Fire and Reconstruction at Lobato Bridge in New Mexico

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The oldest bridge in New Mexico was devastated by fire, leaving an operating railroad with difficult choices for balancing its reconstruction with historic preservation.

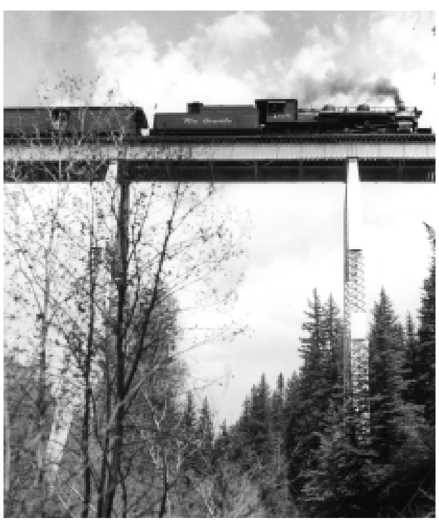

Fig. 1. K-36 Class engine #488 crossing Lobato Bridge with passenger train. View from below prior to the fire. Photographer and date unknown. From Richard L. Dorman Collection, Catalog No. RD012-122, of Historic Narrow Gauge Railroad Photographs located at the Friends of the Cumbres and Toltec Scenic Railroad Library in Albuquerque, N.M., and Historic American Engineering Record, HAER No. NM-16, Lobato Bridge, Wolf Creek.

Introduction

Fire broke out at the Lobato Bridge, a vital feature of the Cumbres and Toltec Scenic Railroad (CTSRR), a National Historic Landmark, on the night of June 23, 2010.[1] By morning it was clear that the bridge had been severely damaged and that operations of historic excursion trains would be interrupted, for how long no one knew.[2] The cause was not clear — the railroad had operated over 120 years without this kind of disaster.

The Railroad, Past and Present

The Lobato Bridge, originally known as the Wolf Creek Trestle, is located on the San Juan Extension of the Denver and Rio Grande Railroad (D&RG).

Gen. William Jackson Palmer founded the Denver and Rio Grande Railway Company in 1870. The original plan for the railroad was to build a line from Denver, Colorado, to El Paso, Texas, a distance of 870 miles. A narrow-gauge system with a 3-foot width was selected, based on reduced cost compared with larger gauges and more favorable operations in mountainous terrain.[3] The 3-foot narrow gauge was more nimble, allowing trains to execute tighter turns and climb steeper grades; it could also accommodate smaller locomotives and rolling stock.

The advent of the San Juan Extension is based on several important occurrences: the Brunot Treaty and subsequent treaties and agreements with the Ute Indians, which led to opening of the San Juan region to mining and agriculture; and the railroad wars between D&RG and the Atchison, Topeka and Santa Fe Railway that culminated in the Treaty of Boston, which restricted the D&RG development in New Mexico. Both of these occurrences ultimately resulted in the D&RG abandoning the goal of building south to El Paso and focusing instead on building west to the mining camps of Leadville, Colorado, and the San Juan Mountains of southwest Colorado.[4]

In 1879 reconnaissance surveying was conducted, and contracts were let for the construction of the San Juan Extension. By April 1880 service was extended from Alamosa, Colorado, to Antonito, Colorado, and by February 1, 1881, service was extended from Antonito over Cumbres Pass to Chama, New Mexico. Building the section of line between Antonito and Chama was a formidable task that included constructing two large bridges and two tunnels, dealing with grades exceeding four percent, maneuvering across the sides of steep canyons, and crossing Cumbres Pass at an elevation of more than 10,000 feet. The line was completed to Durango, Colorado, by July 1881 and to Silverton, Colorado, in July 1882.[5]

The two large bridges of the Cumbres and Toltec Scenic Railroad are the Cascade Bridge, with a span of more than 400 feet at a height of more than 130 feet, and the Lobato Bridge, with a span of more than 300 feet at a height of more than 100 feet. Originally, both bridges were constructed of wood. Noted nineteenth-century engineer C. Shaler Smith was retained by the D&RG to design both bridges. The earliest date of Smith's design on surviving drawings for the Lobato Bridge is November 1880, although records indicate the crossing was built in wrought iron in 1883.[6]

