Steel Bridge Design Handbook

CHAPTER 1

Bridge Steels and Their Mechanical Properties

February 2022

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Foreword

The Steel Bridge Design 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 has a long history, dating back to the 1970s in various forms and publications. The more recent editions of the Handbook were developed and maintained by the Federal Highway Administration (FHWA) Office of Bridges and Structures as FHWA Report No. FHWA-IF-12-052 published in November 2012, and FHWA Report No. FHWA-HIF-16-002 published in December 2015. The previous development and maintenance of the Handbook by the FHWA, their consultants, and their technical reviewers is gratefully appreciated and acknowledged.

This current edition of the Handbook is maintained by the National Steel Bridge Alliance (NSBA), a division of the American Institute of Steel Construction (AISC). This Handbook, published in 2021, has been updated and revised to be consistent with the 9th edition of the AASHTO LRFD Bridge Design Specifications which was released in 2020. The updates and revisions to various chapters and design examples have been performed, as noted, by HDR, M.A. Grubb & Associates, Don White, Ph.D., and NSBA. Furthermore, the updates and revisions have been reviewed independently by Francesco Russo, Ph.D., P.E., Brandon Chavel, Ph.D., P.E., and NSBA.

The Handbook consists of 19 chapters and 6 design examples. The chapters and design examples of the Handbook are published separately for ease of use, and available for free download at the NSBA website, www.aisc.org/nsba.

The users of the Steel Bridge Design Handbook are encouraged to submit ideas and suggestions for enhancements that can be implemented in future editions to the NSBA and AISC at solutions@aisc.org.

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7. Supplementary Notes

The previous edition of this Handbook was published as FHWA-HIF-16-002 and was developed to be current with the 7th edition of the AASHTO LRFD Bridge Design Specifications. This edition of the Handbook was updated to be current with the 9th edition of the AASHTO LRFD Bridge Design Specifications, released in 2020.

8. Abstract

This chapter 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/A709M Specification. This includes both a general introduction to steelmaking 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 ASTM A709 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 describe how the Charpy V-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 uncoated applications.

9. Keywords Steel Bridge, Steel Plate, Steel Manufacturing, Welding, Weldability, Steel Bridge Fabrication, Steel Mechanical Properties, Charpy V-Notch Impact Test, Fracture Toughness	**10. AISC Publication No.** B901-22

Steel Bridge Design Handbook: Bridge Steels and Their Mechanical Properties

Table of Contents

List of Figures

List of Tables

1.0 INTRODUCTION

Structural steels for bridges generally have more stringent performance requirements compared to steels used in other structural applications. Bridge steels are subjected to relatively large temperature changes; millions of cycles of live loading; and often to corrosive environments containing chlorides. In addition to strength and ductility (i.e., toughness) requirements, bridge steels must satisfy additional service requirements with respect to fatigue. Bridge steels also must 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 satisfy fracture toughness requirements and provide a level of corrosion resistance that generally exceeds the requirements necessary for structural steels used in other applications.

This chapter is written from a structural engineer's perspective and focuses on performance aspects of structural steel. A general overview of the steelmaking 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 A709/A709M Specification. This includes both a general introduction to steelmaking 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 ASTM A709 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 describe how the Charpy V-notch test relates to fracture resistance in structures. The weldability and fabrication (i.e., thermal cutting, machining, product tolerances, cold bending, and heat curving and straightening) of bridge steels is also discussed. Finally, the methodology for determining atmospheric corrosion resistance is presented along with the requirements for classification as "weathering steels" for use in uncoated applications.

2.0 PRODUCT SPECIFICATIONS

There are two organizations that publish standards for structural steel in the United States. The first, the American Society for Testing and Materials (ASTM) is a non-profit voluntary standards organization that develops consensus standards for steel products. ASTM Committee A01 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 second, 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. By maintaining independent standards, AASHTO maintains the right to modify the ASTM requirements if it is determined to be in the public's interest. The counterparts to the ASTM A709/A709M standards for structural steels for bridges are the AASHTO M270M/M270 standards.

Most bridge owners specify adherence to the AASHTO material specifications in their construction documents. Some specify adherence to the 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	M270M/M270	A709/A709M
Structural Stainless Steel		A1010 A709/A709M Grade 50CR
Cold-Formed Welded or Seamless Tubing		A1085 A500/A500M Grade B or C A847
Hot-Formed Welded or Seamless Tubing		A501 A618
Pins, Rollers, and Rockers	M169 M102M/M102	A108 A668/A668M
Structural Bolts		A307 Grade A or B F3125/F3125M Grade A325 Grade F1852 Grade A490 Grade F2280
Galvanizing of Structural Bolts (Grades A325 and F1852)	M232M/M232 Class C	A153/A153M B695
Anchor Rods	M314	F1554
Nuts		A563/A563M A194/A194M
Washers		F436/F436M F959/F959M (DTIs)
Shear Studs	M169	A108 Grades G10100 through G10200
Cast Steel	M103M/M103 Grade 70-36 M163M/M163 Grade CA15	A27/A27M A743/A743M
Ductile Iron		A536 Grade 60-40-18
Malleable Castings		A47/A47M Grade 35018
Cast Iron	M105 Class 30	A48/A48M
Stainless Steel		A240/A240M A276/A276M A666
Bright Wire Cables		A510/A510M
Galvanized Wire Cables		A641/A641M
Epoxy Coated Wire Cables		A99
Bridge Strand Cables		A586
Wire Rope Cables	M277	A603
Seven-Wire Strand	M203M/M203	A416/A416M
High Strength Steel Bar	M275M/M275	A722/A722M

2.1 Structural Plate and Rolled Shapes

The ASTM A709/A709M 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 ASTM A709 provisions are identical to those in the individual structural steel specifications applicable for more general use; however, the ASTM

A709 specification includes the additional toughness requirements specified for bridge steels. Table 2 provides an overview of the various steel grades covered by the ASTM A709 specification. The individual structural steel specifications for more general use of each grade, where applicable, are indicated in Table 2 for informational purposes only. The number in the grade designation indicates the nominal yield strength in ksi. The "W" indicates that the steel is a so-called "weathering steel". Minimum mechanical properties of the more commonly used ASTM A709 structural steels are specified in Table 6.4.1-1 of the AASHTO *LRFD Bridge Design Specifications* (hereafter referred to as the *AASHTO LRFD BDS*) [4]. Thickness limitations relative to rolled shapes and groups are to satisfy the provisions of ASTM A6/A6M [5]. The ASTM A709M specification is the metric version of ASTM A709.

Table 2 Overview of Bridge Steels Available in the A709 Specification

M270 A709 GRADE	ASTM Specification	Description	Atmospheric Corrosion Resistance	Product Categories			
				Plates	Shapes	Bars	Sheet Piles
36	A36	Carbon Steel	No	X	X	X	
50	A572	HSLA Steel	No	X	X	X	X
50S	A992	Structural Steel	No		X (***)		
QST 50	A913	HSLA Steel (QST*)	No		X (***)		
QST 50S	A913	HSLA Steel (QST*)	No		X (***)		
50W	A588	HSLA Steel	Yes	X	X	X	
HPS 50W	A709	HSLA Steel (**)	Yes	X			
50CR	A1010	Martensitic Stainless Steel	Yes	X			
QST 65	A913	HSLA Steel (QST*)	No		X (***)		
QST 70	A913	HSLA Steel (QST*)	No		X (***)		
HPS 70W	A709	Heat Treated HSLA Steel (**)	Yes	X			
HPS 100W	A709	Q&T Cu-Ni Steel (**)	Yes	X			

(*) High Strength Low-Alloy Steel shapes produced by quenching and self-tempering process (QST)
(**) High Performance Steel (HPS) grades with enhanced weldability and toughness
HSLA - High Strength Low-Alloy
Q&T - Cu-Ni Quenched & Tempered Copper-Nickel Steel
(***) Rolled I-sections and tee-sections only

2.1.1 Grade 36

The ASTM A36 specification [6] was originally adopted in 1960 as the final evolution of weldable carbon-manganese structural steel. Of all the steels in the ASTM A709 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 steel has higher residual elements than iron ore the resulting steel strength can be much higher. The steels being delivered today as Grade 36 may 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 ASTM A572 specification [7] 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 ASTM A36 steel. This resulted in a 39% increase in yield strength compared to ASTM A36 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, galvanized, or metalized in service.

2.1.3 Grade 50S

The ASTM A992 specification [8] 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 ASTM A992 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 ASTM A992 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. Grade 50S in the ASTM A709 specification is equivalent to ASTM A992, but includes the additional toughness requirements specified for bridge steels. Non-weathering steel rolled I-shapes and structural tees should be specified as Grade 50S. Other non-weathering rolled shapes (e.g., angles and channels) are typically only available as either Grade 36 or Grade 50 and should be specified as such.

2.1.4 QST Grades 50, 50S. 65, and 70

The ASTM A913 *Standard Specification for High-Strength Low Alloy Steel Shapes of Structural Quality, Produced by Quenched and Self-Tempering Process (QST)* [9] was introduced into the standards in 1993, and was added more recently into A709 to offer more options for rolled I-shapes; four grades – 50, 50S, 65, and 70 – are available according to the standard. Grade QST 50S has an upper limit of 65 ksi on the yield strength similar to Grade 50S, whereas Grade QST 50 has no upper limit.

The shapes are produced by an in-line heat treatment and cooling-rate control process comprised of three primary stages: rapid water cooling of the shape, interruption of the cooling before the core is quenched, and self-tempering of the outer layers from the retained heat within the core. The rapid cooling leads to refinement of the microstructure and the attainment of higher yield and tensile strengths. The self-tempering maintains the ductility and toughness of the steel. The carbon maximum of ASTM A913 is roughly half of ASTM A992. Metallurgical benefits include enhanced brittle fracture resistance, increased ductility, and improved weldability. Phosphorus, sulfur, and copper maximums are also lower than ASTM A992.

Since these grades have not yet been included in the AASHTO specifications or in the AASHTO/AWS D1.5 Bridge Welding Code [10], projects utilizing these grades for rolled I-shapes would require special provisions for fabrication, particularly welding, based on recommendations from the manufacturer.

2.1.5 Grade 50W

Grade 50W is a special version of 50 ksi steel that was developed to have enhanced atmospheric corrosion resistance. Grade 50W, commonly referred to as "weathering" steel, has been shown to perform well without paint or other coatings in many bridge applications. Weathering steel achieves its corrosion resistance through the development of a tightly adhered layer of rust (sometimes called the "patina") that acts to inhibit further corrosion of the steel beneath. Different steel companies initially developed competing proprietary grades that were included in the ASTM A588 specification in 1968 [11]. 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 coat the steel.

