on the steel heat (H frequency) and one set of test results is used to qualify all plates produced from the heat. Some applications require additional testing to be performed on each plate (P frequency) invoking supplemental requirement S4. The need for CVN testing is covered in supplemental requirement S5. At a minimum, the mill certificates are required to report the following information:

- Specification Designation
- Heat Number
- Chemical Analysis (chemical composition of the heat)
- Nominal Plate Sizes
- Tension Test Results (F_y, F_u, and percent elongation, including gage length)
- Heat Treatments (Including final tempering temperature if applicable)
- Supplementary Testing Requirements (Most commonly CVN)

4.13.1 Tension Testing

Tension testing procedures are proscribed in the ASTM A 370 specification. For H-frequency sampling, two tension tests are required to characterize all plates or shapes within the heat. For plates wider than 24 in., the test coupons are oriented so the longitudinal axis of the test specimens are transverse to the primary rolling direction of the plate. The sampling location is selected at one corner of the plate. One test is performed on the thickest plate, and one on the thinnest plate produced from the heat. For shapes, the axis of the test specimens is parallel to the longitudinal axis of the shape. The sample location for W and HP shapes is in the flanges, 2/3 of the distance between the web and flange tip. The sample location for other shapes is taken from the web, or from one of the legs for angles, as applicable. As previously discussed in the section on property variability, there is a small chance that a given tension test result will fall below the nominal specification requirements. Recognizing this, the A 6 specification allows one re-test from a different location as long as the failed test is within 1 ksi of the nominal yield strength, 2 ksi of the nominal tensile strength, or 2% of the required percent elongation.

Heat treated steel grades in the A 709 specification are required to have an individual tension test performed on each plate (P-frequency). This recognizes that the final properties are dependent on the specific heat treatments applied to each plate. Grades requiring P-frequency testing are HPS 70W and HPS 100W, and heat treated versions of HPS 50W.

4.13.2 Charpy V-Notch Testing

Tension members in bridges are required to meet the CVN testing requirements shown in Table 7. All primary components subject to tension under the Strength I load combination are by default classified as non-fracture critical and CVN testing is required. Certain critical members must be further designated as fracture critical (FCM) and are subject to higher CVN testing requirements. Fracture critical members are generally defined as members whose failure may reasonably be expected to cause collapse of the structure. It is the responsibility of the design engineer to designate which members are fracture critical (FCM) on the design drawings. At present, the decision to designate FCM is left up to the judgment of the engineer and bridge owner. Research is underway to develop improved guidance on how to analyze bridges to make this decision.

Once members are designated as either non-fracture critical (T) or fracture critical (F), steel is required to be ordered with supplemental provision S5. The member classification (T or F) followed by the temperature zone (1, 2, or 3) must be designated to invoke the proper requirements from Table 7. For example, a grade 50 non-fracture critical plate for use in temperature zone 2 is designated as A 709 Grade 50-T2. A HPS 70W fracture critical plate for use in temperature zone 3 is designated as A 709 Grade HPS 70W-F3.

The ASTM A 673 specification governs the CVN sampling and testing requirements (27). Similar to the tension test sampling requirements, CVN testing is required to be performed at either H or P frequency depending on the grade and application. In addition, P frequency sampling is required at two locations (each end) in some plates depending on grade and heat treatment. This requirement is added for grade and heat treatment combinations that were determined to be subject to property variability at different locations (28). The sampling frequency requirements that are specified in section 6.6.2 of the LRFD Bridge Design Specifications are summarized in Table 8.

Table 8 Required CVN sampling frequency for fracture critical and non-fracture critical steel members.

Steel Grades	Sampling Frequency	Sampling Locations Per Plate		
		As-Rolled	Normalized	Q&T or TMCP
NON-FRACTURE CRITICAL				
36, 50, 50S, HPS 50W	H	N/A	N/A	N/A
HPS 70W, HPS 100W	P	N/A	N/A	One End
FRACTURE CRITICAL				
50S	P	One End	One End	N/A
36, 50, 50W, HPS 50W	P	Both Ends	One End	N/A
HPS 70W, HPS 100W	P	N/A	N/A	Both Ends

