Historical Changes to Steel Bridge Design, Composition, and Properties

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FOREWORD

This report documents a historical review of the evolution of steel composition, steel properties, and steel bridge design from 1900 to 2015. This effort was conducted in support of the Federal Highway Administration Long-Term Bridge Performance Program to identify changes in steel bridges that may be linked to structural performance. The results documented herein may benefit those generally interested in all aspects of steel bridge design, fabrication, and construction, including State transportation departments, researchers, and design consultants.

Cheryl Allen Richter, Ph.D., P.E.
Director, Office of Infrastructure
Research and Development

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<tr><td colspan="7">16. Abstract
This report presents a succinct view of historical steel bridge design with an emphasis on the evolution of the steel itself, including a summary of American Society for Testing and Materials and American Association of State Highway Officials/American Association of State Highway and Transportation Officials steel specifications used in steel bridges and their tensile, chemical, and impact energy requirements. A timeline of major design changes, such as welding and bolting, composite design, and fatigue design, is presented.</td></tr>
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SI* (MODERN METRIC) CONVERSION FACTORS				
APPROXIMATE CONVERSIONS TO SI UNITS				
Symbol	**When You Know**	**Multiply By**	**To Find**	**Symbol**
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1,000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2,000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa
APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	**When You Know**	**Multiply By**	**To Find**	**Symbol**
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2,000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	2.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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LIST OF ABBREVIATIONS AND SYMBOLS

Abbreviations

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ASD	allowable stress design
ASTM	American Society for Testing and Materials
AWS	American Welding Society
CE	carbon equivalency
CVN	Charpy V-notch
EGW	electrogas welding
ESW	electroslag welding
FC	fracture critical
FCAW	flux cored arc welding
FCM	fracture-critical member
FHWA	Federal Highway Administration
GMAW	gas metal arc welding
HPS	high-performance steel
LAST	lowest anticipated service temperature
LFD	load factor design
LRFD	load and resistance factor design
LTBPP	Long-Term Bridge Performance Program
NFC	non-fracture critical
RCRBSJ	Research Council on Riveted and Bolted Structural Joints
SAW	submerged arc welding
SMAW	shielded metal arc welding
S-N	stress–life

Symbols

A_{sc}	cross-sectional area of welded stud
B	fatigue coefficient for channel shear connector
C	carbon content
Cr	chromium content
Cu	copper content
d	diameter of stud
E_c	modulus of concrete
f_c	compressive strength of concrete
F_u	tensile strength
F_y	yield strength
H	height of stud
h	average flange thickness of channel
Mn	manganese content
Mo	molybdenum content
Ni	nickel content
Si	silicon content
t	web thickness of channel

V	vanadium content
w	length of channel
Z_r	shear connector force range resistance
α	fatigue coefficient for stud shear connector

INTRODUCTION

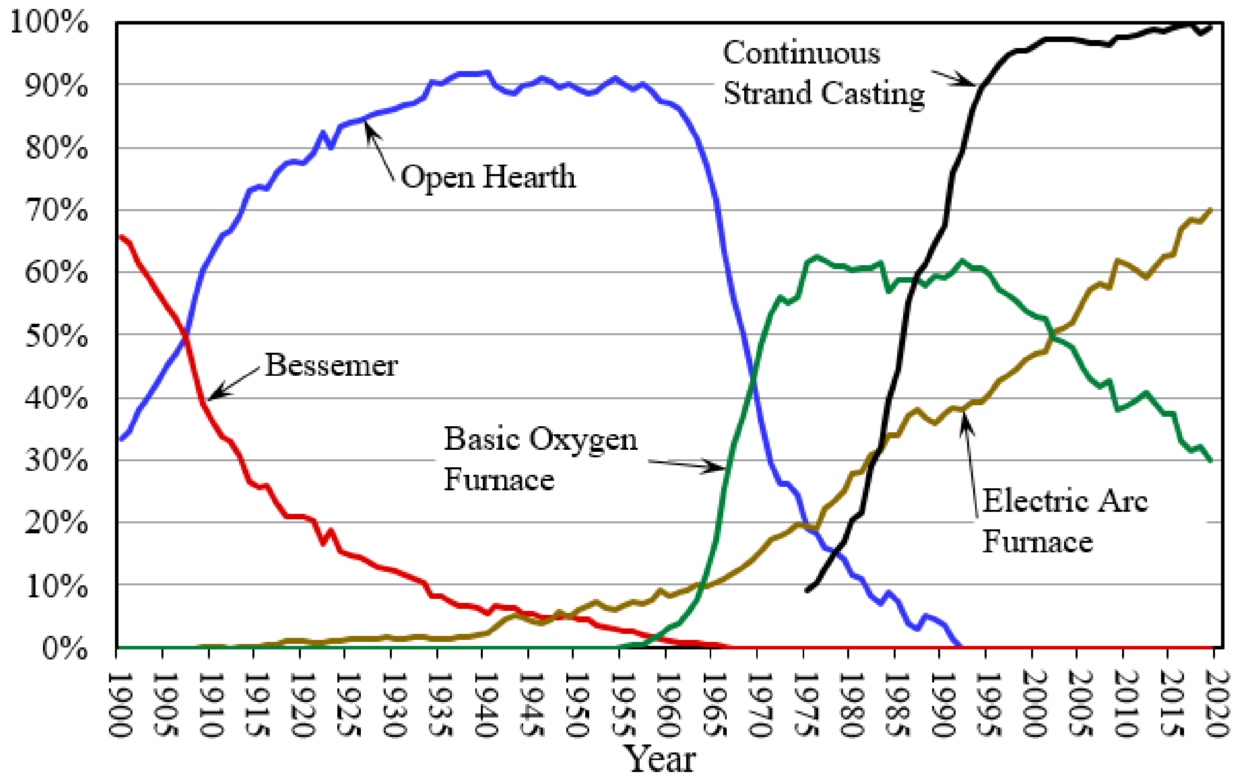
This report was written as part of the Federal Highway Administration (FHWA) Long-Term Bridge Performance Program (LTBPP). The LTBPP is a minimum 20-yr research effort authorized by the U.S. Congress to collect high-quality bridge data from a representative sample of highway bridges nationwide to help the bridge community better understand bridge deterioration and performance. The results and products from this program include a collection of data-driven tools such as predictive and forecasting models that will enhance bridge owners' abilities to optimize their management of bridges.

The LTBPP is collecting field data from bridges constructed from 1960 to the present. This is a long timeframe—more than 50 yr. Because the LTBPP not only collects but also analyzes the data, it is important to view the data in their proper context. Nationally, bridge technologies have changed, and new innovations have arisen that have advanced the state of the art for bridge engineering. It is important to record when these innovations and changes in bridge technology occurred to better understand why the performance data may differ for bridges built in 1960 and the present. For example, if a bridge built in 1965 is outperforming a bridge built in 1978 (or vice versa), it would be helpful to understand what innovations and changes in practice that could affect bridge performance occurred between these two dates.

This document was written to provide context to the evolution of steel bridge design and steel bridge materials. It discusses the evolution of national practices about how steel is made, what types of steel are used in bridges (including chemical composition and mechanical properties), steel bridge design specifications, and welding and mechanical joining. This report examines data between 1900 and 2015 (which is beyond the scope of the LTBPP) because this longer timeframe was required to more fully understand the context of steel bridge evolution.

STEEL PRODUCTION

In the United States, steel has been produced by melting ore or scrap metal using one of five major processes: crucible, Bessemer, open hearth, basic oxygen furnace, and electric arc furnace. Each of these melting processes was most common during a particular time period and then fell out of use as another method became more popular. The rise and fall of these production methods is shown in figure 1. The data come from the U.S. Geological Survey *Minerals Yearbook* published annually and its predecessor, *Mineral Resources of the United States*, before 1932.[1,2]



Source: FHWA.
Note: Continuous strand casting is a casting process; therefore, it should not be considered in the summation of the five melting processes for creating molten steel.

Figure 1. Graph. Proportion of United States crude steel production by year.[1,2]

CRUCIBLE MELTING PROCESS

The crucible process was an early method for making steel. In this method, iron and carbon are melted together in containers, or crucibles, typically made from clay-type materials that hold their form at the temperatures required to melt steel. The process is not shown in figure 1 because in 1900 it was only 1 percent of the total proportion of steel production, and by 1925 its proportion was no longer tracked. It is doubtful that any steel bridge built after 1900 would have been made from crucible steel.

BESSEMER MELTING PROCESS

The Bessemer process was named after Henry Bessemer, who found that blowing air through the molten steel helped to rapidly and completely oxidize (i.e., burn away) the contaminants within the steel. The oxidation of carbon and silicon in the molten steel also contributes to heat generation that further increases the molten metal temperatures, which further increases the steel's purity. Furthermore, the refractory linings of the Bessemer converter react chemically with the molten steel to reduce the phosphorus out of the metal. The process is fast, typically requiring only about 20 min to produce pure molten iron, and then other ferromanganese alloys are added to attain the desired steel chemistry.(3) The Bessemer process was a critical development that drastically reduced the cost of making steel because scale and speed could be increased. However, blowing atmospheric air through the steel tends to create brittleness because of the nitrogen content in the air. Therefore, the Bessemer process was not allowed in specifications for structural steel. The Bessemer process offered great advantages over the crucible process in terms of productivity; however, by the early 1900s, its use began to decline in favor of the open hearth process, as shown in figure 1.

OPEN HEARTH MELTING PROCESS

The open hearth process involves a shallow pool of molten metal with an open flame above it. The process is regenerative, using two regenerators that are large masses of refractory brick below the pool of metal. In one regenerator, the exhaust gases heat up refractory brick, while in the other regenerator, fuel is preheated by the refractory brick within it. This process increases the efficiency of fuel use and allows the fuel to burn hotter. Once the steel is molten, additional iron oxide ores can be metered in, providing a source of oxygen to purify the steel and preventing the introduction of nitrogen, a problem with the Bessemer process. A downside of the open hearth process is that it is slow, typically requiring about 8 h for one cycle; however, the pool sizes can be quite large. Eventually the open hearth process became more popular than the Bessemer process—it was the dominant means of making steel from roughly 1910 to 1960—and the last U.S. open hearth furnace closed in 1990.(3,4)

BASIC OXYGEN FURNACE MELTING PROCESS

The basic oxygen furnace process is not unlike the Bessemer process; however, it blows pure oxygen through the molten metal in lieu of air. This eliminates the nitrogen problem with Bessemer steel and allows for the greater productivity of the Bessemer process compared to the open hearth process.(4) The proportion of production using the basic oxygen furnace process began to increase in the early 1960s, coinciding with the ability to produce industrially pure oxygen at large scale.

ELECTRIC ARC FURNACE MELTING PROCESS

As shown in figure 1, the electric arc furnace process has been around for more than a century, and its use has steadily increased over time. In roughly 1995, production using the basic oxygen furnace process began to decline in favor of the electric arc furnace process. As of 2015, the electric arc furnace process produces a little more than 60 percent of the steel; the rest is produced by the basic oxygen furnace process. This process involves slowly lowering large electrodes into ferrous raw materials, causing an electric arc hot enough to melt the raw materials. The primary advantage of the electric arc furnace process is that it can work with 100-percent scrap metal feedstock (or scrap supplemented with raw ore), whereas the basic oxygen furnace process can only work efficiently with no more than 30-percent scrap.(4) Electric arc furnaces can also be turned on and off quickly, allowing them to operate when electricity is cheap or to rapidly adjust to various changes in feedstock.

CONTINUOUS STRAND CASTING PROCESS

Before the 1970s, molten steel was cast into large ingots, allowed to cool, reheated in a rolling mill, and then rolled out to final product dimensions. The problem with ingots is that their mass causes slow cooling rates, and the slow cooling results in segregation of elements toward the core of the ingot, which cools the slowest. This leads to deleterious chemistry at the center of the ingot, which gets rolled into the product during final rolling.(4) In the early 1970s, the continuous strand casting process was introduced. This process allows the cast product to have a smaller cross section, and because the strand is cast continuously, slabs can be cut to any length from the cast strand (compared to ingots, which had three fixed dimensions). Slabs cut from the continuous strand are not as susceptible to the segregation problem. The continuous strand casting process also allows the cast product to be closer in size to the final product dimension, reducing the amount of rolling required and therefore the cost. As shown in figure 1, tracking of the process began in 1974, and 20 yr after that, 90 percent of all steel was produced by the continuous strand casting process.

