Variability of the Mechanical Properties of Wrought Iron from Historic American Truss Bridges

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Abstract: The mechanical properties of wrought iron from multiple elements of six late 19th-century truss bridges is evaluated by a program of destructive and nondestructive testing, including hardness testing and tension tests to evaluate the yield stress (F_y), tensile strength (F_u), and ductility of the material. The yield stress and tensile strengths are found to be in accordance with those published in period reports and in other modern evaluations of the mechanical properties of wrought iron. The main findings of this work come from a statistical analysis of the test results and are (1) that hardness is a poor predictor of yield stress and tensile strength but has some predictive ability for ductility; (2) that there is a statistically significant difference in the distribution of yield stress and tensile strength between material samples from different bridges and, in some cases, between material samples from different members within a single bridge; and (3) that a size effect is present in the material that results in lower yield stress and tensile strength for larger members. These results provide guidance to engineers in the evaluation of historic iron trusses for rehabilitation and suggest that although nondestructive hardness testing is of limited value, a limited program of destructive testing can provide an adequate characterization of the mechanical properties throughout the bridge. **DOI: 10.1061/(ASCE)MT.1943-5533.0000220.** *© 2011 American Society of Civil Engineers.*

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Introduction

Wrought iron was the primary structural metal used in bridge construction throughout the 19th century; however, by the turn of the 20th century, its use in bridges had been largely replaced by steel. Although it has not been used in new bridge construction for nearly a century, many truss bridges that contain wrought-iron members or components are still being used throughout the United States. The ability for engineers to effectively assess the integrity of these bridges for continued use or rehabilitation depends on a sound understanding of the mechanical properties of wrought iron. This paper examines the mechanical properties of wrought iron through mechanical testing of samples acquired from six 19th-century iron truss bridges. The variability of these properties was investigated and the use of nondestructive testing methods, such as hardness testing, was considered as a field technique for predicting the properties of wrought iron.

Background

Unlike other structural metals, wrought iron is a composite material that contains both metallic and nonmetallic constituents (Gordon 1988). The microstructure of wrought iron consists of a ferrite matrix, consisting mostly of iron with small amounts of phosphorus, carbon, and silicon (each less than 1%), with slag inclusions dispersed throughout it. Slag, a result of the manufacturing process of wrought iron, is a nonmetallic, glasslike substance comprised of fayalite, an iron-silicon-oxygen mineral, or glass with a composition similar to fayalite (Gordon 2008). Slag is unique to wrought iron and is found in well-made material as long strands dispersed throughout the ferrite matrix, dividing it into columns and giving it a fibrous appearance. Although once thought to be an impurity, slag adds to the toughness of wrought iron and is considered an essential component (Kemp 1993).

Nineteenth-century wrought iron was produced in a type of reverberatory furnace known as a puddling furnace. In a puddling furnace, the iron was kept separate from the fuel source, and air was used to blow flames onto the iron, raising it to very high temperatures; the molten iron, called a puddle, was then manually stirred to oxidize carbon from it (Kemp 1993). The result was a spongy mass that would then be removed and mechanically worked into structural shapes. Although more efficient than previous methods of production, this method was still labor intensive, and these furnaces were limited as to how much iron could be produced at any given time. Furthermore, the quality of wrought iron produced through this method was not regulated to any substantial degree, was difficult to control, and relied heavily on the skill of the makers, thus possibly contributing to the variability of its mechanical properties. Nineteenth-century producers of wrought iron were not required to meet any national standards or provide chemical composition or stress strain curves of the iron they produced (Kemp 1993). In effect, they were self-regulating, producing a sufficiently strong material at the least cost. For these reasons, it is likely that wrought-iron's mechanical properties may vary not just between bridges and between manufacturers, but even within the structural components of a single bridge produced by the same manufacturer.

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Table 1. Bridge, Truss Type, Manufacturer, and Source Members for Each Specimen Group

Specimen group	Bridge	Truss type	Manufacturer	Member type (thickness)
BV-H	Bondsville	Pratt pony truss	Wrought Iron Bridge Co.	Round beam hanger (38 mm, 1 1/2 in.)
RB-H	Reeds Bridge	Warren pony truss	R.F. Hawkins Iron Works	Round beam hanger (38 mm, 1 1/2 in.)
GH-H	Golden Hill	Lenticular pony truss	Berlin Iron Bridge Co.	Square beam hanger (28.5 mm, 1 1/8 in.)
GA-H	Galvin	Lenticular through truss	Unknown	Square beam hanger (28.5 mm, 1 1/8 in.)
CB-H	Chester	Pratt pony truss	R.F. Hawkins Iron Works	Round beam hanger (44.5 mm, 1 3/4 in.)
CB-L	Chester	Pratt pony truss	R.F. Hawkins Iron Works	Rectangular lacing (6.3 mm, 1/4 in.)
SV-H	Shattuckville	Pratt through truss	Groton Bridge Co.	Round beam hanger (25.4 mm, 1 in)
SV-L	Shattuckville	Pratt through truss	Groton Bridge Co.	Rectangular lacing (6.3 mm, 1/4 in.)
SV-E	Shattuckville	Pratt through truss	Groton Bridge Co.	Square eye-bar (22.2 mm, 7/8 in.)
SV-B	Shattuckville	Pratt through truss	Groton Bridge Co.	Square looped-bar (22.2 mm, 7/8 in.)

The quality may also have varied with time, as manufacturing changed.

Bridge Iron Owned by the University of Massachusetts

Mechanical testing was conducted on wrought-iron specimens taken from six 19th-century truss bridges that were donated to the Department of Civil and Environmental Engineering at the University of Massachusetts, Amherst (UMass). Table 1 gives a summary of the six bridges used in this study. Built within a span of 15 years, the first built in 1880 and the last built in 1895, these bridges were mostly taken from rural town roads and have relatively short span lengths, ranging from 12 m (40 ft) to 31 m (103 ft). Built by several different manufacturers, the bridges include five different truss configurations: a Warren pony truss, two Pratt pony trusses, a Pratt through-truss, a lenticular pony truss, and a lenticular through-truss. As it is intended that these bridges will be rehabilitated and placed in various locations around the UMass campus for pedestrian use, test coupons were taken from easily replaceable members, mainly beam hangers. For two of these bridges, iron was tested from several structural members in addition to the beam hangers to quantify the variability of the iron.