In the A 673 specification, a CVN impact test is defined as testing three replicate CVN specimens from the same location at the same testing temperature. The average of the three specimens must be greater than the specified minimum requirements in Table 7. In addition, there are limits placed on how much an individual specimen can fall below the specified minimum. This prevents acceptance of plates that have large variability between individual CVN tests. The specimen orientation and sample location requirements are also specified in A 673. Unless otherwise specified, specimens are taken with LT orientation, meaning the longitudinal axis of the specimen is parallel to the rolling direction and the notch is transverse to the rolling direction. Since bridge plates are generally loaded with tension parallel to the rolling direction, this places the notch perpendicular to the expected tension stress field. The engineer may decide that TL orientation is more appropriate for some applications. The through-thickness location of the centerline of the 10 mm x 10 mm specimens is located at the 1/4 thickness for thicker plates.

5.0 WELDABILITY AND FABRICATION

5.1 AWS D1-5

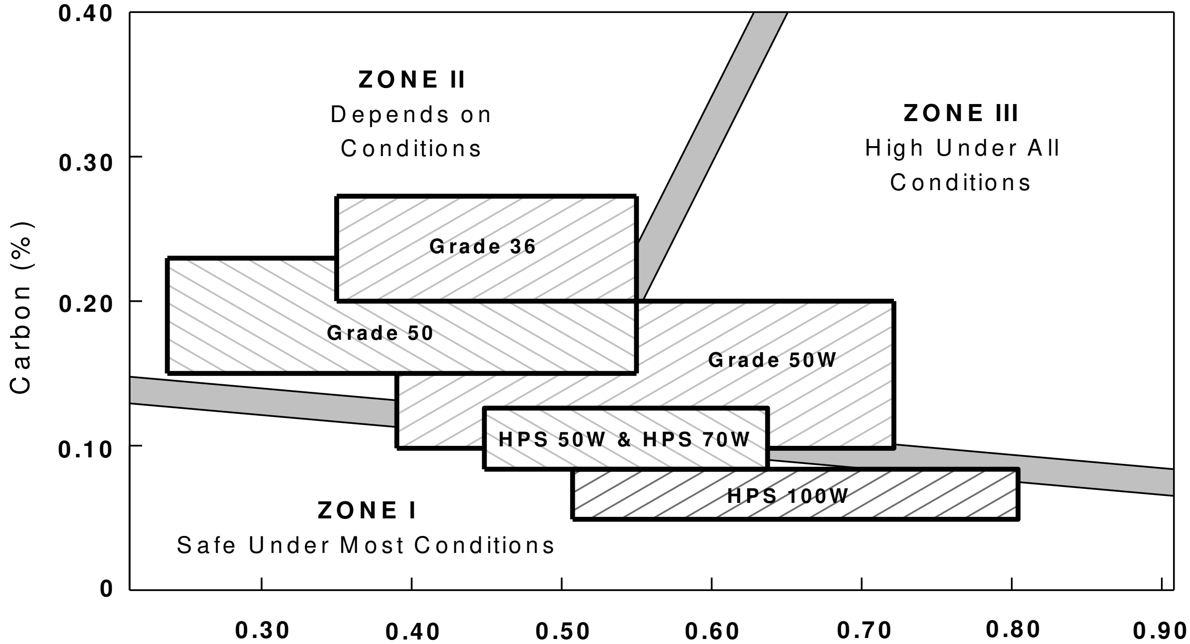
All modern structural bridge steels are weldable following the procedures of the AASHTO/AWS D1.5 welding specifications (29). The D1.5 specification is more restrictive in some aspects compared to the more general D1.1 Structural Welding Code since bridge welds must perform relative to the fatigue and fracture limit states. Weldability can be generally defined as the ability of a steel to be welded to serve its intended application. Following the D1.5 provisions, all bridge steels in the A 709 specification can be considered weldable. The weldability of steel grades 36, 50, 50W, 50S, HPS 50W, HPS 70W and HPS 100W have been well established through a combination of research and experience. Few weldability problems are expected when the D1.5 procedures are followed for these grades.

