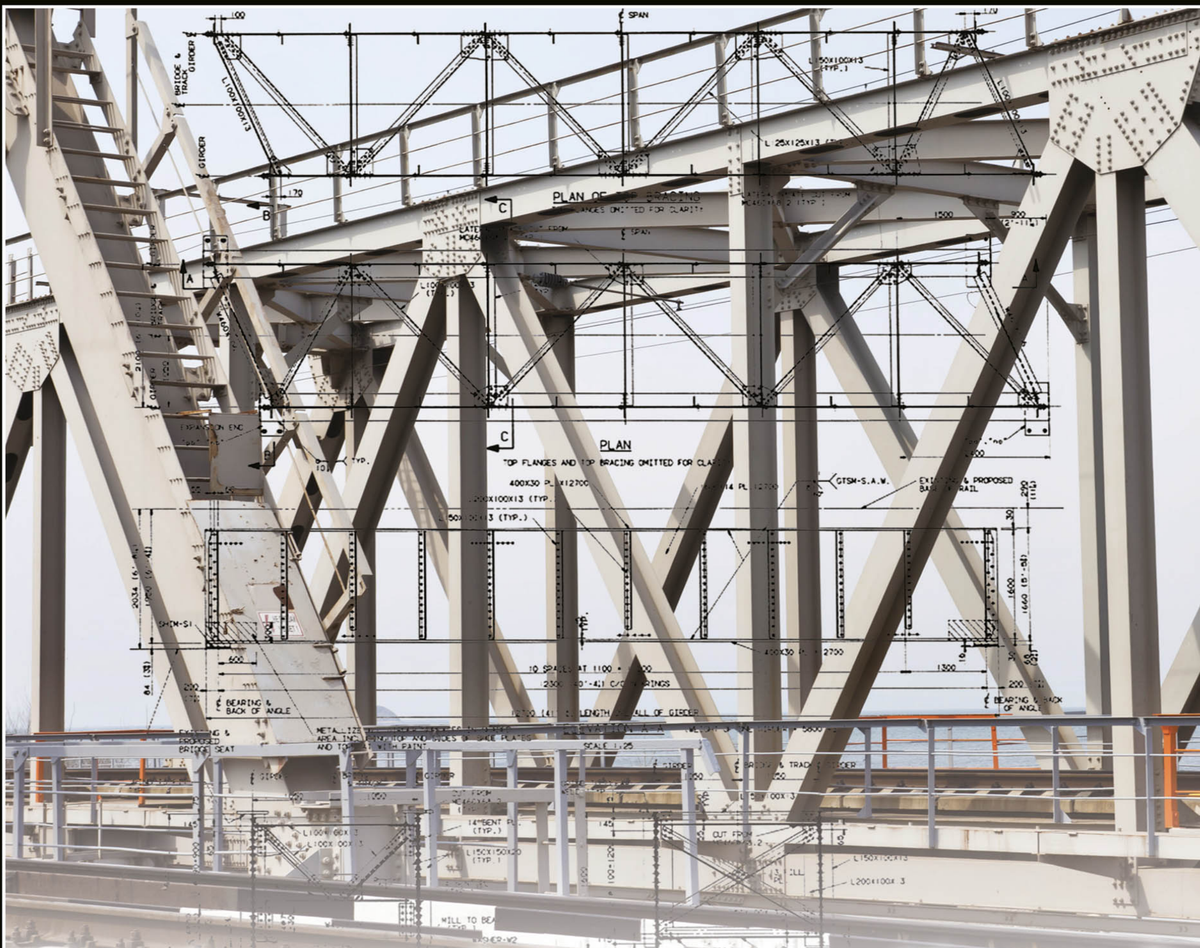
Second Edition



Design and Construction of Modern Steel Railway Bridges

John F. Unsworth

CRC Press
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By
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Dedication

To my extraordinary wife, Elizabeth, whose steadfast support and patience made this book and most other good things in my life possible.

Contents

Acknowledgment

I respectfully acknowledge my father, who provided opportunities for an early interest in science and engineering, and my mother, whose unconditional support in all matters has been acutely treasured. My wife, Elizabeth, deserves singular recognition for everything she does, and the kind and thoughtful way in which she does it. She, my daughters, Tiffany and Genevieve, and granddaughter, Johanna, are perpetual sources of joy and inspiration. In addition, the professional guidance and friendship provided by many esteemed colleagues, in particular Dr. R. A. P. Sweeney, W. G. Byers, and W. B. Conway, must be gratefully acknowledged.

Preface to the Second Edition

The first edition of this book provided nine chapters with a focus on the design of new steel superstructures for modern railway bridges referencing the recommended practices of Chapter 15—Steel Structures in the 2008 edition of the American Railway Engineering and Maintenance-of-way Association (AREMA) Manual for Railway Engineering (MRE). This second edition updates the first edition by including changes precipitated by subsequent revisions to Chapter 15 of the MRE and the many valuable comments received from steel railway bridge design engineers, fabricators, students, academics, and researchers. Grateful appreciation is due to all those who offered comments and suggestions and, in particular, to the many members of AREMA Committee 15—Steel Structures, whose experience and expertise, helped improve the technical content of this second edition.

A notable amendment to this second edition is the use of Système Internationale (SI) units in addition to US Customary or Imperial units throughout[*] the book. Moreover, in this second edition, attention has been expanded to include two new chapters on the construction (fabrication and erection) of new steel superstructures for modern railway bridges.

This second edition is divided into eleven chapters. The first three chapters deliver introductory and general information as a foundation for the subsequent six chapters examining the detailed analysis and design, which precede two chapters concerning fabrication and erection, of modern steel railway superstructures.

Chapter 1 is retained as a brief history of iron and steel railway bridges. The chapter concludes with the evolution and advancement of structural mechanics and design practice precipitated by steel railway bridge development.

A discussion regarding the manufacture of structural steel (steel making) has been included in Chapter 2 as a prelude to the material concerning the engineering properties and types of structural steel used in modern railway superstructure design and fabrication.

The information in Chapter 3 concerning the planning of steel railway bridges is enhanced with additional material regarding bridge scour investigation in accordance with AREMA (2015). Added to the discussion about preliminary design is a brief introduction to probabilistic structural design in terms of modern steel railway superstructure design issues.

The next two chapters concerning the development of loads and structural analysis of modern steel railway bridge superstructures have been substantially updated.

The discussion of railway live loads in Chapter 4 is enhanced with a discussion of the historical development of modern freight train design live loads. Material concerning the fatigue design load, which was included in Chapter 5 of the first edition, is now more appropriately included in Chapter 4 as it specifically relates to the modern freight train design live load. The discussion of the freight train live load in Chapter 4 has been extensively revised using an approach originating with modern vehicle–bridge interaction (VBI) dynamics concepts. The VBI models are reduced to dynamic moving sprung mass, mass and force problems to examine the theoretical foundations of railway live load impact. The load combination table at the end of Chapter 4 has been updated based on thoughtful review by many members of AREMA Committee 15. Chapter 5 now also includes material concerning the lateral deflection of steel superstructures based on recent revisions to AREMA Chapter 15 that provide for better control of track geometry.[†] The material concerning fatigue strength or resistance remains in Chapter 5.

[*] In a very few cases only, US Customary or Imperial units are used. This typically occurs when equations or formulas are developed empirically in US Customary or Imperial units.

[*] As a safety measure, as train speeds increase, track geometry tolerances decrease (become more stringent).

Chapters 6–9 remain to outline the design of members and connections in accordance with AREMA (2015). The postbuckling shear strength of plate girder web plates is not included in AREMA (2015) due to intolerance of such behavior in railway superstructures. Nevertheless, a brief introduction to plate girder web plate postbuckling strength is included in Chapter 7 of the second edition as information concerning the ultimate behavior of plate girder superstructures. Chapter 9 includes updated information regarding the design shear strength of slip-critical connections. Other information in these four chapters has also been updated in accordance with the applicable revisions to AREMA Chapter 15 since the first edition of this book.

Chapters 10 and 11 are new to the second edition. Much of the subject matter considered in Chapters 2, 5, 6, 7, and 9 is affected by fabrication. Consequently, and because steel superstructure design ultimately culminates in fabrication, Chapter 10 concerning the planning, processes, execution, and inspection of fabricated members and assemblies has been incorporated. Since, steel superstructure erection logically trails fabrication and concludes the project, Chapter 11 outlining some typical practices of steel railway superstructure erection planning, equipment, engineering, and execution, follows Chapter 10 to conclude the book.

Appendices outlining the design of a ballasted through plate girder (BTPG) and a ballasted deck plate girder (BDPG) superstructure are included in the second edition to complement the material presented in the book. An appendix has also been included as a précis of the common engineering unit conversions used in the book. Conversions between SI and US Customary or Imperial units and vice versa are presented.

This second edition remains as only one constituent of the information essential for the design and construction of safe and reliable modern steel railway superstructures. Other sources of technical information are also necessary and, again, it is anticipated that, where such material is referenced in this book, proper attribution has been appropriately expressed.

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John F. Unsworth is a professional engineer (P Eng). Since his completion of a bachelor of engineering degree in civil engineering and a master of engineering degree in structural engineering, he has held professional engineering and management positions concerning track, bridge, and structures maintenance, design, and construction at the Canadian Pacific Railway.

He is a former president of the American Railway Engineering and Maintenance-of-way Association (AREMA) and has served as Chairman of AREMA Committee 15—Steel Structures. He currently serves as an emeritus member of AREMA Committee 15. In addition, he is the current Chair of the Association of American Railroads (AAR) Bridge Research Advisory Group and is a former member of the National Academy of Sciences Transportation Research Board (TRB) Steel Bridges Committee. He is also a member of the Canadian Society for Civil Engineering (CSCE), the American Society of Civil Engineers (ASCE), the American Institute of Steel Construction (AISC), and the International Association of Bridge and Structural Engineers (IABSE). He is a licensed professional engineer in six Canadian provinces.

He has written papers and presented them at AREMA Annual Technical Conferences, the International Conference on Arch Bridges, TRB Annual Meetings, CSCE Bridge Conferences, the ASCE Structures Congress, and the International Bridge Conference (IBC). He has also contributed to the fifth edition of the *Structural Steel Designer's Handbook* and the *International Heavy Haul Association (IHHA) Best Practices* books.

1 History and Development of Steel Railway Bridges

1.1 INTRODUCTION

The need for reliable transportation systems evolved with the industrial revolution. By the early 19th century, it was necessary to transport materials, finished goods, and people over greater distances in shorter times. These societal requirements, in conjunction with the development of steam power,* heralded the birth of the railroad. The steam locomotive with a trailing train of passenger or freight cars on iron rails became the principal means of transportation. Accordingly, as transportation improvements were required, the railroad industry became the primary catalyst in the evolution of materials and engineering mechanics in the latter half of the 19th century.

The railroad revolutionized the 19th century. Railroad transportation commenced in the UK on the Stockton to Darlington Railway in 1823 and on the Liverpool and Manchester Railway in 1830. The first commercial railroad in the United States was the Baltimore and Ohio (B&O) Railroad, which was chartered in 1827.

Construction of the associated railroad infrastructure required that a great many wood, masonry, and metal bridges be built. Bridges were required for live loads that had not been previously encountered by bridge builders.† The first railroad bridge in the United States was a wooden arch-stiffened truss built by the B&O in 1830. Rapid railroad expansion‡ and increasing locomotive weights, particularly in the United States following the Civil War, provoked a strong demand for longer and stronger railway bridges. In response, many metal girder, arch, truss, and suspension bridges were built to accommodate railroad expansion, which was occurring simultaneously in the United States and the UK following the British industrial revolution.

In the United States, there was intense competition among emerging railroad companies to expand west. Nevertheless, crossing the Mississippi River was the greatest challenge to planned railroad growth. The first railway bridge across the Mississippi River was completed in 1856 by the Chicago, Rock Island, and Pacific Railroad.§ The efforts of the B&O Railroad company to expand its business and to cross the Mississippi River at St. Louis, MO, commencing in 1839¶ and finally realized in 1874, proved to be a milestone in steel railway bridge design and construction. Although the St. Louis Bridge** never served the volume of the railway traffic anticipated in 1869 at the start of construction, its engineering involved many innovations that provided the foundation for long-span railway bridge design for many years following its completion in 1874.

* Nicolas Cugnot is credited with production of the first steam-powered vehicle in 1769. Small steam-powered industrial carts and trams were manufactured in the UK in the early years of the 19th century and George Stephenson built the first steam locomotive, the "Rocket," for use on the Liverpool and Manchester Railway in 1829.

† Before steam locomotives, bridges carried primarily pedestrian, equestrian, and light cart traffic. Railroad locomotive axle loads were about 50 kN (11,000 lbs) on the Baltimore and Ohio Railroad in 1835.

‡ For example, in the 1840s charters to hundreds of railway companies were issued by the British government.

§ The bridge was constructed by the Rock Island Bridge Company after US railroad companies received approval to construct bridges across navigable waterways. The landmark Supreme Court case that enabled construction of this bridge across the Mississippi River also provided national exposure to the Rock Island Bridge Company solicitor, Abraham Lincoln.

¶ In 1849, Charles Ellet, who designed the ill-fated suspension bridge at Wheeling, WV, was the first engineer to develop preliminary plans for a railway suspension bridge to cross the Mississippi River at St. Louis, MO. Costs were considered prohibitive, as were subsequent suspension bridge proposals by J. A. Roebling, and the project was never commenced.

** Now known as the Eads Bridge in honor of its builder, James Eads.

The need for longer and stronger railway bridges precipitated an evolution of materials from wood and masonry to cast and wrought iron, and eventually to steel. Many advances and innovations in engineering mechanics and construction technology can also be attributed to the development of the railroads and their need for more robust bridges of greater span.

1.2 IRON RAILWAY BRIDGES

1.2.1 Cast Iron Construction

A large demand for railway bridges was generated as railroads in the UK and the United States prospered and expanded. Masonry and timber were the principal materials of early railway bridge construction, but new materials were required to span the greater distances and carry the heavier loads associated with railroad expansion. Cast iron had been used in 1779 for the construction of the first metal bridge, a 30.5 m (100 ft) arch span over the Severn River at Coalbrookdale, UK. The first bridge to use cast iron in the United States was the 24.5 m (80 ft) arch, built in 1839, at Brownsville, PA. Cast iron arches* were some of the first metal railway bridges constructed, and their use expanded with the rapidly developing railroad industry. Table 1.1 indicates some notable cast iron arch railway bridges constructed between 1847 and 1861.

The oldest cast iron railway bridge in existence is the 14 m (47 ft) trough girder at Merthyr Tydfil in South Wales, which was built in 1793 to carry an industrial rail tram. The first iron railway bridge for use by the general public on a chartered railroad was built in 1823 by George Stephenson on the Stockton to Darlington Railway (Figure 1.1). The bridge consisted of 3.8 m (12.5 ft) long lenticular spans† in a trestle arrangement. This early trestle was a precursor to the many trestles that would be constructed by railroads to enable almost level crossings of wide and/or deep valleys. Table 1.2 summarizes some notable cast iron railway trestles constructed between 1823 and 1860.

George Stephenson's son, Robert, and Isambard Kingdom Brunel were British railway engineers, who understood cast iron material behavior and the effects of moving railway loads on arches. They successfully built cast iron arch bridges that were designed to act in compression. However, the

FIGURE 1.1 Gaunless River Bridge of the Stockton and Darlington Railway built in 1825 at West Auckland, UK. (From Chris Lloyd, The Northern Echo, Darlington.)

* Cast iron bridge connections were made with bolts because the brittle cast iron would crack under pressures exerted by rivets as they shrank from cooling.

† Also referred to as Pauli spans.

TABLE 1.1
Notable Iron and Steel Arch Railway Bridges Constructed during 1847–1916

Location	Railroad	Engineer	Year	Material	Hinges	Span m (ft)
Thirsk, UK	Leeds & Thirsk	—	1847	Cast iron	0	—
Newcastle, UK	Northeastern	R. Stephenson	1849	Cast iron	0	38 (125)
Oltwn, Switzerland	Swiss Central	Etzel & Riggenbach	1853	Wrought iron	0	31 (103)
Paris, France	Paris-Aire	—	1854	Wrought iron	2	45 (148)
Victoria, Bewdley, UK	—	J. Fowler	1861	Cast iron	—	
Albert, UK	—	J. Fowler	1861	Cast iron	—	
Coblenz, Germany	—	—	1864	Wrought iron	2	—
Albert, Glasgow, Scotland	—	Bell & Miller	1870	Wrought iron	—	—
St. Louis, MO	Various	T. Cooper & J. Eads	1874	Cast steel	0	159 (520)
Garabit, France		G. Eiffel	1884	Wrought iron	2	165 (540)
Paderno, Italy	—	—	1889	Iron	—	150 (492)
Stony Creek, BC	Canadian Pacific	H. E. Vautelet	1893	Steel	3	102 (336)
Keefers, Salmon River, BC	Canadian Pacific	H. E. Vautelet	1893	Steel	3	82 (270)
Surprise Creek, BC	Canadian Pacific	H. E. Vautelet	1897	Steel	3	88 (290)
Grunenthal, Germany	—	—	1892	Steel	2	156 (513)
Levensau, Germany	—		1894	Steel	2	163 (536)
Mungsten, Prussia	—	A. Rieppel	1896	Steel	0	170 (558)
Niagara Gorge[a], NY	—	L. L. Buck	1897	Steel	2	168 (550)
Viaur Viaduct, France	—	—	1898	Steel	0	220 (721)
Worms, Germany	—	Schneider & Frintzen	1899	Steel	—	66 (217)
Yukon, Canada	Whitepass & Yukon	—	—	Steel	0	73 (240)
Passy Viaduct, France	Western Railway of Paris	—	—	Steel	—	86 (281)

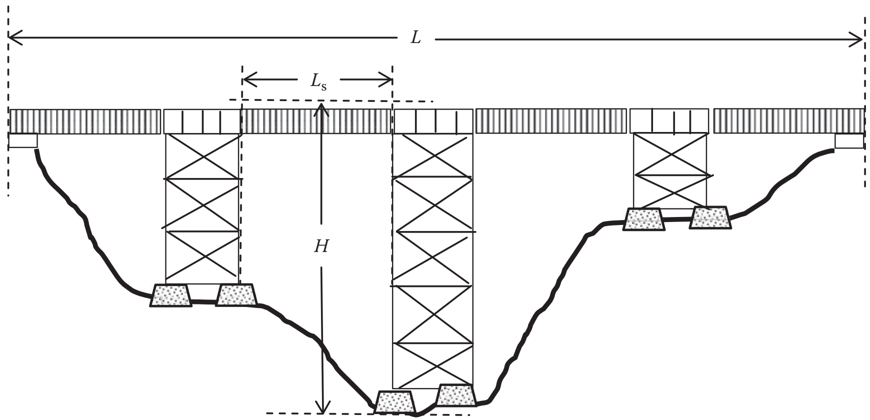
(*Continued*)

TABLE 1.1 (*Continued*)
Notable Iron and Steel Arch Railway Bridges Constructed during 1847–1916

Location	Railroad	Engineer	Year	Material	Hinges	Span m (ft)
Rio Grande, Costa Rica	Narrow gage	—	1902	Steel	2	137 (448)
Birmingham, AL	Cleveland & Southwestern Traction	—	1902	Steel	—	—
Kingsford, WI	Chicago, Milwaukee & St. Paul	—	1902	Steel	3	63 (207)
Mainz, Germany	—	—	1904	Steel	—	—
Estacada, OR	Oregon, Water, Power and Railway	G. Brown	1904	Steel	—	61 (200)
Paris, France	Metropolitan	—	1905	Steel	—	140 (460)
Song-Ma, China	Indo-China	—	—	Steel	3	162 (532)
Iron Mountain, MI	Iron ore	—	—	Steel	3	—
Zambesi, Rhodesia	—	G. A. Hobson	1905	Steel	—	152 (500)
Thermopylae, Greece	—	P. Bodin	1906	Steel	3	80 (262)
Nami-Ti Gorge, China	Yunnan	—	1909	Steel	3	55 (180)
Bend, OR	Spokane, Portland & Seattle	R. Modjeski	1911	Steel	2	107 (350)
Stillwater, MN	Wisconsin Central	C. Turner	1911	Steel	3	107 (350)
Lytton, BC	Canadian Northern	J. A. L. Waddell	1915	Steel	2	137 (450)
Hell Gate, NY	New England Connecting	G. Lindenthal	1916	Steel	2	298 (978)

[a] Indicates the second bridge constructed at the site.