Wrought iron was sampled from square and round beam hangers of several bridges to investigate variability of mechanical properties across bridges within a single member type, and iron was sampled from multiple members of a single bridge to provide insight into how these properties vary across different member types from a single bridge. Mechanical testing included tension tests and hardness tests; properties examined include yield stress, tensile strength, percent reduction in area, and Rockwell hardness. In addition to the results of mechanical testing, the effect of working on tensile strength and the correlation (ρ) between hardness and strength were examined.

This paper discusses both modern and historical sources of information on the strength of wrought iron. Methods of testing and specimen design are described along with testing results. A subsequent discussion examines these results and the analyses performed on them, specifically quantifying size effect in wrought iron and the correlation between hardness and strength.

Historic and Modern Testing of Wrought Iron

In the late 19th century, as steel production increased, wrought iron was gradually phased out of use as a structural metal for bridges. As a result, the mechanical properties of wrought iron have not been documented as well as those of structural metals still used in new construction. Nevertheless, there was great interest in the properties of wrought iron manufactured in the second half of the 19th century, and several significant contemporary studies on wrought iron were conducted (Beardslee 1879; Kirkaldy 1862). Although the testing methods and equipment used by 19th-century researchers did not have the capacity to measure certain properties that may be of interest today, such as yield stress and elastic modulus, these studies remain a valuable reference for those interested in the properties of wrought iron.

David Kirkaldy, a Scottish researcher, conducted some of the most noteworthy research on wrought iron in the 19th century. Using standardized testing procedures and specimens machined from contemporary iron, Kirkaldy performed more than 1,000 tensile tests on wrought-iron coupons taken from bars, plates, and angle irons, publishing the results in his book *Experimental Inquiry into the Comparative Tensile Strength and Other Properties of Various Kinds of Wrought-Iron and Steel* (Kirkaldy 1862). Kirkaldy was careful to randomly select his source material from supplier's stocks, eliminating the possibility of supplier's providing him with their best iron that might not have been representative of typical wrought iron. Although unable to measure a yield stress with his testing equipment, values of tensile strength, elongation, and reduction in area were reported for all tests.

Kirkaldy also investigated the effect of working on wrought-iron strength. The degree of working correlates closely with the size of the iron member, with more working required to fabricate members with smaller cross sections, so the effect of working on strength appears as a size effect, with smaller members possessing greater strength. As iron is worked down to smaller thicknesses, impurities within the iron are better distributed, thus improving the overall quality of the iron. Through his testing of many different bars of varying thickness, he observed a decrease in strength of specimens tested from larger bars compared with those from smaller thickness bars, although this effect was found to vary among the different makes of iron he tested. For lower quality iron, he observed an 8% increase in strength between bars of 31.8-mm (1 1/4-in.) thickness and bars of 15.9-mm (5/8-in.) thickness, whereas for higher quality iron with the same change in dimension, the increase in strength was found to only be 1%. To examine this further, Kirkaldy cut four pieces of iron off a single 38.1-mm (1 1/2-in.) diameter bar, and then rerolled those pieces down to sizes of 31.8, 25.4, 19.0, and 12.7 mm (1 1/4, 1, 3/4, and 1/2 in.) and tested the tensile strength of these bars. From this testing, he found that the bars with the smaller cross sections had greater tensile strength, with the 12.7-mm (1/2-in.) bars being 5% stronger than the 31.8-mm (1 1/4-in.) bars.

In the latter part of the 19th century, a government-sponsored program was begun in the United States to investigate the mechanical properties of wrought iron for use in chain cables for ships. The findings of the program, overseen by Commander L. A. Beardslee, USN, were published under the title *Experiments on the Strength of Wrought-Iron and of Chain-Cables* (Beardslee 1879). Although the primary goal of the program was to investigate the strength of wrought-iron bars that could be forged into chain cables for the U.S. Navy, the bars tested could also have been used as bridge tension members. Iron was acquired from 14 major producers of wrought iron in the United States, but the samples were not randomly selected by Beardslee from the iron producers' stocks. It is therefore possible the quality of the iron may be better than what was typical of the time. Regardless, the Beardslee report is still a useful source of information that contains results from hundreds of wrought-iron tension tests, which include both yield stress and tensile strength values. Unlike Kirkaldy's work a decade earlier, Beardslee was able to measure yield stress by using a method in which the first observable elongation without a load increase was taken as the yield strength. In addition to mechanical testing, the Beardslee report also examined the chemical composition of wrought iron and the effects of phosphorus, carbon, and silicon content on strength and ductility.

Beardslee also investigated the size effect in wrought iron. In addition to performing tensile tests on wrought-iron bars of varying size, he also visited iron mills and carefully observed the process of rolling. Included in his report are detailed tables listing bar diameter and strength, original pile size, and number of times passed through rollers for bars of different thickness. Like Kirkaldy, he also observed that bars of lesser thickness had greater tensile strength than bars of greater thickness. He found the smallest bars he tested, 9.5 mm (3/8 in.), had 16% greater tensile strength than the largest bars he tested, 102-mm (4-in.) thickness, and a 42% greater value of yield strength. Numerical values of strength from both the Beardslee and Kirkaldy references will be examined subsequently in this paper when compared with the testing results of the UMass bridge iron.

More recently, there has been renewed interest in understanding the properties of wrought iron to improve the evaluation of historic structures for rehabilitation. Several researchers have examined wrought-iron specimens sampled from historic structures, with the primary goal of providing material properties to aid in the evaluation of these structures. Gordon and Knopf (2005) examined the properties of wrought iron acquired from three 19th-century truss bridges in upstate New York. Properties such as strength, ductility, hardness, and chemical composition were investigated as well as their relation to one another. Gordon and Knopf (2005) also emphasized the importance of a balance between wrought iron's ductility and strength for it to continue to safely serve in a load-bearing structure. Iron with good toughness will deform plastically before failure, whereas very strong iron may lack toughness and is more susceptible to brittle failure (Gordon and Knopf 2005).