HPS steels were developed with a primary goal of improving weldability compared to the conventional grades 70W and 100W. While these conventional high strength steels are considered weldable, they have little tolerance of variation in welding parameters from those specified in D1.5. Many weld cracking problems were reported in fabrication that drove up the cost of fabrication with these grades. As a consequence, grades 70W and 100W developed a bad reputation and designers were reluctant to utilize them for bridge design. Fabricator experience with the new HPS grades has been excellent and many fabricators report they are at least as weldable as the conventional lower strength grades. Grade HPS 70W has now replaced grade 70W in A 709 and designers are encouraged to use HPS 100W in lieu of 100W for primary bridge members.

The welding provisions for the HPS grades have been developed through the research activities of the FHWA/USN/AISI Welding Advisory Group. The group develops and evaluates research and fabrication experience with the HPS grades and prepares documents for consideration by AASHTO (30). These documents serve as an initial vehicle to disseminate the latest information while it is being considered for inclusion in the D1.5 specification.

5.2 Base Metal Chemistry and Carbon Equivalent

The weldability of structural steels is largely dependent on the chemical composition. Graville categorized the susceptibility to heat affected zone (HAZ) cracking is dependent on both the carbon content and the carbon equivalent calculated by a formula that considers other alloying elements in addition to carbon (31). Figure 12 shows that weldability can be divided into three general classification zones depending on chemical composition as denoted by the gray bands. Conventional grades 36 and 50 tend to fall into Zone II indicating they are weldable if proper procedures are followed. Grade 50W can range between Zones II and III based on alloy content allowed by the A 709 specification but typically falls within Zone II. The new HPS grades, primarily due to their low carbon formulations, now tend to fall into Zone I indicating improved weldability compared to conventional grades. This indicates that HPS steels have a low probability of heat affected zone (HAZ) cracking under most conditions.



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

Figure 12 Graville weldability diagram to indicate the relative susceptibility to HAZ cracking of bridge steels.

The x-axis in Figure 12 shows the carbon equivalent (CE) equation recommended by the D1.5 Bridge Welding Code to assess weldability based on chemical composition.

5.3 Thermal Cutting

All structural steels in the A 709 specification are suitable for thermal cutting. The oxy-fuel gas process is the most widely used process in bridge fabrication since it is capable of cutting all plate thicknesses used in bridge fabrication. Plasma arc and carbon air-arc processes are also used in some cases for thinner plates. Laser cutting is another thermal cutting method that is gaining attention due to potentially high cutting speeds. However, laser cutting is limited to relatively thin plates. This limits the usefulness of laser cutting for bridge fabrication where flange plates up to 4 in. thick are sometimes required.

The effect of thermal cutting on A36, A572, and A588 plate was studied in the 1980's (32). No visual edge cracking was observed for the oxy-fuel or plasma cutting processes. Bend tests were performed on the cut edges to investigate the effect of material properties. Lower edge hardness, higher CVN toughness, and lower carbon levels in the plates had the effect of elevating the bend test rating. However the absence of visual cracking indicates that thermal cutting did not degrade the fitness for service of the plates.

5.4 Machining

All structural steels can be considered machinable using standard shop practices, including grinding, milling, and drilling. Some fabricators have reported that it is more difficult to drill holes in HPS steels versus conventional steels. Other fabricators report no difference. It seems to depend on the drill pressure and coolant used during drilling.

Many fabricators report that the weathering steel grades, including HPS, tend to have a tightly adhering mill scale that is more difficult to remove by blast cleaning. This does not have any adverse structural implications, but it may be a cost factor in fabrication.

5.5 Product Tolerances

All steel products are produced to meet the geometric tolerances proscribed in the ASTM A 6 specification. It is impossible to produce plates that are perfectly flat or shapes that are perfectly straight and free from cross-sectional distortions. Residual stresses are always present that affect plate distortion. The A 6 limits have been established to ensure steel products are dimensionally and aesthetically adequate for use in bridges.

5.5.1 Plate Thickness

Plates are ordered to the required nominal thickness for their intended purpose. The permitted variation below the specified thickness in 0.010 in. The permitted over-thickness variation varies between 0.03 in. to 0.17 in. depending on plate thickness and width. It is common for manufacturers to roll plates with a slight over-thickness to avoid rejects at the under-thickness limit.