The San Juan Extension between Alamosa and Silverton continued in operation into the mid-1960s. In 1967 the Denver and Rio Grande Western Railroad (D&RGW), successor to the D&RG, applied to the Interstate Com-

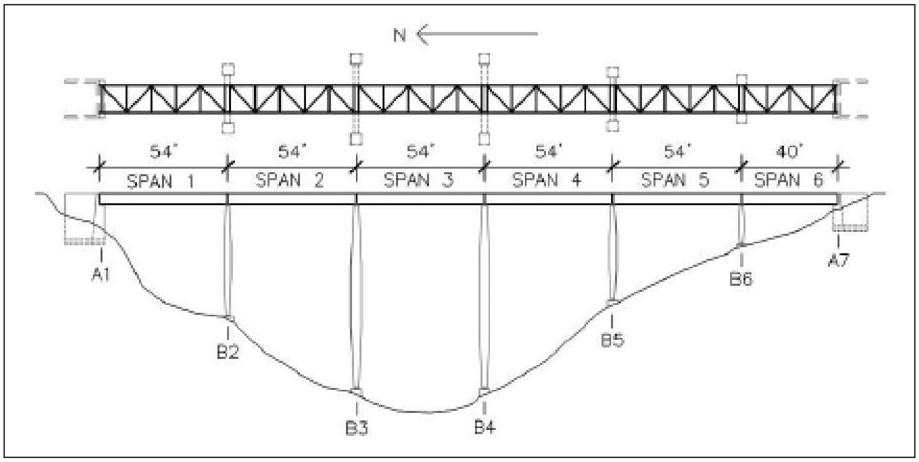

Fig. 2. Bridge arrangement, Lobato Bridge. Sketch by J. R. Harris & Co., Denver.

merce Commission to cease operation on the railroad, except for the Silverton Branch that runs from Durango to Silverton. Operations ceased, and the line was abandoned from Antonito to Durango on December 29, 1969.[7]

On July 1, 1970, the states of Colorado and New Mexico purchased the line and its associated right-of-way, rolling stock, and real property from the D&RGW. Currently the line between Antonito and Chama is operated as the Cumbres and Toltec Scenic Railroad based on the Cumbres and Toltec Scenic Railroad Compact that was authorized by an act of Congress in 1974 and is still jointly owned by the states of Colorado and New Mexico.[8] The Cumbres and Toltec Scenic Railroad Commission manages the railroad. The commission is comprised of four commissioners, two from each state. The governors of each state appoint the commissioners.

The Bridge

Lobato Bridge was fabricated by the Andrew Carnegie's prolific Keystone Bridge Company, headquartered in Pittsburgh. The D&RG installed the bridge over Wolf Creek a few miles north of Chama, as part of their line across the mountains of northern New Mexico and southern Colorado (Figs. 1 and 2).

The six spans — riveted plate girders, five measuring 54 feet and one measuring 40 feet totaling 310 feet in length — were supported on stone-masonry abutments and iron bents as much as 100 feet above the ground. The built-up plate girders, which were 49 1/2 inches deep, were fabricated from metal plate with continuous angle connections riveted between the web and flange plates. Cover plates were used in the mid-span portion of each girder, and X-bracing served as cross bracing between the girders, located 10 feet apart. Additionally, lateral bracing of steel angles provided support of the top compression flange. The ties served a two purposes: to support rails, as with all railroads, and, at 15 inches deep, to span the 10 feet between the parallel girders. Post-fire examination revealed that the rails were offset 2 1/2 inches from the centerline between the girders, resulting in slightly greater loading applied to one girder.

The iron bents, also referred to as trestles, columns, or piers, were fabricated from built-up columns of rolled channels, connected with lacing bars and batten plates, with two such columns cross-connected with transverse struts spaced vertically every 30 feet. The bents are not cross-connected to each other; rather, the girders of the bridge provide all the longitudinal support for the structure. At each interior bent and at each column, the two adjacent girders use one bearing assembly, which rests on a pin. The pin allows for the rotation effects caused by the deflections of the girders. The girders have continuity plates at their top flanges that connect each span to the next. Thus, the anchor bearing at the south abutment offers the only longitudinal restraint.