OTHER REFINEMENTS TO STEEL PRODUCTION

The largest proportion of the steel produced in the United States goes into the production of automobiles, so the requirements of the automotive industry dictated the evolution of steel processing. Automobiles are made by the pressing, forming, and deep drawing of thin gauge steel, and these fabrication processes require higher quality steel with tighter chemistry controls than structural steel to be effective and efficient. To meet these requirements, steel mills took several actions. First, they drastically reduced the sulfur content of steel because sulfur is one of the primary causes of nonmetallic inclusions in steel that cause reduced toughness and create laminar defects.(4) Second, in the 1980s, steel producers began sulfide shape control processing, which refined residual sulfide inclusions into more globular shapes rather than the elongated stringer structures that had deleterious effects on mechanical properties.(4) Lastly, there was a more cognizant effort to protect molten steel during casting. Before the early 1970s, molten steel was often cast in the open atmosphere, but as time went on, more steel producers began to shroud molten steel in argon atmospheres during casting to prevent oxidation of the molten metal. Steel quality has increased over time, and by the early 1980s, the quality of steel being used in steel bridge fabrication was much higher than that produced before the early 1970s.

MATERIAL SPECIFICATIONS

The American Society for Testing and Materials (ASTM) produced the first steel specifications, and after the American Association of State Highway Officials (AASHO) was formed, it also started maintaining its own material specifications (although these were verbatim copies of the equivalent ASTM specifications). In 1973, AASHO changed its name to the American Association of State Highway and Transportation Officials (AASHTO), but this change did not affect the numbering of its material specifications. There are two other important facts to note to describe the specification nomenclature used in this document. One, in 2001, ASTM changed its name to ASTM International, and "ASTM" is no longer considered an acronym. Two, historically, ASTM specifications were represented by an alphanumeric code starting with a letter representing the committee developing the specification followed by a space and a number assigned sequentially based on the next available number from that committee. For instance, ASTM A 709, *Standard Specification for Structural Steel for Bridges*, was first published in 1974.(5) This specification still exists; however, in 2009, ASTM renumbered all its specifications, eliminating the purposeful space between the letter and number, so this is now referred to as ASTM A709, not ASTM A 709. Therefore, throughout this document, references to various ASTM specifications may have a space or may not, depending on the specification's publication year.

In addition to ASTM A 709, the most common ASTM steel specifications historically used for bridges were the following:

- ASTM A 7, *Standard Specification for Steel for Bridges and Buildings*.(6)
- ASTM A 8, *Specification for Structural Nickel Steel*.(7)
- ASTM A 94, *Specification of Structural Silicon Steel*.(8)
- ASTM A 242, *Standard Specification for High-Strength Low-Alloy Structural Steel*.(9)
- ASTM A 373, *Specification for Structural Steel for Welding*.(10)
- ASTM A 440, *Specification for High-Strength Structural Steel*.(11)
- ASTM A 36, *Standard Specification for Carbon Structural Steel*.(12)
- ASTM A 441, *Standard Specification for High-Strength Low-Alloy Structural Manganese Vanadium Steel*.(13)
- ASTM A 514, *Standard Specification for High-Yield-Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding*.(14)
- ASTM A 517, *Standard Specification for Pressure Vessel Plates, Alloy Steel, High-Strength, Quenched and Tempered*.(15)

- ASTM A 572, *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*.[16]
- ASTM A 588, *Standard Specification for High-Strength Low-Alloy Structural Steel, up to 50 ksi [345 MPa] Minimum Yield Point, with Atmospheric Corrosion Resistance*.[17]
- ASTM A 852, *Standard Specification for Quenched and Tempered Low-Alloy Structural Steel Plate with 70 ksi [485 MPa] Minimum Yield Strength to 4 in. [100 mm] Thick*.[18]

Table 1 presents the year these common material specifications were published and withdrawn (if currently inactive) as well as the equivalent AASHO/AASHTO specifications. ASTM tended to first publish "tentative" specifications for further debate until fully adopted. Likewise, AASHO/AASHTO sometimes published "interim" material specifications that were approved by the Subcommittee of Materials but not the full AASHO/AASHTO committee. The columns in table 1 covering the year first published include tentative ASTM specifications and interim AASHO/AASHTO specifications; the year provided merely indicates the first time the specification became publicly available.

Table 1. Table of common ASTM and AASHO/AASHTO material specifications.

ASTM Specification	Year First Published by ASTM	Year Withdrawn by ASTM	Equivalent AASHO/AASHTO Specification	Year First Published by AASHO/AASHTO	Year Withdrawn by AASHO/AASHTO
A 7(6)	1901	1966	M 94(19)	1939	1966
A 8(7)	1912	1962	M 96(20)	1939	1966
A 94(8)	1925	1966	M 95(21)	1939	1966
A 242(9)	1941	Active	M 161(22)	1957	1986
A 373(10)	1954	1966	M 165(23)	1957	[a]
A 440(11)	1959	1975	M 187(24)	1960	1981
A 36(12)	1960	Active	M 183(25)	1960	2000
A 441(13)	1960	1985	M 188(26)	1960	1986
A 514(14)	1964	Active	M 244(27)	1974	2000
A 517(15)	1964	Active	None	None	None
A 572(16)	1966	Active	M 223(28)	1968	2000
A 588(17)	1968	Active	M 222(29)	1968	2000
A 709(5)	1974	Active	M 270(30)	1977	Active
A 852(18)	1985	2010	M 313(31)	1990	2000

[a]Exact year of withdrawal not identified. It was published in the 1966 9th edition *Standard Specifications for Highway Materials and Methods of Sampling and Testing*; however, it was not published in the 1971 10th edition, and the 10th edition did not report when it was withdrawn.(32,33)

Over time, it became cumbersome to work with so many material specifications, so ASTM A 709 and AASHTO M 270 were developed to cover all bridge steels in a single specification. ASTM A 709 was not a newly developed specification but rather a consolidation of ASTM A 36, A 514, A 572, and A 588 into a unified specification. Similarly, AASHTO M 270 was an assembly of AASHTO M 183, M 222, M 223, and M 244. Until publication of the 14th edition of the AASHTO *Standard Specifications for Highway Bridges* in 1989, engineers were provided design values for individual ASTM and AASHTO material specifications.[34] The 14th edition unified bridge design solely around ASTM A 709 and AASHTO M 270, and this remains the case. In 2000, AASHTO withdrew all its individual steel material specifications except for M 270.

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 not used anymore. Some names that may be seen on bridge plans are Chromansil, Cor-Ten, Man-Ten, Sil-Ten, R.D.S., AW 70-90, Centralloy, Yoloy, Granite City HS, Jal-Ten, Konik, Inland Hi-Steel, Armco HT-50, T-1, and Mayari R.[35–37] This report cannot delve into the uniqueness of all these proprietary steels, and it is suspected that their use in bridge design was not mainstream. If proprietary alloys are identified on bridge plans, engineers should consult the manufacturer's original literature for mechanical and chemical properties, not this report.

TENSILE PROPERTIES

Table 2 presents the minimum tensile properties of the 13 (not including A 709) unique steel specifications used in steel bridges and outlined in table 1. The table presents the evolution of each specification by showing how the tensile properties changed over time. Only yield strength, tensile strength, elongation, and reduction in area are shown. When condensing this information for specifications that have existed for multiple decades and contain numerous grades of steel, it is inevitable that some information will be lost. Therefore, the table is only informational and meant to identify trends; specific specifications should be consulted for exact properties.

Several observations can be noted in the table:

- Early specifications (i.e., A 7, A 8, and A 94) sometimes listed yield strength and elongation as a function of tensile strength; over time, these properties evolved into firm numbers.

- Many specifications from the early 1940s onward recognized the effects of plate thickness and that yield strength, tensile strength, and elongation degrade as plate thickness increases.

- ASTM A 7, A 8, and A 94 were never developed for welding considerations; their intent was for bridges riveted or bolted together. These specifications were withdrawn in 1966 and are a good indicator that by that time welding had become the preferred practice for steel bridge construction.

- High-strength steel (e.g., ASTM A 8 or ASTM A 94) has existed since 1912, so there was no general increase in yield and tensile strengths over time. Rather, a variety of steel strengths was possible early on when bridges were mechanically fastened together. The integration of welding into the mainstream caused yield strengths to stabilize at lower numbers, and then higher strength weldable steels were developed from the late 1960s to the modern era.

Table 2. Combined table of tensile properties.

ASTM	Year	Yield Strength, F_y (ksi)	Tensile Strength, F_u (ksi)	Elongation (Percent)	Reduction in Area (Percent)
A 7	1900	Soft steel: 32 Medium steel: 35	Soft steel: 52–62 Medium steel: 60–70	Soft steel: 25[a] Medium steel: 22[a]	—
A 7	1901	$\frac{1}{2}F_u$	60–70	22[a]	—
A 7	1924	$\frac{1}{2}F_u > 30$	55–65	$1{,}500/F_u$[a] or 22[b]	—
A 7	1966	33	≤1½ inches thick: 60–72 >1½ inches thick: 60–75	21[a] or 24[b]	—
A 8	1938	$\frac{1}{2}F_u$	90–115	$1{,}600/F_u$[a,c] or $1{,}700/F_u$[b,c]	30[c]
A 8	1961	55	90–115	14[a]	30[c]
A 94	1925–1952	45	80–95	$1{,}500/F_u$[a,c] $1{,}600/F_u$[b,c] added in 1939	30
A 94	1952–1962	45	80–95	16[a,c] or 19[b,c]	30
A 94	1962–1966	≤1⅛ inches thick: 50 >1⅛ and ≤2 inches thick: 47 >2 inches thick: 45	≤1⅛ inches thick: 75 >1⅛ and ≤2 inches thick: 72 >2 inches thick: 70	≤1⅛ inches thick: 17[a,c] >1⅛ and ≤2 inches thick: 18[a,c] >1⅛ and ≤2 inches thick: 21[b] >2 inches thick: 20[b,c]	—

ASTM	Year	Yield Strength, F_y (ksi)	Tensile Strength, F_u (ksi)	Elongation (Percent)	Reduction in Area (Percent)
A 36	1960–2000	36	Between 1960 and 1962: 60–80 Between 1962 and 2000: 58–80	20[a] or 23[b]	—
A 242	1941–1955	≤¾ inches thick: 50 >¾ and ≤1½ inches thick: 45 >1½ inches thick: 40	≤¾ inches thick: 70 >¾ and ≤1½ inches thick: 66 >1½ inches thick: 63	$1{,}500/F_u$[a] >1½ inches thick: $1{,}600/F_u$[b] Specific values published in 1952	—
A 242	1955–2000	≤¾ inches thick: 50 >¾ and ≤1½ inches thick: 46 >1½ inches thick: 42	≤¾ inches thick: 70 >¾ and ≤1½ inches thick: 67 >1½ inches thick: 63	≤¾ inches thick: 18[a,c] >¾ and ≤1½ inches thick: 18–19[a,c,d] >1½ inches thick: 16–19[a,c,d] >1½ inches thick: 21–24[b,c,d]	—
A 373	1954–1966	32	58–75	21[a] or 24[b]	—
A 440	1959–1963	≤¾ inches thick: 50 >¾ and ≤1½ inches thick: 46 >1½ inches thick: 42	≤¾ inches thick: 70 >¾ and ≤1½ inches thick: 67 >1½ inches thick: 63	≤¾ inches thick: 18[a] >¾ and ≤1½ inches thick: 19[a] >1½ and ≤4 inches thick: 19[a] >1½ and ≤4 inches thick: 24[b]	—
A 440	1963–1970	≤¾ inches thick: 50 >¾ and ≤1½ inches thick: 46 >1½ inches thick: 42	≤¾ inches thick: 70 >¾ and ≤1½ inches thick: 67 >1½ inches thick: 63	≤¾ inches thick: 18[a] >¾ and ≤1½ inches thick: 19[a] >1½ and ≤4 inches thick: 16[a] >1½ and ≤4 inches thick: 24[b]	—
A 440	1970–1975	≤¾ inches thick: 50 >¾ and ≤1½ inches thick: 46 >1½ inches thick: 42	≤¾ inches thick: 70 >¾ and ≤1½ inches thick: 67 >1½ inches thick: 63	≤4 inches thick: 18[a] >¾ and ≤4 inches thick: 21[b]	—