5.5.2 Plate Flatness

Plates have requirements for both flatness and waviness as shown in Figure 13. Overall flatness is measured with the plates lying flat in a horizontal position. The A 6 specification has a table that lists the flatness requirement for both HSLA and carbon steel plates based on plate thickness and width. In general, the flatness requirements are more liberal for thinner plates that are more flexible. Waviness is another measure of flatness. Again, waviness is measured with the plate in the horizontal position. There are limits on both the wave amplitude and number of waves across the plate depending on plate width and thickness. Because plate deformation is typically introduced or eliminated in the fabrication process, the flatness and waviness of plates in fabricated members may be substantially different than the A 6 requirements.

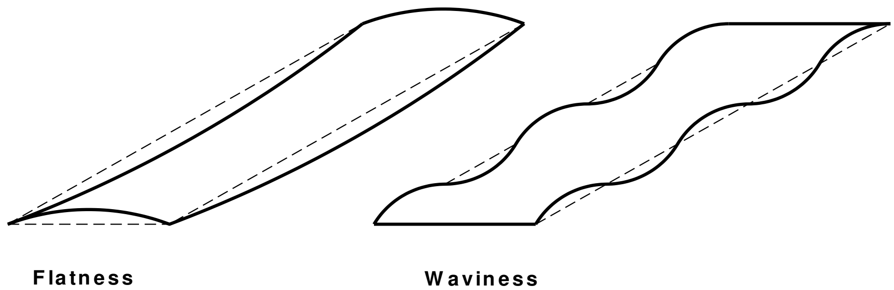


Figure 13 Illustration of plate flatness and waviness.

5.5.3 Rolled Shape Tolerances

Rolled shape tolerances are also proscribed in the ASTM A 6 specification. Thickness tolerances for the web and flange elements are not prescribed. Instead, tolerances are placed on the cross-sectional weight. For shapes less than 100 lbs/ft the weight tolerance varies between -2.5% and +3%. Heavier shapes have a +/- tolerance of 2.5%. Combined with tolerances for the overall width and depth of the section, this provides a reasonable assurance that section properties are being met without having to perform complicated measurements of the elements making up the shapes and the radius between elements. Additional requirements are also provided to control angular distortion between perpendicular elements in a given cross section.

The straightness of rolled shapes measured about both the strong axis (camber) and the weak axis (sweep) are prescribed for rolled shapes. Table 9 shows the maximum out-of-straightness tolerance limits for the most common rolled shapes used for bridge members. Although out-of-straightness is considered in the derivation of the compression and flexural buckling capacity equations in AASHTO, these limits may be useful for engineers performing analysis of various members in compression.

Table 9 Straightness tolerance limits for most rolled shapes used in bridges.

Shape	Use	Direction	+/- Tolerance (in)[(2)]
S, M, C, MC, L, T, Z[(1)]	All	Camber	$\frac{1}{8}\left(\frac{L}{5}\right)$
		Sweep	Not Specified
W, HP	General	Camber	$\frac{1}{8}\left(\frac{L}{10}\right)$
		Sweep	
	Columns ($L \le 45$ ft)	Camber	$\frac{1}{8}\left(\frac{L}{10}\right) \le \frac{3}{8}$
		Sweep	
	Columns ($L > 45$ ft)	Camber	$\frac{3}{8}+\left(\frac{1}{8}\frac{(L-45)}{10}\right)$
		Sweep	

(1) Element sizes greater or equal to 3 in.
(2) Length L measured in feet.

5.6 Cold Bending

As discussed in the section on strain aging, the mechanical properties of structural steel change after the steel undergoes plastic deformation. The A 6 specification recommends limits on the minimum radius for cold bending as shown in Table 10. These limits are set to minimize the potential for cracking on the outside surface of the bend radius due to the reduction in ductility of plastically deformed material.

Table 10 Minimum bend radius specified in ASTM A 6.

Steel Grade	Group Designation	Minimum Inside Bend Radius			
		$t \leq 0.75$	$0.75 < t \leq 1.0$	$1.0 < t \leq 2.0$	$t > 2.0$
36	B	$1.5t$	$1.5t$	$1.5t$	$1.5t$
50, 50W, HPS 50W	C	$1.5t$	$1.5t$	$2.0t$	$2.5t$
HPS 70W	D	$1.5t$	$1.5t$	$2.5t$	$3.0t$
100, 100W, HPS 100W(1)	F	$1.75t$	$2.25t$	$4.5t$	$5.5t$

(1) Requirements for HPS 100W have not yet been established.