In another recent study, Bowman and Piskorowski (2004) tested wrought iron from components of two 19th-century truss bridges from Indiana. Test coupons were sampled from eye-bars and tension bars of these bridges and uniaxial tension tests, Charpy V-notch impact, hardness, chemical analysis, and several fatigue tests were performed.

Other testing of historic bridge wrought iron includes the work of Elleby et al. (1976), Fu and Harwood (2000), and Keller and Kirkpatrick (2006). These researchers only tested a small number of specimens (ranging from six to 17), as the primary goal of testing was to obtain some information on material properties to aid in the design of rehabilitation strategies for the bridges from which the iron was taken.

Materials and Methods

Mechanical testing was conducted on wrought-iron tension coupons machined from six 19th-century truss bridges [see Table 1 for bridge descriptions and Fig. 1(a) for a photograph of one of the bridges prior to dismantling]. These bridges form part of the inventory owned by UMass through the Adaptive Use Bridge Project (http://www.ecs.umass.edu/adaptive_bridge_use). The goals of the project are to acquire historic truss bridges that are being decommissioned from regular use, bring them to UMass, and rehabilitate them for use as pedestrian, bicycle, or light-traffic bridges on campus. Furthermore, testing and analysis of the bridge materials and structures are being incorporated into the UMass civil engineering curriculum. Coupons were cut from the beam hangers of all bridges and from additional components of the Shattuckville and Chester bridges. From the Shattuckville bridge, specimens were cut from three additional members: an eye-bar that was originally part of the bottom chord of the bridge, a looped bar that acted as a tension diagonal in one of the panels, and several lacing members, which made up a portal frame that acted as lateral bracing for the two trusses comprising the bridge. From the Chester bridge, additional coupons were cut from lacing that made up the guard rail of the bridge. Although not part of the primary load-carrying system, the railing was built integrally into the bridge trusses, spanned the entire length of the bridge, and would have had to be able to resist impact loads from wayward traffic. It was therefore assumed for the present study that the iron used is not compositionally different from the iron used in the main load-bearing members of the bridge. Specimens were grouped according to bridge and original member (Table 1). Fig. 1(b) shows the different members sampled and their relative locations on a typical truss bridge. Note that the truss configuration shown is not representative of all bridges from which iron was tested but is rather a composite of the various forms drawn to allow the depiction of all member locations.

It must be acknowledged that the primary structural members tested in this study are limited to the eye-bar and looped bar from the Shattuckville bridge, with the remainder of the specimens being load-bearing but noncritical beam hangers and non load-bearing lacing members. The selection of members for destructive testing was guided by a desire to, as much as possible, preserve key structural members for adaptive use of the bridges. The Shattuckville bridge is incomplete and in generally poor condition, so out of the current inventory, it is the least likely to be reconstructed. It is for this reason that the eye-bar and looped bar from Shattuckville were selected for destructive testing. Although beam hangers are not primary structural members, they are load bearing, and therefore are representative of wrought iron that would have been used in load-bearing members.

To account for the different cross-sectional geometries of the specimens' source material, four different tension coupon designs were adopted. The standard coupon, used for material from beam hangers, was a cylindrical coupon with a total length of 127 mm (5 in.), a reduced section length of 50.8 mm (2 in.), and a reduced section cross section of 12.7-mm (1/2-in.) diameter. For the thinner lacing elements, a plate specimen was used with a total length of 178 mm (7 in.), having a 57-mm (2 1/4-in.) reduced section length with a thickness of 5.1 mm (2/10 in.) and reduced section width of 12.7 mm (1/2 in.). The cylindrical and plate coupons were both designed in accordance with ASTM E8 (2003).

As a result of the larger cross-sectional area of the looped and eye-bars, a larger specimen design was desired to preserve as much of the cross section as possible for testing. A different coupon design was used for each member; these coupons were designed

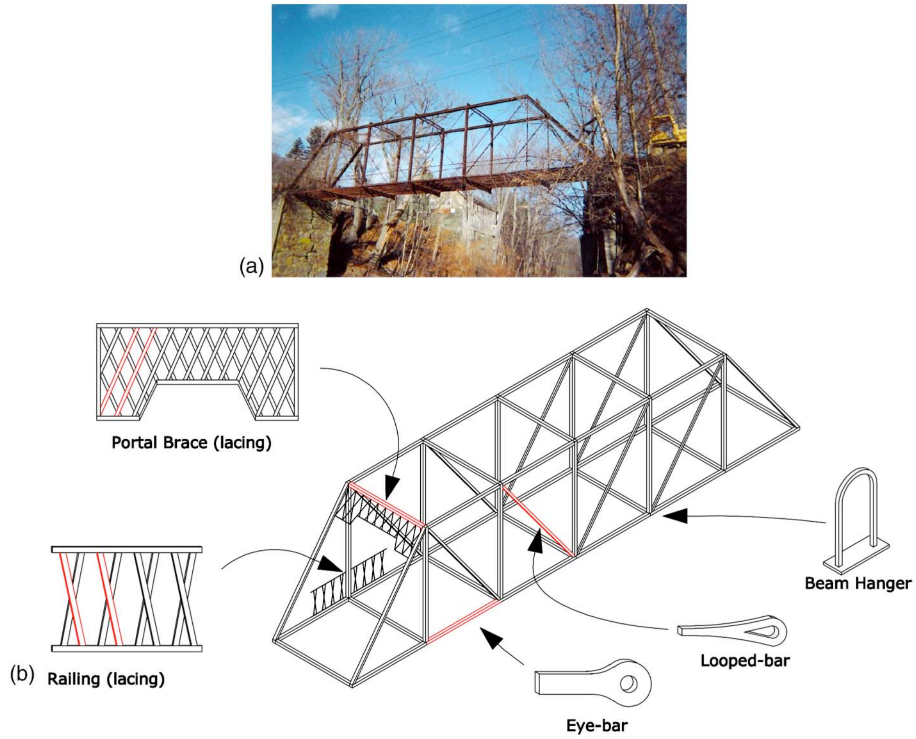

Fig. 1. Shattuckville, Massachusetts, bridge prior to dismantling and schematic member locations: (a) Shattuckville bridge prior to dismantling (photo by A. J. Lutenegger); (b) relative locations on a schematic truss bridge of the different members sampled for testing (note that this drawing is a composite of the bridges studied that is designed to show the location of all members tested)