Some drawings of the original 1881 design have been preserved and were made available. These proved valuable in evaluating the post-fire condition and reconstruction decisions.

The Fire

Nearly all of the timber ties were consumed in the fire, which created great heat at the top of the structure. The rails buckled. The upper flanges of the girders became distorted, buckled, and cracked in numerous places. The upper parts of the girders were raised to higher temperatures than their lower parts, evidenced by discoloration of the paint on the heated surfaces. It is believed that the lower portions of the girders, which were not distressed, and the supporting bents below, sustained no permanent damage. However, the upper portions of the plate girders, the X-bracing, and the top-flange lateral bracing all sustained significant distress (Figs. 3 and 4).

At the north abutment, the bearing separated, and the girder was observed to be about 7 inches above the masonry plate. The top of that girder as it cooled after the fire contracted sufficiently to develop a negative camber, raising it off of its bearing. That meant the girder was now cantilevered from its interior support, 54 feet away, significantly stressing its continuity plate in tension.

The east girder of each span suffered more damage than the west girder, suggesting that prevailing winds from the west may have concentrated more heat on the east side. Spans 1 through 4 suffered irreparable damage; span 5 suffered damage to the top lateral bracing but was considered repairable; and span 6 had no apparent damage. At bents number 2 and 3, the bolts that connect the girders to the bearings were either sheared off or distorted.

Having lost the service of Lobato Bridge, CTSRR was forced to temporarily cease operations from Cumbres Pass to Chama. Service from Antonito to Cumbres Pass was continued. As the bridge is located only three miles north of the Chama terminal, buses were used to transport passengers from the terminal at Chama to Cumbres Pass. Still, publicity affected tourist numbers; business and revenues decreased at a

Fig. 3. Fire damage at a girder. Distortion at the upper portions of the girder flange can be seen. Paint has been burned off the upper flange and has been discolored at the upper part of the web. Photograph by Roger Hogan for CTSRR.

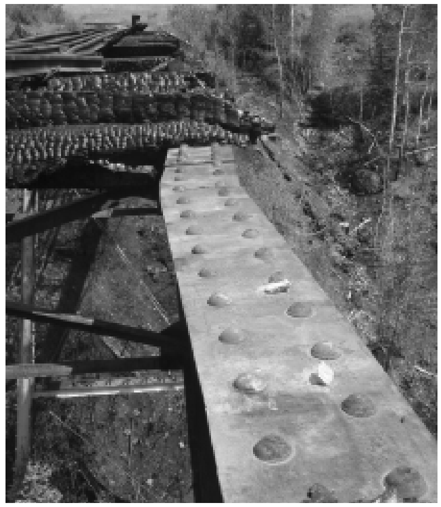

Fig. 4. Fire damage. The riveted plate girder is distorted, ties are burned, and track displaced. Photograph by Roger Hogan for CTSRR.

time when significant extra capital was needed, as insurance did not fully cover the necessary bridge repairs.

Studies

An initial inspection and assessment resulted in a recommendation for total replacement of Lobato Bridge. However, the railroad called in specialists to review the conditions and appraise the damage, in part because of the historic nature of the entire railroad. CTSRR posed the following questions:

1. Could the damaged metal be repaired in place?
2. If not, could replacement be limited to damaged members only?
3. If total replacement were necessary, should the historic appearance be retained, or should a modern railroad bridge be constructed?
4. If repaired in place, could the repaired structure meet modern structural requirements of American Railway Engineering and Maintenance-of-Way Association (AREMA)?[9] Further, could it meet the requirements of the Federal Railroad Administration (FRA)?[10]

The choices had to be weighed against the very real concerns regarding reestablishment of service in a timely fashion; responsibilities to the joint owners of the railroad, the states of Colorado and New Mexico; the historic nature of the railroad itself; and limited finances. While this paper primarily addresses technical issues, these administrative concerns weighed heavily on the management.