ASTM	Year	Yield Strength, F_y (ksi)	Tensile Strength, F_u (ksi)	Elongation (Percent)	Reduction in Area (Percent)
A 441	1960–1985	≤¾ inches thick: 50 >¾ and ≤1½ inches thick: 46 >1½ inches thick: 42	≤¾ inches thick: 70 >¾ and ≤1½ inches thick: 67 >1½ inches thick: 63	≤¾ inches thick: 18[a] (1960–1971) >¾ and ≤1½ inches thick: 19[a] (1960–1971) >1½ and ≤4 inches thick: 19[a] (1960–1963) >1½ and ≤4 inches thick: 16[a] (1963–1971) >1½ and ≤4 inches thick: 24[b] (1960–1971) >1½ inches thick: 18[a] or 21[b] (after 1971)	—
A 514	1964–1974	≤2½ inches thick: 100 >2½ and ≤4 inches thick: 90	≤2½ inches thick: 115–135 >2½ and ≤4 inches thick: 105–135	≤2½ inches thick: 18 >2½ and ≤4 inches thick: 17	40–50, depending on thickness
A 514	1974–2000	≤2½ inches thick: 100 >2½ and ≤4 inches thick: 90	≤2½ inches thick: 110–130 >2½ and ≤4 inches thick: 100–130	≤2½ inches thick: 18[b] >2½ and ≤4 inches thick: 16[b,e]	40–50, depending on thickness
A 517	1964–1967	100	115–135	18[b]	Rectangular specimen: 40 ½-inch-diameter specimen: 50
A 517	1967–1978	100	115–135	16[b]	Rectangular specimen: 35 ½-inch-diameter specimen: 45

ASTM	Year	Yield Strength, F_y (ksi)	Tensile Strength, F_u (ksi)	Elongation (Percent)	Reduction in Area (Percent)
A 517	1978–1990	≤2½ inches thick: 100 >2½ inches thick: 90	≤2½ inches thick: 115–135 >2½ inches thick: 105–135	≤2½ inches thick: 16[b] >2½ inches thick: 14[b]	Rectangular specimen up to 2½ inches thick: 35 ½-inch-diameter specimen: 45
A 572	1966–2000	Grade 42: 42 Grade 45[f]: 45 Grade 50: 50 Grade 55: 55 Grade 60: 60 Grade 65: 65	Grade 42: 60 Grade 45[f]: 60 Grade 50: 65 Grade 55: 70 Grade 60: 75 Grade 65: 80	Grade 42: 20[a] or 24[b] Grade 45[f]: 19[a] or 22[b] Grade 50: 18[a] or 21[b] Grade 55: 17[a] or 20[b] Grade 60: 16[a] or 18[b] Grade 65: 15[a] or 17[b,g]	—
A 588	1968–2000	50	70	19[a,c] or 21[b]	—
A 852	1985–2000	70	90–110	19	—

—Not specified.
[a]Using 8-inch gauge length.
[b]Using 2-inch gauge length.
[c]May be reduced depending on thickness.
[d]Varies by publication year.
[e]Requirement added in 1977.
[f]Grade 45 removed in 1978.
[g]Not specified between 1966 and 1974.
Note: All values represent a minimum unless a range is provided.

The evolution of tensile properties for ASTM A709 is shown in table 3. In 1997, the high-performance steel (HPS) grades of steel were introduced, which are unique to ASTM A709. These were an outgrowth of fabrication frustrations with conventional grades 70 and 100, which were notoriously difficult to weld. In the early 1990s, FHWA teamed with the U.S. Navy and the American Iron and Steel Institute to develop these HPS grades to have great weldability but also enhanced impact energy.[38] In 2000, the grade HPS 70W replaced the conventional grade 70 (i.e., ASTM A 852). In 2001, HPS 50W was added, which was effectively the HPS 70W chemistry without the heat treatment. In 2004, HPS 100W was added, and by 2010, it had fully replaced the conventional grade 100 and 100W (i.e., ASTM A 514). Because this report limited its review to plate material, table 3 does not reflect the addition of grade 50S in 2001 because this grade is meant for only rolled shapes. ASTM A 709 grade 50S mimics ASTM A 992. ASTM A 992 was created in the late 1990s because of the dominance of mini-mills that tended to make hot-rolled shapes from scrap feed.[39] Over time, continued recycling led to greater amounts of residual chemical elements, which caused increased yield and tensile strengths, decreased ductility, and potential weldability problems. Therefore, ASTM A 992 provided tighter controls on chemistry and strength specific to hot-rolled shapes for structural use.

Table 3. Tensile properties of ASTM A709.

Year	Grade	Yield Strength, F_y (ksi)	Tensile Strength, F_u (ksi)	Elongation (Percent)	Reduction in Area (Percent)
1974[5]	36	36	58–80	20[a] or 23[b]	—
	50	50	65	18[a] or 21[b]	—
	50W	50	70	18[a] or 21[b]	—
	100 and 100W	≤2½ inches thick: 100 >2½ inches thick: 90	≤2½ inches thick: 110–130 >2½ inches thick: 100–130	≤2½ inches thick: 18[b] >2½ inches thick: 17[b]	≤2½ inches thick: 40[c] or 50[d] >2½ inches thick: 50[d]
1985[40]	36	36	58–80	20[a] or 23[b]	—
	50	50	65	18[a] or 21[b]	—
	50W	50	70	18[a] or 21[b]	—
	100 and 100W	≤2½ inches thick: 100 >2½ inches thick: 90	≤2½ inches thick: 110–130 >2½ inches thick: 100–130	≤2½ inches thick: 18[b] >2½ inches thick: 16[b]	≤2½ inches thick: 40[c] or 50[d] >2½ inches thick: 50[d]
1987b[41]	36	36	58–80	20[a] or 23[b]	—
	50	50	65	18[a] or 21[b]	—
	50W	50	70	18[a] or 21[b]	—
	70W	70	90–110	19[b]	—
	100 and 100W	≤2½ inches thick: 100 >2½ inches thick: 90	≤2½ inches thick: 110–130 >2½ inches thick: 100–130	≤2½ inches thick: 18[b] >2½ inches thick: 16[b]	≤2½ inches thick: 40[c] or 50[d] >2½ inches thick: 50[d]

Year	Grade	Yield Strength, F_y (ksi)	Tensile Strength, F_u (ksi)	Elongation (Percent)	Reduction in Area (Percent)
1997b[42]	36	36	58–80	20[a] or 23[b]	—
	50	50	65	18[a] or 21[b]	—
	50W	50	70	18[a] or 21[b]	—
	70W	70	90–110	19[b]	—
	HPS 70W	70	90–110	19[b]	—
	100 and 100W	≤2½ inches thick: 100 >2½ inches thick: 90	≤2½ inches thick: 110–130 >2½ inches thick: 100–130	≤2½ inches thick: 18[b] >2½ inches thick: 16[b]	≤2½ inches thick: 40[c] or 50[d] >2½ inches thick: 50[d]
2000a[43]	36	36	58–80	20[a] or 23[b]	—
	50	50	65	18[a] or 21[b]	—
	50W	50	70	18[a] or 21[b]	—
	HPS 70W	70	85–110	19[b]	—
	100 and 100W	≤2½ inches thick: 100 >2½ inches thick: 90	≤2½ inches thick: 110–130 >2½ inches thick: 100–130	≤2½ inches thick: 18[b] >2½ inches thick: 16[b]	40[c] or 50[d]
2001a[44]	36	36	58–80	20[a] or 23[b]	—
	50	50	65	18[a] or 21[b]	—
	50W	50	70	18[a] or 21[b]	—
	HPS 50W	50	70	18[a] or 21[b]	—
	HPS 70W	70	85–110	19[b]	—
	100 and 100W	≤2½ inches thick: 100 >2½ inches thick: 90	≤2½ inches thick: 110–130 >2½ inches thick: 100–130	≤2½ inches thick: 18[b] >2½ inches thick: 16[b]	40[c] or 50[d]

Year	Grade	Yield Strength, F_y (ksi)	Tensile Strength, F_u (ksi)	Elongation (Percent)	Reduction in Area (Percent)
2004a[45]	36	36	58–80	20[a] or 23[b]	—
	50	50	65	18[a] or 21[b]	—
	50W	50	70	18[a] or 21[b]	—
	HPS 50W	50	70	18[a] or 21[b]	—
	HPS 70W	70	85–110	19[b]	—
	100, 100W, and HPS 100W	≤2½ inches thick: 100	≤2½ inches thick: 110–130	≤2½ inches thick: 18[b]	40[c] or 50[d]
	100 and 100W	>2½ inches thick: 90	>2½ inches thick: 100–130	>2½ inches thick: 16[b]	40[c] or 50[d]
2010[46]	36	36	58–80	20[a] or 23[b]	—
	50	50	65	18[a] or 21[b]	—
	50W and HPS 50W	50	70	18[a] or 21[b]	—
	HPS 70W	70	85–110	19[b]	—
	HPS 100W	≤2½ inches thick: 100 >2½ inches thick: 90	≤2½ inches thick: 110–130 >2½ inches thick: 100–130	≤2½ inches thick: 18[b] >2½ inches thick: 16[b]	40[c] or 50[d]

—Not specified.
[a]Using 8-inch gauge length.
[b]Using 2-inch gauge length.
[c]Using 1½-inch-wide specimen.
[d]Using ½-inch-diameter round specimen.
Note: All values represent a minimum unless a range is provided.

IMPACT ENERGY REQUIREMENTS

Impact energy requirements for steel bridges originated from the collapse of the Silver Bridge over the Ohio River between West Virginia and Ohio in 1967.[(47)] The Silver Bridge was an eyebar suspension bridge, and one eyebar developed a small stress corrosion crack near a pin that eventually developed into a brittle fracture, resulting in total collapse of the bridge and the death of 46 people. This was a wake-up call to the bridge engineering profession that such a small, virtually uninspectable defect could result in complete structural failure. The eyebar was made from steel that had low fracture toughness, meaning it could not tolerate an internal flaw under stress without going unstable (i.e., brittle fracture). As a result, numerous studies were conducted to try to quantify the fracture toughness of bridge steels and derive specification changes that could ensure a minimum level of fracture toughness. The Charpy V-notch (CVN) test became the quantifying measure used in specifications because it was a cheap and quick surrogate for rigorous fracture toughness testing.

THE FIRST REQUIREMENTS

Until 1968, AASHO steel material specifications were copies of ASTM specifications, although AASHO had the flexibility to provide exceptions if required. CVN impact energy was one area where AASHO first provided exceptions to ASTM specifications. In 1968, AASHO M 222 and AASHO M 223 were adopted, copying the 1968 versions of ASTM A 588 and A 572, respectively, except for a supplementary requirement of one longitudinal CVN test sample per lot of steel.[(28,29)] The sample was required to have at least 15 ft-lbf of energy absorption at 40°F. These first requirements are also shown in table 4. No information was provided in the references describing the basis behind these requirements. Supplementary impact energy requirements were only applicable if invoked, meaning merely specifying a material (e.g., M 222) did not automatically provide minimum impact energy requirements. However, in bridge design specifications, engineers were instructed to specify steel with supplementary CVN requirements, making CVN requirements mandatory for steel bridge fabrication. For instance, the 1969 10th edition of the *Standard Specifications for Highway Bridges* mandated the supplemental CVN requirements when welding M 222 and M 223 steels.[(48)]

Table 4. First AASHO CVN requirements.

Year	AASHO Specification	Energy[a] (ft-lbf)	Test Temperature (°F)
1968	M 222	15	40
1968	M 223	15[b]	40

[a]The energy value is for acceptance if the steel was to be welded. If the steel was not welded, the energy value is informational.
[b]Only grades 42, 45, and 50 were allowed for welding.

Comprehensive impact energy requirements were first adopted in 1974, when all AASHTO steel material specifications were assigned supplementary impact energy requirements.[(49)] The supplementary requirements were only applicable to "main load carrying member components subject to tensile stress."[(49)] The requirements varied based on temperature zones 1, 2, and 3 with lowest anticipated service temperatures (LASTs) of 0°F, −30°F, and −60°F, respectively (table 5). Interestingly, AASHTO has never written in any of their specifications how an engineer should determine the LAST for the bridge site for which they are designing. Many different measures could be used, such as record minimum, mean minimum (either daily, monthly, or annually), and minimum normal. The only place AASHTO ever defined how to determine the LAST was in the commentary for the 1976 *Interim Specifications Bridge*, which stated it was an "average minimum service temperature."(50) The impact energy requirements as adopted are summarized in table 6, and they were also added as supplementary requirements in ASTM A 709 (albeit without the caveat for mechanically fastened plates). The energy values were based on correlations between rigorous fracture toughness testing and companion CVN testing, as described by Barsom.(51) The CVN test is performed at temperatures higher than the LAST because of a temperature shift correlation between rigorous fracture toughness testing and the CVN surrogate test. The goal was to prevent steel from being on the lower shelf of impact energy and at a minimum ensure it was in the lower part of the transition from brittle to ductile behavior.

