Engineering for Rehabilitation of Historic Metal Truss Bridges

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Figure 1. The rehabilitated 133rd Avenue bridge, installed at the Calhoun County Historic Bridge Park.

Introduction

The Calhoun County Historic Bridge Park southeast of Battle Creek, Michigan, displays a collection of rehabilitated metal truss bridges for use and enjoyment by pedestrians. From the perspective of a structural engineer, it was instructive to investigate the general feasibility of rehabilitating century-old metal truss highway bridges for pedestrian service consistent with modern standards for safety[1,2,3] and historic integrity[14]. Engineering aspects of rehabilitation are discussed for bridges that are now in the Park, specifically:

- 133rd Avenue bridge (Figure 1), a pin-connected half-hip Pratt pony truss spanning 64 ft. (19.5 m), erected in 1897 by the Michigan Bridge Company to cross the Rabbit River in Allegan County, Michigan.
- Twenty Mile Road bridge (Figure 2), a 70 ft. (21 m) long riveted Pratt pony truss that spanned the St. Joseph River in Calhoun County. Physical features hint that this bridge was designed for railway service. The builder has not been identified and several sources date construction to the early twentieth century.
- Gale Road bridge, a pin-connected skewed Pratt through truss built in 1897 by the Lafayette Bridge Company. Originally spanning 122 ft. (37 m) over the Grand River in Ingham County, Michigan, this bridge currently is being re-erected in the Park.

Six other bridges have been procured and are awaiting rehabilitation before being put in the Park, including these that also will be discussed

- Tallman Road and Bauer Road bridges, nearly identical pin-connected Pratt through trusses that spanned about 90 ft. (27 m) over the Looking Glass River in Clinton County. Manufactured by the Penn Bridge Company and erected in 1880, they are two of Michigan's oldest through trusses[9].
- Charlotte Highway bridge, manufactured by the Buckeye Bridge Company and erected in 1886. Prior to its recent removal (Figure 3), it crossed the Grand River in Ionia County with a span of 177 ft. (54 m) and was one of very few double-intersection Pratt truss bridges remaining in Michigan[9].

Feasibility

Investigation of feasibility involves comparing historic and modern specifications for bridge design, particularly those governing materials and loads. During the period when the project bridges were built, standards were promulgated by individual iron and steel producers, bridge designers and manufacturers, owners (typically municipal governments) and textbook authors. These standards were numerous and varied; those cited are representative rather than comprehensive.

Strength of Metals

Although the quality of structural steel has been perfected over the past century, the strength of low carbon steels usually used in bridges has not changed significantly (Table 1). However, the allowable stresses used by bridge designers increased as confidence and understanding developed. This is reflected in the trend toward lower factors of safety illustrated by Tables 1 and 2. Early bridge designers used factors of safety as high as six to compensate for lack of quantitative information. Today, based on results of a century of research and experience, factors of safety of two or less are typical. Modern specifications may allow larger stresses in the old steel and wrought iron members of a historic bridge than did its designer.

Figure 2. The rehabilitated Twenty Mile Road bridge, shown in its new position at the Historic Bridge Park.

Live Load

An old highway bridge may have become deficient in strength due to the increased weight of trucks. In 1916 Waddell[17] advocated designing Class C bridges for a single 6 ton (53 kN) truck weight, and Class A bridges for an 18 ton (160 kN) truck, noting that "Almost all of the old highway bridges are incapable of carrying these new live loads with safety." The smallest

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design vehicle load currently recognized is a two-axle truck weighing 15 tons[1] (133 kN). However, historic metal highway bridges were designed to carry uniformly distributed loads in addition to, or in lieu of, concentrated axle loads to assure safety for lines of wagons or automobiles, livestock, and crowds of people, the latter being the larger, or governing, distributed load.

Figure 3. Lifting the Charlotte Highway bridge from its original abutment. This end was lowered onto a barge prior to hauling the bridge across the river and up the other bank.