by scaling up the dimensions of the plate coupon used for the lacing and then making several adjustments for the coupon to fit better in the testing machine. Both coupons were designed as plate-type specimens with a total length of 457 mm (18 in.), with a 127-mm (5-in.) reduced section length and a 16.5-mm (0.65-in.) thickness, whereas different reduced cross-section widths were used for the eye-bar and looped-bar specimens to account for the different cross sections of the original members. Specimens machined from the eye-bar had a 12.7-mm (1/2-in.) reduced section width, whereas the specimens from the looped bar had a 20.3-mm (8/10-in.) reduced section width.

Prior to tension testing, hardness tests were performed on each tensile specimen in the Rockwell B scale, following ASTM E18 (2003). For hardness testing, a 1.6-mm (1/16-in.) ball indenter was used, with a minor load of 10 kgf (4.5 lb) and a major load of 100 kgf (45 lb). Eight hardness readings were taken per specimen. First, three readings were taken in the reduced gauge section of the specimen, and two readings were taken on each side in the unreduced grip area. Next, the specimen was rotated 180° and two more readings were taken in the reduced section, and one more reading was taken in the grip area. Tension testing of round specimens from beam hangers and plate specimens from lacing members was performed in an Instron screw-driven testing machine. All tests were displacement controlled, with two load rates used: first, a rate of 0.127 mm/min (5/100 in./min) was used until a stress of 138 MPa (20 ksi) was reached; the rate was then increased to 1.27 mm/min (50/100 in./min) until failure. Axial strains were measured using an extensometer with a 50.8-mm (2-in.) gauge length; the extensometer was removed during testing at a stress value of 138 MPa (20 ksi), since it was used solely to measure elastic strains so that elastic modulus could be estimated. Testing of the oversized plate specimens from the eye-bar and looped bars, specimen groups SV-E and SV-B, was conducted using a Tinius Olsen 2,200 kN (500 kip) hydraulic testing machine. Tests were displacement controlled, and a single load rate of 2.54 mm/min (1/10 in./min) was used throughout the duration of the test. Again strains were measured through the use of a 50.8-mm (2-in.) gauge length extensometer, which was removed during testing once a stress value of 138 MPa (20 ksi) was reached. For all tests, yield stress was determined using the 0.2% offset method, whereas tensile strength was taken as the greatest stress applied to the specimen before failure.

Ductility for each specimen was measured by percent reduction in cross-sectional area. Reduction in area was calculated by measuring the final cross-sectional dimensions at the fracture of each coupon after testing. Measurements were taken on each piece of the broken coupon and an average value was used to determine the final area.

Results

Test results for each specimen are summarized in Table 2, which lists measured values of hardness, percent reduction in area, and tensile strength for each specimen. Stress-strain plots for each specimen were similar to those of a typical ductile metal. First, there was an elastic region ending at an abrupt yield point, followed by plastic deformation with little increase in stress, then strain hardening, and finally necking and failure. The stress-strain curves of specimens SV-H8, SV-L-1, and GH-H-3 are shown in Fig. 2. Five specimens did not exhibit a distinct lower yield point; most of those specimens were from groups CB-L and SV-L.

Overall, it was found that the values of tensile strength reported in Table 2 were in the lower range of strength values reported in historical sources of wrought-iron testing. The average value of tensile strength for all tension tests on UMass bridge iron was found to be 335 MPa (48.7 ksi), with values ranging from 273 to 399 MPa