Table 5. AASHTO temperature zones.

Zone	LAST
1	0°F and above
2	−1°F to −30°F
3	−31°F to −60°F

Note: Adapted from AASHTO *Standard Specifications for Transportation Materials and Methods of Sampling and Testing – Part I Specification*.(49)

Table 6. 1974 AASHTO steel CVN requirements.

ASTM Specification (AASHTO Specification)	Thickness	Energy, Zone 1 (ft-lbf)	Energy, Zone 2 (ft-lbf)	Energy, Zone 3 (ft-lbf)
A 242 (M 161)	≤4 inches	15 at 70°F	15 at 40°F	15 at 10°F
A 36 (M 183)	≤4 inches	15 at 70°F	15 at 40°F	15 at 10°F
A 440 (M 187)	≤4 inches	15 at 70°F	15 at 40°F	15 at 10°F
A 441 (M 188)	≤4 inches	15 at 70°F	15 at 40°F	15 at 10°F
A 588 (M 222)	≤4 inches mechanically fastened	15 at 70°F[a]	15 at 40°F[a]	15 at 10°F[a]
	≤2 inches welded	15 at 70°F[a]	15 at 40°F[a]	15 at 10°F[a]
	>2 and ≤4 inches welded	20 at 70°F[a]	20 at 40°F[a]	20 at 10°F[a]
A 572 (M 223)	≤4 inches mechanically fastened	15 at 70°F[a]	15 at 40°F[a]	15 at 10°F[a]
	≤2 inches welded[b]	15 at 70°F[a]	15 at 40°F[a]	15 at 10°F[a]
A 514 (M 244)	≤4 inches mechanically fastened	25 at 30°F	25 at 0°F	25 at −30°F
	≤2½ inches welded	25 at 30°F	25 at 0°F	25 at −30°F
	>2½ and ≤4 inches welded	35 at 30°F	35 at 0°F	35 at −30°F

[a]If the yield point of the material exceeds 65 ksi, the temperature for the CVN value for acceptability shall be reduced by 15°F for each increment of 10 ksi above 65 ksi.
[b]Only grades 42, 45, and 50 may be used for welding.
Note: Adapted from AASHTO *Standard Specifications for Transportation Materials and Methods of Sampling and Testing – Part I Specification*.(49)

FRACTURE-CRITICAL REQUIREMENTS

The Silver Bridge collapse identified a need for minimum impact energy requirements, but it also began a discussion of bridge design redundancy. In the case of the Silver Bridge, the two parallel eyebar chain was nonredundant, meaning the sole intact eyebar could not resist the loading on the chain after failure of its neighbor. Therefore, non-load-path-redundant steel members were given a special designation of "fracture critical (FC)," indicating that failure of these tension members could lead to collapse of the bridge. This term was first used in the AASHTO *Guide Specification for Fracture Critical Non-Redundant Steel Bridge Members* in 1978.(52) This document provided additional criteria for FC members (FCMs). One of these criteria was more stringent impact energy requirements for FCMs. These requirements are shown in table 7 as published in 1978. Compared to the requirements in table 6, they excluded certain materials from FC design and required an additional 10 ft-lbf of energy in those materials still allowed.

Table 7. 1978 impact energy requirements for FCMs.

ASTM Specification (AASHTO Specification)	Thickness	Energy, Zone 1 (ft-lbf)	Energy, Zone 2 (ft-lbf)	Energy, Zone 3 (ft-lbf)
A 36 (M 183)	≤4 inches	25 at 70°F	25 at 40°F	25 at 10°F
A 572 (M 223)	≤4 inches mechanically fastened	25 at 70°F[a]	25 at 40°F[a]	25 at 10°F[a]
	≤2 inches welded	25 at 70°F[a]	25 at 40°F[a]	25 at 10°F[a]
A 588 (M 222)	≤4 inches mechanically fastened	25 at 70°F[a]	25 at 40°F[a]	25 at 10°F[a]
	≤2 inches welded	25 at 70°F[a]	25 at 40°F[a]	25 at 10°F[a]
	>2 and ≤4 inches welded	30 at 70°F[a]	30 at 40°F[a]	30 at 10°F[a]
A 514 (M 244)	≤4 inches mechanically fastened	35 at 0°F	35 at 0°F	35 at −30°F
	≤2½ inches welded	35 at 0°F	35 at 0°F	35 at −30°F
	>2½ and ≤4 inches welded	45 at 0°F	45 at 0°F	45 at −30°F

[a]If the yield point of the material exceeds 65 ksi, the temperature for the CVN value for acceptability shall be reduced by 15°F for each increment of 10 ksi above 65 ksi.
Note: Adapted from AASHTO *Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members*.(52)

UNIFIED REQUIREMENTS

As mentioned previously, ASTM A 709 was first published in 1974, and AASHTO adopted the equivalent, M 270, in 1977.(5,30) ASTM A 709 and AASHTO M 270 included CVN requirements, but FC CVN requirements were published in the 1978 *Guide Specification for Fracture Critical Non-Redundant Steel Bridge Members*.(52) Impact energy requirements as presented in table 6 and table 7 were unified into non-fracture-critical (NFC) and FC requirements, as shown in table 8, in 1985 for A 709 and 1986 for M 270.(53,54) In 1987, grade 70W was introduced, which was derived from ASTM A 852.(55,18) ASTM A 852 had a CVN requirement of 20 ft-lbf at 50°F, and this was translated as a zone 1 requirement in ASTM A 709. The test temperatures were reduced an additional 30°F and 60°F for zones 2 and 3, respectively. Furthermore, the FC impact energy requirements for grade 70W were increased 10 ft-lbf. Table 9 and table 10 show further evolution of the ASTM A709 CVN requirements from the 1990s up to this publication. The primary change was the introduction of the HPS grades of steel and the retirement of the conventional grade 70 and 100 steels they replaced. HPS CVN requirements were not developed based on what the steel could achieve; rather, the steel was developed to achieve a CVN performance requirement. The HPS CVN performance requirement was to attain the zone 3 FC requirements of the conventional grade 50, 70, and 100 steels, so the test temperature was held constant across all the temperature zones (e.g., steel meeting zone 3 requirements would automatically fulfill zone 1 requirements). The 70W NFC impact energies were reduced by 10 ft-lbf from the FC requirements, commensurate with the other grades. The only other notable changes between table 8 through table 10 were the 100-ksi-yield steel's testing temperature, its exclusion from zone 3, and its thickness effects.

Table 8. ASTM A 709 impact energy requirements from 1985 to 1995.

Grade	Thickness	Energy, NFC Zone 1 (ft-lbf)	Energy, NFC Zone 2 (ft-lbf)	Energy, NFC Zone 3 (ft-lbf)	Energy, FC Zone 1 (ft-lbf)	Energy, FC Zone 2 (ft-lbf)	Energy, FC Zone 3 (ft-lbf)
36	≤4 inches	15 at 70°F	15 at 40°F	15 at 10°F	25 at 70°F	25 at 40°F	25 at 10°F
50 and 50W	≤4 inches mechanically fastened	15 at 70°F[a]	15 at 40°F[a]	15 at 10°F[a]	25 at 70°F[a]	25 at 40°F[a]	25 at 10°F[a]
	≤2 inches welded	15 at 70°F[a]	15 at 40°F[a]	15 at 10°F[a]	25 at 70°F[a]	25 at 40°F[a]	25 at 10°F[a]
	>2 to 4 inches welded	20 at 70°F[a]	20 at 40°F[a]	20 at 10°F[a]	30 at 70°F[a,b]	30 at 40°F[a,b]	30 at 10°F[a,b]
70W[c]	≤1½ inches mechanically fastened or welded	20 at 50°F[d]	20 at 20°F[d]	20 at −10°F[d]	30 at 20°F[d]	30 at 20°F[d]	30 at −10°F[d]
	>1½ to 4 inches mechanically fastened	20 at 50°F[d]	20 at 20°F[d]	20 at −10°F[d]	30 at 20°F[d]	30 at 20°F[d]	30 at −30°F[d]
	>1½ to 2½ inches welded	20 at 50°F[d]	20 at 20°F[d]	20 at −10°F[d]	30 at 20°F[d]	30 at 20°F[d]	30 at −30°F[d]
	>2½ to 4 inches welded	25 at 50°F[d]	25 at 20°F[d]	25 at −10°F[d]	35 at 20°F[d]	35 at 20°F[d]	35 at −30°F[d]
100 and 100W	≤4 inches mechanically fastened	25 at 30°F	25 at 0°F	25 at −30°F	35 at 0°F[e]	35 at 0°F	35 at −30°F
	≤2½ inches welded	25 at 30°F	25 at 0°F	25 at −30°F	35 at 0°F[e]	35 at 0°F	35 at −30°F
	>2½ to 4 inches welded	35 at 30°F	35 at 0°F	35 at −30°F	45 at 0°F[e]	45 at 0°F	—

—Not allowed.
[a]If the yield point of the material exceeds 65 ksi, the temperature for the CVN value for acceptability shall be reduced by 15°F for each increment of 10 ksi above 65 ksi.
[b]Not permitted for grade 50 until 1986.
[c]Grade 70 was introduced in 1987b.[(41)]
[d]If the yield point of the material exceeds 85 ksi, the temperature for the CVN value for acceptability shall be reduced by 15°F for each increment of 10 ksi above 85 ksi.
[e]Test temperature was 30°F in 1985 and 1986.

Table 9. ASTM A 709 impact energy requirements from 1995 to 2009.

Grade	Thickness	Energy, NFC Zone 1 (ft-lbf)	Energy, NFC Zone 2 (ft-lbf)	Energy, NFC Zone 3 (ft-lbf)	Energy, FC Zone 1 (ft-lbf)	Energy, FC Zone 2 (ft-lbf)	Energy, FC Zone 3 (ft-lbf)
36	≤4 inches	15 at 70°F	15 at 40°F	15 at 10°F	25 at 70°F	25 at 40°F	25 at 10°F
50, 50W, and 50S[a]	≤4 inches mechanically fastened	15 at 70°F[b]	15 at 40°F[b]	15 at 10°F[b]	25 at 70°F[b]	25 at 40°F[b]	25 at 10°F[b]
	≤2 inches welded	15 at 70°F[b]	15 at 40°F[b]	15 at 10°F[b]	25 at 70°F[b]	25 at 40°F[b]	25 at 10°F[b]
	>2 to 4 inches welded	20 at 70°F[b]	20 at 40°F[b]	20 at 10°F[b]	30 at 70°F[b]	30 at 40°F[b]	30 at 10°F[b]
70W[c]	≤2½ inches mechanically fastened or welded	20 at 50°F[d]	20 at 20°F[d]	20 at −10°F[d]	30 at 50°F[d]	30 at 20°F[d]	30 at −10°F[d]
	>2½ to 4 inches mechanically fastened	20 at 50°F[d]	20 at 20°F[d]	20 at −10°F[d]	30 at 50°F[d]	30 at 20°F[d]	30 at −10°F[d]
	>2½ to 4 inches welded	25 at 50°F[d]	25 at 20°F[d]	25 at −10°F[d]	35 at 50°F[d]	35 at 20°F[d]	35 at −10°F[d]
HPS 50W[e]	≤4 inches	20 at 10°F[b]	20 at 10°F[b]	20 at 10°F[b]	30 at 10°F[b]	30 at 10°F[b]	30 at 10°F[b]
HPS 70W[f]	≤4 inches	25 at −10°F[c]	25 at −10°F[c]	25 at −10°F[c]	35 at −10°F[c]	35 at −10°F[c]	35 at −10°F[c]
HPS 100W[g]	≤2½ inches	25 at −30°F	25 at −30°F	25 at −30°F	35 at −30°F	35 at −30°F	[h]
100 and 100W	≤4 inches mechanically fastened	25 at 30°F	25 at 0°F	25 at −30°F	35 at 30°F	35 at 0°F	35 at −30°F
	≤2½ inches welded	25 at 30°F	25 at 0°F	25 at −30°F	35 at 30°F	35 at 0°F	35 at −30°F
	>2½ to 4 inches welded	35 at 30°F	35 at 0°F	35 at −30°F	45 at 30°F	45 at 0°F	—

—Not allowed.
[a]Grade 50S added in 2001.
[b]If the yield point of the material exceeds 65 ksi, the temperature for the CVN value for acceptability shall be reduced by 15°F for each increment of 10 ksi above 65 ksi.
[c]Grade 70W removed in 2000.
[d]If the yield point of the material exceeds 85 ksi, the temperature for the CVN value for acceptability shall be reduced by 15°F for each increment of 10 ksi above 85 ksi.
[e]HPS 50W introduced in 2001.
[f]HPS 70W introduced in 1997.