2.1.6 Grades HPS 50W, HPS 70W, and HPS 100W

The high-performance steel (HPS) grades were developed in the 1990s 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 [12]. Prior to HPS, steels with yield strengths greater than 50 ksi (i.e., ASTM A852 and A514) 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. Rolled shapes are not currently available in HPS grades.

Because of the enhanced properties, the original Grade 70W steel (A852) has been replaced in the ASTM A709 specification with Grade HPS 70W, which is the only 70-ksi option available for bridge use. The ASTM A852 specification has been withdrawn. For similar reasons, the HPS 100W grade has replaced ASTM A514 Grades 100 and 100W in the ASTM A709 specification for fabrication of structural bridge members where 100 ksi strength is desired. ASTM A514 steel [13] is a high strength, 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 ASTM A514 steels are weldable, 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 HPS grades. Engineers may still specify ASTM A514 Grades 100 and 100W for bearings and other secondary components in bridges. Since ASTM A514 steels are used in other industries, there may be better availability in small quantities.

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 U.S.

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. Hybrid girders utilizing Grade HPS 70W steel for the flanges and Grade 50W steel for the web have been shown to be a particularly economical option in regions of negative flexure. Because it costs more than conventional steel, use of HPS should be carefully considered by the designer to verify that the benefits outweigh the additional cost of the product.

Grade 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, Grade 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 Grade 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 considered beneficial for certain fracture-critical members with low redundancy such as the tension ties in tied-arch bridges. Since Grade HPS 50W is a higher cost material compared to Grade 50W, engineers should carefully consider the need for higher toughness before specifying Grade HPS 50W.

2.1.7 Grade 50CR

In the past, stainless steels have occasionally been used to fabricate bearings and other parts for bridges where high corrosion resistance is required. Traditionally, the relatively high initial cost of stainless steel has limited its use in primary bridge members. However, given the expanding trend toward life-cycle cost analysis, stainless steels merit consideration for expanded use in some corrosive structural applications where weathering steel is not recommended. According to Article 6.4.7 of the *AASHTO LRFD BDS*, stainless steels must conform to either ASTM A240 [14], A276 [15], or A666 [16], unless it can be shown that the steel conforms to the chemical and mechanical requirements of one of the above-listed ASTM specifications or other published specifications that establish its properties and suitability for bridges based on analyses, tests and controls consistent with one of the above-listed ASTM specifications.

To date, one promising product for bridge-use is ASTM A709 Grade 50CR, a dual phase stainless steel with a 10.5-12.5% chromium content, which was previously specified as ASTM A1010 [17] prior to its incorporation into the ASTM A709 specification. The product has been shown to have greatly enhanced corrosion resistance compared to weathering steel grades [18, 19, 20, 21, 22] and can provide adequate performance without paint in higher chloride bridge environments. It has mechanical properties that are more like other bridge steels than austenitic stainless steels and it is more economical than duplex stainless steels. Grade 50CR is currently available in thicknesses up to 2 inches. Although the ASTM A1010 specification has been used in the past for bridges, specifying ASTM A709 50CR for bridge applications is more appropriate than ASTM A1010 because the ASTM A709 specification includes bridge CVN requirements.

Currently, some special provisions are required to utilize this grade within the existing bridge specifications. Grade 50CR steel is weldable using all processes currently employed for bridge fabrication. It can be successfully welded using 309L or 316L austenitic stainless-steel consumables, with the "L" in the consumable designating low carbon [23]. However, this product is not currently included in the AASHTO/AWS D1.5 Bridge Welding Code [10], therefore supplemental provisions need to be invoked based on recommendations by the steel manufacturer or welding consumable manufacturer. The grade can be fabricated using standard fabrication practices including cold bending, heat curving, drilling, and machining. One exception is that the material is not suitable for cutting using oxy-fuel processes. Plasma or laser cutting is required; however, it is not necessary to specify the cutting process. Rather, fabricators will use cutting processes that are effective based on the steel manufacturer's recommendation, and it is widely known that oxy-fuel processes are not used on stainless steel grades. Precautions must also be taken to prevent carbon contamination of Grade 50CR areas to be welded. This can be accomplished through using grinding wheels, wire brushes, chipping hammers, etc. that have either not been used on carbon steel or are made of stainless steel.

Another difference when fabricating with Grade 50CR is blast cleaning. If aesthetics of the Grade 50CR steel are not important, it is recommended that the steel be left as-is without blast cleaning to reduce cost. If aesthetics are important, it is recommended that the steel be blast cleaned with a steel shot/grit mixture similar to what is used on other ASTM A709 bridge steels. The steel shot/grit blast cleaning will create a uniform patina that is reddish brown, similar to Grade 50W. Note that if blast cleaning is desired, it must be done after all welding is completed to avoid carbon contamination. In the past, Grade 50CR has been blast-cleaned with non-metallic garnet blast media, which leaves the steel surface with a light grey color. This leaves the steel vulnerable to staining in high chloride areas such as near expansion joints or close to water, although this is only a concern for aesthetics and not for corrosion performance. When considering blast cleaning, although surface condition factors for slip resistance of Grade 50CR are not yet included in the *AASHTO LRFD BDS* for bolted connection design, testing has shown that Grade 50CR can be expected to perform similar to other ASTM A709 steels [0].

When stainless steels are connected to dissimilar metals in the presence of an electrolyte, there is the potential for galvanic corrosion to occur. Galvanic corrosion is when the less noble (i.e., less corrosion resistant) of the dissimilar metals being joined corrodes at an accelerated rate [25]. This should be considered when selecting secondary members such as cross-frames or diaphragms to be used with Grade 50CR girders. In more corrosive environments, stainless steel fasteners and Grade 50CR secondary members should be specified. Since stainless steel fasteners are more noble than Grade 50CR, no accelerated corrosion is expected to occur. Note that Grade 50CR is currently only produced in hot-rolled plate form, so if secondary members made of Grade 50CR are desired, they must be constructed with bent plates or built-up members [22]. In less corrosive environments, galvanized fasteners and galvanized secondary members can be specified because the zinc provides cathodic protection for the underlying steel. Galvanized steel has also shown good long-term corrosion performance when bolted to Grade 50CR [26]. Since Grade 50CR has yet to be included in the AASHTO specifications or the AASHTO/AWS D1.5 Bridge Welding Code [10], projects will require special provisions for fabrication, particularly welding, and may require supplemental testing at the discretion of the engineer. Also, since Grade 50CR steel has a higher initial cost compared to conventional bridge steels, engineers should consider its use in applications where the higher initial cost can be overcome by the anticipated lack of future maintenance funds spent over the structure's service life. Examples of these applications include areas where uncoated weathering steel is not expected to perform well or where future maintenance could incur significant owner or user cost.

2.2 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 has traditionally referred to cold-formed welded or seamless structural steel tubing produced under the A500 specification [27]). 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 using the electric resistance welding (ERW) process. Round, square, and rectangular shapes are available with various cross-sectional dimensions and wall thicknesses. Cold-formed welded or seamless steel tubing is also available under the ASTM A847 specification (for weathering steel applications) [28]. Hot-formed steel tubing is to conform to ASTM A501 [29] or A618 [30].

The suitability of traditional HSS grades for bridge members subjected to the fatigue and fracture limit states has not been well established, and cold bending of the corners of rectangular shapes can lead to reduced notch toughness in the corner regions. As indicated in Article C6.4.1 of the *AASHTO LRFD BDS*, the ASTM A500 specification cautions that structural tubing manufactured to that specification may not be suitable for applications involving "dynamically loaded elements in welded structures where low-temperature notch-toughness properties may be important." Where this material is contemplated for use in applications where low-temperature notch-toughness properties are important, consideration should be given to requiring that the material satisfy the Charpy V-notch toughness requirements specified in Article 6.6.2.1. The Owner should also be consulted regarding the use of this material. ASTM A1085 (see below) is an improved specification for cold-formed welded carbon steel HSS that is more suitable for dynamically loaded structures.

In 2013, a new ASTM specification for HSS was released titled ASTM A1085 [31], which has since been included in the *AASHTO LRFD BDS*. The ASTM A1085 specification provides an HSS with improved performance for dynamically loaded structures. Shapes are available in square, round, and rectangular sections. ASTM A1085 has more stringent wall tolerances and the addition of a mass tolerance which means that the full nominal wall thickness can be used for design of HSS. This means that designers no longer need to reduce the nominal wall thickness by 0.93 as prescribed in Article 6.12.1.2.4 of the *AASHTO LRFD BDS* for HSS produced to the other standards specified in Article 6.4.1. The ASTM A1085 standard specifies a single minimum yield stress of 50 ksi and a maximum yield stress of 70 ksi. ASTM A1085 further prescribes that all HSS is to meet a minimum CVN value of 25 ft-lb at 40° Fahrenheit, which corresponds to AASHTO requirements for Zone 2 fracture critical steel. A minimum corner bend radius is included in the specification; the ASTM A500 and A501 specifications only provide a maximum bend radius.

Another possible concern related to the use of HSS for bridges is the possibility of internal corrosion within the tubes since the interior of the tube cannot be accessed for visual inspection. Sealing of the tube ends or galvanization are possible options to control internal corrosion. In addition, HSS requires different connection details for which limited fatigue data currently exists. Designers specifying HSS should consider the connection design procedures given in Chapter K of the AISC *Specification for Structural Steel Buildings* [32]. Resistances for fatigue design of round, square, and rectangular HSS may be found in Section 10.2.3 of the *AWS D1.1/D1.1M Structural Welding Code – Steel* [33], or in Section 11 of the *AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries and Traffic Signals* [34]. Where these members are used in fracture-critical applications, refer to Article 8.2.3 of the *LRFD Guide Specifications for the Design of Pedestrian Bridges* [35].

Note that the AASHTO/AWS D1.5 Bridge Welding Code [10] specifically excludes tubular members and tubular joints. Therefore, it is incorrect to specify the use of this Code for these members, including for bridges, since it fails to provide the appropriate information for welding. However, Article 8.2.3 of the *LRFD Guide Specifications for the Design of Pedestrian Bridges* does require the application of the Fracture Control Plan in Clause 12 of the Code for fracture-critical members with some specified modifications. Provisions for the inclusion of tubular materials in the Code is underway and is anticipated for a future edition of the Code.

2.3 Bolts, Rivets, and Anchor Rods

2.3.1 Bolts

High-strength bolts are heavy hexagonal-head bolts used with heavy semi-finished hexagonal nuts. The threaded portion of high-strength bolts is shorter than for bolts used for nonstructural applications, which reduces the probability of having the threads present in the shear plane. High-strength bolts produce large and predictable tension when tightened. Initial tensioning of high-strength bolts results in more rigid joints and greater assurance against nut loosening in connections subjected to slip or vibration.