Table 2. Testing Results for Each Specimen

Specimen	Member thickness (mm)	(in.)	Reduction in area (%)	Avg. hardness	Hardness standard deviation	Yield stress (MPa)	(ksi)	Tensile strength (MPa)	(ksi)
BV-H-1	38.1	1 1/2	48	52	1	154	22.4	273	39.6
BV-H-2	38.1	1 1/2	36	55	4	184	26.7	315	45.7
BV-H-3	38.1	1 1/2	39	55	4	199	28.9	331	48.1
BV-H-4	38.1	1 1/2	46	58	9	206	29.8	334	48.5
BV-H-5	38.1	1 1/2	42	58	7	230	33.3	331	48.0
BV-H-6	38.1	1 1/2	42	54	5	188	27.3	319	46.2
BV-H-7	38.1	1 1/2	36	52	8	187	27.2	316	45.8
BV-H-8	38.1	1 1/2	42	59	9	212	30.8	329	47.7
RB-H-1	38.1	1 1/2	22	49	3	200	29.0	316	45.8
RB-H-2	38.1	1 1/2	23	53	8	208	30.1	320	46.4
RB-H-3	38.1	1 1/2	43	55	7	208	30.2	318	46.1
RB-H-4	38.1	1 1/2	41	48	6	187	27.1	305	44.2
RB-H-5	38.1	1 1/2	43	54	3	196	28.5	314	45.5
RB-H-6	38.1	1 1/2	34	59	5	218	31.6	341	49.4
RB-H-7	38.1	1 1/2	43	62	6	223	32.4	335	48.6
GH-H-1	28.6	1 1/8	50	56	5	242	35.1	341	49.4
GH-H-2	28.6	1 1/8	53	56	4	245	35.5	346	50.1
GH-H-3	28.6	1 1/8	47	57	6	228	33.0	339	49.2
GH-H-4	28.6	1 1/8	47	61	3	239	34.6	345	50.0
GA-H-1	28.6	1 1/8	50	55	6	247	35.9	354	51.4
GA-H-2	28.6	1 1/8	53	55	13	224	32.5	347	50.4
GA-H-3	28.6	1 1/8	47	59	5	253	36.7	356	51.6
GA-H-4	28.6	1 1/8	47	60	4	219	31.7	351	50.9
CB-H-1	44.5	1 3/4	33	62	3	189	27.4	328	47.6
CB-H-2	44.5	1 3/4	41	61	9	185	26.9	330	47.8
CB-H-3	44.5	1 3/4	40	59	6	189	27.4	331	48.0
CB-H-4	44.5	1 3/4	40	63	5	189	27.4	329	47.7
CB-L-1	6.4	1/4	3	72	4	295	42.8	361	52.3
CB-L-2	6.4	1/4	12	78	3	312	45.2	383	55.5
CB-L-3	6.4	1/4	9	82	2	320	46.5	390	56.5
CB-L-4	6.4	1/4	8	76	2	297	43.1	356	51.6
CB-L-5	6.4	1/4	6	79	4	317	46.0	365	52.9
SV-H-1	25.4	1	46	59	9	234	33.9	323	46.9
SV-H-2	25.4	1	46	56	12	247	35.8	334	48.4
SV-H-3	25.4	1	43	46	2	204	29.5	300	43.5
SV-H-4	25.4	1	47	47	6	211	30.6	309	44.8
SV-H-5	25.4	1	49	51	3	219	31.7	322	46.7
SV-H-6	25.4	1	53	53	10	221	32.1	322	46.8
SV-H-7	25.4	1	48	46	6	210	30.5	313	45.4
SV-H-8	25.4	1	50	49	5	218	31.6	316	45.9
SV-L-1	6.4	1/4	26	69	3	298	43.3	381	55.3
SV-L-2	6.4	1/4	25	67	2	305	44.3	399	57.9
SV-L-3	6.4	1/4	32	72	4	294	42.6	380	55.1
SV-L-4	6.4	1/4	17	67	1	268	38.9	360	52.1
SV-E-1	22.2	7/8	46	59	5	232	33.7	334	48.4
SV-E-2	22.2	7/8	42	58	6	235	34.1	340	49.3
SV-E-3	22.2	7/8	44	56	7	243	35.3	337	48.9
SV-E-4	22.2	7/8	46	52	6	214	31.1	325	47.2
SV-B-1	22.2	7/8	41	58	4	236	34.3	339	49.2
SV-B-2	22.2	7/8	40	57	5	243	35.3	341	49.4
SV-B-3	22.2	7/8	37	55	6	220	31.9	328	47.6
SV-B-4	22.2	7/8	41	53	2	227	32.9	330	47.8
SV-B-5	22.2	7/8	43	55	6	226	32.8	323	46.8

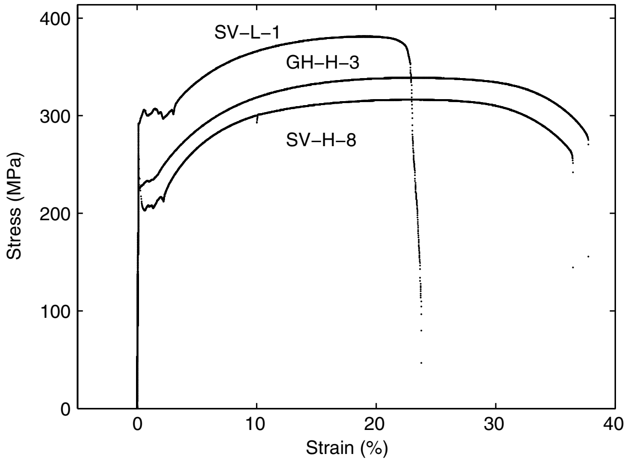

Fig. 2. Stress-strain curves of specimens SV-H-8, SV-L-1, and GH-H-3 (1 MPa = 0.145 ksi)

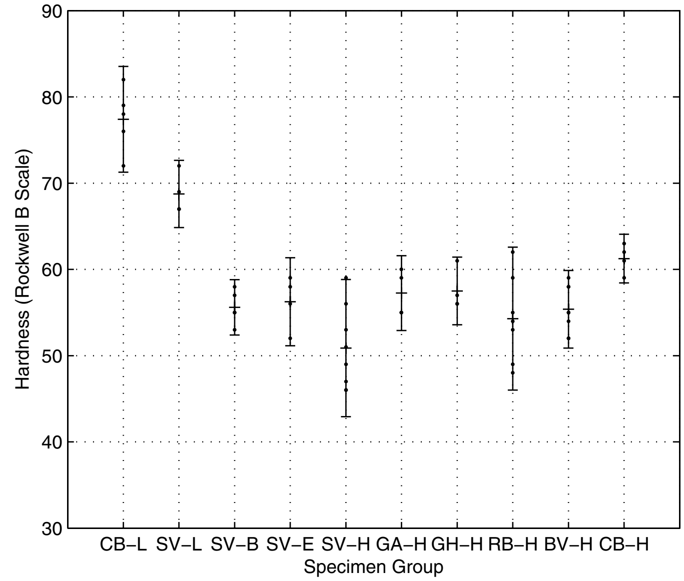

Fig. 3. Individual hardness values for each specimen group plotted with error bars

(39.6 to 57.9 ksi) and a standard deviation of 23 MPa (3.4 ksi), giving a coefficient of variation of 7%. American wrought-iron bars tested by Beardslee (1879) had an average strength of 364 MPa (52.9 ksi), with values ranging from 317 to 427 MPa (46 to 62 ksi). Tests conducted by Kirkaldy (1862) on British wrought-iron bars, plates, and angle irons had an average tensile strength of 358 MPa (52 ksi) and ranged from 255 to 468 MPa (37 to 68 ksi). Finally, *Johnson's Materials of Construction* (Johnson 1939) reports a range of 327 to 358 MPa (47.5 to 52 ksi) for the tensile strength of wrought iron.

Discussion

Hardness

Hardness testing is an easy-to-execute nondestructive test that can be performed in the field using portable hardness testers, and in some homogeneous metals, such as steel, there is a reasonable correlation between hardness and strength (Gordon and Knopf 2005). The ability to use hardness as an indicator of mechanical properties, such as strength or ductility, in wrought iron would be useful in the field assessment of historic wrought-iron bridges.