In addition to the previously discussed effect on strength and ductility, steel subjected to high plastic strains also shows a reduction in CVN toughness. For bridges, this could have an effect on the performance of tension members designated as fracture critical or non-fracture critical. The current AASHTO provisions currently allow the use of cold bending following the A 6 radius limit recommendations. More research is needed to establish if different requirements are needed to address the fatigue and fracture limit states.

5.7 Heat Curving and Straightening

It is common for fabricators to utilize heat curving techniques to introduce camber, sweep, and correct distortion of fabricated bridge members. This involves heating the steel in a controlled pattern with controlled temperature to induce the required movement in the steel. For non-heat treated steels (36, 50, 50W, 50S, and HPS 50W) the maximum temperature is limited to 1,200°F. For heat treated steels (HPS 70W and HPS 100W) the maximum temperature is limited to 1,100°F. Additionally, the heat curving temperature cannot exceed the tempering temperature reported on the mill certificate, if it is less than 1,100°F.

6.0 CORROSION RESISTANCE

Steel grades with the "W" suffix are called "weathering" steels since they demonstrate enhanced atmospheric corrosion resistance. In many environments, weathering steels can be used without paint coatings in bridge structures. It can generally be said that the rate of section loss for weathering steel grades is between 2 and 4 times lower than non-weathering grades 36 and 50. However, corrosion rates are very dependent on the local environment. This can vary widely, even in the same structure, considering that details can trap local moisture and concentrations of chloride from road salts. It is therefore difficult to establish a performance-based requirement for corrosion resistance in terms of section loss.

The ASTM G 101 specification was developed to standardize a methodology for classification of steels as weathering (33). A corrosion index (I) is calculated based on the chemical composition of the steel. The ASTM A 709 specification indicates that steel grades with $I \geq 6$ qualify as weathering steels and have the W suffix appended to the grade. The original corrosion index equation in G 101 was developed by Legault and Leckie to be valid for steels with composition close to grade 50W (A 588):

$$I = 26.01(\%Cu) + 3.88(\%Ni) + 1.20(\%Cr) + 1.49(\%Si) + 17.28(\%P) - 7.29(\%Cu)(\%Ni) - 9.10(\%Ni)(\%P) - 33.39(\%Cu)^2$$

More recently, Townsend introduced an alternate equation in G 101 for steels that exceed the validity limits of the Legault-Leckie equation. This became important with the introduction of the HPS grades (HPS 50W, HPS 70W, and HPS 100W). The HPS grades have higher % Cu compared to grade 50W and the last term in the above equation severely penalizes the HPS compositions. The Townsend equation provides much better correlation with experimental data and should be used for evaluation of the HPS steel grades. Calculation of the corrosion index using the Townsend equation requires a more involved procedure involving the summation of tabulated constants and the reader is referred to the G 101 specification.

Classification of a steel as "weathering" indicates it is suitable for use in bridge structures without paint or other protective coatings. However, weathering steel may not perform well in bridge locations subjected to high time-of-wetness or exposure to high levels of chlorides. It is important for the designer to follow the usage and detailing guidance recommended by the FHWA when weathering steels are specified in bridge structures (34). Figure 14 shows the Townsend corrosion index value for various weathering steel grades as predicted by the A 709 chemical composition requirements (35).