TABLE 1.2
Notable Iron and Steel Viaduct Railway Bridges Constructed during 1823–1909



Viaduct	Railroad	Engineer	Year	Material	L_S m (ft)	L m (ft)	H m (ft)
Gauntless, UK	Stockton to Darlington	G. Stephenson	1823	Cast iron	3.8 (12.5)	15 (50)	~4.5 (~15)
Newcastle, UK	Northwestern	I. K. Brunel	1849	Cast and wrought iron	38 (125)	229 (750)	25(83)
Tray Run	Baltimore & Ohio	A. Fink	1853	Cast iron	—	136 (445)	18 (58)
Buckeye	Baltimore & Ohio	A. Fink	1853	Cast iron	—	107 (350)	14 (46)
Crumlin, UK	Newport & Hereford	Liddell & Gordon	1857	Wrought iron	46 (150)	549 (1800)	64 (210)
Guth, PA, Jordan Creek	Catasauqua & Fogelsville	F. C. Lowthorp	1857	Cast and wrought iron	30.5 (100), 33.5 (110)	342 (1122)	27 (89)
Belah, UK	—	Sir T. Bouch	1860	Cast and wrought iron	13.5 (45)	293 (960)	55 (180)
Weston, ON	Grand Trunk		1860	Iron	22 (72)	198 (650)	21 (70)
Fribourg, Switzerland	—	Mathieu	1863	Iron	48 (158)	396 (1300)	76 (250)
Creuse, Busseau, France	—	Nordling	1865	Iron	—	287 (940)	48 (158)
La Cere, France	Orleans	Nordling	1866	Iron	—	236 (775)	53 (175)
Assenheim, Germany	—	—	~1866	Iron	—	—	—
Angelroda, Germany	—	—	~1866	Iron	30.5 (100)	92 (300)	—

(*Continued*)

TABLE 1.2 (*Continued*)
Notable Iron and Steel Viaduct Railway Bridges Constructed during 1823–1909

Viaduct	Railroad	Engineer	Year	Material	L_S m (ft)	*L* m (ft)	*H* m (ft)
Bullock Pen	Cincinnati & Louisville	F. H. Smith	1868	Iron		143 (470)	18 (60)
Lyon Brook, NY	New York, Oswego & Midland	—	1869	Wrought iron	9 (30)	250 (820)	49 (162)
Rapallo Viaduct	New Haven, Middletown & Willimantic	—	1869	Iron	9 (30)	421 (1380)	18 (60)
St. Charles Bridge over Mississippi	—	—	1871	—	—	—	—
La Bouble, France	Commentary-Gannat	Nordling	1871	Wrought iron	49 (160)	396 (1300)	66 (216)
Bellon Viaduct, France	Commentary-Gannat	Nordling	1871	Steel	40 (131)	—	49 (160)
Verragus, Peru	Lima & Oroya	C. H. Latrobe	1872	Wrought iron	33.5 (110), 38 (125)	175 (575)	78 (256)
Olter, France	Commentary-Gannat	Nordling	1873	Steel	—	—	—
St. Gall, France	Commentary-Gannat	Nordling	1873	Steel	—	—	—
Horse Shoe Run	Cincinnati Southern	G. Bouscaren	~1873	Wrought iron	—	274 (900)	27 (89)
Cumberland	Cincinnati Southern	G. Bouscaren	~1873	Wrought iron	—	—	30.5 (100)
Tray Run[b]	Baltimore & Ohio	—	1875	Steel			18 (58)
Fishing Creek	Cincinnati Southern	G. Bouscaren	1876	Wrought iron	—	—	24 (79)
McKees Branch	Cincinnati Southern	G. Bouscaren	1878	Wrought iron	—	—	39 (128)
Portage, NY	Erie	G. S. Morison	1875	Iron	15.3 (50), 30.5 (100)	249 (818)	62 (203)
Staithes, UK	Whitby & Loftus	J. Dixon	1880	—	—	210 (690)	45 (150)
Oak Orchard, Rochester, NY	Rome, Watertown and Western	—	~1881	Steel	9 (30)	210 (690)	24 (80)
Kinzua[a], PA	New York, Lake Erie and Western	Clarke, Reeves & Co.	1882	Wrought iron	—	626 (2053)	92 (302)
Rosedale, Toronto, ON	Ontario & Quebec	—	1882	—	9 (30),18 (60)	—	—
Dowery Dell, UK	Midland	Sir T. Bouch	~1882				
Marent Gulch, MT	Northern Pacific	—	1884	Steel	35 (116)	244 (800)	61 (200)
Loa, Bolivia	Antofagasta	—	1885–1890	—	—	244 (800)	102 (336)
Malleco, Chile	—	A. Lasterria	1885–1890	—	—	366 (1200)	95 (310)
Souleuvre, France	—	—	1885–1890	—	—	366 (1200)	75 (247)
Moldeau, Germany	—	—	1885–1890	—	—	270 (886)	65 (214)

(*Continued*)

TABLE 1.2 (*Continued*)
Notable Iron and Steel Viaduct Railway Bridges Constructed during 1823–1909

Viaduct	Railroad	Engineer	Year	Material	L_S m (ft)	L m (ft)	H m (ft)
Schwarzenburg, Germany	—	—	1889	Steel	—	—	—
Panther Creek, PA	Wilkes-Barre & Eastern	—	1893	Steel	—	503 (1650)	47 (154)
Pecos, CA	—	—	1894	Steel	—	665 (2180)	98 (320)
Grasshopper Creek	Chicago & Eastern Illinois	—	1899	Steel	—	—	—
Lyon Brook[b], NY	New York, Ontario & Western	—	1894	Steel	30	250 (820)	49 (162)
Kinzua[b], PA	New York, Lake Erie and Western	C. R. Grimm	1900	Steel	—	626 (2052)	92 (302)
Gokteik, Burma	Burma	Sir A. Rendel	1900	Steel	—	690 (2260)	100 (320)
Boone, IA	Chicago & Northwestern	G. S. Morison	1901	Steel	13.5 (45), 23 (75), 92 (300)	819 (2685)	56 (185)
Portage, NY[b]	Erie	—	1903	Steel	15.2 (50), 30.5 (100)	249 (818)	62 (203)
Richland Creek, IN	—	—	1906	Steel	12 (40), 23 (75)	—	48 (158)
Moodna Creek	Erie	—	1907	Steel	12 (40), 24.5 (80)	976 (3200)	56 (182)
Colfax, CA	—	—	1908	Steel	—	247 (810)	58 (190)
Makatote, New Zealand	—	—	1908	Steel	—	262 (860)	92 (300)
Cap Rouge, QC	Transcontinental	—	1908	Steel	12 (40), 18.3 (60)	—	53 (173)
Battle River, AB	Grand Trunk Pacific	—	1909	Steel	—	~823 (~2700)	56 (184)
Lethbridge, AB	Canadian Pacific	Monsarrat & Schneider	1909	Steel	20 (67), 30.5 (100)	1625 (5328)	96 (314)

[a] Indicates the first bridge constructed at the site.
[b] Indicates the second bridge constructed at the site.

relatively level grades required for train operations (due to the limited tractive effort available to early locomotives) and use of heavier locomotives also provided motivation for the extensive use of cast iron girder and truss spans for railway bridges.

Commencing about 1830, Robert Stephenson built both cast iron arch and girder railway bridges in the UK. Cast iron plate girders were also built in the United States by the B&O Railroad in 1846, the Pennsylvania Railroad in 1853, and the Boston and Albany Railroad in 1860. The B&O Railroad constructed the first cast iron girder trestles in the United States in 1853. One of the first cast iron railway viaducts in Europe was constructed in 1857 for the Newport to Hereford Railway line at Crumlin, UK. Nevertheless, while many cast iron arches and girders were built in the UK and the United States, American railroads favored the use of composite trusses of wood and iron.

American railroad trusses built after 1840 were often constructed using cast iron, wrought iron, and timber members. In particular, Howe trusses with wood and cast iron compression members and wrought iron tension members were widely used in early American railroad bridge construction.

The failure of a cast iron girder railway bridge in 1847[*] stimulated an interest in wrought iron construction among British railway engineers.[†] British engineers were concerned with the effect of railway locomotive impact on cast iron railway bridges, and they were beginning to understand that, while strong, cast iron was brittle and prone to sudden failure. Concurrently, American engineers were becoming alarmed by cast iron railway bridge failures, and some even promoted the exclusive use of masonry or timber for railway bridge construction. For example, following the collapse of an iron truss bridge in 1850 on the Erie Railroad, some American railroads dismantled their iron trusses and replaced them with wood trusses. However, the practice of constructing railway bridges using iron was never discontinued on the B&O Railroad.

European and American engineers realized that a more ductile material was required to resist the tensile forces developed by heavy railroad locomotive loads. Wrought iron[‡] provided this increase in material ductility, and it was integrated into the construction of many railway bridges after 1850. The use of cast iron for railway bridge construction in Europe ceased in about 1867.[§] One of the last major railway bridges in Europe to be constructed using cast iron was Gustave Eiffel's 488 m (1600 ft) long Garonne River Bridge built in 1860. However, cast iron continued to be used in the United States (primarily in compression members), even in some long-span bridges for more than a decade after its demise in Europe.

1.2.2 Wrought Iron Construction

Early short- and medium-span railway bridges in the United States were usually constructed from girders or propriety trusses (e.g., the Bollman, Whipple, Howe, Pratt, and Warren trusses shown in Figure 1.2). An example of a Whipple truss is also shown in Figure 1.3. US patents were granted for small- and medium-span iron railway trusses after 1840, and they became widely used by American railroads. The trusses typically had cast iron or wood compression members and wrought iron tension members.[¶]

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[*] This was Stephenson's cast iron girder bridge over the River Dee on the London-Chester-Holyhead Railroad. In fact, Stephenson had recognized the brittle nature of cast iron before many of his peers and reinforced his cast iron railway bridge girders with wrought iron rods. Nevertheless, failures ensued with increasing railway loads.

[†] Hodgekinson, Fairbairn, and Stephenson had also performed experiments with cast and wrought iron bridge elements between 1840 and 1846. The results of those experiments led to a general acceptance of wrought iron for railway bridge construction among British engineers.

[‡] Wrought iron has much lower carbon content than cast iron and is typically worked into a fibrous material with elongated strands of slag inclusions.

[§] In the United States, J. H. Linville was a proponent of all wrought iron truss construction in the early 1860s.

[¶] Wrought iron bridge construction provided the opportunity for using riveted connections instead of bolts. The riveted connections were stronger due to the clamping forces induced by the cooling rivets.

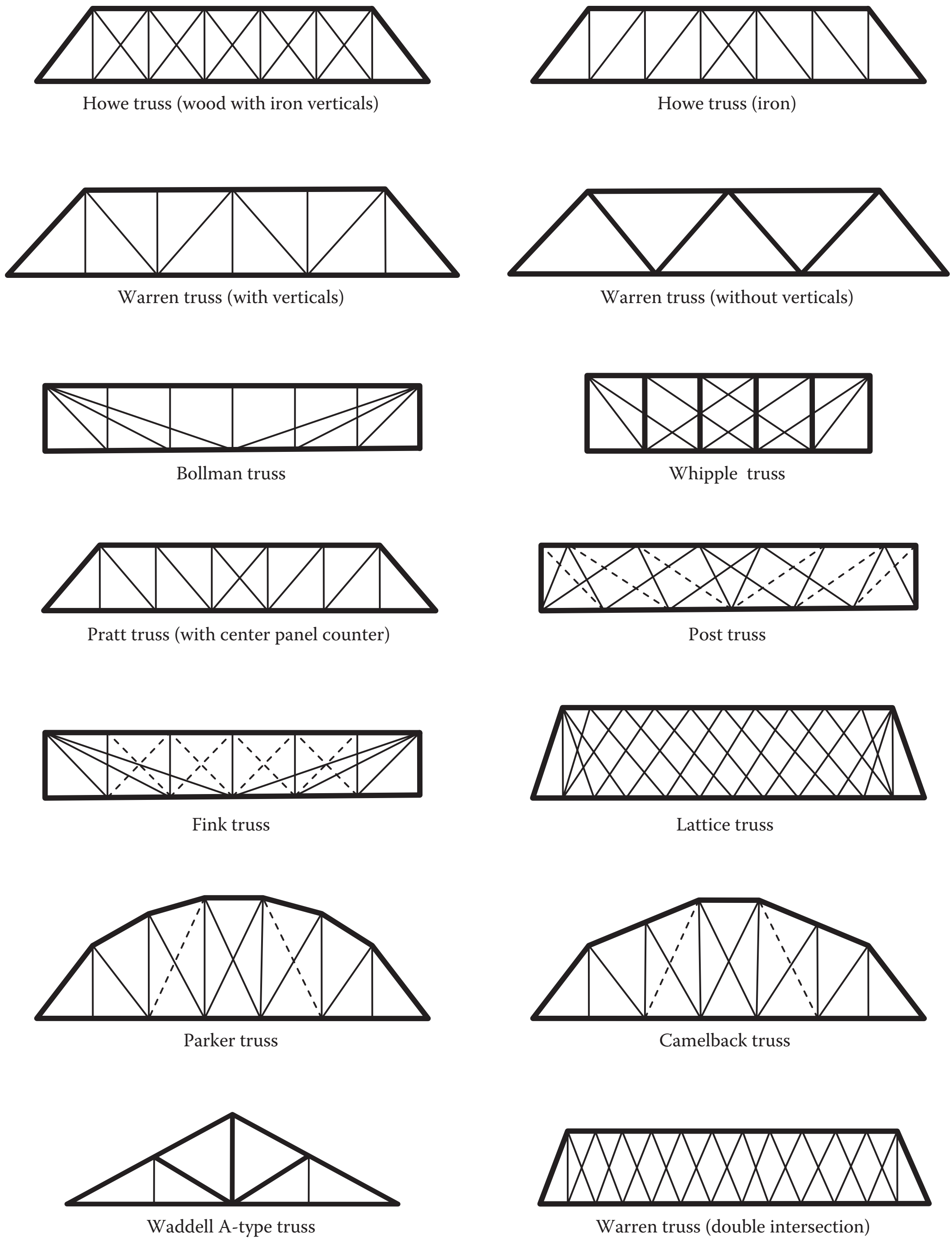


FIGURE 1.2 Truss forms used by railroads in the United States.

FIGURE 1.3 Whipple truss span. (Courtesy of the author, Canadian Pacific Engineering.)

The wooden Howe truss with wrought iron vertical members (patented in 1840) was popular on American railroads up to the 1860s.* The principal attraction of the Howe truss was the use of wrought iron rods that did not permit the truss joints to come apart when diagonal members were in tension from railway loading. However, the Howe truss form is statically indeterminate, and, therefore, many were built on early American railroads without the benefit of applied scientific analysis.†

The first railway bridge in the United States constructed entirely of iron was a Howe truss with cast iron compression and wrought iron tension members built by the Philadelphia and Reading Railroad in 1845 at Manayunk, PA. Iron Howe trusses were also constructed on the Boston and Albany Railroad in 1847 near Pittsfield, MA, and on the Harlem and Erie Railroad in 1850. Following this, iron truss bridges became increasingly common as American railroads continued their rapid expansion. Early examples of Pratt truss use were the Pennsylvania Railroad's cast and wrought iron arch-stiffened Pratt truss bridges of the 1850s. An iron railway bowstring truss, also utilizing cast iron compression and wrought iron tension members, was designed by Squire Whipple‡ for the Rensselaer & Saratoga Railway in 1852. Fink and Bollman, both engineers employed by the B&O Railroad, used their own patented cast and wrought iron trusses extensively between 1840 and 1875.§ Noteworthily, iron trusses were also built by the North Pennsylvania Railroad in 1856 (a Whipple truss) and the Catasauqua and Fogelsville Railroad in 1857. The Erie Railroad pioneered the use of iron post truss bridges in 1865, and they remained a standard of construction on the B&O Railroad for the next 15 years.

However, due to the bridge failures (predominantly of cast iron members) in the 30 years after 1840, the use of cast iron ceased, and wrought iron was used exclusively for railway girders and

* During construction of the railroad between St. Petersburg and Moscow, Russia (c. 1842), American Howe truss design drawings were used for many bridges. Timber Howe trusses were also used for the rapid construction of temporary bridges on the Canadian Pacific Railroad in the 1880s.

† Scientific analysis by engineers was becoming prevalent to ensure safety following the many railway bridge failures that occurred in the middle of the 19th century.

‡ In 1847, Whipple published *A Treatise on Bridge Building*, the first book on the scientific or mathematical analysis of trusses.

§ The first all-iron trusses on the B&O were designed by Fink in 1853.

trusses. Isambard Kingdom Brunel used thin-walled wrought iron plate girders in his designs for short and medium railway spans on the Great Western Railway in the UK during the 1850s. Between 1855 and 1859, Brunel also designed and constructed many noteworthy wrought iron lattice girder, arch, and suspension bridges for British railways. In particular, the Royal Albert Railway Bridge across the Tamar River, completed at Saltash in 1859, is a significant example of a Brunel wrought iron railway bridge using large lenticular trusses (Figure 1.4). Other important railway bridges built by Brunel on the Great Western Railway were the Wharncliffe Viaduct, Maidenhead, and Box Tunnel bridges. Table 1.3 lists some notable wrought iron truss railway bridges constructed between 1845 and 1877.