[g]HPS 100W introduced in 2005.
[h]Not permitted from 2005 to 2008. In 2008, the requirement was set at 35 ft-lbf at −30°F.

Table 10. ASTM A709 impact energy requirements from 2009 onward.

Grade	Thickness	Energy, NFC Zone 1 (ft-lbf)	Energy, NFC Zone 2 (ft-lbf)	Energy, NFC Zone 3 (ft-lbf)	Energy, FC Zone 1 (ft-lbf)	Energy, FC Zone 2 (ft-lbf)	Energy, FC Zone 3 (ft-lbf)
36	≤4 inches	15 at 70°F	15 at 40°F	15 at 10°F	25 at 70°F	25 at 40°F	25 at 10°F
50, 50W, and 50S	≤2 inches	15 at 70°F[a]	15 at 40°F[a]	15 at 10°F[a]	25 at 70°F[a]	25 at 40°F[a]	25 at 10°F[a]
	>2 to 4 inches	20 at 70°F[a]	20 at 40°F[a]	20 at 10°F[a]	30 at 70°F[a]	30 at 40°F[a]	30 at 10°F[a]
HPS 50W	≤4 inches	20 at 10°F[a]	20 at 10°F[a]	20 at 10°F[a]	30 at 10°F[a]	30 at 10°F[a]	30 at 10°F[a]
HPS 70W	≤4 inches	25 at −10°F[b]	25 at −10°F[b]	25 at −10°F[b]	35 at −10°F[b]	35 at −10°F[b]	35 at −10°F[b]
HPS 100W	≤2½ inches	25 at −30°F	25 at −30°F	25 at −30°F	35 at −30°F	35 at −30°F	35 at −30°F
	>2½ to 4 inches	35 at −30°F	35 at −30°F	35 at −30°F	—	—	—

—Not allowed.
[a]If the yield point of the material exceeds 65 ksi, the temperature for the CVN value for acceptability shall be reduced by 15°F for each increment of 10 ksi above 65 ksi.
[b]If the yield point of the material exceeds 85 ksi, the temperature for the CVN value for acceptability shall be reduced by 15°F for each increment of 10 ksi above 85 ksi.

WELD METAL REQUIREMENTS

A complete history of the evolution of weld impact energy requirements cannot be succinctly presented; rather, highlights are presented here. As will be described in the chapter on evolution of steel bridge design, the American Welding Society (AWS) first published welding codes specific to bridges. Then, AASHTO published supplementary specifications to work along with the AWS codes, and finally a joint AASHTO/AWS code was developed. Through this evolution, the CVN requirements varied based on weld process and steel grade.

The first reference to weld metal impact energy requirements was published in the 10th edition of the *Standard Specifications for Highway Bridges* 1970 interims.(56) This edition required bridge designers to specify weld impact energy of 20 ft-lbf at 0°F. In 1972, AWS published the D1.1 welding code, which was meant to cover both bridges and buildings.(57) However, in 1974, AASHTO published the 1st edition of the *Standard Specification for Welding of Structural Steel Highway Bridges*, which was meant to provide overriding supplementary requirements to AWS D1.1 for welding steel bridges.(58) These 1974 specifications made the weld CVN requirements of AWS D1.1 (15 ft-lbf at 0°F for all steels) mandatory.

The 1978 AASHTO *Guide Specification for Fracture Critical Non-Redundant Steel Bridge Members* introduced impact energy requirements for weld metal for only FCMs, as shown in table 11.(52) The 1978 guide specification was updated six times through its last revision in 1989 and was ultimately retired in 1995 when most of its requirements were migrated to clause 12 of the AASHTO/AWS D1.5 *Bridge Welding Code*.(59,60) The weld metal impact requirements from the 2015 AASHTO/AWS D1.5 *Bridge Welding Code* are shown in table 12. The table shows that there has not been much evolution of FC weld metal impact energy requirements over time beyond the inclusion of requirements for grade 70 steel.

Table 11. 1978 impact energy requirements for FCM weld metal.

ASTM Specification (AASHTO Specification)	Thickness	Energy, Zone 1 (ft-lbf)	Energy, Zone 2 (ft-lbf)	Energy, Zone 3 (ft-lbf)
A 36 (M 183), A 572 (M 223), and A 588 (M 222)	All	25 at −20°F	25 at −20°F	25 at −20°F
A 514 (M 244) and A 517	All	35 at −30°F[a]	35 at −30°F[a]	35 at −30°F[a]

[a]Except for fillet welds made with filler metal normally used for welding A 36, A 572, and A 588 (AASHTO M 183, M 223, and M 222), whose impact energy shall be 25 ft-lbf at −20°F.
Note: Adapted from AASHTO *Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members*.(52)

Table 12. 2015 impact energy requirements for FCM weld metal.

ASTM A709 and AASHTO M 270 Steel Grade	Thickness	Energy, Zone 1 (ft-lbf)	Energy, Zone 2 (ft-lbf)	Energy, Zone 3 (ft-lbf)
36	All	25 at −20°F	25 at −20°F	25 at −20°F
50	All	25 at −20°F	25 at −20°F	25 at −20°F
50W and HPS 50W	All	25 at −20°F	25 at −20°F	25 at −20°F
HPS 70W	All	30 at −20°F[a]	30 at −20°F[a]	30 at −20°F[a]
HPS 100W	All	35 at −30°F[a]	35 at −30°F[a]	35 at −30°F[a]

[a]Values are for matching strength weld metal. For undermatching strength weld metal, the requirement is reduced to 25 ft-lbf at −20°F.
Note: Adapted from AASHTO/AWS D1.5 *Bridge Welding Code.*(61)

The 3rd edition of the *Standard Specification for Welding of Structural Steel Highway Bridges* presented a mix of CVN requirements for NFC weld metal.(62) However, the only firm ones were the following:

- Electroslag welding (ESW) and electrogas welding (EGW) require 15 ft-lbf at 0°F.
- ASTM A 514/AASHTO M 244 and ASTM A 517 filler metals required 20 ft-lbf at 0°F when using submerged arc welding (SAW), gas metal arc welding (GMAW), and flux cored arc welding (FCAW). There were no requirements when using shielded metal arc welding (SMAW).
- When using bare exposed ASTM A588/AASHTO M 222, filler metals required 20 ft-lbf at 0°F when using FCAW, specific 80-ksi GMAW electrodes, or specific 80-ksi SMAW electrodes.

The 1st edition of the AASHTO/AWS D1.5 *Bridge Welding Code* published in 1988 established more rigorous weld impact energy requirements, as shown in table 13.(63) The current requirements for NFC weld metal in the 2015 *Bridge Welding Code* have evolved as shown in table 14, with the major differences being the process dependency and some slight changes in test temperature.(61)

Table 13. 1988 impact energy requirements for NFC weld metal.

ASTM Specification (AASHTO Specification)	Welding Process[a]	Energy, Zone 1 (ft-lbf)	Energy, Zone 2 (ft-lbf)	Energy, Zone 3 (ft-lbf)
A 36 (M 183)	SAW, GMAW, FCAW	20 at 0°F[b]	20 at 0°F[b]	20 at −20°F
A 36 (M 183)	ESW/EGW	15 at 0°F	15 at 0°F	15 at 0°F
A 572 (M 223)	SAW, GMAW, FCAW	20 at 0°F[b]	20 at 0°F[b]	20 at −20°F
A 572 (M 223)	ESW/EGW	15 at 0°F	15 at 0°F	15 at 0°F
A 852	SAW, FCAW	25 at −10°F	25 at −10°F	25 at −25°F
A 514 (M 244) and A 517	SAW, GMAW, FCAW	20 at −40°F	20 at −40°F	As approved

[a]SMAW was exempt from CVN requirements for all steel grades.
[b]Specific filler metals using GMAW required 20 ft-lbf at −20°F for zones 1 and 2.
Note: Adapted from AASHTO/AWS D1.5 *Bridge Welding Code.*(63)

Table 14. 2015 impact energy requirements for NFC weld metal.

ASTM A709 and AASHTO M 270 Steel Grade	Energy, Zone 1 (ft-lbf)	Energy, Zone 2 (ft-lbf)	Energy, Zone 3 (ft-lbf)[a]
36	20 at 0°F	20 at 0°F	20 at −20°F
50	20 at 0°F	20 at 0°F	20 at −20°F
50W and HPS 50W	20 at 0°F	20 at 0°F	20 at −20°F
HPS 70W	25 at −10°F	25 at −10°F	25 at −20°F
HPS 100W	20 at −40°F	20 at −40°F	As approved

[a]ESW not permitted in zone 3. EGW requirement as approved by the engineer.
Note: Adapted from AASHTO/AWS D1.5 *Bridge Welding Code.*(61)

CHEMISTRY REQUIREMENTS

To further understand the evolution of steel material specifications, it is important to understand that steel is often composed of approximately 98-percent iron and 2-percent other alloying elements. The alloying elements greatly influence the overall strength, hardness, toughness, ductility, and weldability of the steel. Table 15 describes the common alloying elements and the pros and cons of each. The primary hardening and strengthening element in steel is carbon; however, excessive amounts of carbon cause the steel to become too hard and brittle to function in a bridge that must sustain fatigue cycling, withstand cold environments, and behave in a ductile fashion. To understand the interrelation of compositional elements in steel, it is important to understand the concept of carbon equivalency (CE). This term is often used to describe weldability using an equation that considers the influence of eight different elements compared to only carbon. Although the evolution of steel material specifications generally led to more desirable properties over time, the need for weldability played a large part in that evolution. CE is shown in equation 1, which demonstrates that chromium, molybdenum, and vanadium have 20 percent of the effect of carbon; manganese and silicon have 17 percent of the effect of carbon; and nickel and copper have about 7 percent of the effect of carbon. All contents in equation 1 are expressed as percent by weight.

$$CE = C + \frac{(Mn + Si)}{6} + \frac{(Cr + Mo + V)}{5} + \frac{(Ni + Cu)}{15} \tag{1}$$

Where:

C = carbon content.
Mn = manganese content.
Si = silicon content.
Cr = chromium content.
Mo = molybdenum content.
V = vanadium content.
Ni = nickel content.
Cu = copper content.

Table 16 presents the chemical compositions of the ASTM specifications first presented in table 1. Table 16 is not meant to be all inclusive because the evolution of 13 different steel specifications over multiple decades cannot be condensed into one table (as before, ASTM A709 is not included because it is an assembly of other specifications already shown in the table). Therefore, the chemical compositions were limited to plate product up to 4 inches thick because this is the limit for bridges. Many of the steel specifications are applicable to both rolled shapes and plate product and to thicknesses greater than 4 inches. Furthermore, some specifications cover numerous grades of steel with different compositions; therefore, ranges may be reported that cover multiple grades within that specification or multiple specification publication cycles. Table 16 should not be used to assess the chemical composition of steel grade; individual specifications should be consulted when specific information is needed. Table 16 is only meant to represent the evolution of chemical requirements over time.

Table 15. Alloys and their advantages and disadvantages on material properties.