High-strength bolts used as structural fasteners on bridges are to conform to the ASTM F3125/F3125M Specification [36] (*AASHTO LRFD BDS* Article 6.4.3.1.1). ASTM F3125 is a combined structural bolt specification that was approved and published by ASTM in 2015. The specification replaced six separate bolt specifications: ASTM A325, 325M, A490, A490M, F1852, and F2280. The intent of the ASTM F3125 specification is to streamline and unify language for structural bolts, and to simplify specification maintenance moving forward. Grades within ASTM F3125 are still referred to by familiar designations, e.g., an A325 bolt is now designated as a Grade

A325 bolt and simply resides within the combined specification. One important change in the F3125 specification is an increase in the specified minimum tensile strength of 1-1/8 in. diameter and larger Grade A325 bolts from 105 ksi to 120 ksi (Table 3). Rotational-capacity testing requirements, which are required as specified in Article 11.5.5.4.2 of the *AASHTO LRFD BDS* [37], are also included as supplementary requirements in Annex A2 of ASTM F3125.

Compatible heavy hexagon-shaped nuts that satisfy the provisions for the appropriate grade in the ASTM A563 specification [38] are to be used in connections using ASTM F3125 bolts. Given the strength of these nuts, failure is controlled by bolt yielding rather than by stripping of the nut threads.

Article 11.5.5.4.3 of the *AASHTO LRFD BDS* specifies the conditions under which hardened steel washers satisfying the ASTM F436 specification [39] are required in connections using ASTM F3125 bolts. Direct tension indicators (DTIs) conforming to the requirements of ASTM F959 [40] may be used in conjunction with bolts, nuts, and washers according to *AASHTO LRFD BDS* Article 6.4.3.1.4. DTIs conforming to ASTM F959 are hardened washers with several formed arches on one face that deform in a controlled manner when subjected to a compressive load. DTIs indicate that the maximum installation tension has been achieved without exceeding the ultimate strength of the bolt. Captive DTI/nuts and DTIs that incorporate a self-indicating feature to replace the use of feeler gauges are also permissible for use when the conditions for their use specified in Article 6.4.3.1.4 are satisfied.

The surface condition and presence of lubrication is important for proper installation of the bolt-nut assemblies. The ASTM F3125 specification requires bolt lots to be subjected to tensile testing and hardness testing to verify that the minimum specified tensile strength shown in Table 3**Error! Reference source not found.** is met.

Table 3 Specified Minimum Tensile Strength of Structural Bolts

Grade	Diameter (in.)	Specified Minimum Tensile Strength (ksi)
A307 Grade A or B	All	60
F3125 Grade A325	All	120
F3125 Grade F1852 (*)	All	120
F3125 Grade A490	All	150
F3125 Grade F2280 (*)	All	150

* ASTM F3125 Grades F1852 and F2280 are the "twist-off" (i.e., tension-control) equivalents of Grades A325 and A490, respectively.

The ASTM F3125 specification has two different chemistry requirements for bolts. Type 1 bolts are basic carbon-manganese steel with required addition of silicon and optional addition of 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 uncoated weathering steel applications where both the bolts and base metal develop a compatible protective rust patina in service.

Unfinished bolts, also referred to as common, machine, ordinary, or rough bolts, are manufactured from low-carbon steel and are designated as ASTM A307 bolts [41]. Two grades – Grades A and B – are covered in the ASTM standard. Grade A is the quality that is typically used for general structural applications. Grade A bolt heads and nuts are manufactured with a regular square shape. Grade B bolts are heavy hex bolts and studs typically used for cast iron flanges in piping-system joints. As indicated in Table 3, the specified minimum tensile strength of these bolts is 60 ksi. These bolts are normally tightened using long-handled manual wrenches and hardened steel washers are not generally used. Since these bolts do not have a specified proof load and should only be used for connecting relatively light auxiliary components or members subject to light static loads or for temporary fit-up. These bolts should not be used in connections subject to slip or vibration because of the tendency of the nuts to loosen. ASTM A307 Grade C bolts were non-headed anchor rods, either bent or straight, intended for structural anchorage purposes. The properties of ASTM A307 Grade C bolts conformed to the properties of ASTM A36 material. The ASTM A307 Grade C designation (which addresses anchor rods) was eliminated in 2007 and has been replaced by the virtually identical ASTM F1554 Grade 36 specification [42], which was originally introduced in 1994 (see Section 2.3.3).Corrosion-resistant coatings may be applied to Grade A325, F1852, and A490 bolts, as specified in ASTM F3125; Type 1 bolts may be either mechanically or hot-dip galvanized. Bolts are required to be tested after galvanizing to verify the strength and ductility is not degraded by the process. The bolts, nuts and washers in any assembly must be galvanized using the same process. When galvanized Grade A325 bolts are used on weathering steel projects, only hot-dip galvanized bolts should be used as the relatively thin sacrificial coating on mechanically galvanized bolts will corrode too quickly in an uncoated weathering steel application. Galvanized Grade A490 and F2280 bolts are not allowed by AASHTO for bridge use. Because of their higher strength, Grade A490 bolts are susceptible to possible stress corrosion cracking and hydrogen embrittlement during galvanizing. However, Grade A490 bolts may be coated with a zinc/aluminum coating in accordance with ASTM F1136/F1136M [43] or F2833 [44].

Galvanizing increases the friction between the bolt and nut threads, as well as the variability of the torque-induced pretension. If the nuts are lubricated, a lower required torque and more consistent results are obtained. Nuts to be galvanized should be heat-treated Grade DH nuts and be lubricated with a lubricant containing a visible dye. To accommodate the relatively thick non-uniform zinc coatings on bolt threads during hot-dip galvanizing, the blank nut is typically hot-dip galvanized and then tapped over-size. This results in a reduction in the thread engagement and the resulting stripping strength. Only the stronger hardened nuts (Grade DH) have adequate strength to meet ASTM thread-strength requirements after over-tapping. Less over-tapping is usually required for mechanically galvanized nuts. Black bolts should be oily to the touch when delivered and installed.

2.3.2 Rivets

Rivets are rarely used today for new construction. However, a significant number of bridges with riveted construction still exist. The ASTM A502specification [45] 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. However, many riveted bridge structures were built prior to this specification and the exact rivet grade and strength may be unknown. General information on rivets and riveted connections may be found in [46]. Further information on the strength of riveted connections is provided in Article 6A.6.12.5 of the AASHTO *Manual for Bridge Evaluation* [47].

2.3.3 Anchor Rods

Anchor rods used to connect steel components to concrete foundations with diameters up to 4 in. are required to comply with the ASTM F1554 specification [42]. Three grades are available (36, 55, and 105) corresponding to the yield strength of the rod in ksi. Similar to structural bolts, anchor rods are required to be used with compatible nuts and washers. Both galvanized and non-galvanized options are available. The ASTM F1554 specification has supplemental provisions for notch toughness that can be invoked by the engineer for anchor rods loaded in tension, if needed. As mentioned previously, the ASTM A307 Grade C specification has been replaced by the F1554 specification in ASTM (and in the AASHTO specifications). ASTM F1554 Grade 36 is manufactured from low-carbon steel just like ASTM A307 Grade C, but in addition to being a bent or straight anchor rod, can also be a headed rod that is embedded in concrete and used for anchoring purposes. Most commercially available all-thread rod that satisfies ASTM A307 will not satisfy ASTM F1554 Grade 36.

2.4 Wires and Cables

Cables used in bridge construction are generally referred to as bridge strand (ASTM A586) or bridge rope (ASTM A603) [48, 49]. 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 are 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 (ASTM A416) is used in some structural steel applications although its primary use is for prestressed concrete [50]. 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 (ASTM 722) are another product that has applications for steel construction although their primary use is in prestressed concrete [51]. Although they do not meet the definition of a wire or cable, high-strength steel 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.

2.5 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. Cast iron castings are to conform to ASTM A48, Class 30 [52]. 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 iron and wrought iron are generally considered to be non-weldable, although some materials can be welded using special techniques.

Steel castings, not to be confused with cast iron or cast ductile iron castings, can have compositions and mechanical properties similar to those of structural steel grades. In bridges, steel castings can be used to create nodes or member end connectors to which conventional steel shapes can be joined by welding or bolting. Steel castings are also commonly employed as connectors and deviators in various wire, cable, and rod systems. They are often used in bearing applications and also as machine parts for movable bridges.

Steel castings need to be engineered (which includes the definition of geometric shaping and tolerances and the specification of production requirements including material selection and non-destructive examination requirements, at a minimum) to provide adequate strength, ductility, weldability, and toughness for the intended application. Several ASTM specifications apply to steel casting manufacturing and non-destructive testing. Commonly specified material grades for steel castings in structural applications include the following: ASTM A216 Grades WCA, WCB or WCC [53], ASTM A352 Grades LCA, LCB or LCC [54], or ASTM A958 Grade SC8620 Class 80/50 [55].

2.6 Pins, Rollers, and Rockers

Steel for pins, rollers, and rockers is to conform to the requirements specified in *AASHTO LRFD BDS* Table 6.4.2-1 or 6.4.1-1, or in *AASHTO LRFD BDS* Article 6.4.7 (stainless steel), as applicable.

2.7 Historical Steels

Steel bridge materials have evolved from 1901 when ASTM A7 steel (with a specified minimum yield strength of 33 ksi) was first introduced into the ASTM Specifications until the present day with the advent of High-Performance Steels (HPS). For further information on historical steels, the reader is referred to *AISC Design Guide 15: Rehabilitation and Retrofit* [56] and the FHWA publication entitled "Historical Changes to Steel Bridge Design, Composition, and Properties" [57]. Historical dimensions and section properties for rolled shapes produced from 1870 until 2010 is available in the *AISC-Shapes-Database* [58]. From roughly 1900 to the mid-1960s, individual steel producers marketed their own grades of steel. Sometimes these grades were loosely based on an ASTM specification or were the precursor to an ASTM specification. Regardless, they were proprietary, and sometimes the specific names of the alloys were written on bridge plans. The companies that produced these steels no longer exist or have been purchased by other companies, and these names are no longer used. If proprietary alloys are identified on bridge plans, engineers should consult the manufacturer's original literature for mechanical and chemical properties.

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 semi-furnished products, and processing these ingots, blooms, billets, or slabs through rollers to form finished plates or structural shapes. This basic process for steelmaking 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 determines 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 (Table 4). 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 A709 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, with potentially detrimental effects to material toughness and fatigue resistance. 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 (Section 3.2.1).