Thorough visual inspection by specialists in engineering railroad bridges was initiated. Material samples were obtained, initially from the web of span 6, where the heat from the fire was presumed to be minimal, and from a lacing bar almost certainly unaffected by heat. Later specimens came from areas susceptible to heat, with those samples from the flanges of spans 1 and 2, directly below the burned ties, which were clearly affected by heat.

Chemical tests revealed that the 1883 metal had a relatively low carbon content but relatively high silicon content, typical of wrought iron, which was commonly used in this period for railroad bridges.[11] The results shown in Table 1 are not a complete metallurgical listing, as other elements were found in very small to trace quantities, but the table does report on the elements discussed in this paper.

Physical properties were also determined by tensile testing (Table 2). As shown in Table 2, samples 1 and 2 were taken from portions relatively unaffected by heat from the fire. Samples 3 and 4, taken from webs of girders, presumably were moderately exposed to heat. Samples 5 and 6, from upper flanges, were greatly affected by heat. Elongation, an indicator of ductility, was measured across gauge marks 2 inches apart on the tensile specimens. Because the bridge was so seriously damaged and not in service, no measures were taken to compensate for the holes created where samples had been removed.

Bridge Analysis and Rating

A rating analysis was performed to determine the structural capacity of the spans and bents, using the as-built plans, field measurements and section properties, and a material yield strength of F_y = 29,400 psi for wrought-iron members. The rating analysis was performed in accordance with Chapter 15, Steel Structures; Part 7, Existing Bridges, of the 2010 edition of the AREMA *Manual for Railway Engineering*.

Information provided by CTSRR indicated that two trains per day, one in each direction, pass across the bridge at a maximum speed of 8 mph. The train consists of a K36 or K37 coal-fired steam-engine locomotive; a tender; and eight viewing cars with a maximum capacity of 44 passengers per car. Rating calculations for normal and maximum loads under pre-fire, as-built conditions were completed. The normal ratings are for loads that can be carried by the structure for its expected service life at a

Table 1. Chemical properties from original metal

Sample No.	Location	Carbon (%)	Silicon (%)	Phosphorus (%)
1	Web, span 6	0.02	0.17	0.19
2	Lacing bar, pier 6	0.01	0.22	0.11
3	Web, span 1	<0.01	0.06	0.20
4	Web, span 2	<0.01	0.21	0.21
5	Flange, span 1	0.01	0.22	0.14
6	Flange, span 2	0.01	0.28	0.18

The list is not complete as other elements were found in very small to trace quantities, but these are the constituents discussed in the paper.

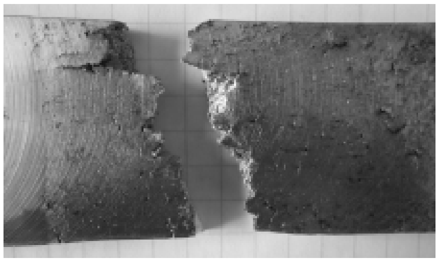

Fig. 5. Specimen, lacing bar after testing in tension failure. Photograph by the author.

standard speed. The intent of a normal rating is to limit the stresses in the structure to those that it would have typically been designed for and to determine the Cooper's equivalent load that it could carry on a daily basis while providing a consistent factor of safety.[12] Maximum ratings are for loads that can be carried at infrequent intervals. The maximum rating provides a reduced factor of safety and, if more frequent maximum loads are allowed, a reduced structure life. Results of the rating analysis determined that the bridge in its pre-fire condition was overstressed from 20 to 30 percent when loaded by the current trains at the bridge spans and at two of the tallest bridge bents.