The English engineer William Fairbairn constructed a tubular wrought iron through-girder bridge on the Blackburn and Bolton Railway in 1846. Later, in partnership with Fairbairn, Robert Stephenson designed and built the innovative and famous wrought iron tubular railway bridges for the London–Chester–Holyhead Railroad at Conwy in 1848 and Menai Straits (the Britannia Bridge) in 1850. The Conwy bridge (Figure 1.5) is a simple tubular girder span of 125.6 m (412 ft) and the Britannia bridge consists of four continuous tubular girder spans of 70.1 m (230 ft), 140.2 m (460 ft), 140.2 m (460 ft), and 70.1 m (230 ft) (Figure 1.6).[*] Spans of 140.2 m (460 ft) were mandated for navigation purposes, making this the largest wrought iron bridge constructed. It was also one of the first uses of span continuity to reduce dead load bending moments in a bridge. Arch bridges were also proposed for the Menai Straits crossing by Stephenson[†] and Brunel.[‡] However, arch bridges were rejected due to concerns about interference with navigation, and the four wrought iron tubular girder continuous spans were built to obtain the stiffness required to resist wind and train loadings.

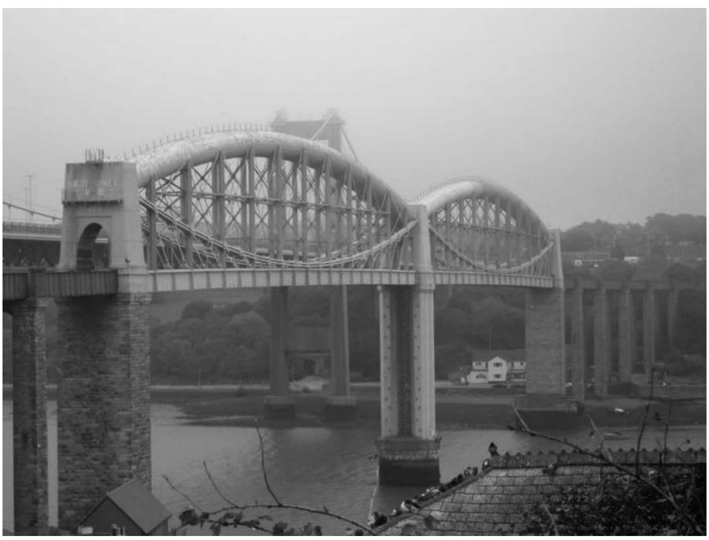


FIGURE 1.4 The Royal Albert Bridge built in 1859 over Tamar River at Saltash, UK. (From Owen Dunn, http://en.wikipedia, June 2005.)

[*] The Britannia Bridge was destroyed by fire in 1970 and only the Conwy Bridge remains as an example of Stephenson's tubular railway bridges. Following the fire, the Britannia Bridge was rebuilt as a steel truss arch bridge, carrying both road vehicle and railway traffic.

[†] Stephenson had studied the operating issues associated with some suspension railway bridges, notably the railway suspension bridge built at Tees in 1830, and decided that suspension bridges were not appropriate for railway loadings. He proposed an arch bridge.

[‡] To avoid the use of falsework in the channel, Brunel outlined the first use of the cantilever construction method in conjunction with his proposal for a railway arch bridge across Menai Straits.

TABLE 1.3
Notable Iron and Steel Simple Truss Span Railway Bridges Constructed during 1823–1907

Location	Railroad	Engineer	Year Completed	Type	Material	*L* m (ft)
West Auckland, UK	Stockton to Darlington	G. Stephenson	1823	Lenticular	Cast iron	3.8 (12.5)
Ireland	Dublin & Drogheda	G. Smart	1824	Lattice	Cast Iron	25.5 (84)
Manayunk, PA	Philadelphia and Reading	R. Osborne	1845	Howe	Cast and wrought iron	10.5 (34)
Pittsfield, MA	Boston & Albany	—	1847	Howe	Cast and wrought iron	9 (30)
Windsor, UK	Great Western	I. K. Brunel	1849	Bowstring	Iron	57 (187)
Newcastle, UK	Northwestern	I. K. Brunel	1849	Bowstring	Cast and wrought iron	38 (125)
—	Harlem & Erie	—	1850	Howe	Iron	—
Various	Pennsylvania	H. Haupt	1850s	Pratt with cast iron arch	Iron	—
Harper's Ferry	Baltimore & Ohio	W. Bollman	1852	Bollman	Cast and wrought iron	38 (124)
Fairmont, West VA	Baltimore & Ohio	A Fink	1852	Fink	Cast and wrought iron	62.5 (205)
—	Rennselaer & Saratoga	S. Whipple	1852	Whipple	Iron	—
Newark Dyke, UK	Great Northern	C. Wild	1853	Warren	Cast and wrought iron	79 (259)
—	North Pennsylvania	—	1856	Whipple	Iron	—
Guth, PA, Jordan Creek	Catasauqua & Fogelsville	F. C. Lowthorp	1857	—	Cast and wrought iron	33.5 (110)
Phillipsburg, NJ	Lehigh Valley	J. W. Murphy	1859	Whipple (pin-connected)	Iron	50 (165)
Plymouth, UK	Cornish (Great Western)	I. K. Brunel	1859	Lenticular	Wrought iron	139 (455)
Frankfort, Germany	—	—	1859	Lenticular	Iron	105 (345)
Various	New York Central	H. Carroll	1859	Lattice	Wrought iron	27.5 (90)
Kehl River, Germany	Baden State	Keller	1860	Lattice	Iron	60 (197)
Schuylkill River	Pennsylvania	J. H. Linville	1861	Whipple	Cast and wrought iron	58.5 (192)
Steubenville, OH	Pennsylvania	J. H. Linville	1863	Murphy-Whipple	Cast and wrought iron	97.5 (320)
Mauch Chunk, PA.	Lehigh Valley	J. W. Murphy	1863	—	Wrought iron	—
Liverpool, UK	London & Northwestern	W. Baker	1863	—	Iron	93 (305)
Blackfriar's Bridge, UK	—	Kennard	1864	Lattice	Iron	—

(*Continued*)

TABLE 1.3 (*Continued*)
Notable Iron and Steel Simple Truss Span Railway Bridges Constructed during 1823–1907

Location	Railroad	Engineer	Year Completed	Type	Material	*L* m (ft)
Orival, France	Western	—	~1865	Lattice	Iron	51 (167)
Various	Baltimore & Ohio	S. S. Post	1865	Post	Iron	—
Lockport, IL	Chicago & Alton	S. S. Post	~1865	Post	Cast and wrought iron	—
Schuylkill River	Connecting Railway of Philadelphia	J. H. Linville	1865	Linville	Wrought iron	—
Burlington[a], IA	Chicago, Burlington & Quincy	M. Hjorstberg	1868	—	Iron	—
Dubuque, IA	Illinois Central	J. H. Linville	1868	Linville	Wrought iron	76 (250)
Quincy, IL	Chicago, Burlington & Quincy	T. C. Clarke	1868	—	Cast and wrought iron	76 (250)
Kansas City[a], MO	Chicago, Burlington & Quincy	J. H. Linville & O. Chanute	1869	—	Iron	71 (234)
Louisville, KY	Baltimore & Ohio	A. Fink	1869	Subdivided Warren & Fink	Wrought iron	119 (390)
Parkersburg & Benwood, WV	Baltimore & Ohio	J. H. Linville	1870	Bollman	Iron	106 (348)
Hannibal[a], MO.	—	—	1871	—	Iron	—
Atcheson	Various	—	1875	Whipple	Iron	79 (260)
Cincinnati, OH	Cincinnati Southern	J. H. Linville & G. L. F. Bouscaren	1876	Linville	Wrought iron	157 (515)
Tay River[a], Scotland		Sir T. Bouch	1877	Lattice	Wrought iron	
Glasgow, MO	Chicago & Alton		1879	Whipple	Steel	732 (2402)
Bismark, ND		G. S. Morison & C. C. Schneider	1882	Whipple	Steel	
Hannibal[b], MO			1886		Steel	
Tay River[b], Scotland			1887		Steel	
Sioux City, IA			1888	—	Steel	122 (400)
Cincinnati, OH		W. H. Burr	1888	—	Steel	168 (550)

(*Continued*)

TABLE 1.3 (*Continued*)
Notable Iron and Steel Simple Truss Span Railway Bridges Constructed during 1823–1907

Location	Railroad	Engineer	Year Completed	Type	Material	*L* m (ft)
Benares, India			1888	Lattice	Steel	109 (356)
Hawkesbury, Australia	—	—	1889	—	Steel	127 (416)
Henderson Bridge	Louisville & Nashville	—	~1889	Subdivided Warren	Steel	160 (525)
Cairo, IL	Illinois Central		1889	—	Steel	—
Ceredo RR Bridge		Doane & Thomson	~1890	—	Steel	159 (521)
Merchant's Bridge, St. Louis		G. S. Morison	1890	Petit	Steel	158 (517)
Kansas City[b], MO		—	1891	—	Steel	
Burlington[b], IA	Chicago, Burlington & Quincy	G. S. Morison	1892	—	Steel	—
Louisville, KY	—		1893	Petit	Steel	168 (550)
Nebraska City, NB	—	G. S. Morison	1895	Whipple	Steel	122 (400)
Sioux City, IA			1896		Steel	149 (490)
Montreal, QC	Grand Trunk		1897		Steel	106 (348)
Kansas City, MO	Kansas City Southern	J. A. L. Waddell	1900	Pratt	Steel	—
Rumford, ON	Canadian Pacific	C. N. Monsarrat	1907	Subdivided Warren	Steel	126 (412)

[a] Indicates the first bridge constructed at the site.
[b] Indicates the second bridge constructed at this site.

FIGURE 1.5 The Conwy Bridges: Stephenson's Tubular Railway Bridge built in 1848 and Thomas Telford's Suspension Highway Bridge built in 1826 at Conwy Castle, Wales. (From Stephen J. Hill, Redwood City, CA. With permission.)

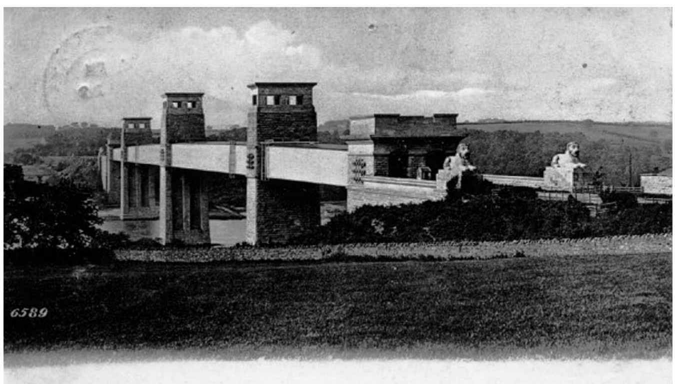


FIGURE 1.6 The Britannia Bridge built in 1850 across the Menai Straits, Wales. (Postcard from the private collection of Jochem Hollestelle.)

The construction of the Conwy and Britannia tubular iron plate bridges also provided the opportunity for further investigations into issues of plate stability, riveted joint construction, lateral wind pressure, and thermal effects. Fairbairn's empirical work on fatigue strength and plate stability during the design of the Conwy and Britannia bridges is particularly significant.*

A small 16.8 m (55 ft) long, simple span tubular wrought iron plate girder bridge was built in the United States by the B&O Railroad in 1847. However, the only large tubular railway bridge constructed in North America was the Victoria Bridge built in 1859 for the Grand Trunk Railway over the St. Lawrence River at Montreal† (Figure 1.7). The Victoria Bridge was the longest bridge in the world upon its completion.‡ The bridge was replaced with steel trusses in 1898 due to the rivet

* In 1864, Fairbairn studied iron plate and box girder bridge models under a cyclical loading representative of railway traffic. These investigations assisted with the widespread adoption of wrought iron, in lieu of cast iron, for railway bridge construction in the latter quarter of the 19th century.

† The Victoria Bridge over the St. Lawrence River at Montreal was also designed by Stephenson.

‡ The longest span in the Victoria Bridge was 100 m (330 ft).

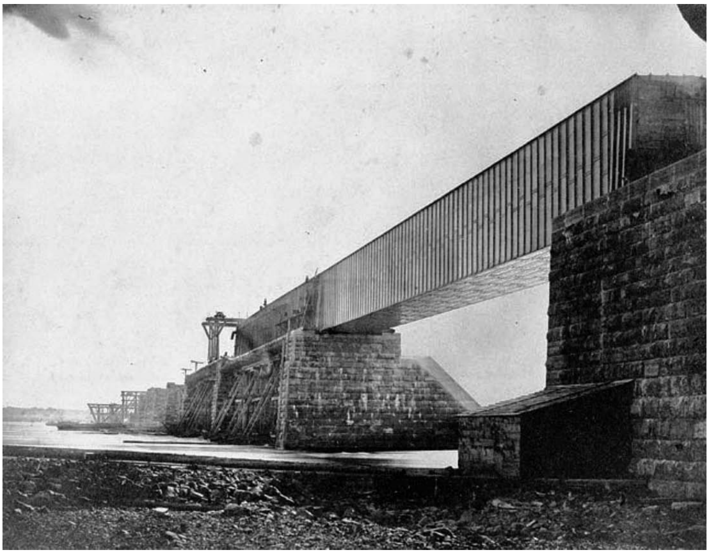


FIGURE 1.7 The Victoria Bridge under construction (completed in 1859) across the St. Lawrence River, Montreal, Canada. (Courtesy of William Notman, Library and Archives Canada.)

failures associated with increasing locomotive weights and ventilation problems harmful to passengers traveling across the 2788 m (9144 ft) river crossing with almost 2000 m (6560 ft) of tubular girders. Table 1.4 indicates some notable continuous span railway bridges constructed after 1850.

These tubular bridges provided the stiffness desired by their designers but proved to be costly. Suspension bridges were more economical but many British engineers were hesitant to use flexible suspension bridges for long-span railroad crossings.* Sir Benjamin Baker's 1867 articles on long-span bridges also promoted the use of more rigid bridges for railway construction. Furthermore, Baker had earlier recommended cantilever trusses for long-span railway bridges.† Also in 1867, Heinrich Gerber constructed the first cantilever bridge at Hanover, Germany; and following this, cantilever arch and truss bridges were built in New England and New Brunswick‡ between 1867 and 1885.

Nevertheless, railway suspension bridges were built in the United States in the last quarter of the 19th century. Unlike the almost universal aversion to railway suspension bridge design and construction that was prevalent among British railway engineers, some American engineers were using iron suspension bridges for long spans carrying relatively heavy freight railroad traffic. Modern suspension bridge engineering essentially commenced with the construction of the 250 m (820 ft) span railway suspension bridge over the Niagara Gorge in 1854. European engineers and many American engineers had expressed concern over the scope of such a suspension bridge.§ Nevertheless, this bridge, designed and constructed by John A. Roebling, was used by the Great Western, New York Central, Grand Trunk Railway, and successor railroads for over 40 years. Roebling had realized the need for greater rigidity

* An effort to construct a suspension bridge for the Stockton and Darlington Railroad in the 1820s had been a failure. The first railway suspension bridge built over the Tees River in 1830 in the UK [with a 91.5 m (300 ft) span] had performed poorly by deflecting in a very flexible manner that even hindered the operation of trains. It was replaced by cast iron and steel girders, respectively, in 1842 and 1905. The Basse-Chaine suspension bridge in France collapsed in 1850 as the suspension bridge at Wheeling, WV in 1854, illustrating the susceptibility of flexible suspension bridges to failure under wind load conditions.

† Baker's 1862 book *Long-Span Railway Bridges* and A. Ritter's calculations of the same year outlined the benefits of cantilever bridge design.

‡ For example, the railway bridge built in 1885 (replaced in 1922) over the reversing falls of the Saint John River in New Brunswick, Canada.

§ Only four American engineers expressed support of the proposal by the Great Western Railroad to connect to the New York Central Railroad with a suspension bridge. These were Charles Ellet, John A. Roebling, Edward Serrell, and Samuel Keefer.

TABLE 1.4
Notable Continuous Span Railway Bridges Constructed during 1850–1929

Location	Railroad	Engineer	Year	Type	Longest Span m (ft)
Torksey, UK	—	J. Fowler	1850	3 Span continuous tubular girder	40 (130)
Britannia Bridge, Menai Straits, UK	London-Chester-Holyhead	R. Stephenson	1850	4 Span continuous tubular	140 (460)
Montreal, QC	Grand Trunk	R. Stephenson	1860	25 Span continuous tubular	100 (330)
Montreal, QC	Canadian Pacific	C. Shaler Smith	1886	4 Span continuous trusses	124 (408)
Sciotoville, OH	Chesapeake & Ohio	G. Lindenthal & D. B. Steinman	1917	2 Span continuous truss	236 (775)
Allegheny River	Bessemer & Lake Erie	—	1918	3 Span continuous truss	158.5 (520)
Nelson River	Bessemer & Lake Erie	—	1918	3 Span continuous truss	122 (400)
Cincinnati, OH	C.N.O. & T.P.	—	1922	3 Span continuous truss	157 (516)
Cincinnati, OH	Cincinnati & Ohio	—	1929	3 Span continuous truss	206 (675)

in suspension bridge design after the failure of the Wheeling* and other suspension bridges. As a consequence, his Niagara Gorge suspension bridge was the first to incorporate stiffening trusses into the design (Figure 1.8). Rehabilitation work was required in 1881 and 1887, and it was replaced with a steel spandrel braced hinged arch bridge, designed by Leffert L. Buck, in 1897, due to capacity requirements for heavier railway loads. The railway suspension bridge constructed in 1840 over the Saone River in France was replaced only 4 years after completion due to poor performance under live load.† The railway suspension bridge constructed in 1860 at Vienna, Austria, was also prematurely replaced with an iron arch bridge in 1884 after concerns over the flexibility of the suspended span. The early demise of these and other suspension bridges generated new concerns among some American engineers over the lack of rigidity of cable-supported bridges under steam locomotive and moving train loads.

The first all-wrought-iron bridge in the United States, a lattice truss, was completed in 1859 by the New York Central Railroad.‡ In the same year, the Lehigh Valley Railroad built the first pin-connected truss. In 1861, the Pennsylvania Railroad pioneered the use of forged eyebars in a pin-connected truss over the Schuylkill River. After this, many American railway bridges were constructed with pinned connections, while European practice still favored the use of riveted construction. Riveted construction was considered superior, but pin-connected construction enabled the economical and rapid erection of railway bridges in remote areas of the United States. The principal exception was the New York Central Railroad, which used riveted construction exclusively for its iron railway bridges.

In 1863, the Pennsylvania Railroad successfully crossed the Ohio River using a 98 m (320 ft) iron truss span. The railroad used the relatively rigid Whipple truss for such long spans. This bridge construction encouraged greater use of longer span iron trusses to carry heavy freight railroad traffic in the United States. Another notable wrought iron railway truss was the 119 m (390 ft) span built by the B&O Railroad at Louisville, KY, in 1869.