As stated previously, eight hardness readings were taken per specimen, and hardness readings displayed a significant amount of variation for many specimens; values of standard deviation and average hardness for each specimen are listed in Table 2. This variability can be attributed to the large length scale associated with heterogeneity in the iron compared with the amount of iron deformed by the indenter ball during hardness testing (Gordon and Knopf 2005). Fig. 3 contains individual values of hardness in the Rockwell B scale for each specimen group plotted with error bars for a 95% level of confidence; specimen groups are arranged in order of decreasing original member thickness.

Although values of hardness generally fell between 45 and 65 Rockwell hardness B, in Fig. 3 it can be seen that hardness readings from groups CB-L and SV-L were higher than all other groups. A Kolmogorov-Smirnov test (K-S test) performed on the hardness data using a confidence level of 95% found that the distributions of hardness values for the groups CB-L and SV-L were statistically different than those of all other groups. The K-S test also showed that hardness values from other specimen groups were not different from each other at a 95% confidence level, with the exception of group SV-H. This test was selected to compare the strength and yield values since it tests difference in distribution, rather than simply difference in either mean or variance. The K-S test is therefore able to detect any statistically significant difference between the distributions of strength and stiffness in different bridge components.

Although there is often a good correlation between hardness and strength in homogenous metals, this relationship does not necessarily hold true in wrought iron because of the scale of the heterogeneity in the iron (Gordon 1998; Gordon and Knopf 2005; Sparks 2007). Values of yield strength and tensile strength for all wrought-iron specimens are plotted against hardness in Fig. 4.

A weak linear relationship between hardness and yield strength can be observed in Fig. 4. Although the scatter appears large, the correlation coefficient, provided in Table 3, indicates a reasonably strong correlation between these two properties for the samples tested. It can be observed that the specimens with the greatest yield strength and hardness values were all from the groups SV-L and CB-L, and removing these groups from the data sample would greatly reduce the strength of the correlation between yield strength

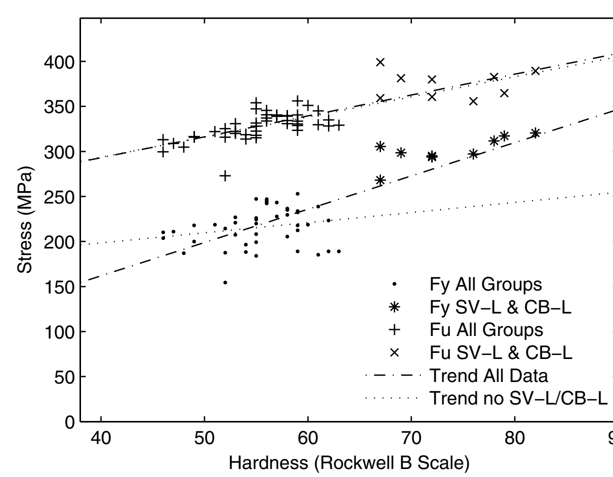

Fig. 4. Yield stress and tensile strength plotted against hardness with trend lines (1 MPa = 0.145 ksi)

Table 3. Results from the Regression Analysis Depicted in Fig. 4, Where the Trend Line Equation Is $F_{u/y} = a \times \text{hardness} + b$

	a (Mpa, ksi)	b (Mpa, ksi)	r^2	ρ correlation coefficient
F_y all	3.7, 0.54	14, 2.0	0.62	0.79
F_y no SV-L/CB-L	1.1, 0.16	150, 22	0.05	0.22
F_u all	2.3, 0.34	200, 29	0.66	0.81
F_u no SV-L/CB-L	2.2, 0.32	200, 29	0.39	0.62

and hardness. Removing any other specimen group from the data set did not have as large an effect on the overall relationship between hardness and yield strength as removing groups SV-L and CB-L did. The specimens from these two groups came from nonstructural lacing members.

Next, the relationship between hardness and tensile strength was investigated, as the data identified in Fig. 4 show a correlation between these two properties as well. The results from this regression analysis are also listed in Table 4. The effect of specimen groups SV-L and CB-L on this relationship was investigated, as these groups again had the highest values of strength and hardness. Without specimen groups SV-L and CB-L, the correlation coefficient and r^2 value decrease, although not as substantially as with yield stress. A smaller change in trend line slope and correlation coefficient for the reduced data indicates that the relationship between hardness and tensile strength is somewhat stronger and more robust, in terms of data included, than the relationship between hardness and yield strength.

Fig. 5 shows a weak inverse relationship between hardness and percent reduction in area. The correlation coefficient for these two properties was found to be −0.75, which suggests there is some correlation between ductility and hardness. A trend line fitted to these data, plotted in Fig. 5, was found to have a slope of −1.14 (nondimensional) and an r^2 value of 0.56. Similar to the trend observed with strength and hardness, specimen groups SV-L and CB-L had the highest values of hardness and lowest values of ductility among all specimens. These two groups were removed from the set and the correlation coefficient was recalculated to be −0.03, which indicates that there is little correlation between these two properties. A trend line plotted to the remaining data was found to have a slope of −0.05 (nondimensional) and an r^2 value of 0.001.

Sparks (2008) has suggested that Brinell hardness number (BHN), in addition to chemical and microstructural analysis, be used as a field screen for low ductility in wrought iron and recommends that BHN > 130 (Rockwell $B > 72$) should trigger additional evaluation of the material for the possibility of low ductility as indicated by percentage reduction in area of less than 25%. The results shown in Fig. 5 are in good agreement with Sparks' recommendation, with the current data indicating a screening threshold of Rockwell $B > 65$ (BHN > 116) to detect material samples in which the percent reduction in area is less than 25%, although Sparks (2007, 2008) reports BHN = 116 as being well within the normal range of wrought-iron hardness.