Table 3 traces the trend and variations in design values for distributed live loads on highway bridges as well as listing current design values for pedestrian bridges[2]. Ranges reflect levels of service. This table demonstrates that, in general, the published design loads for old highway bridges exceed the current requirement for pedestrian bridges. Bridges with long spans and designed for rural service may be exceptions.

Wind Load

In contrast to distributed live loads, design wind loads have increased significantly. In 1901 Waddell advocated design loads of 250 and 150 lb/ft. (3.65 and 2.19 kN/m) on the loaded and unloaded chords, respectively, for class A bridges with spans of 150 ft. (46 m) or less[16], but by 1916 he had

Table 1. Tensile strengths of steel and factors of safety for tension fracture at net section.

Source	Year	Grade of Steel	Yield stress, minimum, ksi (MPA)	Ultimate stress, minimum, ksi (MPA)	Allowable stress on net section, ksi (MPA)	Factor of safety for fracture
Pottsville Iron & Steel Co.[7]	1887				15.6 (108)	
Carnegie Phipps & Co.[7]	1889-1893	for bridges			12.5 (86)	
IATM[10]	1900	medium	35 (241)	60 (414)		
Waddell[16]	1901	medium	35 (241)	60 (414)	16 (110) 18 (124)	3.8 3.3
Burr and Falk[4]	1901					3.5 to 6.0@
Copper[12]	1909	medium			10 to 25 (69 to 720)#	2.4 to 6.0#
Michigan[13]	1910	medium	30 (207)	60 (414)	15 (103)	4.0
Bethlehem Steel Co.[7]	1907-11	moving loads			12.5 (86)	
Waddell[17]	1916	medium	35 (241)	60 (414)	16 (110)	3.8
Ketchum[12]	1920	medium			16 (110)	
AASHTO[3]	pre 1905 1905-36		26 (179) 30 (207)	52 (358) 60 (414)	26 (179)* 30 (207)*	2.0* 2.0*
AASHTO[1]	current	ASTM A36	36 (248)	58 (400)	29 (200)*	2.0*

* for inventory rating, less than 100,000 load cycles
@ depending on span # depending on type of load, including impact factor

Table 2. Tensile strengths of wrought iron and factors of safety for tension fracture.

Source	Year	Grade of Steel	Yield stress, minimum, ksi (MPA)	Ultimate stress, minimum, ksi (MPA)	Allowable stress ksi (MPA)	Factor of safety for fracture
Carnegie Kloman & Co.[7]	1873	wrought iron			14 (97)	3
Waddell[15]	1883	iron	26 (179)	50 (345)	8 to 12.5 (55 to 86)#	4.0 to 6.2#
Phoenix Iron Co.[7]	1885				12 (83)	
IATM[11]	1900	refined iron test iron class A test iron class B stay-bolt iron	25 (172) 25 (172) 25 (172) 25 (172)	48 (331) 48 (331) 50 (345) 46 (317)		
Waddell[16]	1901	wrought iron	26 (179)	50 (345)	13 (90)	3.8
AASHTO[3]		wrought iron			14.6 (101)*	

* for inventory rating # depending on service class and influence area

increased those values to 320 and 180 lb/ft.[17] (4.67 and 2.63 kN/m). The Illinois Highway Department designed for the larger of 25 lb/ft.^{2} (1.2 kN/m^2) on the vertical projection of each truss and of the deck, or 300 and 150 lb/ft. (4.38 and 2.19 kN/m) on the loaded and unloaded chords, respectively[12]. Modern specifications[1,2] are much more demanding, requiring design for wind loads of 75 lb/ft.^{2} (3.6 kN/m^2) on the vertical projection of each truss and of the deck, plus 300 and 150 lb/ft. (4.38 and 2.19 kN/m) on the loaded and unloaded chords, respectively (this lineal load is not required for pedestrian bridges), plus 20 lb/ft.^{2} (0.96 kN/m^2) upward on the deck. Clearly, historic bridges are unlikely to have been designed for the wind loads currently mandated.