In the 1870s, the Pratt Truss (patented in 1844) became predominant for short- and medium-span railway bridges in the United States. Pratt trusses are statically determinate, and their form is well

* The 308 m (1010 ft) wire rope suspension bridge, designed by Charles Ellet, over the Ohio River at Wheeling, WV collapsed due to wind loads in 1854, just 5 years after completion of construction.

† The suspension bridge was replaced by a stone masonry bridge.

‡ The New York Central Railroad also initiated the use of iron stringers (instead of wood stringers) in railway trusses in the 1860s.

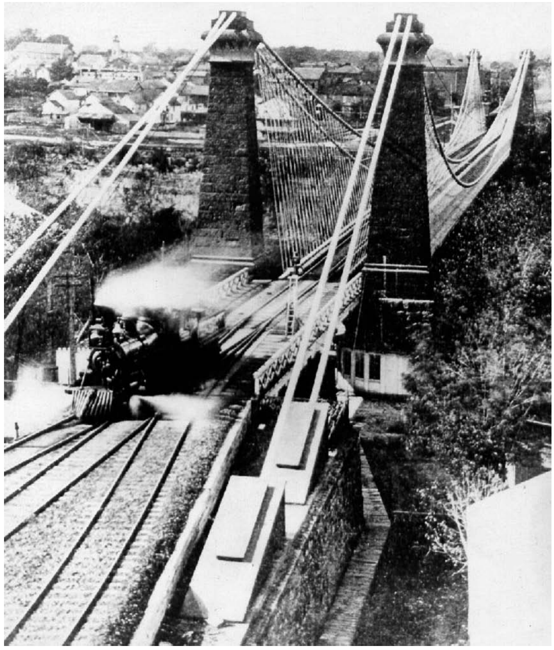


FIGURE 1.8 The Railway Suspension Bridge built in 1854 across the Niagara Gorge between New York, USA and Ontario, Canada. (From Niagara Falls Public Library.)

suited for use in iron bridges. Whipple, Warren, and post trusses were also used by US railroads in the 1870s. The Bollman truss bridge, patented in 1852 and used by the B&O and other railroads until 1873, was an example of the innovative* use of wrought iron in American railway bridge construction. Baltimore, Petit, or Pennsylvania truss spans were often used for longer wrought iron railway bridge spans (Figure 1.9).† The first use of a Baltimore truss (a Pratt truss with subdivided panels) was on the Pennsylvania Railroad in 1871.

Large railway viaduct bridges were also constructed using wrought iron. The 66 m (216 ft) high and 396 m (1300 ft) long Viaduc de la Bouble was built in France in 1871. In 1875, the Erie Railroad completed construction of a 249 m (818 ft) long wrought iron viaduct at Portage, New York‡ (Figure 1.10). This was followed in 1882 by the 92 m (300 ft) high and over 600 m (2000 ft) long wrought iron Kinzua Viaduct, PA (Figure 1.11).§ Also in France, Gustave Eiffel designed the wrought iron Garabit Viaduct, which opened to railroad traffic in 1884 (Figure 1.12).

A large number of iron railway bridges built after 1840 in the United States and the UK failed under train loads. It was estimated that about one-fourth of the railway bridges in the American railroad infrastructure were failing annually between 1875 and 1888. Most of these failures were related to fatigue and fracture, and the buckling instability of compression members (notably top chords of trusses). Although most of the failures were occurring in cast iron truss members and

* Bollman trusses used wrought iron tension members and cast iron compression members. The redundant nature of the truss form reduced the possibility of catastrophic failure.

† The Petit truss was used extensively by American railroad companies.

‡ The 1875 viaduct was designed by G. S. Morison and O. Chanute. It was extensively strengthened using steel in 1903 and is currently planned for replacement commencing in 2016 with a 147 m (483 ft) two-hinged spandrel braced steel arch (Irwin et al., 2013).

§ Both the Portageville Viaduct and the Kinzua Viaduct were designed by Morison and Chanute for the Erie Railroad.

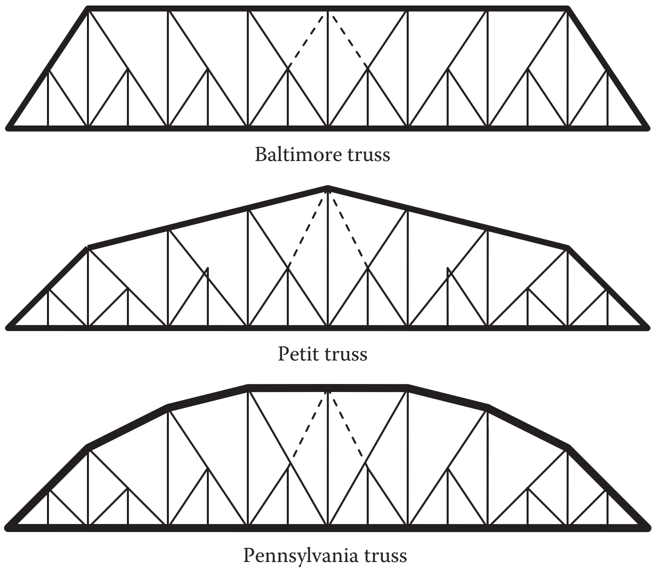


FIGURE 1.9 Baltimore trusses (inclined chord truss is also called Petit truss).

FIGURE 1.10 Portageville Viaduct 1875 (strengthened 1903) over Genesee River Gorge, New York. (From James N. Carter, Norfolk Southern Corp., Atlanta, GA. With permission.)

girders, by 1850 many American engineers had lost confidence in even wrought iron girder, truss, and suspension railway bridge construction.*

At this time, railway construction was not well advanced in Germany, and these failures interested Karl Culmann during construction of some major bridges for the Royal Bavarian Railroad. He proposed that American engineers use lower allowable stresses to reduce fatigue failures of iron truss railway bridges, and he recognized the issue of top chord compressive instability. Culmann also proposed the use of stiffening trusses for railroad suspension bridges after learning of the distress expressed by American bridge engineers concerning their flexibility under moving live loads.

* For example, following the collapse of an iron bridge in 1850, all metal bridges on the Boston and Albany Railroad were replaced with timber bridges.

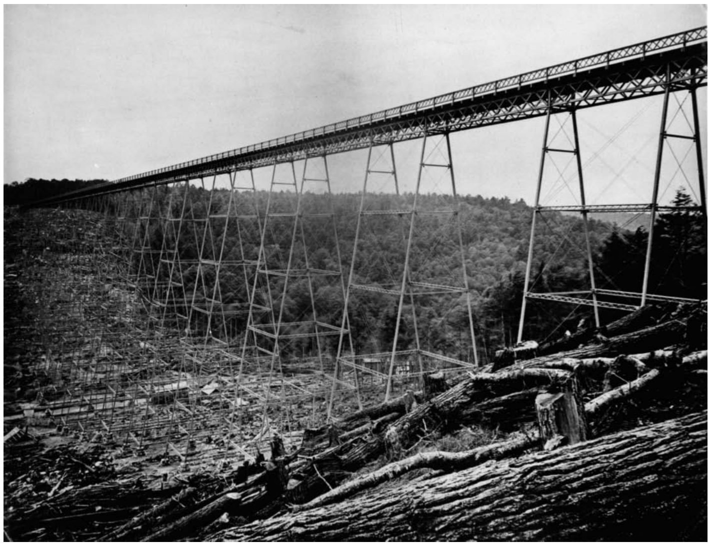


FIGURE 1.11 Kinzua Viaduct 1882, Pennsylvania. (Courtesy of Historic American Engineering Record.)

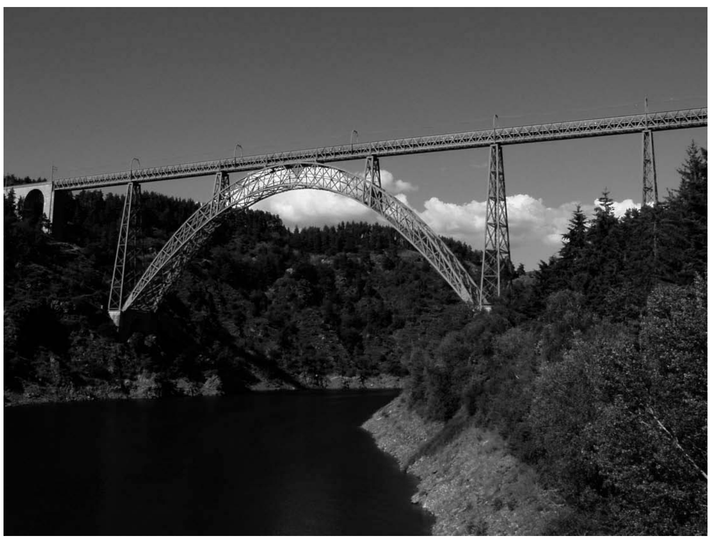
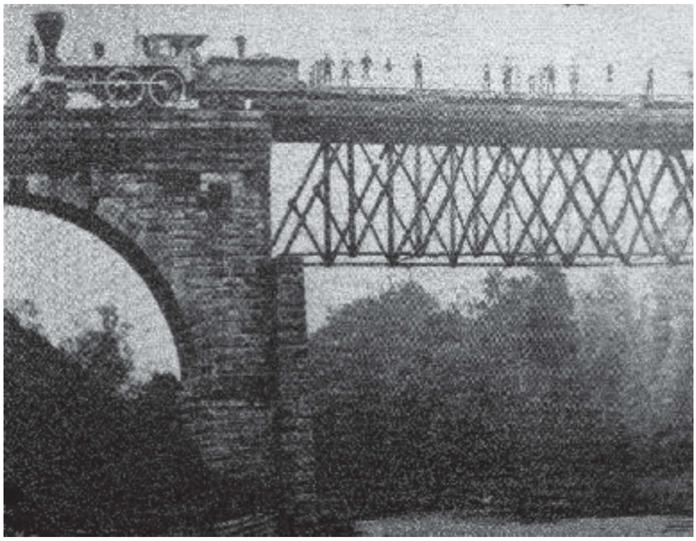


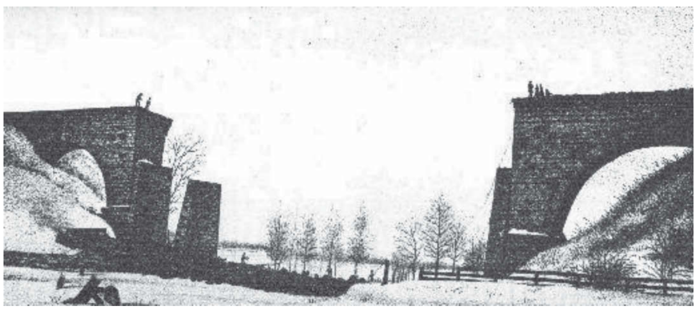
FIGURE 1.12 The Garabit Viaduct built in 1884 over the Tuyere River, France. (From GFDL J. Thurion, http://fr.wikipedia, July 2005.)

A railroad Howe truss collapsed under a train at Tariffville, CT, in 1867 and a similar event occurred a decade later at Chattsworth, IL. However, the most significant railway bridge failure, due to the considerable loss of life associated with the incident, was the collapse of the cast iron Howe deck truss span on the Lake Shore & Michigan Southern Railroad at Ashtabula, OH, in 1876 (Figure 1.13a and b). The Ashtabula bridge failure provided further evidence that cast iron was not appropriate for heavy railway loading conditions and caused American railroad companies to abandon the use of cast iron elements for bridges.* This was, apparently, a wise decision as modern forensic analysis indicates the likely cause of the Ashtabula failure was a combination of fatigue and brittle fracture initiated at a cast iron flaw.

* With exception of cast iron bearing blocks at ends of truss compression members.



(a)



(b)

FIGURE 1.13 (a) The Ashtabula Bridge, OH, before 1876 collapse. (b) The Ashtabula Bridge, OH, after 1876 collapse. (Courtesy of Ashtabula Railway Historical Foundation.)

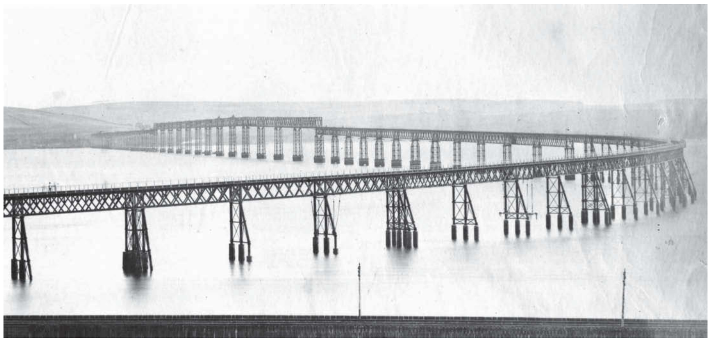
In addition, the collapse of the Tay Railway Bridge in 1879, only 18 months after completion, promoted a renewed interest in wind loads applied to bridges (Figure 1.14a and b). The Tay bridge collapse also reinforced the belief, held by many engineers, that light and relatively flexible structures are not appropriate for railway bridges.

These bridge failures shook the foundations of bridge engineering practice and created an impetus for research into new methods (for design and construction) and materials to ensure the safety and reliability of railway bridges. The investigation and specification of wind loads for bridges also emerged from research conducted following these railway bridge collapses. Furthermore, in both Europe and the United States, a new emphasis on truss analysis and elastic stability was developing in response to railway bridge failures.

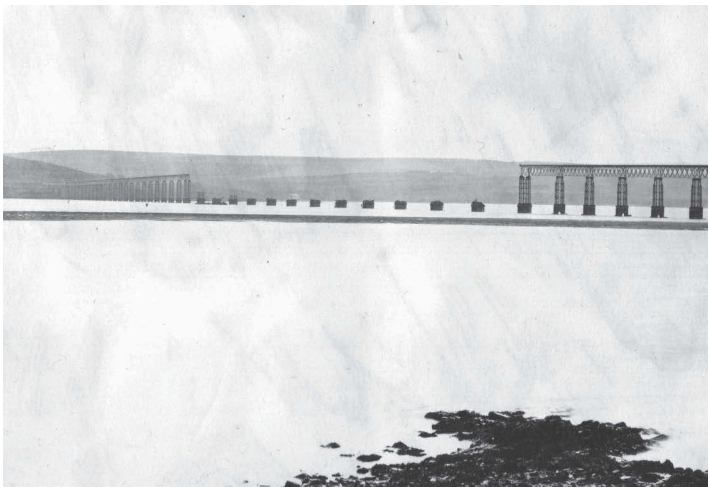
A revitalized interest in the cantilever construction method occurred, particularly in connection with the erection of arch bridges. Early investigations by Stephenson, Brunel, and Eads had illustrated that the erection of long arch spans using the cantilever method* was feasible and precluded the requirement for falsework as temporary support for the arch. The cantilevered arms were joined to provide fixed or two-hinged arch action† or connected, allowing translation of members

* Often using guyed towers and cable stays as erection proceeds.

† Depending on whether fixed or pinned arch support conditions were used.



(a)



(b)

FIGURE 1.14 (a) The Tay River Bridge, Scotland, before 1879 collapse. (From http://en.wikipedia, January 2007). (b) The Tay River Bridge, Scotland, after 1879 collapse. (From http://en.wikipedia, January 2007.)

to provide a statically determinate structure. The cantilever construction method was also proposed for long-span truss erection, where the structure is made statically determinate after erection by retrofitting to allow appropriate members to translate. This creates a span suspended between two adjacent cantilever arms that are anchored by spans adjacent to the support pier, providing a statically determinate structure.* Alternatively, the cantilever arms may progress only partially across the main span and be joined by a suspended span erected between the arms.† Other benefits of cantilever construction are smaller piers (due to a single line of support bearings) and an economy of material for properly proportioned cantilever arms, anchor spans, and suspended spans.

Iron trusses continued to be built in conjunction with the rapid railroad expansion of the 1860s. However, in the second half of the 19th century, steel started to replace iron in the construction of railway bridges.‡ For example, the iron Kinzua Viaduct of 1882 was replaced with a similar structure of steel only 18 years after construction due to concerns about the strength of wrought iron bridges under increasing railroad loads.

* Statically indeterminate structures are susceptible to stresses caused by thermal changes and support settlements. Therefore, statically indeterminate cantilever bridges must incorporate expansion devices and be founded on unyielding foundations to ensure safe and reliable behavior.

† This was the method used in the 1917 reconstruction of the Quebec bridge.

‡ In 1895, steel completely replaced wrought iron for the production of manufactured structural shapes.

1.3 STEEL RAILWAY BRIDGES

Steel is stronger and creates lighter structures than wrought iron, but it was expensive to produce in the early 19th century. Bessemer developed the steelmaking process in 1856, and Siemens further advanced the steel industry with open-hearth steelmaking in 1867. These advances enabled the economical production of steel. These steelmaking developments, in conjunction with the demand for railway bridges following the American Civil War, provided remarkable stimulus to the extensive use of steel in the construction of railway bridges in the United States. In the latter part of the 19th century, North American and European engineers favored steel arches and cantilever trusses for long-span railway bridges which, due to their rigidity, were considered to better resist the effects of dynamic impact, vibration, and concentrated moving railway loads.

The first use of steel in a railway bridge* was during the 1869–1874 construction of two 152 m (500 ft) flanking spans and 158.5 m (520 ft) central span of the St. Louis Bridge (now named the Eads Bridge after its builder, James Eads†) across the Mississippi River at St. Louis, MO. Eads did not favor the use of a suspension bridge for railway loads‡ and proposed a cast steel arch bridge. Eads' concern for stiffness for railway loads is illustrated by the trusses built between the railway deck and main steel arches of the St. Louis Bridge (Figure 1.15). The Eads Bridge features not only the earliest use of steel but also other innovations in American railway bridge design and construction. The construction incorporated the initial use of the pneumatic caisson method§ and the first use

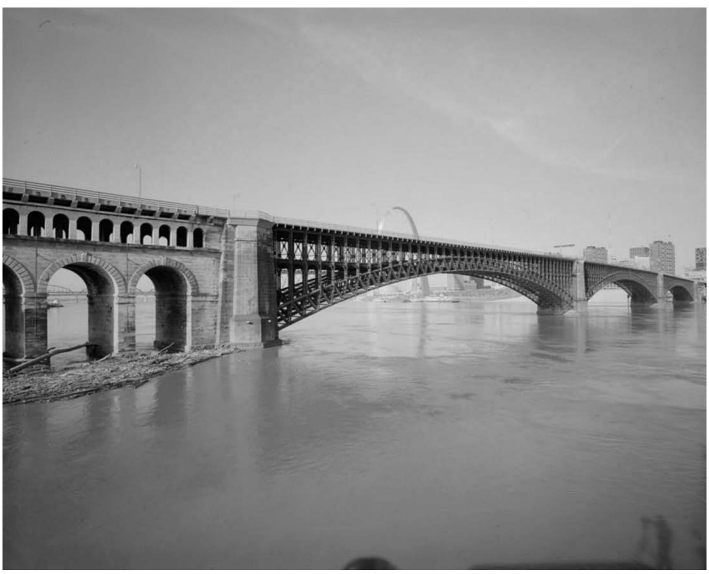


FIGURE 1.15 The St. Louis (Eads) Bridge built across the Mississippi River in 1874 at St. Louis, MO. (Courtesy of Historic American Engineering Record.)