Element	Advantages	Disadvantages
Carbon	Increases strength and hardness	Decreases ductility, toughness, and weldability
Manganese	Increases strength; binds to sulfur, reducing its harmful effects	Decreases weldability
Phosphorus	Increases strength, hardness, and corrosion resistance	Decreases ductility and toughness; has a strong tendency to segregate and form cracks
Sulfur	Increases machinability	Decreases ductility, toughness, and weldability; has a strong tendency to segregate and form cracks
Silicon	Increases strength and hardness; used to deoxidize molten steel	At high levels (>0.30 percent) can reduce weldability
Vanadium	Small additions increase strength	Excessive amounts can decrease toughness
Columbium	Small additions increase strength	Excessive amounts can decrease toughness
Nickel	Increases strength, ductility, and toughness	—
Chromium	Increases strength and corrosion resistance	—
Molybdenum	Increases strength and hardenability	—
Copper	Increases atmospheric corrosion resistance	At high levels (>1.50 percent) can reduce weldability
Nitrogen	Increases strength and hardness	Decreases ductility and toughness

—No data to report.
Note: Adapted from FHWA *Steel Bridge Design Handbook.*[65]

Table 16. ASTM chemistry requirements.

ASTM	Year(s)	C	Mn	P	S	Si	V	Cb	Ni	Cr	Mo	Cu
A 7	1900–1905	[a]	[a]	0.08	0.06	—	—	—	—	—	—	—
A 7	1905–1966	[a]	[a]	0.04	0.05	—	—	—	—	—	—	0.20 min.[b,c]
A 8	1939–1962	0.43	0.80	0.04	0.05	—	—	—	3.00–4.00	—	—	0.20 min.[b]
A 36	1960–2000	0.25–0.28[d]	0.80–1.20[e]	0.04	0.05	0.40[f]	—	—	—	—	—	0.20 min.[b]
A 94	1925–1962	0.40	[a]	0.04	0.05	0.20 min.	—	—	—	—	—	0.20 min.[b,g]
A 94	1962–1966	0.33	1.10–1.60	0.04	0.05	0.30[h]	—	—	—	—	—	0.20 min.[b]
A 242	1941–1967	0.20[i]	1.25	[a]	0.05	—	—	—	—	—	—	—
A 242 Type 1	1968–1985	0.15	1.00	0.15	0.05	[j]	—	—	—	[j]	—	0.20 min.[j]
A 242 Type 2	1968–1985	0.20	1.35	0.04	0.05	[j]	—	—	—	[j]	—	0.20 min.[j]
A 373	1954–1965	0.25–0.27[d]	0.50–0.90[k]	0.04	0.05	0.15–0.30[k]	—	—	—	—	—	0.20 min.[b]
A 440	1959–1975	0.28	1.10–1.60	0.04	0.05	0.30	—	—	—	—	—	0.20 min.
A 441	1960–1968	0.22	1.25	0.04	0.05	0.30	0.02 min.	—	—	—	—	0.20 min.

ASTM	Year(s)	C	Mn	P	S	Si	V	Cb	Ni	Cr	Mo	Cu
A 441	1968–1985	0.22	0.85–1.25	0.04	0.05	0.30[l]	0.02 min.	—	—	—	—	0.20 min.
A 514	1964–2000	0.08–0.21[m]	0.40–1.50[m]	0.035	0.04[n]	0.15–0.90[m]	0.03–0.08[o]	0.06[p]	0.30–1.50[m,o]	0.35–2.00[m,o]	0.10–0.65[m]	0.15–0.50[m,o]
A 517	1964–1990	0.10–0.21[m]	0.40–1.50[m]	0.035	0.040	0.10–0.90[m]	0.03–0.08[o]	—	0.30–1.50[o]	0.40–2.00[m,o]	0.15–0.65[m]	0.15–0.50[m,o]
A 572 Type 1	1966–2000	0.21–0.26[d]	1.35–1.65[d]	0.04	0.05	0.15–0.40[q]	—	0.005–0.05	—	—	—	—
A 572 Type 2	1966–2000	0.21–0.26[d]	1.35–1.65[d]	0.04	0.05	0.15–0.40[q]	0.01–0.15	—	—	—	—	—
A 572 Type 3	1966–2000	0.21–0.26[d]	1.35–1.65[d]	0.04	0.05	0.15–0.40[q]	[r]	[r]	—	—	—	—
A 572 Type 4	1966–2000	0.21–0.26[d]	1.35–1.65[d]	0.04	0.05	0.15–0.40[q]	[s]	—	—	—	—	—
A 588	1968–2000	0.10–0.20[m]	0.50–1.35[m]	0.04	0.05	0.15–0.90[m]	0.01–0.10[m,o]	0.005–0.05[m,o]	1.25[t]	0.10–1.00[m]	0.08–0.25[m,o]	0.20–1.00[m,s]
A 852	1985–2007	0.19[e]	0.80–1.35[e]	0.035	0.04	0.20–0.65	0.02–0.10	—	0.50	0.40–0.70	—	0.20–0.40

—No requirement.
[a]Not specified, although actual value had to be reported on mill test report until 1949 when ASTM A 6 was first published. ASTM A 6 dictated that at a minimum C, Mn, P, and S must be reported.
[b]If specified.
[c]Requirement for Cu first appeared in 1929.
[d]Is a maximum amount but is shown as a range because it varies depending on product (e.g., plate or shape), plate thickness, and/or grade.
[e]Mn specified only for plates over ¾ inch thick.
[f]Varies by publication year. Sometimes it was not applicable for thinner plates, was specified as a range for a certain plate thickness, or was just shown as a maximum.
[g]Requirement for Cu first appeared in 1949.
[h]Range of 0.15–0.30 for plates over 1½ inches thick.
[i]Increased to 0.22 in 1955.

[j]If Cr and Si contents are each 0.50 min., the Cu requirement does not apply.
[k]Not applicable for thin plate ranges.
[l]0.40 from 1981 to 1985.
[m]Specification covers numerous grades with defined range for element. This value represents the range for all grades over all years of publication.
[n]Some grades specify a lower value, and the values decreased over time.
[o]Not applicable for all grades.
[p]Only for grade S.
[q]Varies depending on plate thickness and publication year. Sometimes is a maximum value or a range.
[r]Cb plus V must be from 0.02 to 0.15. Cb limited to 0.05 maximum until 1997. After 1997, Cb specified in the range of 0.005–0.05 and V in the range of 0.01–0.15.
[s]N (0.015 maximum) added as supplement to V; maximum ratio of V to N shall be 4:1. After 1997, N specified at 0.01–0.15.
[t]This is a maximum value across all grades. Some grades use a lower maximum value or specify a range.
Min. = minimum
Note: The amounts of different elements are expressed in percentages. This table covers only plate product and plates less than 4 inches thick. All values are maximum values unless otherwise noted.

The analysis of table 16 is not trivial; however, the chronology of when the material specifications were introduced is represented as one moves down the table rows. The first specification in the table, ASTM A 7, was clearly a chemically uncontrolled specification, providing only maximum levels for sulfur and phosphorus.[6] Carbon contents could be high for some of the early steels because carbon was needed for strength at the time. However, the early nickel (ASTM A 8) and silicon (ASTM A 94) steels were considered unweldable.[7,8] ASTM A 7 was weldable if carbon and manganese were controlled; thus, carbon and manganese had to be specified by the engineer because relying on the ASTM alone was insufficient.[6] Generally, the steel specifications established after about 1940 provided limits or ranges for carbon, manganese, sulfur, and phosphorus. It is believed that the inclusion of these was driven by requirements to maintain strength and weldability at the same time. As time progressed, higher strength steels were needed, so ranges and limits were applied to a greater number of elements.

Atmospheric corrosion resistant steels have been available as far back as 1905. Steels bearing a copper content greater than 0.20 percent have moderate corrosion resistance, and many specifications allowed copper to be optionally specified. In the mid-1960s, weathering steels that had significant corrosion resistance (e.g., A 588, A 514, and later A 852) were introduced; these steels relied on silicon, nickel, and chromium along with copper to attain corrosion resistance.[37]

Because ASTM A709 was excluded from table 16, some of the compositional evolution of carbon, manganese, sulfur, and phosphorus is presented in table 17 through table 20. The tables show the compositional requirements starting in 1974 and at key years where grades were either introduced or removed from the specification. In terms of carbon, the requirements for grades 36, 50, and 50W have been static through time, ranging from 0.20 to 0.27 percent. The carbon requirements for the early grade 100 steels were reduced to less than 0.20 percent because the steels were quench and tempered and carbon had to be limited so the steel did not overharden. The conventional grade 70W and 100W steels were notoriously difficult to weld, and the development of HPS demonstrated that carbon limits had to be further reduced to maintain weldability. The HPS grades thus relied on alloying and processing for strength gain in lieu of carbon. Manganese contents have stayed in the same range throughout the life of ASTM A709 over all the grades. However, there is a clear trend of reduction of sulfur and phosphorus composition over time, especially in the high-strength HPS grades. In elevated concentrations, these two elements can wreak havoc on weldability and toughness, and keeping their levels low was a key to creating a high-strength steel that remains weldable, as is the case with the HPS grades.

Table 17. Maximum carbon composition (percent) in ASTM A709.

Grade	1974	1985	1987b[41]	1993a[66]	1997b[42]	2000a[43]	2001a[44]	2004a[45]	2010	2015
36	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27
50	0.23	0.23	0.23	0.23	0.23	0.23	0.23	0.23	0.23	0.23
50W	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
100 and 100W	0.10–0.21	0.10–0.21	0.10–0.21	0.10–0.21	0.10–0.21	0.10–0.21	0.10–0.21	0.10–0.21	—	—
70W	—	—	0.19	0.19	0.19	—	—	—	—	—
HPS 70W	—	—	—	—	0.11	0.11	0.11	0.11	0.11	0.11
HPS 50W	—	—	—	—	—	—	0.11	0.11	0.11	0.11
50S	—	—	—	—	—	—	0.23	0.23	0.23	0.23
HPS 100W	—	—	—	—	—	—	—	0.08	0.08	0.08

—Grade did not exist.

Table 18. Maximum manganese composition (percent) in ASTM A709.

Grade	1974	1985	1987b(41)	1993a(66)	1997b(42)	2000a(43)	2001a(44)	2004a(45)	2010	2015
36	0.80–1.20[a]	0.80–1.20[a]	0.80–1.20[a]	0.80–1.20[a]	0.80–1.20[a]	0.80–1.20[a]	0.80–1.20[a]	0.80–1.20[a]	0.80–1.20[a]	0.80–1.20[a]
50	1.35	1.35	1.35	1.35	1.35	1.35	1.35	1.35	1.35	1.35
50W	1.35	0.50–1.35[b]	0.75–1.35[b]	0.75–1.35[b]	0.75–1.35[b]	0.75–1.35[b]	0.75–1.35[b]	0.75–1.35[b]	0.75–1.35[b]	0.75–1.35[b]
100 and 100W	0.40–1.50	0.40–1.50[b]	0.40–1.50[b]	0.40–1.50[b]	0.40–1.50[b]	0.40–1.50[b]	0.40–1.50[b]	0.40–1.50[b]	—	—
70W	—	—	0.80–1.35	0.80–1.35	0.80–1.35	—	—	—	—	—
HPS 70W	—	—	—	—	1.15–1.30	1.10–1.35	1.10–1.35	1.10–1.35	1.10–1.50[c]	1.10–1.50[c]
HPS 50W	—	—	—	—	—	—	1.10–1.35	1.10–1.35	1.10–1.50[c]	1.10–1.50[c]
50S	—	—	—	—	—	—	0.50–1.50	0.50–1.50	0.50–1.60	0.50–1.60
HPS 100W	—	—	—	—	—	—	—	0.95–1.50	0.95–1.50	0.95–1.50

—Grade did not exist.
[a]Exact range is dependent upon plate thickness range.
[b]Range varies by type; range shown is maximum across all types.
[c]Range varies by thickness; range shown is maximum across all thicknesses.

Table 19. Maximum sulfur composition (percent) in ASTM A709.

Grade	1974	1985	1987b[41]	1993a[66]	1997b[42]	2000a[43]	2001a[44]	2004a[45]	2010	2015
36	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.030
50	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.030
50W	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.030
100 and 100W	0.04	0.04	0.04	0.035	0.035	0.035	0.035	0.035	—	—
70W	—	—	0.05	0.04	0.04	—	—	—	—	—
HPS 70W	—	—	—	—	0.006	0.006	0.006	0.006	0.006	0.006
HPS 50W	—	—	—	—	—	—	0.006	0.006	0.006	0.006
50S	—	—	—	—	—	—	0.045	0.045	0.045	0.045
HPS 100W	—	—	—	—	—	—	—	0.006	0.006	0.006

—Grade did not exist.

Table 20. Maximum phosphorus composition (percent) in ASTM A709.