Material

Bridges of the early 1880s were typically fabricated from wrought iron, as bridge engineers were reluctant to specify the new material, steel, in their designs. Most railroad bridges built prior to 1890 are of wrought iron, while steel was phased in over a transition period from about 1890 to about 1894, after which steel was typically used.[13]

Wrought iron manufactured by the puddling process that replaced earlier production by bloom smelting or fining of pig iron tended to produce ferrous metal with a carbon content to concentrations less than 0.1 percent by weight.[14] Phosphorus was sometimes added to improve strength, although, except for wrought iron of very high purity, it would decrease ductility.[15]

Slag is a characteristic feature of wrought iron manufactured by the nineteenth-century puddling process, unlike modern wrought iron. Slag formed in small (mostly microscopic, but sometimes visible), often thread-like deposits oriented in the direction of rolling. While the ferrous component of wrought iron has a crystalline structure similar to that of steel, it is the slag that gives wrought iron its fibrous appearance (Fig. 5).[16] Slag may be present in concentrations of 1 to 3 percent by weight or more.[17] A typical chemical analysis of historic wrought iron with a slag content of 3 percent by weight was found to contain 0.15 percent silicon.[18] While the slag percentage was not specifically determined from the Lobato samples, the silicon contents were, and they compare well to the findings from typical historic wrought iron (Table 1). For the Lobato samples, the carbon, silicon, and phosphorus contents all indicate wrought iron.

Similarly, from physical testing at the University of Colorado Denver, a fibrous appearance is readily apparent along the fracture surfaces (Fig. 5). The dark spots are slag inclusions. Both the fibrous appearance on fracture surfaces and slag inclusions are characteristic features of historic wrought iron. The conclusion from the chemical tests, physical tests, and visible appearance of post-test fracture surfaces is that the material at Lobato Bridge is wrought iron.

Design Decisions

The initial physical test results from samples 1 and 2 suggested that repair in place might be possible (Table 2). This approach would have had the advantages of probable lowest cost and least removal of historic fabric; the disadvantages would have included a probable maximum engineering effort to design many specific details and probable inability to improve the overall structural strength. Later structural analysis revealed that the bridge was originally designed for narrow-gauge rolling stock that weighed less than the 1926 engines and tenders currently in use. Thus a need for strengthening the superstructure was identified.

While such strengthening might be done along with repairs in place, later physical analyses eventually eliminated this approach as an option. Samples 3 and 4 and particularly samples 5 and 6, taken from heat-affected regions, revealed higher yield strength than samples 1 and 2, but they also indicated drastically reduced ductility, which can be seen in Table 2 in the columns labeled "Elongation" and "F_u/F_y." (For comparison, the ratio F_u/F_y using minimum values for ASTM A36, a ductile structural steel, is 1.61.) These tests suggested that the metallurgical properties of material subject to the fire's heat had been affected, resulting in an increase in strength but a decrease in ductility. It was the reduced ductility that eliminated further consideration for repairs in place. Modern structural design methodologies, including the AREMA standard for railroad-bridge design, are based on the use of ductile metals because of the serious consequences of brittle failures. The loss of ductility in the primary structural members rendered them unsuitable for repair.

Thus, the decision was made to replace the superstructure (i.e., the girders and associated bracing that were affected by heat) but to allow the substructure (the support bents that were not affected by heat) to remain in place. Several minor repairs, as well as strengthening of the bent towers based on condition assessments and structural

Table 2. Physical properties from original metal

Sample No.	Location	Exposure to Heat	Yield Strength (psi)	Ultimate Strength (psi)	Elongation at gauge marks (%)	F_u/F_y
1	Web, span 6	low	29,400	45,100	40	1.53
2	Lacing bar, pier 6	nil	29,400	45,100	40	1.53
3	Web, span 1	moderate	37,000	49,600	18	1.34
4	Web, span 2	moderate	34,600	49,700	18	1.43
5	Flange, span 1	high	32,500	37,300	4	1.15
6	Flange, span 2	high	37,900	40,300	3.5	1.06

Samples 1 and 2 were taken from portions relatively unaffected by heat from the fire. Samples 3 and 4, from webs of girders, presumably were moderately exposed to heat. Samples 5 and 6, from upper flanges, were greatly affected by heat. Elongation, an indicator of ductility, was measured across gauge marks 2 inches apart on the tensile specimens. F_y = yield strength, F_u = ultimate strength, psi = pounds per square inch. The ratio F_u/F_y is an indication of ductility, with a high ratio indicative of ductile material and a low ratio indicative of brittle material.