* The first use of steel in any bridge was in the 1828 construction of a suspension bridge in Vienna, Austria, where open-hearth steel suspension chains were incorporated into the bridge.

† Eads was not an academically trained engineer and was assisted with design by Charles Pfeiffer and construction by Theodore Cooper.

‡ A suspension bridge was proposed for this site by John Roebling in 1864.

§ This method of pier construction was also used by Brunel in the construction of the Royal Albert Bridge at Saltash, UK, in 1859.

of the cantilever method of bridge construction in the United States.* It was also the first arch span over 150 m (500 ft) and incorporated the earliest use of hollow tubular chord members.† The plethora of innovations associated with this bridge caused considerable skepticism among the public and press. In response, before it was opened, Eads tested the bridge using 14 of the heaviest locomotives available. The construction of the Eads Bridge almost depleted the resources of the newly developed American steelmaking industry.

The initial growth of the American steel industry was closely related to the need for steel railway bridges, particularly those with long spans. The American railroads' demand for longer spans, and their use of increasingly heavier locomotives and freight cars encouraged Andrew Carnegie‡ and others to invest considerable resources toward the development of improved steels of higher strength and ductility. These improved steels elicited the construction of the first all-steel railway bridge (comprising Whipple trusses) by the Chicago and Alton Railway in 1879 at Glasgow, MO.

Despite concerns with suspension bridge flexibility under train and wind loads, some American bridge engineers continued to design and construct steel suspension railway bridges. The famous Brooklyn Bridge, when completed in 1883, carried two railway lines. However, lingering concerns with suspension bridge performance and increasing locomotive weights hastened the general demise of this relatively flexible type of railway bridge construction.

The structural and construction efficacy of cantilever-type bridges for carrying heavy train loads led to the erection of many long-span steel railway bridges of trussed cantilever design after 1876. The Cincinnati Southern Railway constructed the first cantilever, or Gerber§ type, steel truss railway bridge in the United States over the Kentucky River in 1877.¶ In 1883, the Michigan Central and Canada South Railway completed the construction of a counterbalanced cantilever deck truss bridge** across the Niagara Gorge parallel to Roebling's 1854 railway suspension bridge. Shortly afterward, in 1884, the Canadian Pacific Railway crossed the Fraser River in British Columbia with the first balanced cantilever steel deck truss (Figure 1.16). Cantilever bridges became customary for long-span railway bridge construction as they provided the rigidity required to resist dynamic train loads, may be made statically determinate, and require no main span (comprising cantilever arms and suspended span) falsework to erect. Table 1.5 summarizes some notable cantilever railway bridges constructed after 1876.

Theodore Cooper promoted the exclusive use of steel for railway bridge design and construction in his 1880 paper to the American Society of Civil Engineers (ASCE) titled "The Use of Steel for Railway Bridges." Following this, almost all railway bridges, and by 1895 many other bridges, in the United States were constructed of steel. Structural steel shape production was well developed for the bridge construction market by 1890.††

The British government lifted its ban on the use of steel in railway bridge construction in 1877. More than a decade later, Benjamin Baker reviewed precedent cantilever bridges constructed in North America (in particular, those on the Canadian Pacific Railway) and proposed a steel cantilever truss for the Firth of Forth Railway Bridge crossing in Scotland.‡‡ It was a monumental

* The cantilever method had been proposed in 1800 by Thomas Telford for a cast iron bridge crossing the Thames River at London and in 1846 by Robert Stephenson for construction of an iron arch railway bridge to avoid falsework in the busy channel of the Menai Straits. Eads used principles developed in the 17th century by Galileo to describe the principles of cantilever construction of arches to skeptics of the method.

† The tubular arch chords used steel with 1.5%–2% chromium content providing for a relatively high ultimate stress of about 100 ksi.

‡ Andrew Carnegie worked for the Pennsylvania Railroad prior to starting the Keystone Bridge Company (with J. H. Linville) and eventually going into the steelmaking business.

§ This type of bridge design and construction is attributed to the German engineer Heinrich Gerber who patented and constructed the first cantilever type bridge in 1867.

¶ At the location of an uncompleted suspension bridge by John Roebling.

** This was the first use of cantilever construction using a suspended span.

†† By 1895, structural shapes were no longer made with iron, and steel was used exclusively.

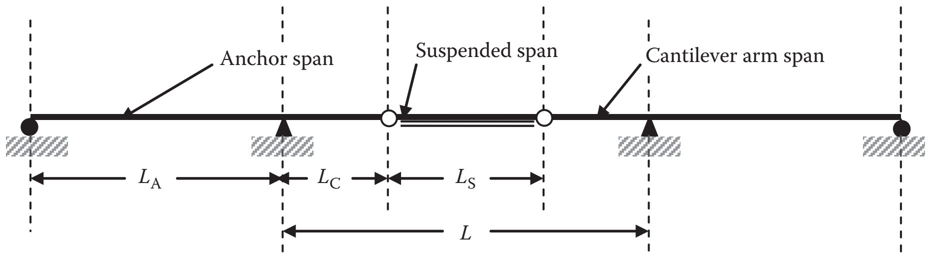
‡‡ Before this, Baker may not have known of the work of engineers C. Shaler Smith or C. C. Schneider who had already designed and constructed cantilever railway bridges in the United States.

FIGURE 1.16 Fraser River Bridge built in 1884, British Columbia, Canada. (Photo by J. A. Brock, Canadian Pacific Archives NS.11416.)

FIGURE 1.17 The Forth Rail Bridge built over the Firth of Forth in 1890, Scotland. (From GFDL Andrew Bell, http://en.wikipedia, January 2005.)

undertaking completed in 1890 (Figure 1.17). It is an example of steel truss cantilever-type railway bridge construction on a grand scale with cantilever arms of 207 m (680 ft) supporting a 107 m (350 ft) suspended span. Baker used the relatively new Bessemer steel in the bridge even though it was an untested material for such large structures, and some engineers thought it too susceptible to cracking. The bridge is very stiff and the 90 mm (3½ in.) deflection, measured by designer Baker

TABLE 1.5
Notable Steel Cantilever Railway Bridges Constructed during 1876–1917



Location	Railroad	Engineer	Year	L_A m (ft)	L_C m (ft)	L_S m (ft)	L m (ft)
Posen, Poland	—	—	1876	23 (74)	—	—	45 (148)
Dixville, KY	Cincinnati Southern	C. Shaler Smith & G. Bouscaren	1877	~73(~240)	49.5 (162.5)	0	99 (325)
St. Paul, MN	Chicago, Milwaukee & St Paul	C. Shaler Smith	1880	74 (243)	49.5 (162)	0	99 (324)
Niagara Gorge, NY	Michigan Central	C. C. Schneider	1883	63 (207.5)	57 (187.5)	36.5 (120)	151 (495)
Fraser River, BC (Figure 1.16)	Canadian Pacific	C. C. Schneider	1884	32 (105)	32 (105)	32 (105)	96 (315)
St. John, NB	Canadian Pacific	G. H. Duggan	1885	43.5 (143), 58 (190)	43.5 (143), 58 (190)	43.5 (143)	145 (476)
Louisville, KY	—	—	1886	55 (180)	49 (160)	49 (160)	146 (480)
Point Pleasant, WV	—	—	1888	73 (240)	43.5 (142.5)	61 (200)	148 (485)
Tyrone, KY	Louisville and Southern	J. W. MacLeod	1889	~68.5 (~225)	—	—	551
Poughkeepsie, NY	Central New England	—	1889	80 (262.5)	~49 (~160)	~69.5 (~228)	167 (548)
Hoogly, India	East India	Sir B. Leslie	1890	—	—	—	—
Firth of Forth, Scotland (Figure 1.17)	North British	Sir B. Baker & Sir J. Fowler	1890	207 (680)	207 (680)	107 (350)	521 (1710)
Pecos River	Southern Pacific	A. Bonzano	1891	—	—	—	~61 (~200)
Red Rock, CO	—	J. A. L. Waddell	1892	50 (165)	50 (165)	100 (330)	200 (660)
Callao, Peru	Lima and Oroya	L. L. Buck	~1892	—	—	—	265
Cernavoda, Romania	—	—	~1892	71 (233.5)	50 (164)	90 (295)	190 (623)

(*Continued*)

TABLE 1.5 (*Continued*)
Notable Steel Cantilever Railway Bridges Constructed during 1876–1917

Location	Railroad	Engineer	Year	L_A m (ft)	L_C m (ft)	L_S m (ft)	L m (ft)
Memphis, TN	—	G. S. Morison	1892	69 (226) and 94.5 (310)	52 (170)	137 (450)	241 (790.5)
Ottawa, ON	Canadian Pacific	G. H. Duggan	1900	75 (247)	37.5 (123.5)	94 (308)	169 (555)
Loch Etive, Scotland	—	Sir J. W. Barry	1903	42.5 (139.5)	44.5 (146)	71 (232)	160 (524)
Pittsburgh, PA	Wabash	—	1904	106 (346)	69 (226)	110 (360)	248 (812)
Mingo Junction, OH	Wabash	—	1904	91 (298)	—	—	213 (700)
Thebes, IL	—	A. Noble & R. Modjeski	1905	79.5 (260.5)(½ of span)	46.5 (152.5)	111.5 (366)	204.5 (671)
Blackwell's Island (Queensboro), NY	City of New York (light rail)	G. Lindenthal	1907	143 (469.5) & 192 (630)	180 (591)	0	360 (1182)
Khushalgarth, India	—	Rendel & Robertson	1908	—	—	—	—
Westerburg, Prussia	Prussian State	—	1908	—	—	33.5 (110)	—
Daumer Bridge, China	Yunnan	—	1909	37.5 (123)	27.5 (90)	51 (168)	106 (348)
Beaver, PA	Pittsburgh & Lake Erie	—	1910	97.5 (320)	74 (242)	87 (285)	234.5 (769)
Quebec, QC (Figure 1.18)	Canadian Government	Duggan, Vautelet, Monsarrat, Modjeski, Schneider	1917	157 (515)	177 (580)	195 (640)	549 (1800)

under the heaviest locomotives available in the North British Railway, compared well with his estimate of 100 mm (4 in.). The bridge was further tested under extreme wind conditions with two long heavy coal trains, and the cantilever tip deflection was <180 mm (7 in.).

However, the Forth Railway Bridge used a large quantity of steel and was costly. This prompted engineers such as Theodore Cooper (who had worked with Eads on the St. Louis Bridge) to consider cantilever construction with different span types using relatively smaller members. Two such statically determinate railway bridges constructed were the 205 m (671 ft) main span bridge crossing the Mississippi River at Thebes, IL, and the 549 m (1800 ft) main span Quebec Bridge. The Thebes bridge, constructed in 1905, consists of five pin-connected through-truss spans, of which two spans are 159 m (521 ft) fixed double anchor spans [anchoring four 46.5 m (152.5 ft) cantilever arms] and three contain 111.5 m (366 ft) suspended spans. The Quebec Bridge, an example of economical long-span steel cantilevered truss construction for railroad loads, was completed in 1917 after two construction failures (Figure 1.18a and b). The initial 1907 failure was likely due to the calculation error in determining dead load compressive stresses in the bottom chord members during construction as the cantilever arms were increased in length. The bridge was redesigned,[*] and a new material, nickel steel,[†] was used in the reconstruction. In 1916, the suspended span truss fell while being hoisted into place. It was quickly rebuilt and the Quebec Bridge was opened to railway traffic in 1917 (Figure 1.18c). Another major cantilever-type bridge was not to be constructed until after 1930. The Quebec Bridge remains as the longest span cantilever bridge in the world.

Continuous spans were often used for long-span steel railway bridge construction in Europe but seldom in North America due to the practice of avoiding statically indeterminate structures. The first long-span continuous steel truss railway bridge was built by the Canadian Pacific Railway over the St. Lawrence River at Montreal in 1886 (Figure 1.19).[‡] The 124.5 m (408 ft) main spans were erected by the cantilever method with careful consideration of the deflections and stresses in the bottom chords of the truss. These were controlled during the cantilever erection procedure with cables and adjustment screws attached to the partially completed truss supported on the center pier of the continuous span. The Viaur Viaduct, built in 1898, was the first major steel railway bridge in France.[§]

Many iron and steel railway bridges were replaced in the first decades of the 20th century due to the development of substantially more powerful and heavier locomotives.[¶] Riveting was used extensively in Europe but only became a standard of American long-span steel railway bridge fabrication after about 1915[**] with construction of the Hell Gate and Sciotoville bridges. Hell Gate is a 298 m (978 ft) two-hinged steel trussed arch bridge in New York. It was built to carry four heavily loaded railroad tracks of the New England Connecting Railroad and Pennsylvania Railroad when it was completed in 1916 (Figure 1.20). It is the largest arch bridge in the world, and it was erected without the use of falsework. It was also the first major bridge to use high carbon steel members in its construction.[††] The Chesapeake & Ohio Railroad completed construction of two 236.5 m (775 ft)

[*] The original designers were Theodore Cooper and Peter Szlapka (Phoenixville Bridge Company). Following the collapse, a design was submitted by H. E. Vautelet; but the redesign of the bridge was tendered to various bridge companies and carried out by G. H. Duggan (St. Lawrence Bridge Company) under the direction of C. C. Schneider, R. Modjeski, and C. N. Monsarrat.

[†] Alloy nickel steel was first used in 1909 on the Blackwell's Island (now Queensboro) Bridge in New York. Nickel steel was also used extensively by J. A. L. Waddell for long-span railway bridge designs. A. N. Talbot conducted tests of nickel steel connections for the Quebec bridge reconstruction.

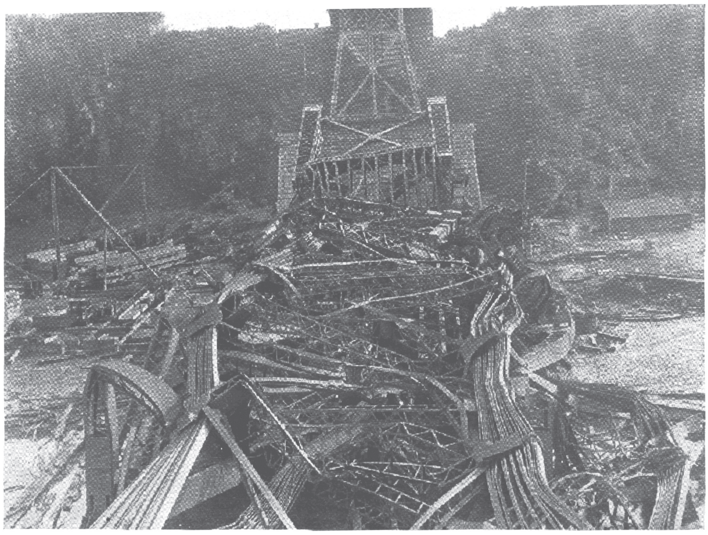
[‡] These spans were replaced in 1912 due to concern over performance under heavier train loads. The lead end of the simple span replacement trusses was supported by falsework on a movable barge during installation on an adjacent alignment.

[§] This cantilever truss arch bridge is unusual, in that it incorporates no suspended span, thereby rendering the structure statically indeterminate. Many engineers believe that the design was inappropriate for railroad loading.

[¶] Locomotive weights were typically about 40 tons in 1860, 70 tons in 1880, 100 tons in 1890, 125 tons in 1900, and 150 tons in 1910.

[**] Riveting was used on smaller spans earlier in the 20th century.

[††] Primarily, because of the high cost of alloy steel.



(a)

(b)

(c)

FIGURE 1.18 (a) The 1907 Quebec Bridge collapse, Canada. (b) The 1916 Quebec Bridge collapse, Canada. (c) The Quebec Bridge completed in 1917 across the St. Lawrence River at Quebec City, Canada. [(a) and (c) Courtesy of Carleton University Civil Engineering Exhibits; (b) Courtesy of A. A. Chesterfield, Library & Archives Canada.]

FIGURE 1.19 The St. Lawrence Bridge built in 1886 at Montreal, Canada. (Photo by J. W. Heckman Canadian Pacific Archives NS.1151.)

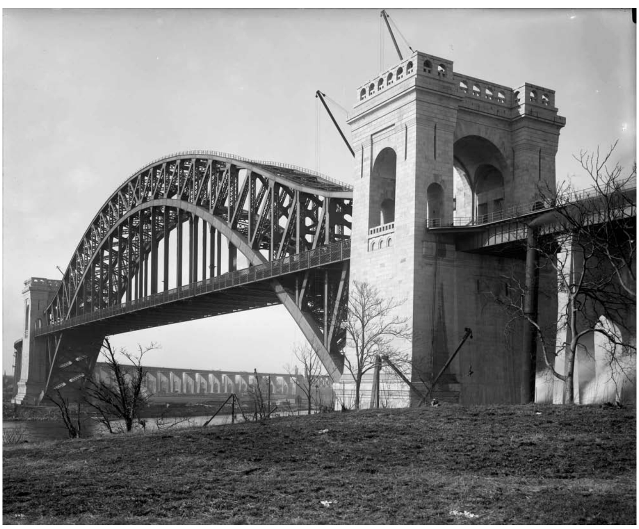


FIGURE 1.20 The Hell Gate Bridge built across the East River in 1916, New York. (Courtesy of Library of Congress from Detroit Publishing Co.)

span continuous steel trusses across the Ohio River at Sciotoville, OH, in 1917. This bridge remains the longest continuous span bridge in the world.

It has been estimated that in 1910 there were 80,000 iron and steel bridges* with a cumulative length of 2250 km (1400 miles) on about 300,000 km (190,000 miles) of track. Railroads were the

* The majority of the bridges were of steel construction by the beginning of the 20th century.

principal incentive for material and construction technology innovation in the latter half of the 19th century as the transition from wood and masonry to iron and steel bridges progressed in conjunction with the construction methods that minimized interference with rail and other traffic.* The art and science of bridge engineering was emerging from theoretical and experimental mechanical investigations prompted, to a great extent, by the need for rational and scientific bridge design to support a rapidly developing and expanding railroad infrastructure.

1.4 THE DEVELOPMENT OF RAILWAY BRIDGE ENGINEERING

1.4.1 Strength of Materials and Structural Mechanics

The early work of Robert Hooke (1678) concerning the elastic force and deformation relation, Jacob Bernoulli (1705) regarding the shape of deflection curves, Leonard Euler (1759) and C. A. Coulomb (1773) about elastic stability of compression members,† and Louis M. H. Navier (1826) on the subject of the theory of elasticity laid the foundation for the rational analysis of structures. France led the world in the development of elasticity theory and mechanics of materials in the 18th century, and produced well-educated engineers who, in many cases, became the leaders of American railway bridge engineering practice.‡ Railroad expansion continued at a considerable pace for another 80 years following inception in the 1820s. During that period, due to frequently increasing locomotive loads, it was not uncommon for railway bridges to be replaced at 10- to 15-year intervals. The associated demand for stronger, longer, and more reliable steel bridges, coupled with the in-service failures that were occurring, compelled engineers in the middle of the 19th century to engage in the development of a scientific approach to the design of iron and steel railway bridges.