Tension

Individual values of yield stress and tensile strength for each specimen group are plotted in Fig. 6, in which specimen groups are arranged in order of decreasing original member thickness. As previously noted, wrought iron is known for its variability in strength, which is illustrated in Fig. 6. Overall, the coefficient of

Table 4. Results from the K-S Test on Yield and Tensile Strength Data

Yield strength										
Group	CB-L	SV-L	SV-B	SV-E	SV-H	GA-H	GH-H	RB-H	BV-H	CB-H
CB-L	0	0	x	x	x	x	x	x	x	x
SV-L	0	0	x	x	x	x	x	x	x	x
SV-B	x	x	0	0	0	0	0	x	x	x
SV-E	x	x	0	0	0	0	0	0	x	x
SV-H	x	x	0	0	0	0	x	0	x	x
GA-H	x	x	0	0	0	0	0	x	x	x
GH-H	x	x	0	0	x	0	0	x	x	x
RB-H	x	x	x	0	0	x	x	0	0	x
BV-H	x	x	x	x	x	x	x	0	0	0
CB-H	x	x	x	x	x	x	x	x	0	0
Tensile strength										
Group	CB-L	SV-L	SV-B	SV-E	SV-H	GA-H	GH-H	RB-H	BV-H	CB-H
CB-L	0	0	x	x	x	0	x	x	x	x
SV-L	0	0	x	x	x	x	x	x	x	x
SV-B	x	x	0	0	x	x	0	0	0	0
SV-E	x	x	0	0	x	x	0	0	0	0
SV-H	x	x	x	x	0	x	x	0	0	x
GA-H	0	x	x	x	x	0	x	x	x	x
GH-H	x	x	0	0	x	x	0	x	x	x
RB-H	x	x	0	0	0	x	x	0	0	0
BV-H	x	x	0	0	0	x	x	0	0	0
CB-H	x	x	0	0	x	x	x	0	0	0

Note: A "0" signifies that the data from the two specimen groups have the same distribution, whereas an "x" signifies the two groups have different distributions with a 95% confidence.

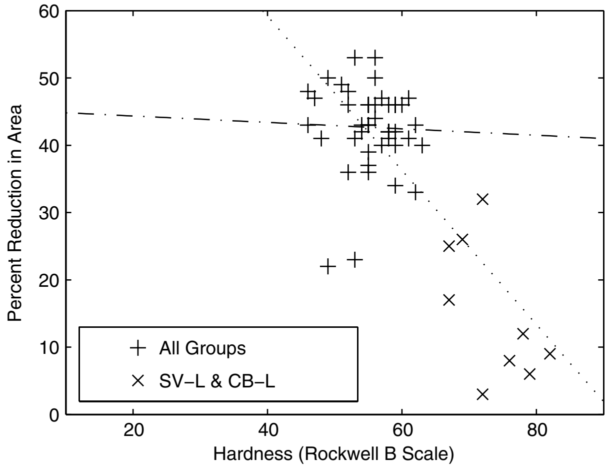

Fig. 5. Percent reduction in area plotted against hardness for each specimen with trend lines

variation (COV) for tensile strength was found to be 6.9%, whereas a COV of 16.5% was found for yield strength. For comparison, Bartlett et al. (2003) reported a COV of 5.6% for testing of A992 steel coupons sampled from wide flange shapes.

Wrought-iron beam hangers were tested from six bridges and are identified in Fig. 6 as the specimen groups ending with the letter H. The COV for yield strength for beam hangers was found to be 10.5%, whereas the COV for tensile strength was 5%. A K-S test was performed, using a 95% level of confidence, to compare the distributions of strength for each group of beam hangers. The results from the K-S test are reported in Table 4.

From Table 4, it can be seen that the different specimen groups have varying strength distributions, and although some groups have comparable distributions for yield strength, they may not for tensile strength. These results show that wrought-iron strength can vary from bridge to bridge, and that yield strength displays more variability than tensile strength.

Iron specimens taken from different elements on the Shattuckville bridge were found to have more variation than iron from the beam hangers of the six bridges. For all specimens from the Shattuckville bridge, COVs of 12.5 and 7.25% were measured for yield strength and tensile strength, respectively. Specimens tested from the lacing members (group SV-L) were found to have a statistically significant higher yield and tensile strength than specimens from other elements, as confirmed by the K-S test. Not including strength data from group SV-L, the COV of yield strength would be reduced to 5.4% and that of tensile strength to 3.4%. This shows that there can be a significant amount of variation in strength of different elements from a single bridge.

Size Effect

Several sources have observed a size effect, or rolling effect, in wrought-iron strength where samples from sections of lesser thickness have been found to have greater tensile strength than those from sections of greater thickness. This effect was observed by Beardslee (1879) in the testing of American wrought-iron bars of different sizes and by Kirkaldy (1862) in testing British iron. *Johnson's Materials of Construction* also makes mention of a size effect and suggests that the increase in strength is attributable to a greater amount of hot work required to make smaller sections, resulting in an increase in density and cohesion between the ferrite grains. Johnson also mentions that rolling/section thickness has a more pronounced effect on yield strength than on tensile strength. In contrast, modern structural steels have quite consistent mechanical properties over the range of sizes used in typical constructions. This consistency breaks down in the case of very thin plates and wires and plates several inches thick, which may have microstructural features or distributions of residual stresses that either increase (thin plates/wires) or decrease (thick plates) the strength or ductility. Even in these cases, however, the mechanism of the size effect is quite different from that in wrought iron. In wrought iron, increased strength at small sizes comes from a more even distribution of the slag phase caused by increased working of the material. In structural steel, whatever size effect is present is caused by changes to the steel grain structure caused by hot or cold work, differential cooling rates, or the presence of residual stresses caused by differential cooling of the material.