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 steelmaking is to pour molten steel into molds and cast it into ingots. These ingots are then removed from their molds, reheated, and rolled into rectangular cross-sections called slabs, blooms, or billets 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 steelmaking today is done by the process of continuous casting. Continuous casting machines were developed to directly cast molten steel into semi-finished slabs, blooms, billets, or beam blanks (also referred to as “dog bones”) 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 or section emerges from the mold. The continuous slab/section 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/sections from the ingots is eliminated. Consequently, continuous casting produces more uniform steel products and improves the cost-effectiveness of steelmaking.

3.1.3 Rolling

The cast slabs/sections are reheated and passed through rollers to form the slabs/sections 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 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 steelmaking. 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 for plates. 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.

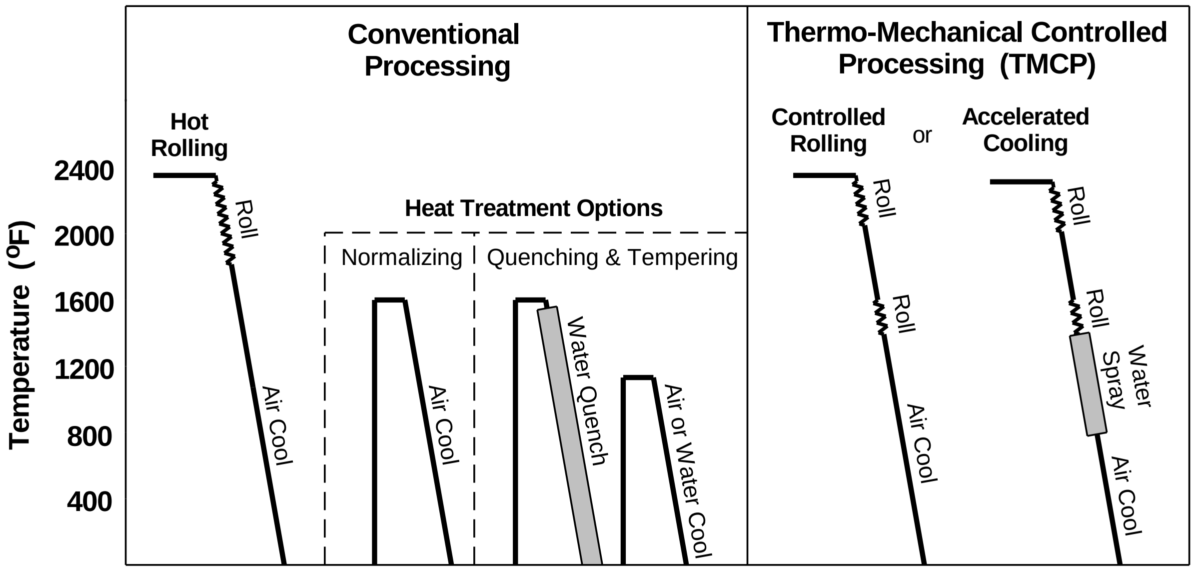


Figure 1 Relative Temperature-time History for Plate-Rolling and Heat-Treating Processes

3.2 Quality Control Measures

The ASTM A6 specification specifies that structural products are to be free of injurious defects and are to have a workmanlike finish [5]. Visual inspection is the usual requirement for inspection of the surface of plates, although the definition of injurious defects is vague. Surface roughness and dimples due to rolling in the 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 obtain adequate performance with respect to the fatigue and fracture limit state. Segregation can lead to variability in mechanical properties.

3.2.1 Deoxidation

Dissolved oxygen combines with carbon to form carbon dioxide and carbon monoxide gas in the molten steel. During solidification, the solubility of these gasses 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, 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. In the ASTM A709 specification, for all grades, the steel is to be killed.

Grades 50W, QST 65, QST 70, 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 A6. 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.

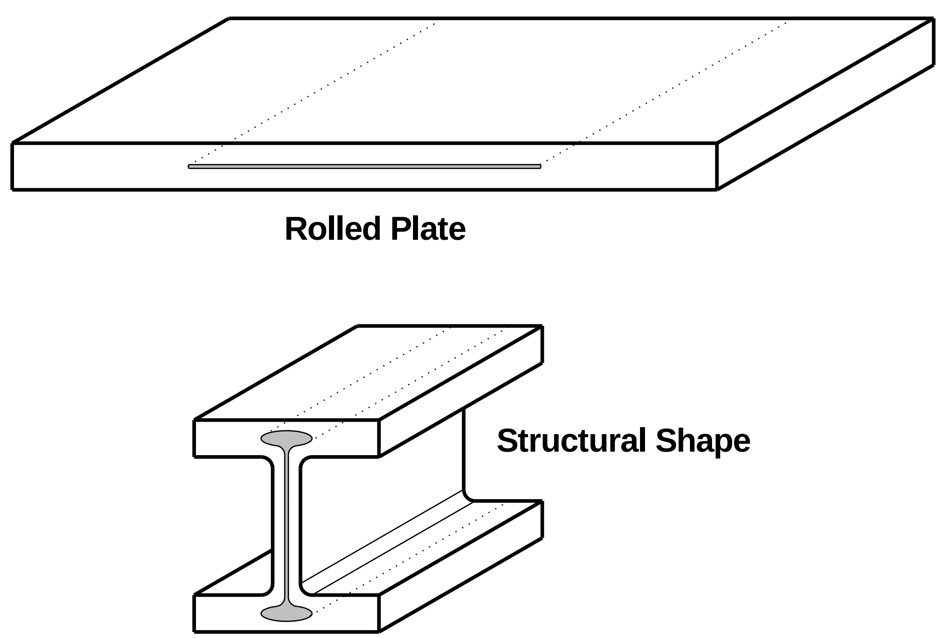


Figure 2 Segregation Causes Planar Inclusions at Mid-thickness Location of Steel Products

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 that may potentially result in lamellar tearing. 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 ASTM A709 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 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 Quenching 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 followed by tempering, where the steel is reheated to between 1,050°F and 1,300°F depending on Grade requirements, 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 for plates 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. Like controlled rolling, TMCP processing is usually limited to plates less than 2 in. thick. 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°F and 1,300°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 to achieve mechanical properties will list the tempering temperature on the mill report. As addressed in the AASHTO/AWS D1.5 Bridge Welding Code [10], fabrication heating options are not allowed to exceed the tempering temperature, thus avoiding degradation of the steel mechanical properties. 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 A370 specification [59] defines requirements for application of the ASTM E8 [60] 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.

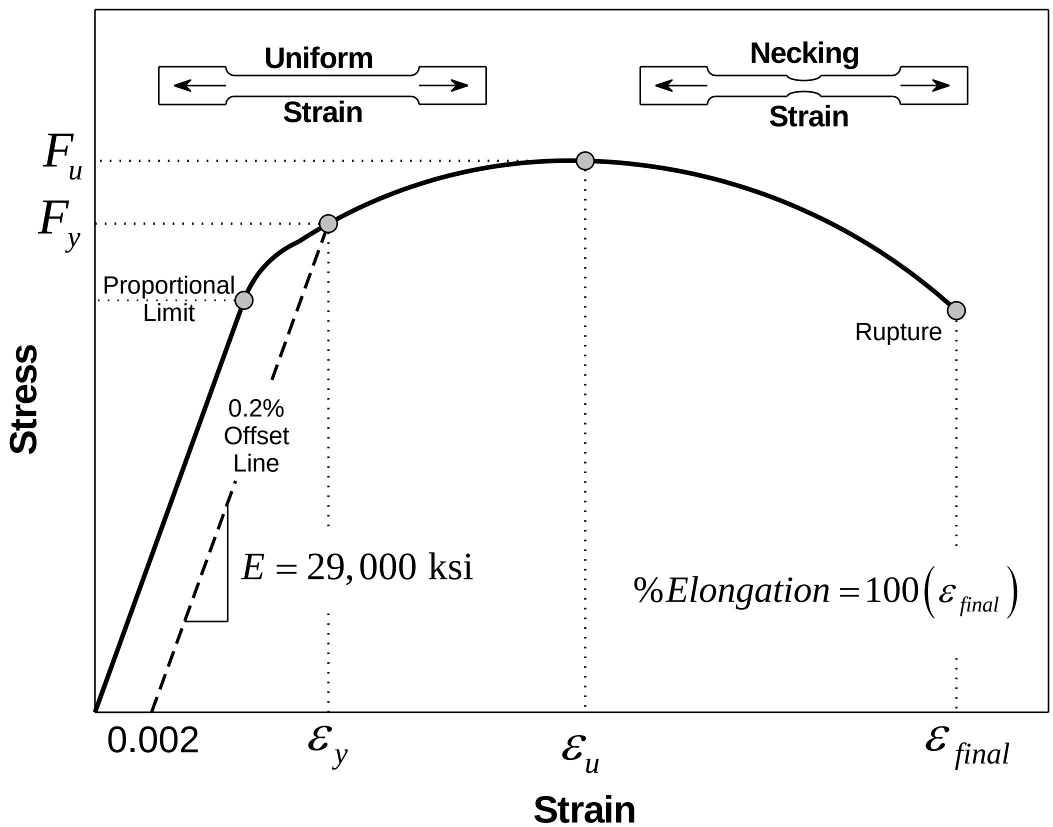


Figure 3 Engineering Stress vs. Strain Curve for Structural Steels without a Defined Yield Plateau

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 E8 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 E111 standard [61] 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, and 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, which are the more common structural steels utilized in plates and shapes. 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.

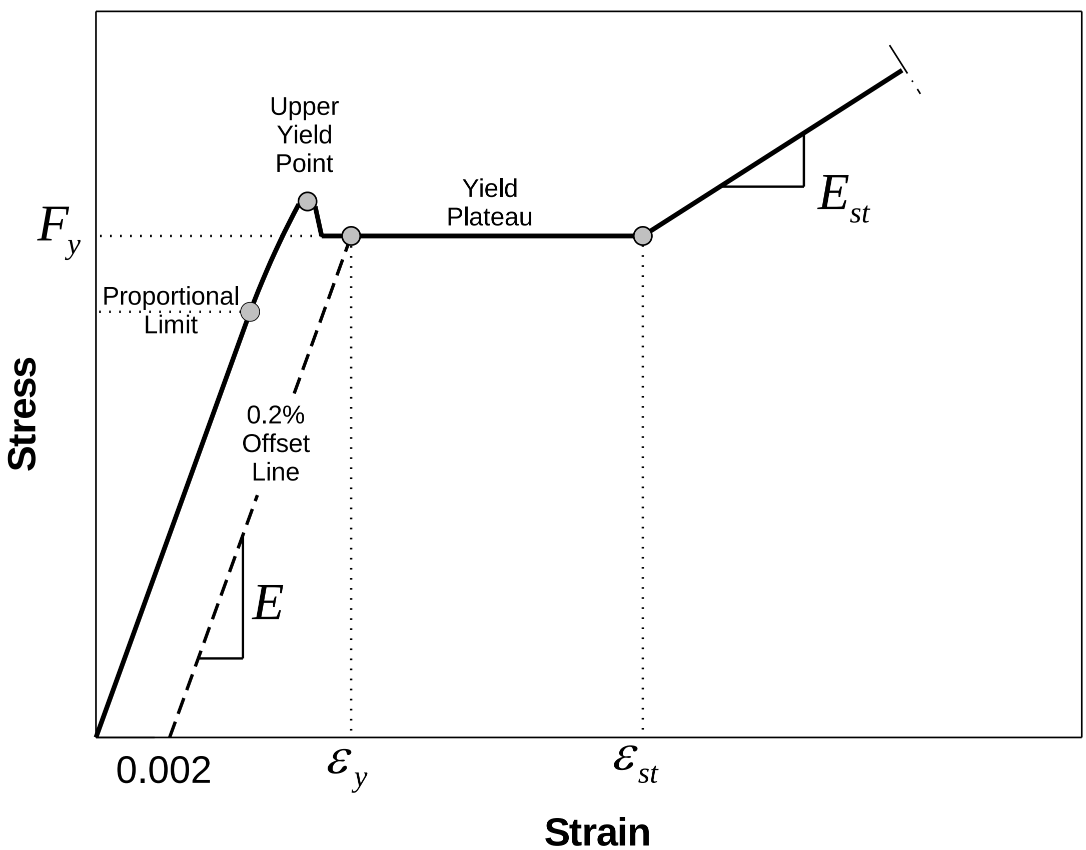


Figure 4 Calculation of Parameters for Steels with a Yield Plateau

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 undeformed 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.