American railway bridge engineering practice was principally experiential and based on the use of proven truss forms with improved tensile member materials. Many early Town, Long, Howe, and Pratt railway trusses were constructed without the benefit of a thorough and rational understanding of forces in the members. This lack of scientific approach was revealed by the many failures of railway bridge trusses between 1850 and 1870. This empirical practice had served the emerging railroad industry until heavier loads and longer span bridges, in conjunction with an increased focus on public safety,§ made a rational and scientific approach to the design of railway bridges imperative. In particular, American engineers developed a great interest in truss analysis because of the extensive use of iron trusses on US railroads. In response, Squire Whipple published the first rational treatment of statically determinate truss analysis (the method of joints) in 1847.

The rapid advancement of elasticity theory and engineering mechanics in Europe in the mid-19th century also encouraged French and German engineers to design iron and steel railway bridges using scientific methods. At this juncture, European engineers were also interested in the problems of truss analysis and elastic stability. B. P. E. Clapyron developed the three-moment equation in 1849 and used it in an 1857 postanalysis of the Britannia Bridge.¶ Concurrently, British railway bridge engineers were engaged in metals and bridge model testing for strength and stability. Following Whipple, two European railway bridge engineers, D. J. Jourawski** and Karl Culmann,

* For example, to not to interfere with railway traffic, the tubular spans of the Victoria Bridge at Montreal were replaced by extension of substructures and erecting steel trusses around the exterior of the tubular girders.

† Between 1885 and 1889, F. Engesser, a German railway bridge engineer, further developed a compression member stability analysis for general use by engineers.

‡ Charles Ellet (1830), Ralph Modjeski (1855), L. F. G. Bouscaren, Chief Engineer of the Cincinnati Southern Railroad (1873), and H. E. Vautelet, Bridge Engineer of the Canadian Pacific Railway (c. 1876) were graduates of early French engineering schools.

§ As a result of the considerable number of train accidents attributed to bridge structural failures.

¶ The design of the Britannia bridge was based on a simple span analysis even though Fairbairn and Stephenson had a good understanding of continuity effects on bending. The spans were erected simply supported, then sequentially jacked up at the appropriate piers and connected with riveted plates to attain span continuity.

** Jourawski was critical of Stephenson's use of vertical plate stiffeners in the Britannia Bridge.

provided significant contributions to the theory of truss analysis for iron and steel railway bridges. Karl Culmann, an engineer of the Royal Bavarian Railway, was a strong and early proponent of the mathematical analysis of trusses. He presented, in 1851, an analysis of the Howe and other proprietary trusses* commonly used in the United States. The Warren truss was developed in 1846,† and by 1850 W. B. Blood had developed a method of analysis for triangular trusses. Investigations, conducted primarily in the UK in the 1850s, into the effects of moving loads and speed were also being initiated. These investigations were preceded by theoretical work on the strength and vibration of railway bridges by Stokes and Willis in the late 1840s. Fairbairn also considered the effects of moving loads on determinate trusses as early as 1857.

J. W. Schwedler, a German engineer, presented the fundamental theory of bending moments and shear forces in beams and girders in 1862. Previously, he had also made a substantial contribution to truss analysis by introducing the method of sections. Also in 1862, A. Ritter improved truss analysis by simplifying the method of sections through development of the equilibrium equation at the intersection of two truss members. James Clerk Maxwell‡ and Culmann§ both published graphical methods for truss analysis. Culmann also developed an analysis for the continuous beams and girders that were often used in the 1850s by railroads. Later, in 1866, he published a general description of the cantilever bridge design method.¶ In subsequent years, Culmann also developed moving load analysis and beam flexure theories that were almost universally adopted by railroad companies in the Unites States and Europe. Developed by E. Winkler in 1867, bridge engineers were also given the powerful tool of influence lines for moving load analysis.

The effects of moving loads, impact (from track irregularities and locomotive hammer blow), pitching, nosing, and rocking of locomotives continued to be of interest to railway bridge engineers and encouraged considerable testing and theoretical investigation. Heavier and more frequent railway loadings were also creating an awareness of, and initiating research into, fatigue (notably by A. Wohler for the German railways).

North American engineers had recognized the need for rational and scientific bridge design, and, in response, J. A. L. Waddell published comprehensive books on steel railway bridge design in 1898 and 1916. Furthermore, Waddell and other engineers promoted independent bridge design in lieu of the usual proprietary bridge design and procurement practice of the American railroad companies. The Erie Railroad was the first to establish this practice and only purchased fabricated bridges from their own scientifically based designs. This soon became the usual practice of all American railroads.

1.4.2 Railway Bridge Design Specifications

Almost 40 bridges (about 50% of them iron) were collapsing annually in the United States during the 1870s. This was alarming as the failing bridges comprised about 25% of the entire American railway bridge inventory at that time. In particular, between 1876 and 1886, almost 200 bridge spans collapsed in the United States.

Proprietary railway bridges were failing, many due to a lack of rigidity and lateral stability. Most of these spans were built by bridge companies without the benefit of independent engineering design, and while some bridge companies had good specifications for design and construction, others did not. Therefore, without independent engineering design, railroad company officials required a good knowledge of bridge engineering to ensure public safety. This was not always the case, as demonstrated by the Ashtabula bridge collapse, where it was learned in the subsequent inquiry that the proprietary

* Culmann also analyzed Long, Town and Burr trusses using approximate methods for these statically indeterminate forms.
† The Warren truss was first used in a railway bridge in 1853 on the Great Northern Railway in the UK.
‡ Truss graphical analysis methods were developed and improved by J. C. Maxwell and O. Mohr between 1864 and 1874. Maxwell and W. J. M. Rankine were also among the first to develop theories for steel suspension bridge cables, lattice girders, bending force, shear force, deflection, and compression member stability.
§ Culmann published an extensive description of graphical truss analysis in 1866.
¶ Sir Benjamin Baker also outlined the principles of cantilever bridge design in 1867.

bridge design had been approved by a railroad company executive without bridge design experience.[*] To preclude further failures, American engineers were proposing the development and implementation of railroad company specifications that all bridge fabricators would use. Developments in the fields of materials and structural mechanics had supplied the tools for rational and scientific bridge design and provided the information required to establish specifications for iron and steel railway bridges.

The first specification for iron railway bridges was made by the Clarke, Reeves and Company (later the Phoenix Bridge Co.) in 1871. This was followed in 1873 by G. S. Morison's[†] "Specifications for Iron Bridges" for the Erie Railroad (formerly the New York, Lake Erie & Western Railroad). L. F. G. Bouscaren of the Cincinnati Southern Railroad published the first specifications with concentrated wheel loads in 1875.[‡] Following this, in 1878, the Erie Railroad produced a specification (at least partially written by Theodore Cooper) with concentrated wheel loads that specifically referenced steam locomotive forces.

By 1876, the practice of bridge design by consulting engineers working on behalf of the railroad companies became more prevalent in conjunction with the expanding railroad business. In particular, Cooper's publications concerning railway loads, design specifications, and construction were significant contributions to the development of a rational basis for the design of steel railway bridges. Cooper produced specifications, intended for use by all railroad companies, for iron and steel railway bridges as early as 1884. These were updated until 1888, and Cooper delivered his first specification for steel railway bridges in 1890. Cooper's specifications for steel railway bridges were updated until 1905. Nevertheless, many railroad companies continued to use their own specifications.[§] The multitude and variety of steel railway bridge specifications prepared by railroad companies, consulting engineers and fabricators, heralded the development of general specifications for steel railway bridges by the American Railway Engineering & Maintenance-of-way Association (AREMA) in 1906. This latter specification has been continuously updated and is the current recommended practice on which most North American railroad company design requirements are based. Other significant milestones in the development of general specifications for iron and steel railway bridges were as follows:

- 1867 St. Louis Bridge Co. specifications for Eads' steel arch[¶]
- 1873 Erie Railway Co. (G. S. Morison)
- 1877 Chicago, Milwaukee & St. Paul Railway Co. (C. Shaler Smith)
- 1877 Lake Shore & Michigan Southern Railway (C. Hilton)
- 1878 New York, Lake Erie, and Western Railroad (T. Cooper).
- 1880 Quebec Government Railways
- 1880 New York, Pennsylvania & Ohio Railroad
- 1886 Philadelphia and Reading Railroad
- 1890 Illinois Central Railroad
- 1894 Baltimore & Ohio Railroad (J. E. Greiner)

The large magnitude dynamic loads imposed on bridges by railroad traffic created a need for scientific design in order to ensure safe, reliable, and economical[**] construction. Railroad and consulting

[*] There were also material quality issues with the cast iron compression blocks, which were not discovered as the testing arranged by the Lake Shore and Michigan Southern Railroad company was inadequate.

[†] Morison and O. Chanute (an engineer educated in France) designed the 249-m (818 ft) long wrought iron viaduct for the Erie Railroad at Portage, New York, in 1875.

[‡] However, it appears that the first use of concentrated wheel loads for bridge design was by the New York Central Railroad in 1862.

[§] Cooper recommended a design live load of E40, but many railroads used their own specifications, which often specified design live loads equivalent to about Cooper's E50 and E60.

[¶] This was not a general specification but was the first use of specification documents in the design and construction of railway bridges in the United States. The specification also included the first requirements for the inspection of material.

[**] This can be a critical consideration as most railway bridge construction projects are privately funded by railroad companies.

engineers engaged in iron and steel railway bridge design were the leaders in the development of structural engineering practice.* Evidence of this governance was the publication by AREMA, in 1906, of the first general structural design specification for steel bridges in the United States, where design loads and material stresses were specified.

Allowable stresses for materials were provided based on generally conservative estimates of the tensile and compressive strength of steel.† Allowable stresses for steel and fasteners have been continuously modified in subsequent editions of the Manual for Railway Engineering (MRE) reflecting the latest materials research and engineering.

In 1906, the design live load was specified as Cooper's E40. The design live loads were increased in 1920, 1935, and 1968 to Cooper's E60, E72, and E80, respectively. The 2015 AREMA Manual of Recommended Practice (MRE) indicates a minimum Cooper's E80 (SI equivalent is Cooper's EM360) live load with an alternate load consisting of four 100 kip (SI equivalent is 445 kN) axles.

The MRE has also specified various formulas for calculating steam and diesel locomotive impact forces (dynamic increment) in various editions of the AREMA specifications and recommended practices. Most North American railroads discontinued steam locomotive use in the early 1960s. The diesel and diesel-electric locomotives that followed, in conjunction with improved track design, construction, and maintenance practices, have allowed bridge designers to use smaller impact loads for design.‡ Figure 1.21 outlines the recommended dynamic increment (impact) in the AREMA and American Railway Engineering Association (AREA) specifications and recommended practices of 1906, 1920, 1935, and 1968.§

Well-maintained steel railway bridges designed prior to the 1960s with relatively conservative allowable stresses¶ for heavy steam locomotives** with large dynamic increment (steam locomotive impact) continue to safely and reliably carry modern railway traffic.†† Many of these bridges are over 100 years old, providing evidence of the exceptional design, fabrication, and erection skills of early steel railway bridge engineers, and the scientific methods and specifications that guided their work. Modern steel railway bridge design practice is able to continue this record of safety and reliability using cost-effective materials, analysis and design methods based on updated design specifications, guidelines, codes, and recommendations such as AREMA MRE Chapter 15—Steel Structures.

1.4.3 Modern Steel Railway Bridge Design

The basic forms of ordinary steel railway superstructures have not changed substantially since the turn of the 20th century. Steel arch, girder, and truss forms are still routinely designed. However, considerable improvements in materials, structural analysis and design, and fabrication and erection technology occurred during the 20th century.

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* The advanced state of steel design and construction knowledge possessed by railway bridge engineers made them a greatly sought after resource by architects from about 1880 to 1900 during the rebuilding of Chicago after the Great Fire.

† The allowable tensile stress for steel was typically specified to be about 110 MPa (16,000 psi) in the AREMA specifications of the first quarter of the 20th century.

‡ Steam locomotive impacts were very large due to eccentric reciprocating wheel motion or "hammer blow."

§ Figure 1.21 is shown in US Customary or Imperial units only as the impact formulae of these older specifications and recommended practices were provided in only US Customary or Imperial units. The AREMA (2015) recommendations for impact loads in Chapter 15 are the same as the 1968 recommendations (see Chapter 4).

¶ Particularly for bridges designed in the early part of the 20th century.

** Steam locomotives used in the early part of the 20th century weighed about the same as modern diesel locomotives.

†† Modern rail car axle loads are typically not greater than modern diesel locomotive or older steam locomotive combined static and dynamic loads. However, older bridge design specifications did not consider fatigue as a design limit state (and did not need to because of the light rail cars pulled by few heavy locomotives). Nevertheless, older railway bridges generally perform well in the modern cyclical railway live load environment due to low allowable design stresses, internal redundancy of riveted connections, and the use of modern methods of fatigue life evaluation. Modern steel bridges must be designed considering fatigue due to the large number of high-magnitude tensile stress ranges experienced by some steel superstructure members (typically short members) and details.

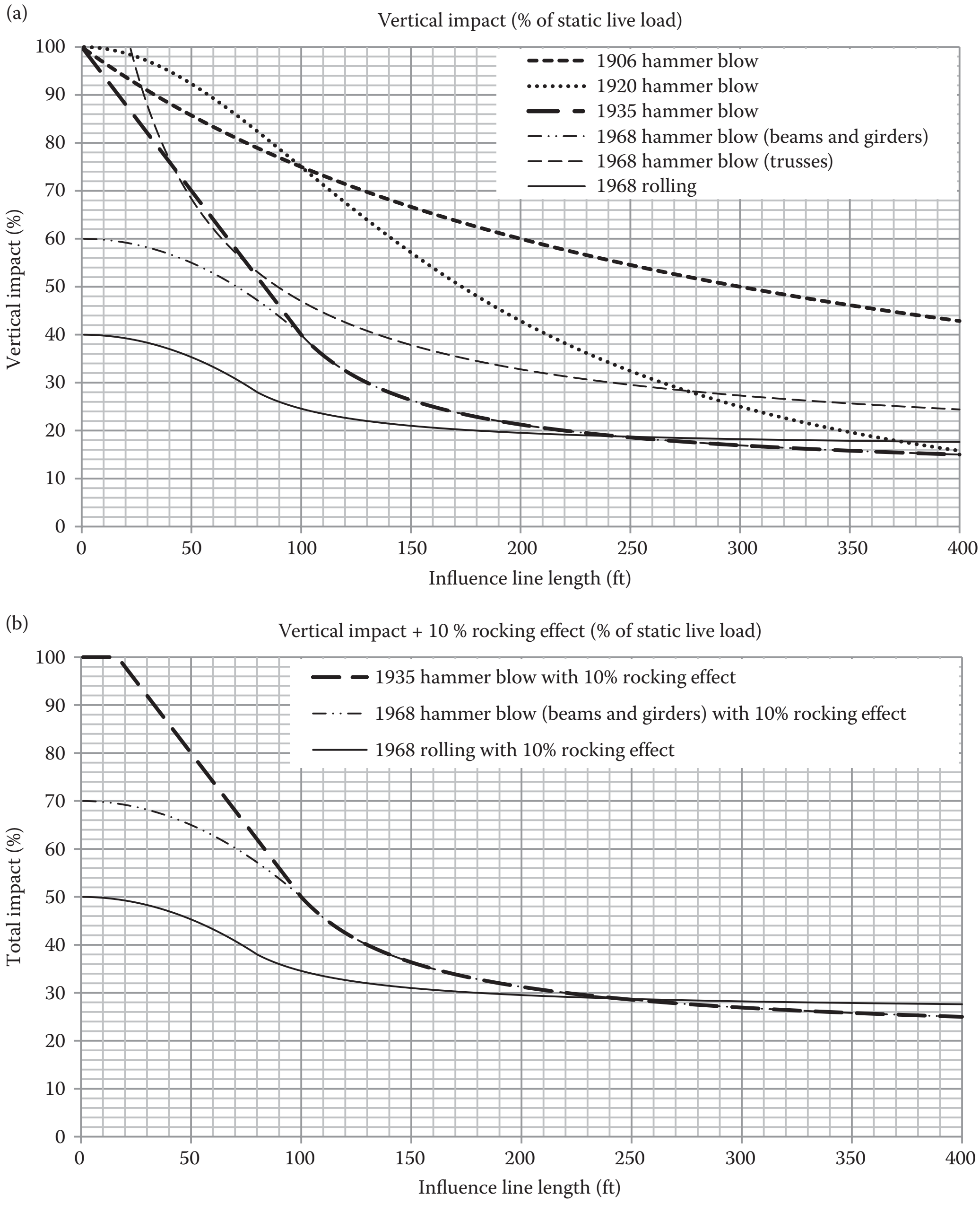

FIGURE 1.21 AREMA and AREA impact loads since 1905. (a) Vertical impact. (b) Total impact (with 10% rocking effect).

The strength, ductility, toughness, corrosion resistance, and weldability properties of structural steel have improved significantly since the middle of the 20th century. These material enhancements, combined with a greater understanding of planning considerations associated with modern bridge design and construction, have enabled the design of economical, reliable, and safe railway superstructures.

Modern structural analysis has also allowed considerable progress regarding the safety and economics of modern railway superstructures. Vast advancements in the theory of elasticity and structural mechanics were made in the 19th century as a result of railroad expansion. Today, the steel railway bridge engineer can take advantage of modern numerical methods, such as the matrix displacement (or stiffness) method, to solve difficult and complex structures. These methods of modern structural analysis have further evolved into multipurpose and specialized finite-element programs capable of linear elastic, nonlinear, static, dynamic (including seismic), stability, fracture, and other analyses using even small digital computers. In addition, modern methods of structural design, such as probabilistic (reliability) methods, that continue to enable the efficient and safe design of modern structures have ensued from recent research and practice.

Advances in manufacturing and fabrication technologies have permitted plates, sections, and members of large and complex dimensions to be fabricated and erected using superior fastening techniques such as welding and high-strength bolting. Modern fabrication with computer-controlled machines performing shop operations such as cutting, punching, drilling, bending, and welding has produced economical, expedient, and reliable steel railway superstructures. Advanced technologies such as radiographic and ultrasonic testing have enhanced modern fabrication quality control and quality assurance execution. Modern steel superstructure erection methods and procedures have also benefitted from technological advances in erection equipment such as large cranes, launching machinery, and transporter units.

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2 Steel for Modern Railway Bridges

2.1 INTRODUCTION

Steel development in the latter part of the 20th century has been remarkable. Modern steel is made of iron with small amounts of carbon, manganese, and traces of other alloy elements added to enhance physical properties. Chemical and physical metallurgical treatment has enabled improvements to many steel properties.