Grade	1974	1985	1987b[41]	1993a[66]	1997b[42]	2000a[43]	2001a[44]	2004a[45]	2010	2015
36	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.030
50	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.030
50W	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.030
100 and 100W	0.035	0.035	0.035	0.035	0.035	0.035	0.035	0.035	—	—
70W	—	—	0.04	0.035	0.035	—	—	—	—	—
HPS 70W	—	—	—	—	0.020	0.020	0.020	0.020	0.020	0.020
HPS 50W	—	—	—	—	—	—	0.020	0.020	0.020	0.020
50S	—	—	—	—	—	—	0.035	0.035	0.035	0.035
HPS 100W	—	—	—	—	—	—	—	0.015	0.015	0.015

—Grade did not exist.

EVOLUTION OF STEEL BRIDGE DESIGN

This chapter describes some of the changes in design procedures that may contribute to performance changes in steel bridges over time. Like any change made to a design specification, any adopted change in a bridge design specification takes time to make its way into practice. This occurs for two reasons. One, there is always inertia for adoption of new procedures, and two, the life of a bridge from conception to construction has always been a multiyear process. Specification changes affect design at the concept stage, and the bridge may be built a couple of years later. Therefore, any specification change described in this chapter cannot be assumed to occur in practice at the same time. Typically it takes a couple of years for a specification change to evolve into real bridge practice, but it is also possible that engineers implemented a change before national specifications were changed.

This chapter frequently references editions of the AASHO or AASHTO bridge design specifications. Seventeen editions of the AASHO/AASHTO *Standard Specifications for Highway Bridges* were published between 1931 and 2002.(34,48,64–80) This chapter will refer to them only by year and edition number. Sometimes interim changes were published individually between editions.

DESIGN METHODOLOGY

Allowable stress design (ASD) was the only design method when AASHO was founded. Then, in 1971, the interims to the 10th edition introduced the concept of load factor design (LFD), and engineers could choose to design per ASD or LFD.(81) Both ASD and LFD were supported through the publication of the 2002 17th edition. The 1st edition of the AASHTO *Load and Resistance Factor Bridge Design Specifications* was introduced in 1994, creating a third possible design methodology, load and resistance factor design (LRFD).(82) In a policy memorandum published in June 2000, FHWA mandated the use of LRFD for all bridges designed with federal aid dollars by October 2007.(83) Conceivably, until 2007, bridges could have been designed using any of the three methods, although by the mid-2000s all states had fully transitioned to LRFD.

FASTENERS

Bridges constructed in the first half of the 1900s were exclusively riveted. Once high-strength bolts were introduced, use of rivets began to decline because of their greater cost and installation time compared with bolts. However, rivets remained in the AASHTO bridge design specifications until the 2002 17th edition, so technically their use may have occurred as late as that, although it is doubtful they were used much beyond the early 1970s (and that late they were primarily used only for field construction of truss members; i.e., welded truss members would have been riveted together at the truss nodes with gusset plates). Furthermore, the use of rivets to construct built-up bridge members likely did not occur beyond 1960 because of the comfort with welding at that time.

In terms of bolts, the 1931 1st and 1935 2nd editions stated that bolts could be used only with approval of the engineer.(64,66) The 1941 3rd edition softened this position, stating that bolts could be a designed element; however, at that time there were no bolt material specifications.(68) The year 1947 was a key year because two things occurred; first, ASTM adopted the A 307 bolt material specification, and second, the Research Council on Riveted and Bolted Structural Joints (RCRBSJ) was formed.(84) The purpose of the RCRBSJ was "To carry on investigations as deemed necessary to determine the suitability of various types of joints used in structural frames" (p. 2).(84) The RCRBSJ was also instrumental in developing the first high-strength bolt material specification, which ASTM published as A 325 in 1949.(85) In 1951, the RCRBSJ published their connection specification, *Specifications for Assembly of Structural Joints Using High-Strength Bolts*.(86) This publication could indicate the first use of high-strength bolted connections in steel bridges, but their first use was likely after publication of the 1957 7th edition, which clearly stated that high-strength bolts are an equivalent substitute for rivets.(72)

The 1961 8th edition directly listed the ASTM A 325 bolt material for use in bridges, and the bolts could be designed for direct tension and in friction connections, two design features not allowed for rivets. In 1964, ASTM published the first version of A 490, which is a bolt with even higher strength than A 325.(87) That same year, the 1964 8th edition interims included A 490 bolts in the design specification.(88) However, when the 1965 9th edition was published, the A 490 bolt provisions had been removed with no commentary describing why.(74) ASTM A 490 bolts were not reintroduced until the 1974 11th edition interims.(89)

In the late 1970s and early 1980s, there were noted issues with high-strength bolts used in bridge construction, particularly with galvanized fasteners that tended to strip. These issues led to an extensive research project funded by FHWA and summarized in the publication *High Strength Bolts for Bridges*, which found that the bolt industry was not regulating itself well and poor-quality product was being delivered to jobsites, with some even coming from foreign sources.(90) At the conclusion of the research, FHWA issued a memorandum titled *High Strength Bolts* on July 18, 1988, which provided supplemental contract requirements for owners to consider on projects that used high-strength bolts "…until such time when revised specifications are available."(91) Additionally, through the 1990s and early 2000s, FHWA sponsored training on the installation, qualification, and inspection of high-strength bolts to assist owners in understanding the nuances associated with proper installation of high-strength bolts.(92) Quality issues with high-strength bolt materials and installation methods would have been encountered during bridge construction and would have been remedied at that time. Therefore, it is not expected that high-strength bolts had much effect on the long-term performance of a bridge, beyond time-dependent problems such as relaxation or stress-corrosion cracking.

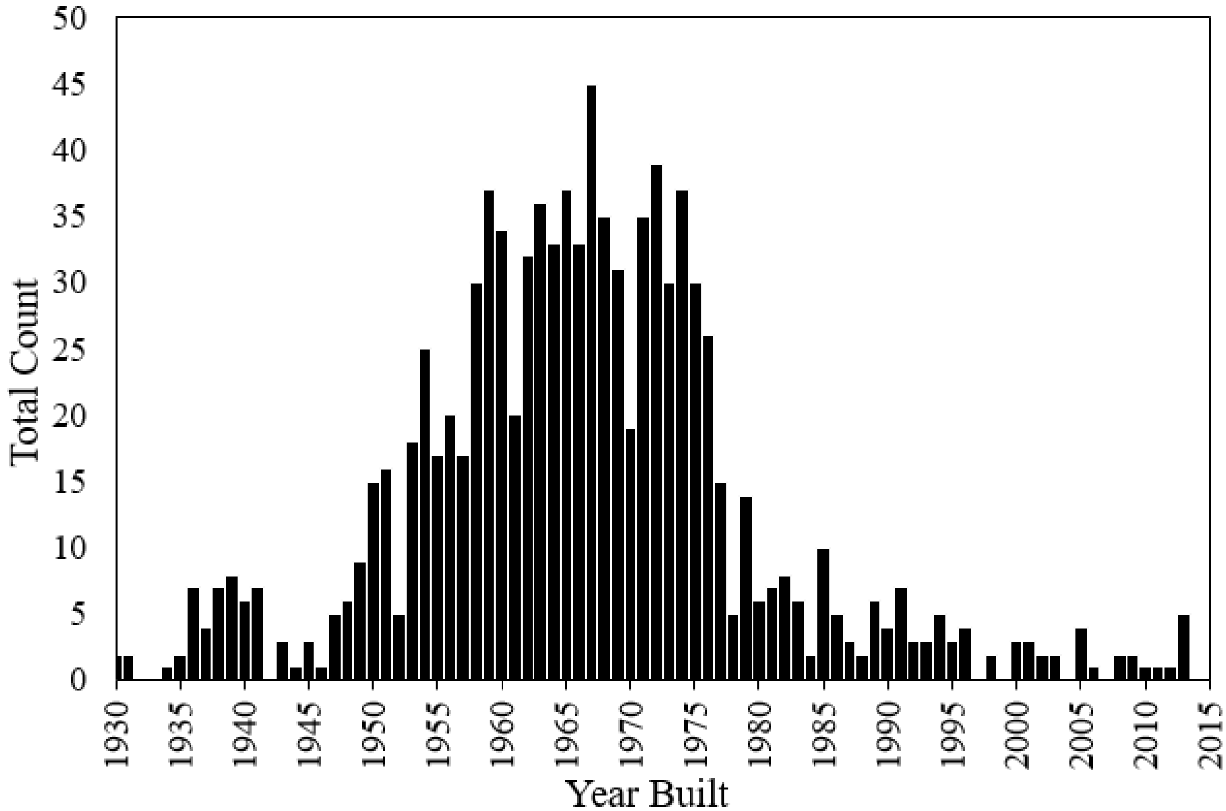
From the 1980s through the 2000s, twist-off bolts, ASTM F 1820 and ASTM F 2280 (effectively equivalent to A 325 and A 490, respectively), were introduced.(93,94) In addition, metric equivalents for A 325 and A 490 were used during the time period when AASHTO provided both imperial and metric design specifications. Through much of the last decade, there were six bolt specifications that were not substantively different, yet each had evolved with slight differences. In 2015, ASTM published F3125, *Standard Specification for High Strength Structural Bolts, Steel and Alloy Steel, Heat Treated, 120 ksi (830 MPa) and 150 ksi (1040 MPa) Minimum Tensile Strength, Inch and Metric Dimensions*, a high-strength bolt specification that combined ASTM A325, A325M, A490, A490M, F1852, and F2280 into one.(95,96) ASTM then withdrew the six prior individual specifications in 2016.

WELDING

The AWS was first established in 1919. They published the first structural welding code in 1928, *Code of Fusion Welding and Gas Cutting in Building Construction*.(97) Coincidently, the first welded bridge in the United States was built that same year to carry railroad traffic in Massachusetts.(98) In 1934, a committee was set up to develop a welding code specific to bridges, and in 1936, AWS published *Specifications for Welded Highway and Railway Bridges, Design, Construction, Alteration and Repair of Highway and Railway Bridges by Fusion Welding*.(99) Over time, these two documents evolved to have slightly different titles, and the one for buildings was deemed AWS D1.0 and the one for bridges was deemed AWS D2.0. In 1972, AWS D1.0 and D2.0 were merged into one document, AWS D1.1, *Structural Welding Code*, which was meant to cover both building and bridge construction.(57) However, AASHTO quickly realized that they would need supplemental requirements to work with AWS D1.1 and in 1974 published the first version of the *Standard Specifications for Welding of Structural Steel Highway Bridges* to supplement AWS D1.1.(57,58) The supplemental requirements provided additional requirements for welding ASTM A 514/A 517 material and more stringent inspection protocols, qualifications, and reporting requirements. These supplemental requirements were revised in 1975, a 2nd edition was published in 1977, and a 3rd edition was published in 1981.(100,101,102) At this point it was clear that the notion to combine the building and bridge welding codes was unworkable, and AASHTO and AWS formed a joint committee to develop and maintain a welding code specific to bridges. The first edition of the AASHTO/AWS D1.5 *Bridge Welding Code* was published in 1988.(63)

The tensile properties chapter discussed the publication of the AASHTO *Guide Specification for Fracture Critical Non-Redundant Steel Bridge Members* in 1978.(52) Although this document did introduce enhanced CVN requirements for base metal and weld metal, its main purpose was outlining more stringent welding requirements for FCMs. These additional requirements focused on hydrogen control through higher preheats, specific hydrogen designators for filler metal, enhanced inspection requirements, restrictions on repair, and others. The last edition of the AASHTO *Guide Specification for Fracture Critical Non-Redundant Steel Bridge Members* was published in 1989.(59) However, the 1995 2nd edition of the AASHTO/AWS D1.5 *Bridge Welding Code* fully integrated the old FC guide specification into a dedicated clause within the code so it could work harmoniously with the rest of the welding code.(60)