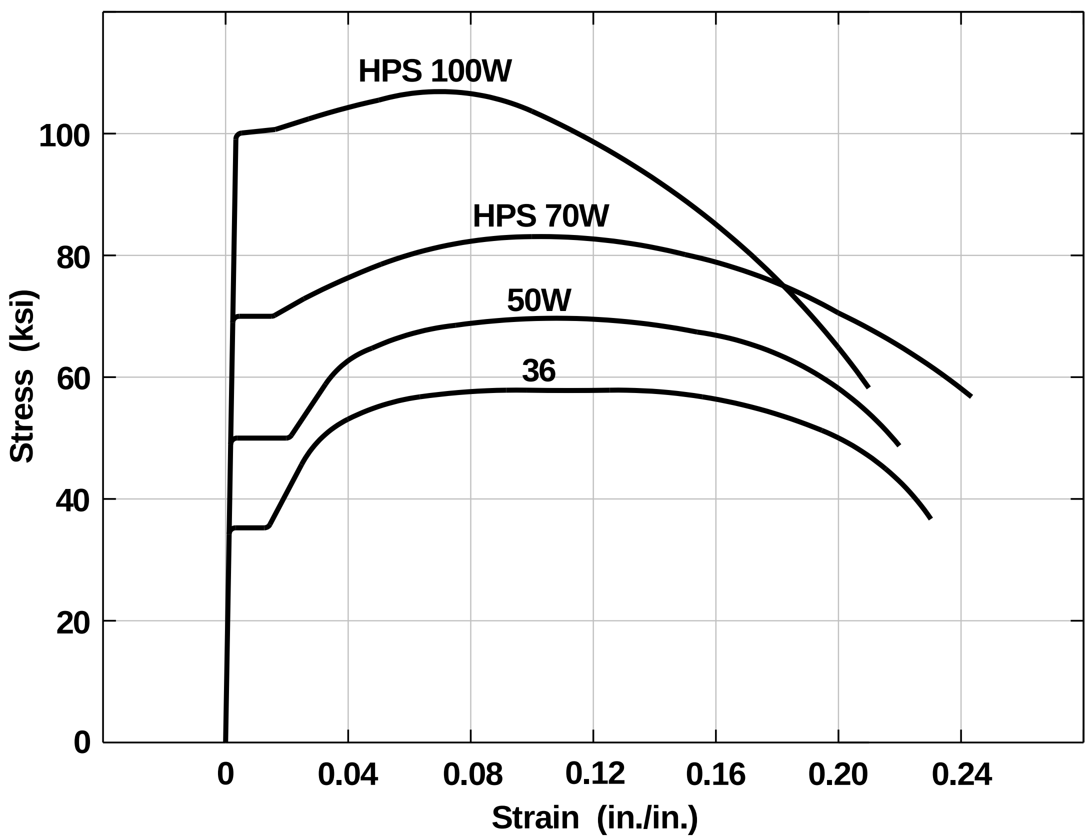


Figure 5 Typical Engineering Stress-Strain Curves for Structural Bridge Steels

Figure 5 shows typical stress-strain curves for steels in the ASTM A709 specification. Steels with $F_y \leq 70$ ksi show definite yield plateaus with similar ductility. The Grade 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 ASTM A709 specification. Plates with thickness up to 4 inches are available in all grades for which plates are available (except for Grade 50CR). As mentioned previously, rolled shapes are not available in the HPS grades or in Grade 50CR.

Table 5 Nominal Strength of ASTM A709 Steel Grades

Grade	**36**		**50**	**QST 50**	**50S QST 50S**	**50W**	**HPS 50W**
Plate Thickness (in.)	$t \leq 4.0$	N/A	$t \leq 4.0$	N/A	N/A	$t \leq 4.0$	$t \leq 4.0$
Shapes	$t \leq 3.0$	$t > 3.0$	All Groups	All Groups	All Groups	All Groups	N/A
F_u (ksi)	58-80	58	65	65	65	70	70
F_y (ksi)	36	36	50	50	50-65	50	50

Grade	**50CR**	**QST 65**	**QST 70**	**HPS 70W**	**HPS 100W**	
Plate Thickness (in.)	$t \leq 2.0$	N/A	N/A	$t \leq 4.0$	$t \leq 2.5$	$2.5 < t \leq 4.0$
Shapes	N/A	All Groups	All Groups	N/A	N/A	
F_u (ksi)	70	80	90	85-110	110-130	100-130
F_y (ksi)	50	65	70	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 € 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 [62] recommended the following equation for estimating the dynamic yield strength as an alternative to direct measurement:

$$\sigma_{YD} = \left[\sigma_{YS} + \frac{174{,}000}{\log\left(2\times10^{10}t\right)\left(T+459\right)} - 27.4 \right]$$

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 E399 Standard [63], 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 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 A370 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, which 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 [64, 65].

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 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. Methodologies for post-fire inspection of bridge structures were developed in NCHRP Project 12-85 [66]. 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 steelmaking practices. This does not present a problem for mechanically fastened or welded plates subjected to in-plane loading. It is possible in welded structures, however, to load plates in the through-thickness direction as shown in Figure 7. If significant inclusions are present, they can create a plane of weakness that can cause a plate to fail along the lamellar plane. Thicker plates are more susceptible to this phenomenon compared to thinner plates. Fortunately, modern steelmaking 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 [67].

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.

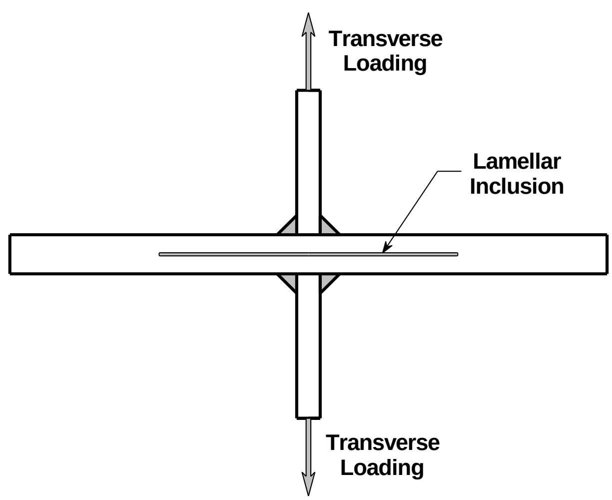


Figure 7 Lamellar Tearing Potential in a Plate Loaded in the Through-Thickness Direction

Lamellar tearing resistance is not addressed in the ASTM A6 and A709 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 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 more commonly used to assess the heat-treated condition of high strength steels when the heating history is not precisely known. For example, Grade 100 (ASTM A514) 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, ASTM A514 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 for steel members and connections to 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 designer's perspective, the ASTM A709 specification requires 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 plastic strain only 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 verify 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 [68]. 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 [68] concludes that the strength equations in the *AASHTO LRFD BDS* are valid to predict behavior for steels with YT ≤ 0.93. He also reports that steels with YT ≈ 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 ASTM A709 specification.

A 2003 study showed that Grade 50 and 50W structural plates produced in North American mills have YT ratios varying between 0.63 and 0.81 [69]. At higher strengths, the YT ratio typically increases, approaching YT ≈ 0.93 for Grade HPS 100W. The current *AASHTO LRFD BDS* does not allow use of an inelastic strength basis for steel with $F_y > 70$ ksi.

As mentioned above, for the steels specified in ASTM A709, there is no need for special consideration of the YT ratio for most bridge structural applications. Steels not covered by A709 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 ASTM A992 specification with a supplemental provision limiting YT to be less than or equal to 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 originally introduced a Fracture Control Plan in 1978 [70] in the aftermath of the Silver Bridge collapse in 1967 due to brittle fracture. The Fracture Control Plan is currently specified in Clause 12 of the AASHTO/AWS D1.5 Bridge Welding Code [10].

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. A primary member is defined in the *AASHTO LRFD BDS* as a steel member or component that transmits gravity loads through a necessary as-designed load path; considered synonymous with the term "main member". A fracture-critical member (FCM) is defined as a steel primary member or portion thereof subject to tension whose failure would probably cause a portion of or the entire bridge to collapse. These members are required to have higher levels of fracture toughness compared to non-fracture critical members. The designer is responsible for determining which, if any, member or component is without load-path redundancy and should therefore be classified as a fracture-critical member (FCM). The location of all FCMs is to be clearly indicated on the contract plans. A secondary member is defined as a steel member or component that does not transmit gravity loads through a necessary as-designed load path. Examples of secondary members include cross-frame and diaphragm members in straight bridges with or without skew, bearing plates, utility conduit hangers, and other members that are not part of the main structural system. Per *AASHTO LRFD BDS* Article 6.6.2.1, secondary members and diaphragm or cross-frame members in horizontally curved bridges (which are designated as primary members) are not required to have any specified fracture toughness and should also not be designated as FCMs.

Primary steel members or elements that are not subject to a net tensile stress under Strength Load Combination I are also not to be classified as FCMs.