Mild carbon and high-strength low-alloy (HSLA) steels have been used for many years in railway bridge design and fabrication. Recent research and development related to high-performance steel (HPS) metallurgy has provided modern structural steels with even further enhancements to physical properties.

The important physical properties of modern structural bridge steels are:

- Strength
- Ductility
- Fracture resistance or toughness
- Weldability
- Corrosion resistance

These physical properties and general steel quality are controlled in the manufacturing process for structural steel shapes and plates used for superstructure fabrication.

2.2 MANUFACTURE OF STRUCTURAL STEEL

Significant advances in the art and science of steelmaking have occurred since the early part of the 20th century. Many of these advances have been related to the need for steels of increasingly higher strength with improved ductility, fracture toughness, corrosion resistance, and weldability properties. Modern high-strength structural steel shapes and plates are manufactured using chemistry* and process† to control these important physical and mechanical properties. Steel chemistry has the greatest influence on strength, ductility, fracture toughness, corrosion resistance, and weldability. Carbon and HSLA steels attain their mechanical properties through chemistry. However, increasing the strength of HSLA steel and HPS also requires supplemental heat treatment processes. HPS attains its mechanical properties through supplemental heat treatment in conjunction with chemistry manipulation.

Carbon and manganese are hardening and strengthening alloys. Carbon is the principal element controlling the mechanical properties of steel. The strength of steel may be increased by increasing the carbon content, but at the expense of ductility and weldability. Steel also contains deleterious elements, such as sulfur and phosphorous, that are present in the iron ore used to manufacture steel. Manganese also combines with sulfur to preclude the detrimental effects associated with the presence of elemental sulfur. Aluminum and silicon are alloyed to promote deoxidization and improve general steel quality. Chromium and copper are alloyed to increase atmospheric corrosion

* Chemical composition ranges for elements in various grades of structural steel are specified in ASTM and other applicable steel material specifications.

† Casting, rolling, and heat treatment operations.

resistance. Table 2.1 indicates the effects of various alloying elements on the physical and mechanical properties of steel.

The modern steelmaking process involves continuous casting of the molten steel (iron carbon, manganese, and other alloy elements) into slabs or blooms with relatively high cooling rates to discourage segregation of the elements.* Continuous casting provides plates with uniform physical properties at low production cost. Nevertheless, structural steel for railway superstructure fabrication requires steel mill process quality control to ensure that properties are appropriate in regard to fatigue and fracture performance. Specifically, measures are necessary to ensure that microscopic crack-like defects† do not occur due to trapped gasses and to minimize alloy element segregation

TABLE 2.1
Effects of Alloying Elements on Physical and Mechanical Properties of Steel

	Effect on Mechanical and Physical Properties	
Element	**Increase or Improve**	**Decrease or Reduce**
Aluminum (Al)	Toughness (with Si-killed steel)	Surface quality, hardness (aging)
Boron (B)	Hardenability (Q&T steels), strength (low-C Mo steels)	
Carbon (C)[a]	Strength, hardenability	Ductility, toughness, weldability
Chromium (Cr)[a]	Strength (high temperature), hardenability, toughness, abrasion resistance, corrosion resistance	Weldability
Columbium (Co)	Strength	Toughness
Copper (Cu)[a]	Corrosion resistance, strength, hardenability	Ductility, surface quality
Hydrogen (H)		Ductility (embrittlement)
Manganese (Mn)[a]	Strength, hardenability, sulfur control, toughness, corrosion resistance, ductility	Weldability
Molybdenum (Mo)[a]	Strength (high temperature), hardenability, abrasion resistance, corrosion resistance, weldability	Toughness, ductility
Nickel (Ni)[a]	Strength, toughness, hardenability, corrosion resistance, ductility	Weldability
Nitrogen (N)	Strength	Ductility
Oxygen (O)		Ductility, toughness
Phosphorus (P)[a]	Strength, hardenability, corrosion resistance	Ductility, weldability
Silicon (Si)[a] with other alloys	Strength, toughness, hardenability, ductility, deoxidation	Weldability, surface quality
Sulfur		Inclusions, weld porosity, and cracking
Titanium (Ti)	Strength, abrasion resistance, deoxidation, grain refinement	
Tungsten (W)	Strength (high temperature), hardenability, toughness, abrasion resistance	
Vanadium (V)[a]	Strength (high temperature, hardenability, abrasion resistance, deoxidation, grain refinement	

[a] Indicates the most common steel alloy elements.

* In particular, carbon segregation during casting may degrade steel uniformity, ductility, fracture toughness, and weldability. New HPSs with lower carbon content preclude excessive carbon segregation during casting.
† These defects occur at grain boundaries that are opened as trapped gasses escape.

during slab solidification and subsequent hot rolling operations. Atmospheric corrosion-resistant (weathering) steel chemistry also requires that production processes yield fine grain-size steel.

Degassing or "killing" steel involves alloying aluminum and/or silicone to reduce the oxygen available for the production of carbon dioxide. Aluminum alloying also promotes fine grain size. Low hydrogen processes such as vacuum degassing* can also be used to further protect against small crack-like defects caused by escaping hydrogen gases. Structural steel for railway superstructures must be killed or semikilled to reduce the creation of gases that affect fatigue strength and fracture resistance.

The cooled cast slabs are reheated and passed back and forth through a succession of rollers to create plates and shapes. Heat and roller pressure plastically deform the plate or shape to final dimensions for fabrication, but segregated alloy elements will tend to form planar inclusions.† Element segregation control is necessary to avoid the possibility of subsequent lamellar tearing.‡ Controlled cooling during the steel hot rolling process is often required to control element segregation, particularly for thicker plates such as those typically used for the flange plates of modern welded girders.

Nevertheless, hot-rolled structural shapes and plates may require postrolling heat treatment to improve physical and mechanical properties. Heat treatments such as normalizing, quenching and tempering (Q&T), and stress relieving may be used to enhance strength, ductility, and/or fracture toughness.

The quenching process following hot rolling increases strength, but at the expense of ductility and toughness.§ Normalizing¶ refines grain size and improves microstructure uniformity, providing increased ductility and fracture resistance.** Normalizing involves reheating the shape or plate between 900°C and 925°C (1650°F and 1700°F) and allowing the steel to cool slowly in air. However, because this postmanufacture heat treatment requires a furnace, shape and plate lengths for normalizing are often practically limited to about 15 m (50 ft).

Higher strength steel plates may be attained through the heat treatment of HSLA steel plates.†† These heat-treated low-alloy steel plates (Q&T steels) are not typically used for steel railway superstructure fabrication due to concerns with weldability.‡‡ Heat-affected zone (HAZ) strength may be detrimentally affected by welding, and welding consumables with equivalent yield and ultimate strength to that of the heat-treated low-alloy steel base metal are difficult to obtain. Thick plates and higher strength Q&T steels may also increase the propensity of the steel to hydrogen crack during welded fabrication.§§ Heat-treated low-alloy steel plates are produced by a Q&A process by reheating the plates to 900°C (1650°F) until an austenitic¶¶ microstructure is achieved. Subsequent rapid cooling provides increased hardness and strength, but at the expense of ductility and fracture toughness. Ductility and toughness may be improved through tempering by reheating between 425°C and 675°C (800°F and 1250°F) and slow cooling. Tempering results in a slight reduction in strength, but with greater ductility and fracture toughness. However, since a furnace is required, the production of heat-treated low-alloy steel (Q&T steels) may also be limited to lengths of about 15 m (50 ft).

Stress relieving is not typically required following the rolling process,*** but if necessary a specified heat can be applied followed by very slow cooling to relax internal stresses.

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* Vacuum degassing is used for the production of modern HPS to further control fatigue strength and fracture resistance.

† Typically at mid-thickness of thicker plates due to lower cooling rate. Element segregation is potentially greater in copper-alloyed atmospheric resistant steels.

‡ Generally occurs due to loading and/or welding operations.

§ In particular, for thick plates.

¶ Normalizing is typically specified by bridge owners for plates thicker than about 38 mm (1–1/2 in.) or 50 mm (2 in.).

** Ductility and toughness are improved by tempering with only a small effect on strength.

†† Many modern 485 MPa (70 ksi) and 690 MPa (100 ksi) yield strength steels attain their increased strength through heat treatment of 345 MPa (50 ksi) steel chemistry.

‡‡ However, some Q&T steels have been developed with low carbon content and good weldability.

§§ Fabrication-induced hydrogen cracking may be precluded by using an under-matching strength filler metal, increasing the preheat or welding heat input (see Chapter 10).

¶¶ The crystal structure of the steel transforms from ferrite to austenite when heated above 900°C (1650°F).

*** Typically required following some welding, cold bending, cutting, or machining operations to relieve residual stresses (see Chapter 10).

Postrolling heat treatments, such as normalizing, may be precluded by controlled hot rolling. Controlled hot rolling involves regulating heating rates, cooling rates, and holding times during the rolling process. Modern controlled hot rolling of plates may be performed precisely using the thermo-mechanically controlled process (TMCP).* TMCP equalizes plate temperature by localized heating and variable cooling rate sprays. TMCP produces plates with a fine and uniform microstructure.†

Controlled rolling heat treatment is not limited by plate length, but by plate thickness. Plate thicknesses greater than 50 mm (2 in.) are precluded by the roll pressures required for thicker plates at the lower rolling temperatures used in portions of the controlled hot rolling process. However, in many cases,‡ controlled hot rolling may preclude the need to normalize and avoid limitations on plate length.§

In some cases, fabricators may need to understand the tempering temperatures used in production heat treatments to ensure that mechanical properties are not altered by shop heating above the tempering temperatures.

2.3 ENGINEERING PROPERTIES OF STEEL

2.3.1 Strength

2.3.1.1 Elastic Yield Strength of Steel

Strength may be defined in terms of tensile yield stress, F_y, which is the point where plastic behavior commences at almost constant stress (unrestricted plastic flow). Strength or resistance may also be characterized in terms of the ultimate tensile stress, F_U, which is attained after yielding and significant plastic behavior. An increase in strength is associated with plastic behavior (due to strain hardening) until the ultimate tensile stress is attained (Figure 2.1). The most significant properties of steel that are exhibited by stress–strain curves are the elastic modulus (linear slope of the initial portion of the curve up to the proportional limit), the existence of yielding, and plastic behavior, with some unrestricted flow and strain hardening, until the ultimate stress is attained.

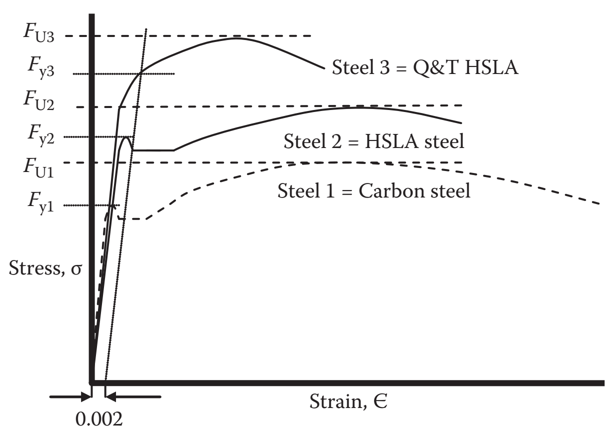

FIGURE 2.1 Engineering tensile stress–strain behavior of typical bridge structural steels.

* Not all steel mills have this technology.
† Grain size reduction and uniformity increase strength, ductility, and toughness.
‡ For economic and technical reasons (see Chapters 7 and 10), girder flange plate thickness is typically limited by bridge owners to less than about 65 mm (2–1/2 in.) or 75 mm (3 in.).
§ Limited plate lengths may require that flange plates of girders be spliced with shop butt welds. Such butt welds, particularly in tensile regions, must be carefully inspected (see Chapter 10).

Yield stress in tension can be measured by simple tensile tests (ASTM, 2015). Yield stress in compression is generally assumed to be equal to that in tension.* Yield stress in shear may be established from theoretical considerations of the yield criteria. Various yield criteria have been proposed, but most are in conflict with experimental evidence that yield stress is not influenced by hydrostatic (or octahedral normal) stress. However, two theories, the Tresca and von Mises yield criteria, meet the necessary requirement of being pressure independent. The von Mises criterion is most suitable for ductile materials with similar compression and tensile strength, and it also accounts for the influence of intermediate principal stress (Chen and Han, 1988; Chatterjee, 1991). It has also been shown by experiment that the von Mises criterion best represents the yield behavior of most metals (Chakrabarty, 2006).

The von Mises yield criterion is based on the octahedral shear stress, τ_h, attaining a critical value, τ_{hY}, at yielding. The octahedral shear stress, τ_h, in terms of principal stresses, $\sigma_1, \sigma_2, \sigma_3$, is

$$\tau_h = \frac{1}{3}\sqrt{(\sigma_1-\sigma_2)^2+(\sigma_1-\sigma_3)^2+(\sigma_2-\sigma_3)^2}. \tag{2.1}$$

Yielding in uniaxial tension will occur when $\sigma_1 = \sigma_Y$ and $\sigma_2 = \sigma_3 = 0$. Substitution of these values into Equation (2.1) provides

$$\tau_{hY} = \frac{\sqrt{2}}{3}\sigma_Y \tag{2.2}$$

or the criterion that, at yielding,

$$\sigma_Y = \frac{1}{\sqrt{2}}\sqrt{(\sigma_1-\sigma_2)^2+(\sigma_1-\sigma_3)^2+(\sigma_2-\sigma_3)^2}, \tag{2.3}$$

where σ_Y is the yield stress from the uniaxial tensile test.

It can also be shown that the octahedral shear stress at yield is (Hill, 1989)

$$\tau_{hY} = \sqrt{\frac{2}{3}}\tau_Y, \tag{2.4}$$

which when substituted into Equation (2.2) provides

$$\tau_Y = \frac{\sigma_Y}{\sqrt{3}}, \tag{2.5}$$

where τ_Y is the yield stress in pure shear. Therefore, a theoretical relationship is established between yield stress in shear and tension.

Example 2.1

Determine the allowable shear stress for use in design, f_v, if the allowable axial tensile stress, f_t, is specified as $0.55F_y$ and $0.60F_y$ (F_y is the axial tensile yield stress).

For $f_t = 0.55F_y$, f_v = allowable shear stress $= \dfrac{0.55F_y}{\sqrt{3}} = 0.32F_y$.

For $f_t = 0.60F_y$, f_v = allowable shear stress $= \dfrac{0.60F_y}{\sqrt{3}} = 0.35F_y$.

AREMA (2015) recommends an allowable shear stress for structural steel of $0.35F_y$.

* It is typically around 5% higher than the tensile yield stress.

2.3.1.2 Fatigue Strength of Steel

Localized material failure can occur when applied cyclical stresses* are greater than a threshold tensile stress range, but below the elastic yield stress. On a microscopic level, cyclical stresses may precipitate the movement of atomic dislocations creating slip bands and surface discontinuities,† particularly at grain boundaries.‡ Progressive microscopic material failure may involve a relatively long time to initiate a macroscopic crack, but some superstructure design details§ and fabrication imperfections¶ may cause more rapid fatigue crack initiation and propagation that could lead to failure.** The fatigue behavior of macroscopic detail stress raisers concerns the bridge design engineer. The macroscopic fatigue strength of steel railway superstructures is related to:

- Cyclical stress state (magnitude and number of cycles)
- Manufacturing residual stresses (casting and rolling)
- Design geometric details (welded attachments and stress concentrations)
- Fabrication quality and process residual stresses [rolling, cutting, welding (see Chapter 10)]
- In-service temperatures and atmospheric environment

A stress-life approach for the fatigue strength design of railway superstructures is appropriate for high-cycle fatigue at stress levels below the yield strength. The macroscopic fatigue behavior of common design details has been investigated by testing at nominal stress ranges that incorporate the stress concentration affects of the design detail. Analysis of the test data reveals a linear logarithmic relationship, with slope $-m$, between the number of cycles to failure and the constant amplitude stress range above a threshold or constant amplitude fatigue limit stress range as shown in Figure 2.2 (see also Chapter 5).

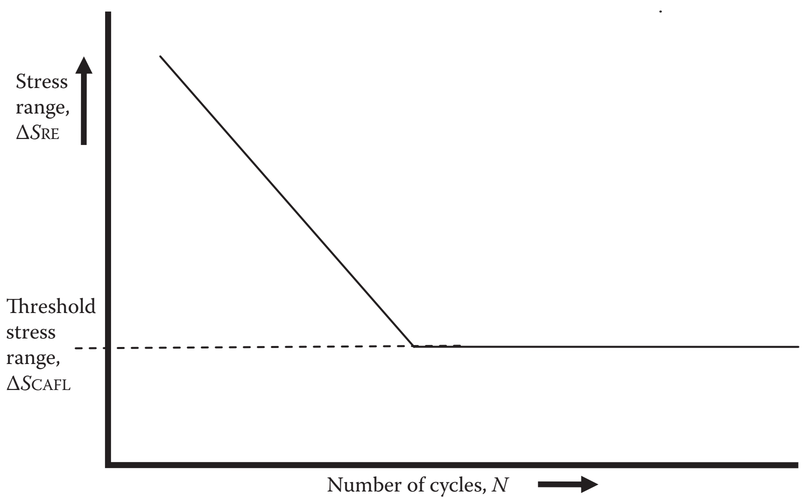

FIGURE 2.2 Fatigue strength behavior of typical bridge structural steels.

* Railway train loads are highly cyclical in nature creating a high-cycle fatigue regime, particularly on members with short influence lines (see Chapter 5).
† Essentially microscopic cracks.
‡ A principal reason for reduced grain size practice in steelmaking.
§ Design details such as welded attachments, intersecting welds, and copes have lower fatigue strength. The lower fatigue strength of design details is reflected by a lower value of the constant in the linear logarithmic relationship between stress range and number of cycles to failure (see Chapter 5).
¶ Many imperfections are avoided or mitigated during design, fabrication, and quality control/quality assurance (QC/QA) testing (see Chapter 10), but some may be unavoidable or undetected.
** Fatigue analysis is probabilistic and, therefore, fatigue "failure" is defined based on statistical criteria.

2.3.2 Ductility

Ductility is the ability of steel to withstand large strains after yielding and prior to fracture. Ductility is necessary in railway bridges and many civil engineering structures to provide advance warning of overstress conditions and potential failure. Ductility also enables the redistribution of stresses when a member yields in redundant systems, in continuous members, and at locations of stress concentrations (i.e., holes and discontinuities). Adequate ductility also assists in the prevention of lamellar tearing in thick elements.* Ductility is measured by simple tensile tests and specified as a minimum percentage elongation over a given gage length [usually 200 mm (8 in.)]. Only ductile steels are used in modern railway bridge fabrication.

2.3.3 Fracture Resistance

Brittle fracture occurs as cleavage failure with little associated plastic deformation. Once initiated, brittle fracture cracks can propagate at very fast rates as elastic strain energy is released (Fisher, 1984; Barsom and Rolfe, 1987). In steel railway bridges, this fracture can be initiated below the yield stress.