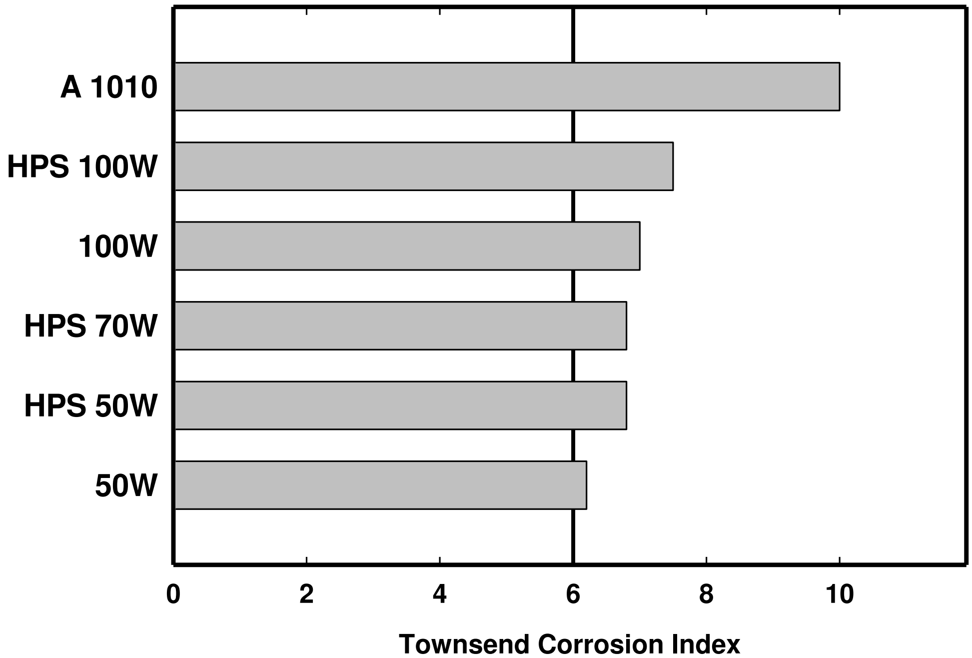


Figure 14 Comparison of the typical corrosion index between different grades of weathering steels based on the Townsend corrosion index.

Applications that are not suitable for use of unpainted weathering steel require other corrosion protection options. Paint coatings are the most common solution. Other options are available, including galvanizing and metalizing. All of the A 709 bridge steels are suitable for use with any of these coating options. In general, there is no need to specify weathering steel grades if they are going to be used with a coating system. Some owners have specified weathering steel for painted structures as a back-up if the coating system fails at some time in the future. The benefits, if any, from using weathering steel with coating systems has not been established.

7.0 REFERENCES

1. ASTM. *Annual Book of Standards, Section 1, Iron and Steel Products.* West Conshohocken : ASTM, 2010. Vol. 01.04.

2. AASHTO. *Standard Specifications for Transportation Materials and Methods of Sampling.* s.l. : American Association of Highway and Transportation Officials, 2011. 31st Ed..

3. ASTM. *Standard Specification for Structural Steel for Bridges.* West Conshohocken : American Society for testing and Materials, 2010. A 709/A709M.

4. Wright, W.J. High Performance Steel: Research to Practice. *Public Roads.* Spring, 1997, Vol. Vol. 60, No. 4.

5. ASTM. *Standard Specifications for Higher Strength Martensitic Stainless steel Plate, Sheet, and Strip.* West Conshohocken : American Society for Testing and Materials, 2009. A 1010/A 1010M-01.

6. *Corrosion Performance of Improved Weathering Steels for Bridges.* Fletcher, F.B., Townsend, H.E., and Wilson, A.D. World Steel Bridge Symposium, Nov. 19-20, Orlando : National Steel Bridge Alliance, 2003. World Steel Bridge Symposium.

7. *Weldability of ASTM A 1010 Steel.* Seradj, H. 8th International Conference on Short & Medium Span Bridges, Niagara Falls : Canadian Society of Civil Engineering, 2010.

8. ASTM. *Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes.* West Conshohocken : American Society for Testing and Materials, 2010. A 500 - 10a/A 500M.

9. ASTM. *Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling.* West Conshohocken : American Society for Testing and Materials, 2010. A 6/A 6M.

10. ASTM. *Standard Test Methods and Definitions for Mechanical Testing of Steel Products.* West Conshohocken : American Society for Testing and Materials, 2010. A 370 - 10a.

11. ASTM. *Standard test Methods for Tension Testing of Metallic Materials.* West Conshohocken : American Society for Testing and Materials, 2008. E 8/E 8M - 08.

12. ASTM. *Standard Test Method for Young's Modulus, Tangent Modulus, and Chord Modulus.* West Conshohocken : American Society for Testing and Materials, 2004. E 111 - 04.

13. *Dynamic KIc testing of Structural Steel.* Madison, R.B., and Irwin, G.R. Paper 10653, Journal of the Structural Division : American Society of Civil Engineers, 1974, Vol. No. ST 7.