AASHTO LRFD BDS Article 6.6.2.1 also requires that primary members or components, or portions thereof, subject to a net tensile stress under Strength Load Combination I be designated on the contract plans. Member or component designations (i.e., primary vs. secondary) are provided in Table 6.6.2.1-1. Proper designation of members or components is essential because this information affects various aspects of fabrication such as material purchasing, welding, and inspection. Arbitrary designation of secondary members or components, as listed in Table 6.6.2.1-1, as primary members or components is discouraged as this will invoke more costly and complex fabrication and testing requirements that do not add significant value.

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.

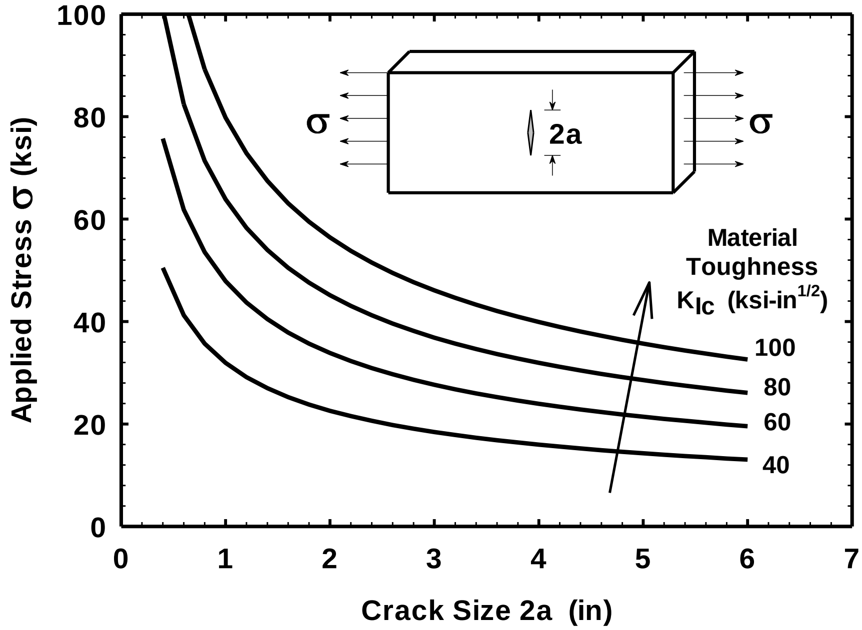


Figure 8 Basic Relationship between Applied Stress, Crack Size, and Material Fracture Toughness Based on LEFM. Increasing Material Toughness Raises the Fracture Prediction Curve

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 [71]. 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 constraint around the fracture zone of the CVN specimen is also typically more severe than found in bridges. Although plates thicker than the thickness of a CVN specimen are used in bridges, a minimum level of fracture toughness is reached in the CVN test, as the test represents a plane-strain condition. No further reduction in fracture toughness than the level attained in the CVN test is realized with increasing plate thickness.

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 [72, 73]. 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.

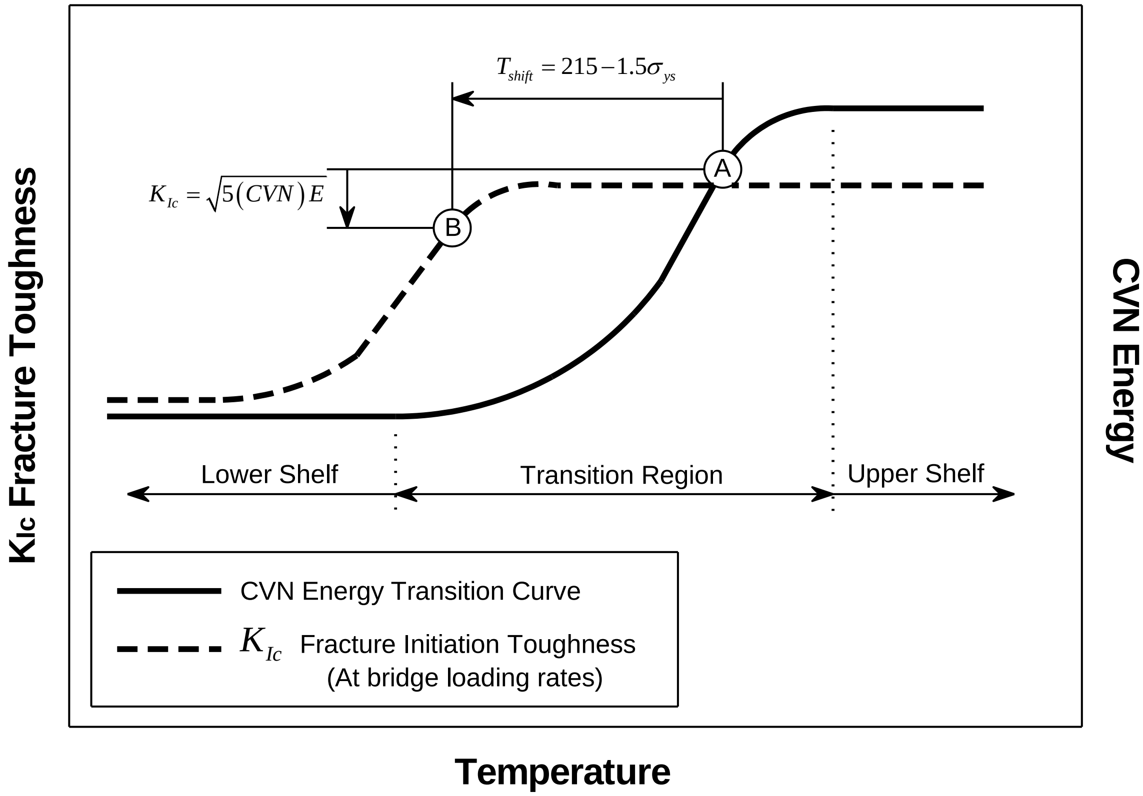


Figure 9 Effect of Temperature and Loading Rate on Fracture Toughness of Structural Steels

Barsom and Rolfe developed a two-step procedure to calculate the K_{Ic} toughness from CVN data [73]. 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 preceding 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 ASTM A709 were originally derived using the Barsom & Rolfe correlation procedure in the original AASHTO Fracture Control Plan [73,74]. 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 BDS* divides 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 (*AASHTO LRFD BDS* Table 6.6.2.1-2). The required fracture toughness increases as the minimum expected service temperature for the bridge decreases.

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 [70]. This document has been discontinued and the CVN toughness requirements for bridge steels are now maintained in the ASTM A709 specification (and are reproduced in Table C6.6.2.1-1 of the *AASHTO LRFD BDS*). The welding and fabrication quality control provisions of the Fracture Control Plan can now be found in Clause 12 of the AASHTO/AWS D1.5 Bridge Welding Code [10].

Table 7 shows the CVN toughness requirements for bridge steels given in Table C6.6.2.1-1 of the 9th Edition *AASHTO LRFD BDS*. These requirements are subject to periodic change and the current specification edition should always be consulted before using these values. Fracture toughness requirements for the various bridge steels are given in terms of the energy (in foot-pounds) absorbed by the CVN specimens at specified test temperatures for the three different temperature zones. The bridge fabricator is 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 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.

Table 7 *AASHTO LRFD BDS* Table C6.6.2.1-1 Fracture Toughness Requirements for Bridge Steels (9th Edition)

Grade (Y.P./Y.S.)	Thickness (in.)	Fracture-Critical				Nonfracture-Critical		
		Min. Test Value Energy (ft-lbf.)	Zone 1 (ft-lbf. @ °F)	Zone 2 (ft-lbf. @ °F)	Zone 3 (ft-lbf. @ °F)	Zone 1 (ft-lbf. @ °F)	Zone 2 (ft-lbf. @ °F)	Zone 3 (ft-lbf. @ °F)
36	$t \leq 4$	20	25 @ 70	25 @ 40	25 @ 10	15 @ 70	15 @ 40	15 @ 10
50/50S/ 50W	$t \leq 2$	20	25 @ 70	25 @ 40	25 @ 10	15 @ 70	15 @ 40	15 @ 10
	$2 < t \leq 4$	24	30 @ 70	30 @ 40	30 @ 10	20 @ 70	20 @ 40	20 @ 10
HPS 50W	$t \leq 4$	24	30 @ 10	30 @ 10	30 @ 10	20 @ 10	20 @ 10	20 @ 10
HPS 70W	$t \leq 4$	28	35 @ −10	35 @ −10	35 @ −10	25 @ −10	25 @ −10	25 @ −10
HPS 100W	$t \leq 2½$	28	35 @ −30	35 @ −30	35 @ −30	25 @ −30	25 @ −30	25 @ −30
	$2½ < t \leq 4$	36	not permitted	not permitted	not permitted	35 @ −30	35 @ −30	35 @ −30

The 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; that is, they are required to meet more stringent Zone 3 requirements in all three temperature zones. The actual toughness of HPS typically exceeds the specification requirements by a large margin.

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 [75, 76]. 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 nominal fatigue resistance for the applicable fatigue category, irrespective of steel grade. All bridge steel grades are therefore considered to have equivalent fatigue resistance. Further information on fatigue resistance is provided in NSBA's *Steel Bridge Design Handbook: Design for Fatigue* [77].

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 LRFD BDS* is based on the nominal yield and tensile strength "minimum" requirements. Most steel products are delivered with strength that exceeds the nominal minimums since steelmakers 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 [69]. 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 [5]. 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 A6 specification allows for a retest if any tensile test falls slightly below the nominal specification value (i.e., 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 *AASHTO LRFD BDS*.

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 complicated process 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 are 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 verify that 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 specified in ASTM A6. 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 LRFD BDS* for flexural and compression members all consider residual stresses in their formulation. Likewise, residual stresses are inherently embedded 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.