Fabrication-induced cracks, notches, discontinuities, or defects can create stress concentrations that may initiate brittle fracture in components in tension. Welding can also create hardened HAZ, hydrogen-induced embrittlement, and high residual tensile stresses near welds. All of these may be of concern with respect to brittle fracture. Rolled sections might contain rolling inclusions and defects that may also initiate brittle fracture. Thick plates are more susceptible to brittle fracture than thinner plates. Other factors that affect brittle fracture resistance are galvanizing (hot-dip), poor heat treatments, and the presence of nonmetallic alloy elements. Brittle fracture most often occurs from material effects in cold service temperatures, high load rates, and/or triaxial stress states (Figure 2.3).

Normal railway bridge strain rate application is relatively slow (in comparison to, e.g., machinery components or testing machines). Brittle fracture can, however, be caused by high strain rates associated with large impact forces from live loads.† Triaxial stress distributions and high stress

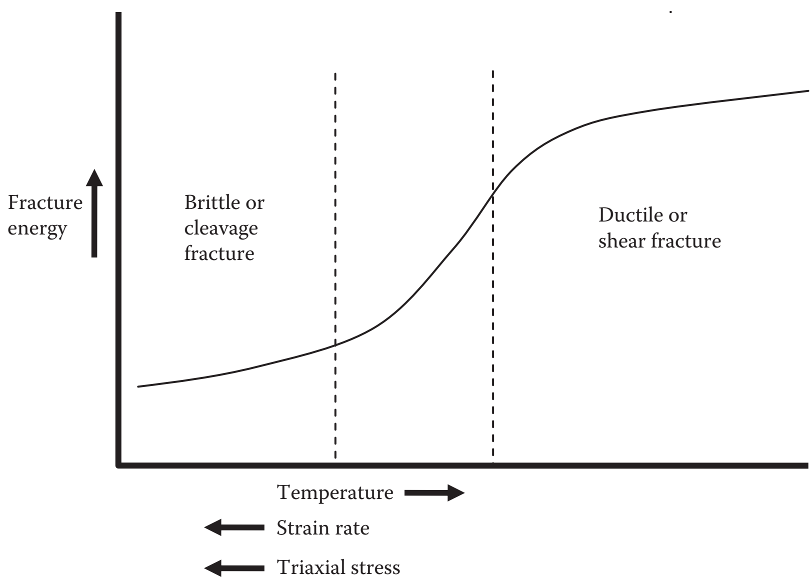

FIGURE 2.3 Fracture toughness behavior of typical bridge structural steels.

* Such as the relatively thick flange plates typically required for railway loads on long-span girders.

† Caused by poor wheel and/or rail conditions, derailment, or other vehicular collision.

concentrations can be avoided by good detailing and welding practice. Thick elements are often more susceptible to brittle fracture due to the triaxial stress state. Normalizing, a supplemental heat treatment, can be beneficial in improving material toughness through grain size reduction in thick elements (Brockenbrough, 2011). Adequate material toughness for the coldest service temperature likely to be experienced by the bridge (generally a few degrees cooler than the coldest ambient temperature) is critically important.

Temperature changes the ductile to brittle behavior of steel. A notch ductility measure, the Charpy V-notch (CVN) test, is used to ensure adequate material toughness against brittle fracture at intended service temperatures. A fracture control plan should ensure that weld metals have at least the same notch ductility as the specified base metal, and some specifications indicate even greater notch toughness requirements for welds in fracture critical members (FCMs). CVN testing is performed to establish notch ductility or material toughness based on energy absorbed at different test temperatures. CVN testing is done at a rapid load rate, so adjustments are made to the specified test temperature to account for the greater ductility associated with the slower strain rate application of railway traffic. For design purposes, temperature service zones are established with a specified minimum energy absorption at a specified test temperature for various steel types and grades. CVN requirements are often specified separately for FCM and non-FCM. Tables 2.2 and 2.3 show the specified CVN test requirements for non-FCM and FCM, respectively, for steel railway bridges recommended by AREMA (2015).

2.3.4 Weldability

If the carbon content of steel is less than 0.30%, it is generally weldable. Higher strength steels, where increased strength is attained through increased carbon and manganese content, will become hard and difficult to weld. The addition of other alloy elements to increase strength (Cr, Mo, and V) and corrosion resistance (Ni and Cu) will also reduce the weldability of steel.

The weldability of steel is estimated from an empirical carbon equivalency equation,* given as

$$CE = C + \frac{Mn + Si}{6} + \frac{Ni + Cu}{15} + \frac{Cr + Mo + V}{5}, \tag{2.6}$$

silicon, nickel, copper, chromium, molybdenum, and vanadium in the steel, respectively. Carbon equivalence, CE, of about 0.5% or greater indicates that special weld treatments may be required.

Weld cracking may result from resistance to weld shrinkage upon cooling. Thicker elements are more difficult to weld. Preheat and interpass temperature control, in conjunction with the use of low hydrogen electrodes, will prevent welding-induced hardening and cracking (see Chapter 10).

Modern high-strength structural steels have been developed with excellent weldability.† The increase in weldability enables limited preheat requirements and postweld treatments (translating into fabrication savings), and may eliminate hydrogen-induced weld cracking.

2.3.5 Corrosion Resistance

Atmospheric corrosion-resistant (weathering) steel chemistry (using chromium, copper, nickel, and molybdenum alloys) is such that a thin iron oxide film forms upon initial wetting cycles and prevents the further ingress of moisture. This type of corrosion protection works well where there are alternate wetting and drying cycles. It may not be appropriate in locations where deicing chemicals and salts are prevalent, in marine environments, or where there is a high level of sulfur content in the atmosphere.

* Other similar formulas also exist such as the Deardon and O'Neill equation and others formulated in Japan.

† For example, HPSs for bridges such as ASTM A709M (A709) HPS 345W (50W), 485W (70W), and 690W (100W).

Weldability is slightly compromised because carbon equivalence, CE (Equation 2.6), is raised through the addition of alloy elements for corrosion resistance. However, these steels have about four times the resistance to atmospheric corrosion as carbon steels (Kulak and Grondin, 2002), which makes their use in bridges economical from a life cycle perspective. Corrosion resistance can be estimated by a Corrosion index (CI), based on an empirical alloy content equation,*

$$CI = 26.01(Cu) + 3.88(Ni) + 1.20(Cr) + 1.49(Si) + 17.28(P) - 7.29(Cu)(Ni) - 9.10(P)(Ni) - 33.39(Cu)^2, \quad (2.7)$$

where Cu, Ni, Cr, Si, and P are the percentage of elemental copper, nickel, chromium, silicon, and phosphorus in the steel, respectively. A CI of 6.0 or higher† is typically required for bridge weathering steels.

Nonweathering steels can be protected with paint or sacrificial coatings (hot-dip or spray-applied zinc or aluminum). Shop applied three-coat paint systems are commonly used by many North American railroads. Two, and even single, coat painting systems are being assessed by the steel coatings industry and bridge owners. An effective modern three-coat paint system consists of a zinc-rich primer, epoxy intermediate coat, and polyurethane top coat. For aesthetic purposes, steel with zinc or aluminum sacrificial coatings can be top-coated with epoxy or acrylic paints.

2.4 TYPES OF STRUCTURAL STEEL

2.4.1 Carbon Steels

Modern carbon steel contains only manganese, copper, and silicon alloys. Mild carbon steel has a carbon content of 0.15%–0.29%, and a maximum of 1.65% manganese (Mn), 0.60% copper (Cu), and 0.60% silicon (Si). Mild carbon steel is not of high strength, but it is very weldable and exhibits a well-defined upper and lower yield stress (Steel 1 in Figure 2.1). Shapes and plates of ASTM A36M (A36) and A709M (A709) Grade 250 (36) are mild carbon steels used in railway bridge fabrication.

2.4.2 High-Strength Low-Alloy Steels

Carbon content must be limited to preclude negative effects on ductility, toughness, and weldability. Therefore, it is not desirable to increase strength by increasing carbon content, and manipulation of the steel chemistry needs to be considered. HSLA steels have increased strength attained through the addition of many alloys.

Alloy elements can significantly change steel phase transformations and properties (Jastrewski, 1977). The addition of small amounts of chromium, columbium, copper, manganese, molybdenum, nickel, silicon, phosphorous, and vanadium in specified quantities results in improved mechanical properties. The total amount of these alloys is less than 5% in HSLA steels. These steels typically have a well-defined yield stress in the 300–415 MPa (44–60 ksi) range (Steel 2 in Figure 2.1). Shapes and plates of ASTM A572M (A572), A588M (A588), and A992M (A992) (rolled shapes only) and A709M (A709) Grade 345 (50), 345S (50S), and 345W (50W) are HSLA steels used in railway bridges.

A572M (A572) Grade 290 (42), 345 (50), and 380 (55) steels are used for bolted or welded construction. Higher strength A572M (A572) steel [Grades 415 (60) and 450 (65)] is used for bolted construction only, due to reduced weldability. A572M (A572), A588M (A588), and A992M (A992) steels are not

* This equation is given in ASTM G101. Other equations, such as the Townsend equation, have also been proposed and may be of greater accuracy.

† ASTM A588M (A588) steel has a CI of about 5.8 (Swanson, 2014), but it is considered acceptable as an atmospheric corrosion resistant steel for railway superstructures (Table 2.5).

material toughness graded at the mills and often require supplemental CVN testing to ensure adequate toughness, particularly for service in cold climates. A588M (A588) and A709M (A709) Grade 345W (50W) steels are atmospheric corrosion-resistant (weathering) steels. ASTM A709M (A709) Grade 345 (50), 345S (50S), and 345W (50W) steel is mill certified with a specific toughness in terms of the minimum CVN impact energy absorbed at a given test temperature (e.g., designations 345T2 (50T2) indicating non-FCM Zone 2 and 345WF3 (50WF3) indicating FCM Zone 3 toughness criteria).

Further increases in strength, ductility, toughness, and corrosion resistance through steel chemistry alteration have been made in recent years. HSLA steels with 485 MPa (70 ksi) yield stress have been manufactured with niobium, vanadium, nickel, copper, and molybdenum alloy elements. These alloys stabilize either austenite or ferrite so that martensite formation and hardening does not occur, as it may for higher strength steel attained by heat treatment. A concise description of the effects of various alloy and deleterious elements on steel properties is given in Brockenbrough (2011).

2.4.3 Heat-Treated Low-Alloy Steels

Higher strength steel plate [with yield stress in excess of 485 MPa (70 ksi)] is produced by heat treating HSLA steels. A disadvantage of higher strength steels is a decrease in ductility. Heat treatment restores loss of ductility through Q&A processes. The quenching of steel increases strength and hardness with the formation of martensite. Tempering improves ductility and toughness through temperature relief of the high internal stresses caused by martensite formation. However, after quenching, tempering, and controlled cooling, these steels will not exhibit a well-defined yield stress (Steel 3 in Figure 2.1). In such cases, the yield stress is determined at the 0.2% offset from the elastic stress–strain relation (Figure 2.4).

Use of these steels may result in considerable weight reductions and precipitate fabrication, shipping, handling, and erection cost savings. High-strength steel can also allow for design of shallower superstructures. ASTM A514M (A514), A852M (A852), and A709M (A709) Grade 485W (70W) and 690W (100W) are quenched and tempered low-alloy steel plates. However, none of these steels are typically used in ordinary railway superstructures due to weldability concerns.

2.4.4 High-Performance Steels

HPS plates have been developed in response to the need for enhanced toughness, weldability, and corrosion resistance of high-strength steels. HPS 485W (70W) and 690W (100W) steels are produced by a

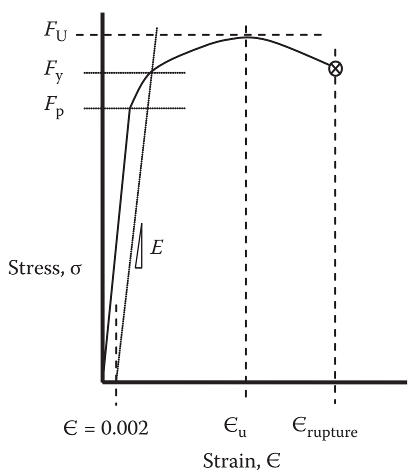

FIGURE 2.4 Engineering tensile stress–strain behavior of typical high strength bridge structural steel.

combination of chemistry manipulation and quench and temper operations or, for longer plates, TMCP. The first HPSs were produced with a yield stress of 485 MPa (70 ksi). However, HPS with 345 MPa (50 ksi) yield stress soon followed due to the weldability, toughness, and atmospheric corrosion resistance property improvements of HPS. HPS 345W (50W) is produced with the same chemistry as HPS 485W (70W), using conventional hot or controlled rolling techniques. HPS plates with 690 MPa (100 ksi) yield stress are also available. HPS 690W (100W) is considered an improvement to A514M (A514) steel plates (Lwin et al., 2005). HPS with 690 MPa (100 ksi) yield stress has been quench and temper heat treated to provide good ductility, weldability, and CVN toughness (Chatterjee, 1991).

Weldability is increased by lowering the carbon content [e.g., below 0.11% for HPS 485W (70W)], therefore, benefiting the carbon equivalence (Equation 2.6). This weldability increase results in the elimination of preheat requirements for thin members and limited preheat requirements for thicker members. Also, postweld treatments are reduced and hydrogen-induced cracking at welds eliminated (provided correct measures are taken to eliminate hydrogen from moisture, contaminants, and electrodes). Welding of HPSs using low hydrogen electrodes is done by submerged arc welding or shielded metal arc welding processes (see Chapters 9 and 10).

Toughness is significantly increased through reductions in sulfur content (0.006% max) and control of inclusions (by calcium treatment of steel). The fracture toughness of HPS is, therefore, much improved with the ductile to brittle transition occurring at lower temperatures (the curve shifts to the left in Figure 2.2). Higher toughness also translates into greater crack tolerance for fatigue crack detection and repair procedure development. HPSs meet or exceed the CVN toughness requirements specified for the coldest climates (Zone 3 in AREMA, 2015).

The corrosion-resistant properties of HPS are based on quenched and tempered ASTM A709M (A709) Grade 485W (70W) and 690W (100W) steels. Chromium, copper, nickel, and molybdenum are alloyed for improved weathering resistance. Improved weathering resistant steels are under development that might provide good service in even moderate chloride environments.

Hybrid[*] applications of HPSs with HSLA steels have proven technically and economically successful on a number of highway bridges (Lwin, 2002) and may be appropriate for some railway bridge projects.

2.5 STRUCTURAL STEEL FOR RAILWAY SUPERSTRUCTURES

There is no increase in stiffness associated with higher strength steels (deflections, vibrations, and elastic stability are proportional to the modulus of elasticity and not strength). Also, because fatigue strength depends primarily on applied stress range and detail (see Chapter 5), there is no appreciable increase in fatigue resistance for higher strength steels.[†] Therefore, the material savings associated with the use of higher strength steels [with greater than 345 MPa (50 ksi) yield stress] may not be available because deflection criteria and fatigue often govern critical aspects of ordinary steel railway superstructure design. The steel bridge designer must carefully consider all design limit states (strength, serviceability, fatigue, and fracture), procurement (availability and cost), and fabrication issues when selecting the materials for railway bridge projects.

2.5.1 Material Properties

The following material properties may be used for steel railway bridge design and construction:

- Density, $\gamma = 7850\,\text{kg/m}^3$ ($490\,\text{lb/ft}^3$)
- Modulus of elasticity (Young's modulus), $E = 200{,}000\,\text{MPa}$ ($29\times10^6\,\text{psi} = 29{,}000\,\text{ksi}$)
- Coefficient of thermal expansion, $\alpha = 12\times10^{-6}/°\text{C}$ ($6.5\times10^{-6}/°\text{F}$)

[*] An example is the use of HPSs for tension flanges in simple and continuous girders.

[†] Recent testing indicated that CVN requirements for HPS grades were only marginally better than current AREMA and AASHTO Zone 2 and Zone 3 specifications (Alstadt et al., 2014).

- Poisson's ratio, $\upsilon = 0.3$ (lateral to longitudinal strain ratio under load)
- In accordance with the theory of elasticity, shear modulus, $G = \dfrac{E}{2(1+\upsilon)} \sim 77{,}000\,\text{MPa}$ ($\sim 11.2 \times 10^6$ psi)

2.5.2 Structural Steel for Modern North American Railway Superstructures

Structural bridge steels have increased in strength and quality over the past century. Table 2.2 indicates the strength of some of the structural steels used in the past century in the United States and Canada (Canadian Institute of Steel Construction, 2004).

Modern structural bridge steels provide good ductility, weldability, and corrosion resistance. Structural steel for use in modern railway superstructures in North America is typically specified as ASTM A36M (A36), A572M (A572), A588M (A588), A709M (A709), and/or A992M (A992), depending on strength, ductility, welding, and corrosion-resistant requirements. Tables 2.3 and 2.4 indicate the toughness requirements for these steels for non-FCM and FCM applications, respectively. Table 2.5 outlines the strength of these steels for use in railway superstructures.

The AREMA (2015) recommendations do not include heat-treated low-alloy steels. The only steel with a yield stress greater than 345 MPa (50 ksi) currently recommended is A709M (A709)

TABLE 2.2
Structural Steel Used in North America Since 1900

			F_y		F_u	
Steel Designation	**Country**	**Date**	**(MPa)**	**(ksi)**	**(MPa)**	**(ksi)**
ASTM A7	USA	1900–1909	$0.5F_u$	$0.5F_u$	410–490	60–70
		1914	$0.5F_u$	$0.5F_u$	380–450	55–65
CSA A16	Canada	1924	$0.5F_u$	$0.5F_u$	380–450	55–65
ASTM A7	USA	1924	$0.5F_u \geq 210$	$0.5F_u \geq 30$	380–450	55–65
		1934	$0.5F_u \geq 230$	$0.5F_u \geq 33$	410–500	60–72
CSA S39	Canada	1935	210	30	380–450	55–65
CSA S40	Canada	1935	230	33	410–500	60–72
CSA G40.4 and G40.5	Canada	1950	230	33	410–500	60–72
CSA G40.6	Canada	1950	310	45	550–650	80–95
ASTM A242	USA	1955	350	50	480	70
ASTM A36	USA	1960	250	36	410–550	60–80
ASTM A440 and A441		1959 and 1960	350	50	480	70
CSA G40.8	Canada	1960	280[a]	40[a]	450–590	65–85
CSA G40.12	Canada	1964	300[b]	44[b]	450	65
ASTM A572 (Grade 50)	USA	1966	345	50	450	65
ASTM A588	USA	1968	345[c]	50[c]	485[c]	70[c]
CSA G40.21	Canada	1973	Incorporated all previous CSA G40 standards			
ATM A992	USA	1998	345–450	50–65	450	65

[a] Less for material thicker than 16 mm (5/8″)
[b] Less for material thicker than 40 mm (1–1/2″)
[c] Less for thicker material