The 1931 1st edition allowed welding only for the repair of injurious defects and only with engineer approval; it is doubtful any highway bridges were welded based on this specification.(64) The 1935 2nd edition included welding in construction; however, its only guidance was welding "…shall be performed in accordance with the Specifications for Welded Steel Structures of the AASHO" (although the Specifications for Welded Steel Structures of the AASHO could not be found and checked for content).(67) Again, it is doubtful that much welding was performed on bridges designed using the 2nd edition.(67) The 1941 3rd edition allowed for welding in numerous applications on bridges, including "floor expansion devices, railings, built-up shoes, pedestals or expansion rockers, connections of diaphragms to beams or other members, stiffeners except at supports of beams and girders, and at connection to tension flanges stressed more than 75 percent capacity, filler plates, stay plate and lacing connections to members, connections and details of bracing, caps and baseplates, for trestle columns, splicing of steel piling, sidewalk brackets except main tie, fastening of cover plates to rolled beams, and other incidental part of the structure" (p. 166).[68] However, welding was "not permissible" on main members, so it is doubtful that welded plate girders were designed using this edition. Engineers were directed to the AWS *Welded Highway and Railway Bridges, Design, Construction, Alteration and Repair of Highway and Railway Bridges by Fusion Welding* for the specification of welding materials, weld design, and overall weld procedure.[99] The AWS specification precluded the application of welding to nickel (ASTM A 8) and silicon (ASTM A 94) steels; only ASTM A 7 steel could be considered for welding. Because ASTM A 7 controlled only phosphorus and sulfur content, the AWS specification limited carbon content to a maximum of 0.25 percent and manganese content to a maximum of 1.00 percent because it was recognized that these two elements greatly affect the weldability of the steel.[99] The 1945 4th edition softened the position such that welding to main members was "not recommended," and the 1949 5th edition did not preclude the use of welding within main members.[69,70] Based on the evidence presented, fully welded plate girders likely became mainstream after 1945; however, rolled beams with welded cover plates have likely existed since 1941. Further evidence to support this timeline is shown in figure 2, which is based on data parsed from the National Bridge Inventory. The figure shows a histogram of steel continuous, girder, and floorbeam bridges with longest spans of between 98 and 1,148 ft. The goal was to capture the inventory of two-girder bridges that would have been fully welded. The 98 ft. minimum length was thought to screen out rolled beams with cover plates because rolled sections of that period likely could not span longer than that. The figure demonstrates that these bridge populations began to increase around 1945 but then started to decline around 1975. The decline was likely due to engineers observing a national debate on redundancy, CVN specifications, and fracture-control plans in the wake of the Silver Bridge collapse. Because two-girder steel bridges have low redundancy (and indeed would eventually be categorized as FCM when the 1978 AASHTO *Guide Specification for Fracture Critical Non-Redundant Steel Bridge Members* was published), they fell out of favor.[52]



Source: FHWA.

Figure 2. Histogram. Steel continuous, girder, and floorbeam bridges with spans between 98 and 1,148 ft.

The remaining historical welding requirements were based on material type that could be welded. The 1957 7th edition and 1961 8th edition stated that welded members should be made from ASTM A 373 steel but secondary members could be made from ASTM A 7 steel, although the A 242, A 440, and A 441 specifications were allowable for mechanically fastened designs.(72,73) The 1964 8th edition interims expanded welding to include A 36, A 441, A 242, or other steels possessing the physical and chemical properties of A 373.(88) The 1965 9th edition listed only A 36, A 242, and A 441 as weldable, and the 1969 10th edition introduced A 572, A 588, and A 514/517 as weldable.(74,48)

FATIGUE DESIGN

Although not always called as such, fatigue has been a design consideration since the 1st edition AASHO specification.(64) Between the 1st edition and the 1962 8th edition interims, fatigue was a design consideration based on the "alternating stress" provisions.(64,103) Members with an algebraic difference in load from the dead, live, and centrifugal load analysis were required to be designed to resist loads that were artificially increased by 50 percent. Without being called fatigue design, this requirement effectively resulted in larger member cross sections to counteract the effects of alternating stresses, or fatigue. This requirement was copied from railroad bridge design specifications predating AASHO, and its exact origin is not known, so consideration of alternating stresses likely predated 1931.

In the 1963 8th edition interims, the alternating stress provisions were replaced with an article titled “Members and Connections Subject to Repeated Variations of Stress,” which introduced adjustments to the allowable stresses based on cycle count and load ratio.(104) The allowable stress was material dependent and based on the load ratio, or the ratio between minimum and maximum stress. The allowable stress and load ratio approach was tweaked over the next 10 yr. Interestingly, fatigue considerations for welded shear connectors were introduced in the 1966 9th edition interims; they were independent of the allowable stress and load ratio approach.(105) The 1974 11th edition interims completely changed the fatigue design approach to the modern stress–life (S-N) approach used today.(58)

The S-N fatigue design procedures introduced in 1974 virtually eliminated all load-controlled fatigue cracking problems commonly seen in welded bridges constructed before this time. However, displacement-controlled fatigue cracking, particularly at web gaps between girder flanges and ends of connection plates, continued to be a fatigue cracking nuisance. This type of cracking is due to a long-standing prohibition of welding to tension flanges for fear of brittle fracture. In the case of connection plates, when they are not connected to girder flanges, the differential displacement between adjacent girders results in flexing of the girder web and growth of fatigue cracks. The 1985 13th edition interims were modified to require connection plates to be rigidly attached to both girder flanges, so bridges built after 1985 would not be expected to develop distortional-induced fatigue cracks.(106)

COMPOSITE DESIGN

Composite design considers that beams and girders are positively connected with the concrete deck using shear connectors behaving together in bending to provide more strength than the beam or girder by itself. In the United States, composite design procedures were based on testing conducted at the University of Illinois in the mid-1940s, although composite design originated in Europe.(107) Composite design provisions were first introduced in the 1945 4th edition, so compositely designed bridges conceivably could have existed as early as 1945.(69) This edition provided criteria for determining the effective width of the concrete slab for use in strength calculations. Effective width was determined as follows:

- For an interior girder or beam, effective width is the minimum of the span length divided by 4, the center-to-center distance between adjacent beams, and 12 times the slab thickness.

- For an exterior girder or beam, effective width is the minimum of the span length divided by 12, the center-to-center distance to the adjacent beam divided by 2, and 6 times the slab thickness.

It is not clear where these criteria came from because the AASHO publication date was before the University of Illinois testing began. This edition did not provide requirements for the strength of an individual shear connector, stating only that the pitch of the connectors was to be based on dividing the elastic shear flow by the strength of an individual shear connector, although the pitch was not to exceed 2 ft. The maximum pitch was editorially changed in the 7th edition to be 24 inches, and this requirement has remained unchanged through all other editions of the Standard and LRFD Specifications.

Engineers have always had the choice between composite and noncomposite design, although since the inception of the LRFD design philosophy, noncomposite flexure design has not been recommended (Article C6.10.1).[82] AASHO/AASHTO has never provided equations for partial composite design.

Shear Connector Ultimate Strength

The 7th edition provided predictive equations for calculating the strength of an individual shear connector, as shown in table 21.[72] These strength equations came from Viest and Siess for channel shear connectors and Veist for welded stud connectors.[108,109] The individual shear connector strengths were to be reduced by a safety factor. The safety factor was calculated from an equation based on moment and shear ratios (which vary depending on whether the member is shored or unshored). If the predictive strength equations reduced by the safety factor were used to proportion the shear connectors across the length of the beam, fatigue would not govern design, based on limited testing.[108] In the 9th edition, the safety factor equation was eliminated and the safety factor value was fixed at 3.0; therefore, all the coefficients in the strength equation were divided by 3.0.[74] The shear connector strength provisions up to this point were based on preventing slip between the steel and concrete, and it was observed that an uneconomical number of shear connectors was required.[109] This observation led to a joint research study initiated in 1960 at Lehigh University to determine whether shear connector pitch could be relaxed if some level of slip could be tolerated.[110] This research resulted in a significant change in shear connector design that was published in the 1966 interims.[105] As shown in table 21, the coefficients in front of the shear connector strength equation became much larger, resulting in a nearly 10-fold increase in shear connector resistance. Welded studs were then simplified into one equation where the height-to-diameter ratio had to be greater than 4.0. The specification also required the shear connector resistance to be reduced by 0.85. According to Slutter and Fisher, this reduction factor was necessary because shear connectors are a "connection" element requiring an extra margin against failure. They further remarked that a reduction factor of 0.85 seemed "reasonable" considering the "ultimate flexural capacity of composite bridge beams is usually 2.5 or more times the working load moment, [so] the corresponding margin for the shear connection would be approximately 3 or greater [with the 0.85 factor]."[111] This 0.85 reduction factor was maintained through all subsequent editions of the Standard and LRFD Specifications. The increase in shear connector resistance would result in fewer shear connectors, and fatigue would become a failure mode that had to be considered in design. Therefore, a shear connector fatigue design procedure was also provided in the 1966 interims that are discussed in the following section.[105] For the remainder of the Standard Specifications, the shear connector strength remained essentially unchanged, with only slight modifications as seen in table 21. In the 1974 interims, the modulus of concrete term was added into the square root term, resulting in a commensurate change in the coefficient to 0.4. In the 1999 interims, the stud strength equation was modified to cap the resistance to the tensile strength of the stud.[112] The 1st edition of LRFD revised the shear connector strengths. Although it appears that the strength of a channel connector was also revised, it was only rewritten to move the modulus of concrete explicitly into the square root term, as was done in the 1974 interims for welded studs. However, the LRFD did adopt a modified welded stud strength equation with a coefficient of 0.5 instead of 0.4. This change resulted from a 1971 revised analysis of stud strength data that considered lightweight concrete that was not considered in the Lehigh University research in the mid-1960s.[110,113]

Table 21. Evolution of individual shear connector strengths.

AASHO/AASHTO Edition	Strength of Individual Connector	
1944 4th Standard Specifications[69]	No specific provisions	
1957 7th Standard Specifications[57]	Channel $180(h + t/2)w\sqrt{f_c'}$	(2)
	Welded stud ($H/d > 4.2$) $330d^2\sqrt{f_c'}$	(3)
	Welded stud ($H/d < 4.2$) $80Hd\sqrt{f_c'}$	(4)
	Helical bars $3840d^4\sqrt{f_c'}$	(5)
1965 9th Standard Specifications[74]	Channel $60(h + t/2)w\sqrt{f_c'}$	(6)
	Welded stud ($H/d > 4.2$) $110d^2\sqrt{f_c'}$	(7)
	Welded stud ($H/d < 4.2$) No change	
	Helical bars $1280d^4\sqrt{f_c'}$	(8)
1966 Standard Specifications Interims[105]	Channel $550(h + t/2)w\sqrt{f_c'}$	(9)
	Welded stud ($H/d > 4.0$) $930d^2\sqrt{f_c'}$	(10)
1974 Standard Specifications Interims[89]	Channel No change	
	Welded stud ($H/d > 4.0$) $0.4d^2\sqrt{f_c' E_c}$	(11)
1999 Standard Specifications Interims[112]	Channel No change	
	Welded stud ($H/d > 4.0$) $0.4d^2\sqrt{f_c' E_c} \leq 60{,}000A_{sc}$	(12)
1994 LRFD Specifications[82]	Channel $0.3(h + t/2)w\sqrt{f_c' E_c}$	(13)

AASHO/AASHTO Edition	Strength of Individual Connector
	Welded stud ($H/d > 4.0$) $0.5d^2\sqrt{f_c' E_c} \leq F_u A_{sc}$ (14)

A_{sc} = cross-sectional area of welded stud; d = diameter of stud; E_c = modulus of concrete; f_c = compressive strength of concrete; H = height of stud; h =average flange thickness of channel; t = web thickness of channel; w = length of channel.

Shear Connector Fatigue Strength

The fatigue design adopted in the 1966 interims calculated the individual shear connector force range resistance, Z_r, as shown in equation 15 for channel connectors and equation 16 for welded stud connectors. The coefficients B and α for these two equations are in table 22. These were based on Lehigh University recommendations, albeit with slightly changed coefficients because of rounding.[110]

$$Z_r = Bw \tag{15}$$

$$Z_r = \alpha d^2 \tag{16}$$

Table 22. Coefficients for equations 15 and 16.

Number of Design Cycles	Coefficient, *B*	Coefficient, *α*
100,000	4,000	13,000
500,000	3,000	10,600
2,000,000	2,400	7,800
Over 2,000,000[a]	2,100	5,500

[a]This condition was added in the 1977 interims.

These fatigue design provisions remained unchanged through all editions of the Standard and LRFD specifications. Although the formulations in LRFD look different, they are equivalent to those existing after 1977 for two reasons. First, the coefficients from table 22 were replaced with an equation to provide a continuous function rather than fixed steps, although the equation predicts the same values as shown in table 22. Second, the LRFD fatigue equation was written in an explicit finite and infinite life formulation, although the infinite life portion uses the same coefficient as in the "Over 2,000,000" row in table 22.

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