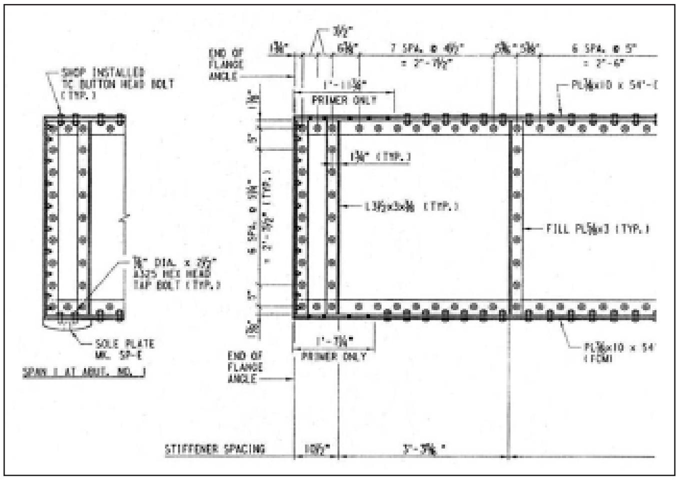

Fig. 6. Lobato Bridge, detail from the design drawings of the end of a girder. Efforts were made to replicate the historic appearance as much as possible within the confines of the project's design criteria. Compare the connections using tension controlled bolts to the original arrangement of rivets shown in Figure 3. Drawing by HDR.

Fig. 7. A temporary gantry was used by the contractor for removing girder segments and installing new girder segments. The gantry rolls on a temporary beam that also served to provide longitudinal bracing for the structure during construction. Photograph by Roger Hogan for CTSRR.

analyses, were also made to the bents. Because structural wrought iron is no longer manufactured and because modern bridge designers are familiar with structural carbon steel, Grade 50 (minimum yield strength of 50,000 psi) structural carbon steel was the final choice for the girders and their associated elements. In the analytical modeling, each member material — either iron or steel — was assigned its appropriate modulus of elasticity and yield strength.

The original 1883 structure had been connected with rivets (Fig. 4). While the eventual decision was to utilize high-strength bolts, the railroad management gave consideration to using rivets to help retain the historic appearance.[19] A search was undertaken to find fabricators with capabilities in rivets, and six such fabricators were located. Unfortunately, none possessed American Institute of Steel Construction (AISC) certification for bridges, while fabricators of structural steel did. Use of tension-controlled bolts (TCBs) was proposed in a layout to match the historic rivet patterns. While not rivets, TCBs do have a rounded head on one side, suggestive of rivets from a distance. The heads were placed on the viewable side of the girders: compare the design drawing of the girder ends per Figure 6 with the rivet patterns in Figure 3.

The structural design was prepared in accordance with AREMA *Manual for Railway Engineering*. The plate girders of all six spans were to be removed and replaced with new plate girders of structural steel, connected with TCBs. Repairs were made to the supporting bents, and some miscellaneous repairs were made at base plates on the original stone piers and footings.

Construction

While the investigations, planning, and design took the better part of a year, construction was able to be completed in approximately one month in the spring of 2011. A fundamental technique designed by the contractor was a gantry supported on temporary beams that were connected to the original support bents (Fig. 7). The gantry could be rolled over all spans and was used to lift and deliver the damaged girder segments to one end, where they were lifted by a crane for transport off site. Likewise, the new girder segments were transported across the bridge to their desired locations via the gantry. The temporary beams also provided lateral support for the overall structure in the longitudinal direction while the girders were not in place. The girders were fabricated off site and assembled into segments of pairs of girders with associated bracing. The segments were transported to the bridge site for installation. New ties and new rails were installed. The new ties replicated the dimensions (nominal 8 inches wide by 15 inches deep) and material of former ties, which were not original. The ties here, as at all railroads, had been replaced as part of ongoing maintenance. They were of Douglas fir and creosote-coated, similar to the ties lost to the fire. During the investigation stage, it was learned that the original track had been installed slightly off the bridge centerline; this fault was corrected during refurbishment, with the new track centered on the bridge.