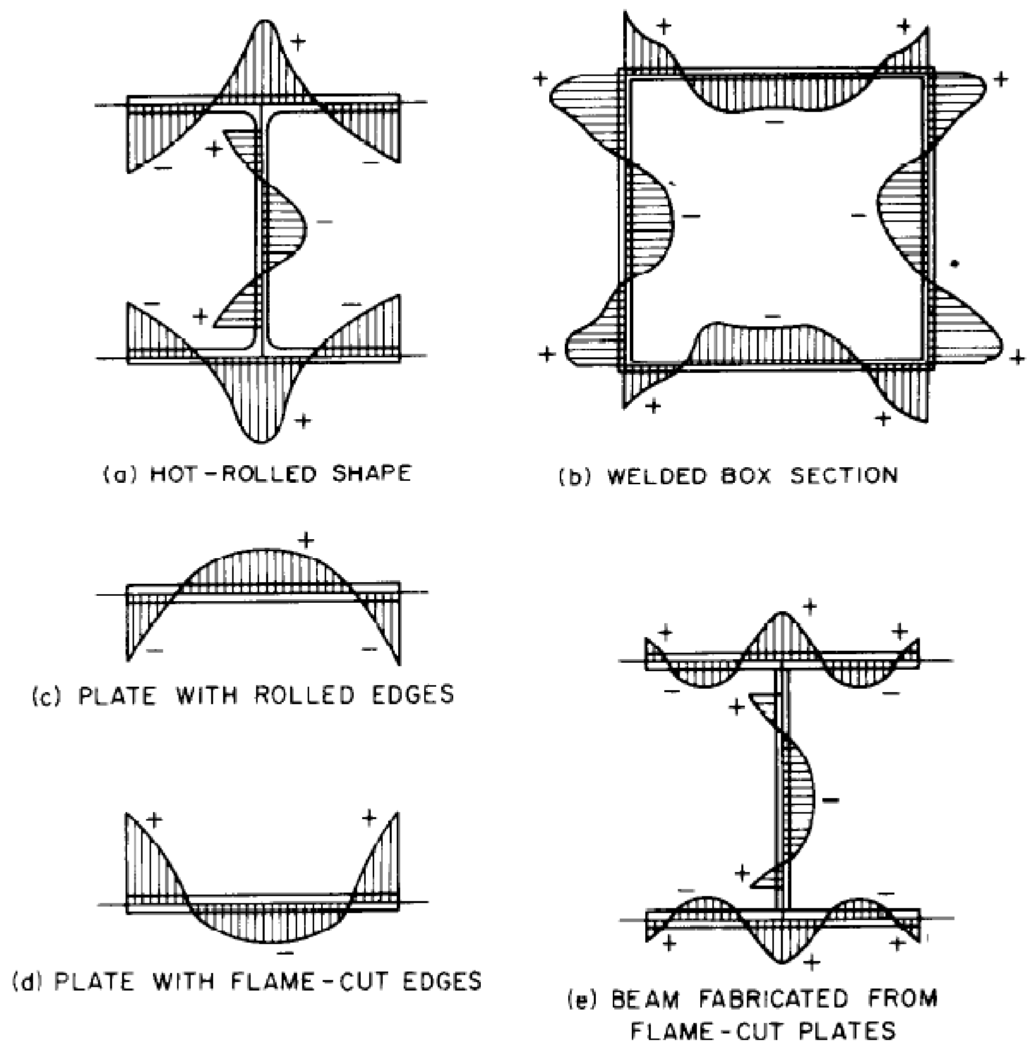


Figure 10 Typical Residual Stress Distributions in Rolled Shapes, Plates, and Built-up Members

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 along 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, the failure strain will be reduced to A4 – A3 indicating a notable loss of ductility.

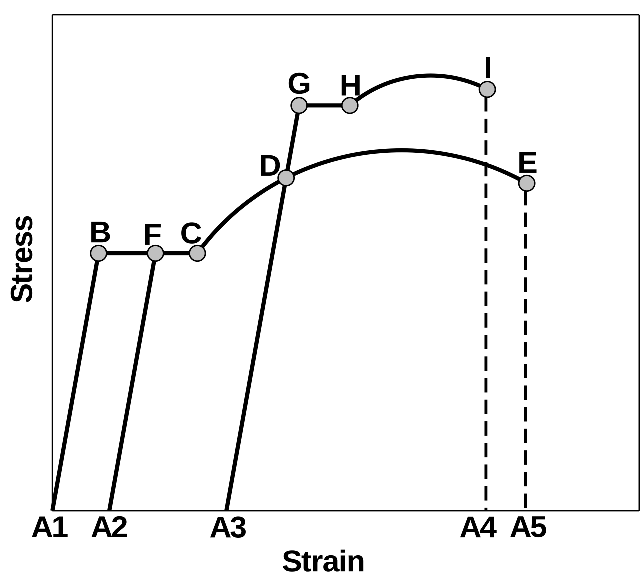


Figure 11 Stress-Strain Behavior Showing the Effects of Strain Hardening and Strain Aging

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 A6 specification [5]. The default tension 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 S60 in the ASTM A709 specification. CVN testing is covered in supplemental requirement S5 of ASTM A6, and is required for structural products ordered for use as tension components of non-fracture-critical or fracture-critical members according to the ASTM A709 specification. 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 prescribed in the ASTM A370 specification [59]. 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 is 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 ASTM A6 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 ASTM A709 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 Grade HPS 100W and quenched and tempered (heat treated) versions of Grades 50CR, HPS 50W and HPS 70W.

4.13.2 Charpy V-Notch Testing

According to *AASHTO LRFD BDS* Article 6.6.2.1, unless otherwise indicated on the contract plans, Charpy V-notch testing is required for primary members or components that are subject to a net tensile stress, or for portions thereof located in designated tension zones, under Strength Load Combination I, except for diaphragm and cross-frame members and mechanically fastened or welded cross-frame gusset plates in horizontally curved bridges. Certain primary members or portions thereof subject to tension may be further designated as fracture-critical members (FCMs) and are subject to more stringent CVN testing requirements. As mentioned previously, it is the responsibility of the design engineer to designate which members or components are fracture critical on the contract plans. All the preceding members or components are required to satisfy the applicable CVN impact toughness requirements shown in Table 7.

Once members are designated as either non-fracture-critical (T) or fracture-critical (F), steel is required to be ordered by the fabricator with the corresponding suffix (T or F) according to the ASTM A709 specification. The suffix (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 A709 Grade 50-T2. A Grade HPS 70W fracture-critical plate for use in Temperature Zone 3 is designated as A709 Grade HPS 70W-F3. Steel grades ordered for use without the suffix (T or F) do not require impact testing.

The ASTM A673 specification governs the CVN sampling and testing requirements [78]. 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 [79]. The sampling frequency requirements that are specified in the ASTM A709 specification 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 Shape/ As-Rolled from a Bloom, Billet or Beam Blank;	Per Plate/Shape As Heat Treated	
		Per Plate/As-Rolled from a Slab; Control-Rolled; or TMCP	Normalized	Q&T
NON-FRACTURE CRITICAL				
36, 50, 50S, 50W, 50CR, HPS 50W	H	One end	One end	One end
HPS70W	P	One End	N/A	One End
QST 50, QST 50S, QST 65, QST 70, HPS 100W	P	N/A	N/A	One End
FRACTURE CRITICAL				
50S	P	One End	One End	N/A
QST 50, QST 50S, QST65, QST70	P	N/A	N/A	Both Ends
50CR	P	N/A	One End	Both Ends
36, 50, 50W	P	Both Ends	One End	N/A
HPS 50W, HPS 70W	P	Both Ends	N/A	Both Ends
HPS 100W	P	N/A	N/A	Both Ends

*H = one plate/shape as-rolled or heat treated per each 50 tons from a heat of steel. P = each plate/shape as-rolled or heat treated.

In the ASTM A673 specification, a CVN impact test is defined as testing three replicate CVN specimens from the same location at the same testing temperature. Per ASTM A709, if the yield point of the structural steel exceeds the specified minimum value by 15 ksi or more, the testing temperature is to be reduced by 15 degrees for each increment or fraction of 10 ksi above the 15 ksi exceedance of the specified minimum value. The average of the three specimens must be greater than the specified minimum average 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 ASTM A673. 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 Weldability

5.1.1 General

Weldability can be generally defined as the ability of a steel to be welded. All modern structural bridge steels are weldable. The welding specification governing highway structures is the AASHTO/AWS D1.5 Bridge Welding Code [10]. The AASHTO/AWS D1.5 specification is more restrictive in some respects compared to the more general AWS D1.1 Structural Welding Code [33] since bridge welds must perform relative to the fatigue and fracture limit states. Following the D1.5 provisions, all bridge steels in the ASTM A709 specification, except for Grade 50CR and the QST steels (which require special provisions), are approved for welding under the AASHTO/AWS D1.5 specification. The weldability of steel Grades 36, 50, 50W, 50S, HPS 50W, HPS 70W and HPS 100W has 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, the conventional Grades 70W and 100W developed a bad reputation and designers were reluctant to utilize them for bridge design. Fabricator experience with the HPS grades has been excellent, and many fabricators report they are at least as weldable as the conventional lower strength grades. As discussed previously, Grades HPS 70W and HPS 100W have now replaced conventional Grades 70W and 100W in the ASTM A709 specification.

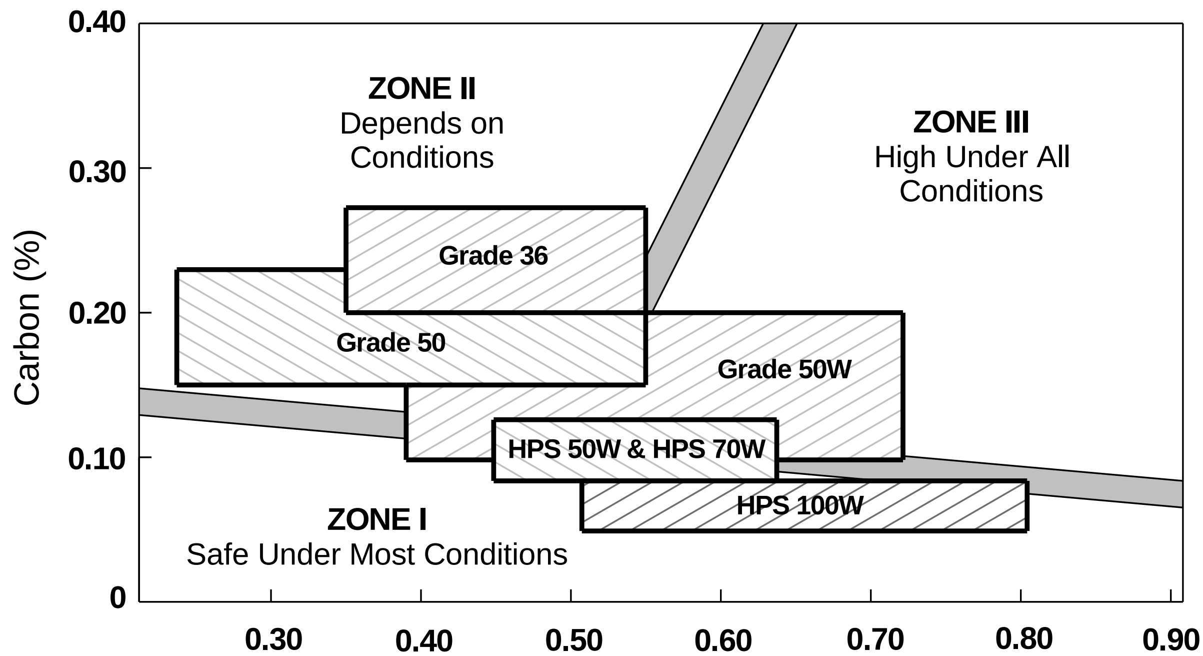
The welding provisions for the HPS grades were developed through the research activities of the FHWA/USN/AISI Welding Advisory Group. The group developed and evaluated research and fabrication experience with the HPS grades and prepared the AASHTO *Guide Specification for Highway Bridge Fabrication with HPS 70W (485W) Steel* [80] (now archived by AASHTO), which eventually formed the basis for the current Annex H in D1.5.