Values of yield stress and tensile strength for each specimen are plotted against original member thickness in Fig. 7, which shows that strength decreases as section thickness increases. The correlation coefficient for tensile strength and thickness is -0.69; a trend line fitted to these data was found to have a slope of -1.4 MPa/mm (-5.2 ksi/in.) and an r^2 value of 0.47. In Fig. 7,

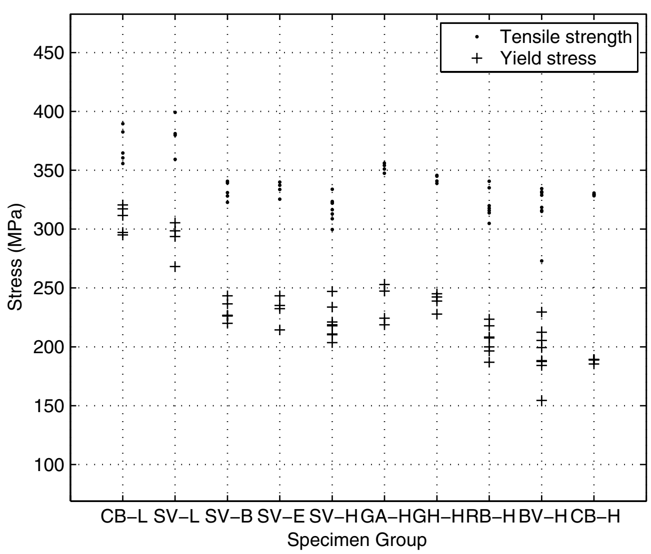

Fig. 6. Values of yield stress and tensile strength for each specimen arranged by group

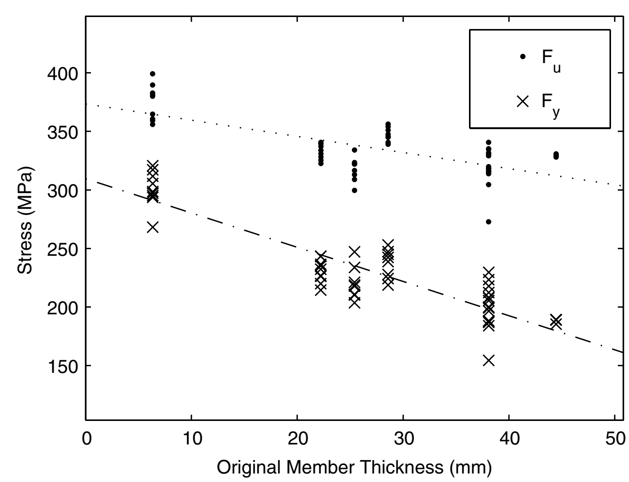

Fig. 7. Values of yield stress and tensile strength plotted against thickness of original member (1 MPa = 0.145 ksi and 1 mm = 0.04 in.)

the relationship between strength and thickness appears more pronounced in yield stress than in tensile strength. This was verified numerically with a correlation coefficient of −0.89 for yield stress and thickness. A trend line fitted to these data had a slope of −2.9 MPa/mm (11 ksi/in.) and an r^2 value of 0.79. Not only does the higher r^2 value indicate that thickness is a more reliable predictor of yield stress than of tensile strength, but the difference in slope between the best fit lines to the yield stress and tensile strength data indicate that the ratio of tensile strength to yield stress changes with specimen thickness. The reserve strength beyond yield is greater for larger specimens, although the strength is lower. Next, a K-S test was used to examine the distributions of strength for each group of specimens. The correlation coefficients between specimen size and yield stress and tensile strength are high, yet there remains substantial scatter in the yield stress and tensile strength at a given specimen size. Much of this uncertainty is attributable to the specimens originating in different bridges. If all specimens were taken from the same bridge, the scatter would be expected to be small. This is evident in the SV- samples in Fig. 6, in which there is no overlap between the range of yield stress for the smallest (SV-L) and largest (SV-H) specimens. The same is true of the tensile strength for these specimens.

Results from this analysis on yield strength are shown in Table 4, where specimen groups are arranged by size and are listed in order of increasing thickness. Table 4 shows that there is little statistical difference in strength between groups of the same thickness. The boxes of zeros clustered around the diagonal indicate that the likelihood of strengths being different is greater when the size difference is greater. The specimen groups with the thinnest original cross section, CB-L and SV-L, both had an original thickness of 6.4 mm (1/4 in.) and had the greatest average yield strength. From Table 4, it can be seen that both of these groups had statistically significant difference in distribution of strength from the other specimen groups tested. The thickest elements tested were from group CB-H, with an original thickness of 44.4 mm (1 3/4 in.); these specimens had the lowest average yield strength measured and had different strength distributions than the other groups, with the exception of group BV. Groups SV-B, SV-E, GA-H, and GH-H did not exhibit a statistically significant difference in distribution of strength. The source material for these groups consisted of square and rectangular bars ranging in thickness from 22.2 mm (7/8 in.) for groups SV-B and SV-E to 28.5 mm (1 1/8 in.) for groups GA-H and GH-H.

Conclusions

A program of material testing was conducted on late 19th-century wrought iron salvaged from six historic truss bridges built and originally used in the New England states. The testing program gathered yield stress, tensile strength, ductility, and hardness for material taken from beam hangers from each of the six bridges and from three other member types for one of the bridges, and one other member type for one of the bridges. This experimental design allowed variability of properties across bridges within a single member type (beam hanger) and across member types within a single bridge to be determined. The data also allowed investigation of size effect in the yield stress and tensile strength of historic wrought iron and correlation between hardness and yield stress and tensile strength. The main conclusions of the study are: (1) a weak correlation exists between hardness and strength/ductility measures, but the samples with very high hardness, greater than about 65 Rockwell B, exhibited substantially lower ductility; (2) a size effect in which larger members have lower yield stress and tensile strength was observed, and this effect was stronger for yield stress than for tensile strength; and (3) property variation was, in general, smaller for different members within a bridge than for a single member type across bridges, with the exception of the somewhat anomalous nonstructural lacing members, which had very different properties from those of beam hangers, eye-bars, or looped-bars. These conclusions allow some qualitative recommendations for engineers working on the preservation of historic iron truss bridges. Nondestructive testing such as hardness testing is not, in itself, sufficient to predict the yield stress or tensile strength of historic wrought iron. Nevertheless, hardness may be useful as a field screening test to identify material with very low ductility. For example, a Rockwell B hardness greater than 65 is a very strong indicator of percent area reduction less than 30% (Fig. 6).

Average yield stress and tensile strength varies substantially from bridge to bridge, so it is not advisable to use tests on iron from one bridge to predict the strength of iron in another bridge. Average yield stress and tensile strength, however, appear to be quite consistent across structural member types within a bridge, excluding nonstructural lacing members, so that it may be possible to avoid destructive testing on some of the more visible structural members, confining testing to less visible and more easily replaceable members. If this is done, the engineer should take account of the effect of member size on yield stress and tensile strength.

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