Conclusion

Laboratory testing of specimens taken from both the heat-affected part of the girders and from portions not affected by heat revealed that the heated wrought iron no longer possessed significant ductility and thus was unsuitable for an active railroad bridge. This discovery led to the decision to remove and replace the girders with new girders. As structural wrought iron is no longer produced, structural steel became the logical choice. Making connections with bolts instead of rivets was a more difficult decision. Rivets are available, as are all the necessary hammers and forges. The difficulty was in finding qualified riveters and, in particular, a fabricator that possessed AISC certification for such bridges using rivets and the establishment of satisfactory inspection criteria. If rivets are to be used on a significant project such as this one, the issues of certification and inspection will need to be addressed.

The management and design team struggled with weighing preservation measures against very real public-safety, financial, and schedule constraints. Historic features were evaluated to determine the appropriate level of intervention. The final decision was to reconstruct the superstructure while the substructure was largely preserved, although some rehabilitation took place. While the reconstructed girders were steel instead of wrought iron, the distinctive features defining the historic character were largely retained, the only visible difference being use of bolts in place of rivets. The bridge is used as it was historically, supporting loaded trains daily.

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PETER FOSTER is a professional civil engineer and vice president of Wright Water Engineers, Inc., specializing in water resources, where he manages the office in Durango, Colorado. He has served as a commissioner of the Cumbres and Toltec Scenic Railroad for three terms, as appointed by three Colorado governors. He can be reached at pfoster@wrightwater.com.

Notes

1. Jeff Brown, "Scenic Railroad to Repair Fire-Damaged Crossing," *Civil Engineering* 81, no. 4 (April 2011): 30.

2. A television news report can be found at http://www.krqe.com/dpp/news/local/northwest/train-trestle-burning-outside-of-chama.

3. Herbert Danneman, "A Ticket to Ride the Narrow Gauge," *Colorado Rail Annual* no. 24, Colorado Railroad Museum, Golden, Colo., (2000).

4. Vernon Glover, Denver & Rio Grande Railroad, San Juan Extension, Wolf Creek Trestle (Cumbres and Toltec Scenic Railroad, Lobato Trestle), HAER No. NM-16, Historic American Engineering Record 2011: 4-6.

5. Ibid., 6.

6. Ibid.

7. Ibid., 7.

8. Ibid., 8.

9. American Railway Engineering and Maintenance-of-Way Association, AREMA *Manual for Railway Engineering* (Lanham, Md.: AREMA, 2010).

10. Federal Railroad Administration, 40 CFR Parts 213 and 237, Bridge Safety Standards, Final Rule, Nov. 22, 2010.

11. Robert Gordon and Robert Knopf, "Evaluation of Wrought Iron for Continued Service in Historic Bridges," *Journal of Materials in Civil Engineering* 17, no. 4, (July/August 1995): 396.

12. "Cooper's Equivalent Load" is a design live load using the Cooper E series–loading axle configuration adopted by the railroad industry. Each axle is given a prescriptive axle loading. Higher loadings are obtained by multiplying each load by the same constant; wheel spacings do not change. Many different axle configurations and loadings currently exist, and the Cooper E loading represents an industry-adopted mechanism to designate a rating.

13. Charles C. Schneider, "The Evolution of the Practice of American Bridge Building," *Transactions of the American Society of Civil Engineers,* v. 54 (New York: American Society of Civil Engineers, 1905): 222.

14. Francis E. Griggs, "Restoration of Cast and Wrought Iron Bridges," Structure 8, no. 7 (Sept. 2001): 18.

15. Gordon and Kompf, 393.

16. M. O. Withey and James Aston, *Johnson's Materials of Construction*, fifth ed. (London: John Wiley and Sons, 1919), 599.

17. James Aston and Edward Story, *Wrought Iron: Its Manufacture, Characteristics and Applications*, second ed. (Pittsburgh, Pa.: A. M. Byers Company, 1949), 2-3.

18. Aston and Story, 42.

19. Riveted connections were used in most iron or steel structures until about 1960, when bolts became more economical. Rivets are still used in certain applications such as railroad rolling stock, but they have given way to bolts in general, and high-strength bolts in particular, in building and bridge structures.

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