Further information on the technical aspects of welding of highway structures, with an emphasis on steel highway bridges, and on the AASHTO/AWS D1.5 Bridge Welding Code [10] may be found in the FHWA *Bridge Welding Reference Manual* [81].

5.1.2 Base Metal Chemistry and Carbon Equivalent

The weldability of structural steels is largely dependent on the chemical composition. Graville categorized the susceptibility of steel to heat affected zone (HAZ) cracking, suggesting it is dependent on both the carbon content and the carbon equivalent. The carbon equivalent (CE) is calculated by a formula which considers other alloying elements in addition to carbon [82]. Graville's weldability diagram (Figure 12) shows that weldability can be divided into three general classification zones (depending on chemical composition) as denoted by the gray bands. The x-axis in Figure 12 shows the CE equation recommended by the AASHTO/AWS D1.5 Bridge Welding Code [10] to assess weldability of various structural steels.

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 ASTM A709 specification but typically falls within Zone II. The HPS grades, primarily due to their low carbon formulations, tend to fall into Zone I indicating improved weldability compared to conventional grades. This indicates that HPS steels have a low probability of 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 Relative Susceptibility to HAZ Cracking of Bridge Steels

5.2 Fabrication

5.2.1 Thermal Cutting

All structural steels in the ASTM A709 specification are suitable for thermal cutting. The oxy-fuel gas process is the most widely used process in bridge fabrication since this process is capable of cutting all plate thicknesses used in bridge fabrication. Plasma arc (Figure 13) and carbon air-arc processes are also used in some cases for thinner plates. The plasma arc cutting process shown in Figure 13 is being performed by a CNC (Computer Numerical Control) machine, which is an important tool utilized in many aspects of modern fabrication. 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.

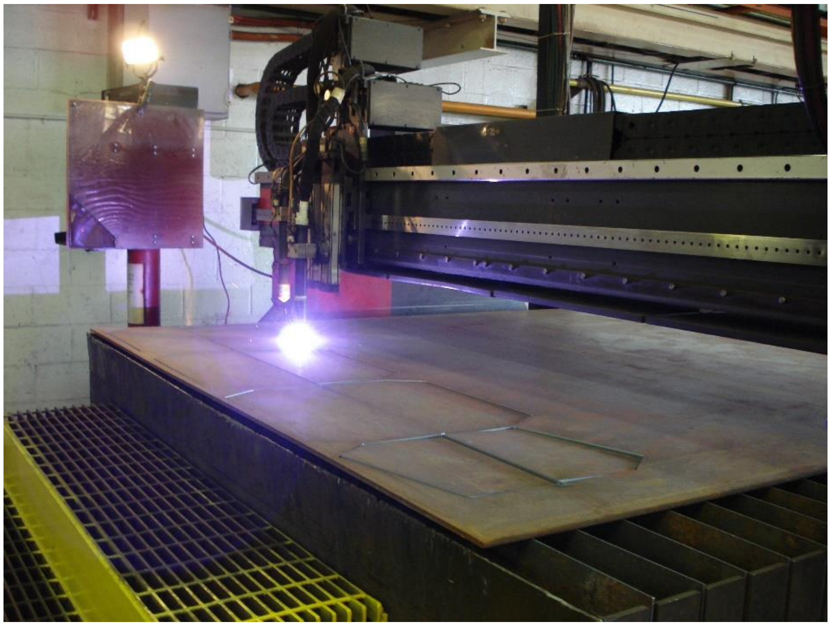


Figure 13 Thermal Cutting of Plate by the Plasma Arc Process using a CNC Machine (Photo Courtesy of High Steel Structures, Inc.)

The effect of thermal cutting on Grade A36, A572, and A588 plate was studied in the 1980s [83]. 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.

5.2.2 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. Holes may be made by punching, plasma-cutting, or water-jetting as described further in [37].

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.

5.2.3 Product Tolerances

All steel products from the mill are produced to meet the geometric tolerances prescribed in the ASTM A6 specification and discussed further in the following subsections. 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 ASTM A6 limits have been established to verify steel products are dimensionally and aesthetically adequate for use in bridges.

Tolerances for fabricated welded steel bridge construction are prescribed in the AASHTO/AWS D1.5 Bridge Welding Code [10], and include dimensional tolerances along with tolerances on alignment, camber, web flatness, offset, variation from straightness, and warpage.

5.2.3.1 Plate Thickness

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

5.2.3.2 Plate Flatness

Plates have requirements for both flatness and waviness as shown in Figure 14. Overall flatness is measured with the plates lying flat in a horizontal position. The ASTM A6 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 ASTM A6 requirements.

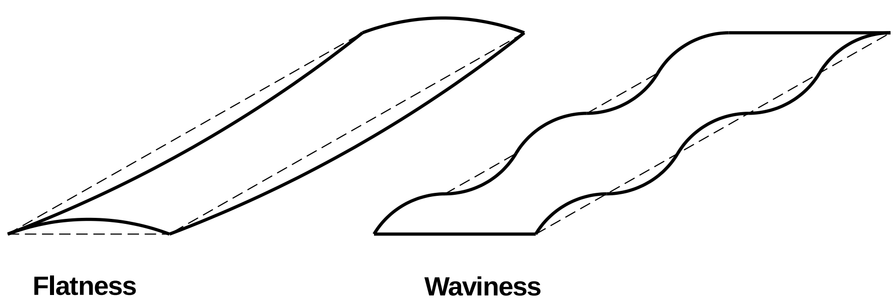


Figure 14 Illustration of Plate Flatness and Waviness

5.2.3.3 Rolled Shape Tolerances

Rolled shape tolerances are also prescribed in the ASTM A6 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 measured about both the strong axis (camber) and the weak axis (sweep) is 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 the *AASHTO LRFD BDS*, 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.

5.2.4 Cold Bending

As discussed in Section 4.12, the mechanical properties of structural steel change after the steel undergoes plastic deformation. The *AASHTO LRFD BDS* [37] prescribe limits on the minimum radius for cold bending measured to the concave face of a fracture-critical or non-fracture-critical plate or bar conforming to ASTM A709, 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 *AASHTO LRFD BDS* Article 11.4.3.3.2.

Application	**Direction of Bend**	**Thickness, in. (t)**	
		$t \leq 0.75$	$t > 0.75$
Cross-frame/ Diaphragm Connection	Parallel to Final Roll	*1.5t*	*7.5t*
	Perpendicular to Final Roll	*1.5t*	*5.0t*
Other	Parallel to Final Roll	*7.5t*	*7.5t*
	Perpendicular to Final Roll	*5.0t*	*5.0t*

Web splice plates, fillers, gusset plates not serving as chord splices, connection plates, and web stiffeners are not included in the rolling direction requirement in Table 9. 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 potentially have an effect on the performance of tension members designated as fracture-critical or non-fracture-critical. Per the *AASHTO LRFD BDS*, hot-bending does not reduce the minimum bending radius limit.

5.2.5 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 (Grades 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 should not exceed the tempering temperature reported on the mill certificate, if it is less than 1,100°F. The girder is not to be artificially cooled until after naturally cooling to 600°F.

6.0 CORROSION RESISTANCE

Steel grades with the "W" suffix are called "weathering" steels because they demonstrate enhanced atmospheric corrosion resistance. In many environments, weathering steels can be used without paint or other protective 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 G101 specification was developed to standardize a methodology for classification of steels as weathering [84]. A corrosion index (I) is calculated based on the chemical composition of the steel. The ASTM A709 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 G101 was developed by Legault and Leckie to be valid for steels with composition close to Grade 50W (ASTM A588):

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

More recently, Townsend introduced an alternate equation in ASTM G101 for steels that exceed the validity limits of the Legault-Leckie equation. This became important with the introduction of the HPS grades (Grades HPS 50W, HPS 70W, and HPS 100W). The HPS grades have a higher percent of copper (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 ASTM G101 specification.

Although classification of a steel as "weathering" indicates enhanced atmospheric corrosion resistance, weathering steel may not perform well in bridge locations subjected to high time-of-wetness or exposure to high levels of chlorides. Grade 50CR steel (Section 2.1.7) may be more appropriate for such applications. 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 [85]. Figure 15 shows the Townsend corrosion index value for various weathering steel grades as predicted by the ASTM A709 chemical composition requirements [86].

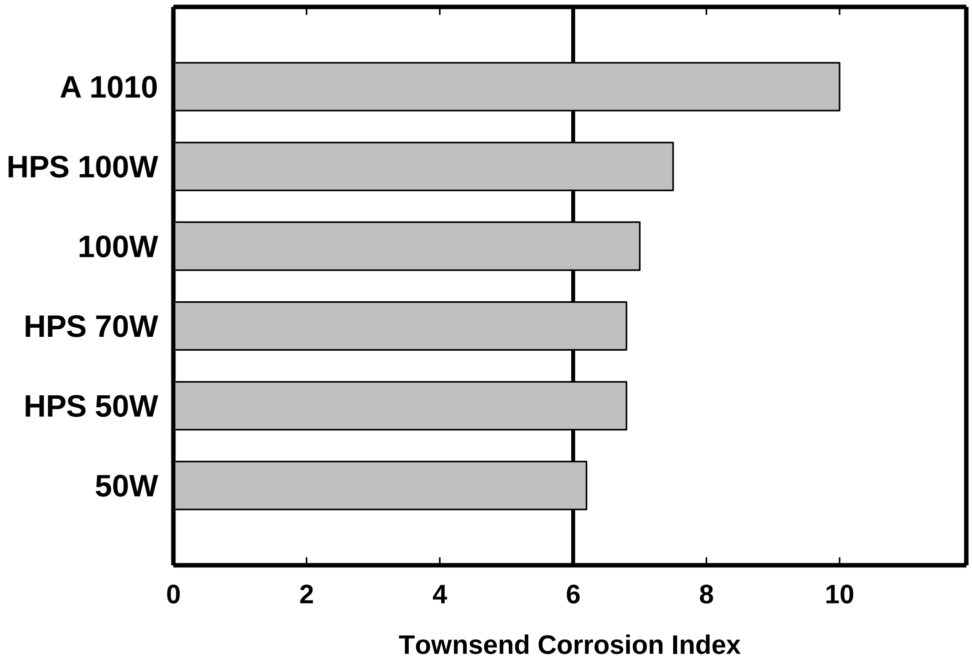


Figure 15 Comparison of the Typical Corrosion Index between Different Grades of Weathering Steels Based on the Townsend Corrosion Index

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. 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.

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