14. ASCE. *Structural Fire Protection.* New York : American Society of Civil Engineers, 1992. Manual No. 78.

15. ECS. *The European Standard: Part 1-2: General Rules for Structural Fire Design.* Brussels : European Committee for Standardization, 2005. EN 1993-1-2.

16. Wilson, A.D., *Clean Steel Technology - Fundamental to the Development of High Performance Steels*, *Advances in the Production and Use of Steel with Improved Internal Cleanliness, ASTM STP 1361*, J.K. Mahaney, Jr. Ed., American Society for Testing and Materials, West Conshohocken, PA, 1999.

17. Brockenbrough, R.L. *Effect of Yield-tensile Ratio on Structural Behavior.* Washington, DC : American Iron and Steel Institute, 1995.

18. Suwan, S., Manual, L., and Frank, K. *Statistical Analysis of Structural Plate Mechanical Properties.* Austin : Fergurson Structural Engineering Laboratory, University of Texas, 2003. Report No. 03-1.

19. AASHTO. *Guide Specification for Fracture Critical Non-Redundant Steel Bridge Members.* Washington, DC : American Association of State Highway and Transportation Officials, 1978.

20. ASTM. *Standard Test Methods for Notched Bar Impact testing of Metallic Materials.* West Conshohocken : American Society for Testing and Materials, 2011. E 23 - 07ae1.

21. ASTM. *Standard test Method for Linear-Elastic Plane-Strain Fracture Toughness KIc of Metallic Materials.* West Conshohocken : American Society for Testing and Materials, 2011. E 399 - 09e2.

22. ASTM. *Standard Test Method for Measurement of Fracture Toughness.* West Conshohocken : American Society for Testing and Materials, 2011. E 1820-09e1.

23. Barsom, J.M., and Rolfe, S.T. *Fracture and Fatigue Control in Structures - Applications of Fracture Mechanics.* s.l. : Prentice Hall, 1999.

24. *Fracture Toughness Requirements for Highway Bridges: Past and Future Trends.* Wright, W.J. 1, s.l. : Progress in Structural Engineering and Materials, 2002, Vol. 4.

25. *Fatigue Strength of Steel Beams with Welded Stiffeners and Attachments.* Fisher, J.W., Albrecht, P., Yen, B.T., Klingerman, D.J., and McNamee, B.M. Washington DC : Transportation Research Board, 1974. NCHRP Report No. 147.

26. Fisher, J.W., Frank, K.H., Hirt, M.A., and NcNamee, B.M. *Effect of Weldments on the Fatigue Strenght of Steel Beams.* Washington, DC : Transportation Research Board, 1970. NCHRP Report No. 102.

27. ASTM. *Standard Specification for Sampling Procedure for Impact Testing of Structural Steel.* West Conshohocken : American Society for Testing and Materials, 2011. A 673/A 673M-07.
28. Frank, K.H., George, D.A., Schluter, C.A., Gealy, S. and Horos, D.R. *Notch Toughness Variability in Bridge Steel Plates.* Washington, DC : Transportation Research Board, 1993. NCHRP 355.

29. AASHTO/AWS. *Bridge Welding Code.* Washington, DC : American Association of State Highway and Transportation Officials, 2010. ANSI/AASHTO/AWS D1.5-10.

30. SMDI. *Guide Specifications for Highway Bridge Fabrication with High Performance Steel.* Washington, DC : Steel Market Development Institute, 2011. 3nd. Ed., April.

31. Graville, B.A. *Cold Cracking in Welds in HSLA Steels.* s.l. : American Society for Metals, 1976. Welding of HSLA (Microalloyed) Structural Steels, Nov 9-12.

32. Wilson, A.D. *Thermal Cutting of HSLA Bridge Steels.* Washington, DC : American Iron and Steel Institute, 1987.

33. ASTM. *Standard Guide for Estimating Atmospheric Corrosion Resistance of Low Alloy Steels.* West Conshohocken : American Society for Testing and Materials, 2004. G 101.

34. FHWA. *Uncoated Weathering Steel in Structures.* Washington DC : Federal Highway Administration, 1989. Technical Advisory T 5140.22.

35. Wilson, A.D. *Current Status of the High Performance Steel Research Program* . Washington DC : Transportation Research Board, 2004. Proceedings of the TRB Annual Meeting, January.