Structural Analysis and Design

The components of each of the rehabilitated project bridges were analyzed to estimate design stresses associated with internal forces caused by specified combinations of loads[1] and acting on the original uncorroded member cross-sections. Allowable stresses were computed from assumed material properties[3] and specified factors of

Table 3. Uniformly distributed design live loads for highway bridge trusses in pounds per square foot (kN/m²).

Source	Year	Span		
		50 feet (15.2 m)	**100 feet (30.5 m)**	**200 feet (61.0 m)**
Whipple[5]	1846	100 (4.79)	100 (4.79)	100 (4.79)
ASCE[5]	1875	100-70 (4.79-3.35)	75-50 (3.59-2.39)	60-40 (2.87-1.92)
Waddell[15]	1883	100-80 (4.79-3.83)	90-80 (4.31-3.83)	70-60 (3.35-2.87)
Waddell*[16]	1901	170-113 (8.14-5.41)	149-98 (7.13-4.69)	120-80 (5.75-3.83)
American Bridge Co.*[4]	1901	125-100 (5.99-4.79)	125-94 (5.99-4.50)	100-69 (4.79-3.30)
Michigan Highway Comm.[13]	1910	100 (4.79)	100 (4.79)	100 (4.79)
Waddell*#[17]	1916	161-107 (7.71-5.12)	144-95 (6.89-4.55)	119-80 (5.70-3.83)
Ketchum*[12]	1920	151-116 (7.23-5.55)	126-89 (6.03-4.26)	103-60 (4.93-2.87)
Illinois Highway Comm.[12]	1920	125 (5.99)	100 (4.79)	85 (4.07)
Wisconsin Highway Comm.[12]	1920	120 (5.74)	93 (4.45)	50 (2.39)
AASHTO (pedestrian)#[2]	1997	67 (3.21)	65 (3.11)	65 (3.11)

* Prescribes an impact factor, which is included in the tabulated values   # For 16 foot (4.88 m) deck width

safety[1]. For each component and load combination, the allowable stress was divided by the design stress. A ratio less than unity indicates need for modification, while a ratio greater than unity suggests that an acceptable level of safety may be achieved without completely restoring corroded sections (in general, significant damage was repaired in the interest of historic integrity and esthetics). The three rehabilitated project bridges were found to have adequate capacity for pedestrian loading.

Unusual Features

The structural analysis of a truss usually is a routine procedure. To simplify computations, the structural engineer assumes that each member transmits force only in the direction of its longitudinal axis. That is, the member is not subject to transverse force (shear) or bending. This assumed behavior is achieved if the members are straight and connected at their ends by frictionless pins, longitudinal axes of members are concentric at connections, and loads are applied to the truss only at connections. Real trusses conform to this idealization only approximately but member forces may be computed with sufficient accuracy if the design approaches the ideal conditions.

The Tallman Road bridge displays two peculiar details that are contrary to the ideal conditions and to subsequent practice. The most obvious is the hip joint, which has two pins rather than one. One pin carries the vertical eyebar and the other carries the diagonal eyebar pair. Because the longitudinal axes of the inclined end post, top chord, vertical and diagonal members do not meet at a common point, bending is induced in the end post and top chord.

The second peculiarity of the Tallman Road bridge is that each lower chord eyebar spans two deck panels and has three eyes: one at each end and one in the middle. When gravity load is applied to a truss, the panel points near midspan typically deflect downward more than those near the ends. If the truss conforms to the ideal conditions, the members rotate but remain straight as the panel points deflect. Obviously this behavior cannot be achieved by a three-hole eyebar. Thus, these unusual lower chord eyebars are subject to bending as well as axial tension.

Strength Not Predicted by Conventional Truss Analysis

Conventional analysis predicts that the lower chord of a single-span through truss is always in tension when the bridge is carrying gravity load. However, the lower chords in the end panels of the Charlotte Highway bridge were observed to be slack (i.e., subjected to compression rather than tension) when the bridge was in service in its original location. Those members remained slack after the vehicular railings and deck were removed in preparation for moving the

Design wind loads have increased significantly

bridge from its masonry abutments. However, when the bridge was freed from its inoperative expansion bearings, that end appeared to move inland several inches and cracks opened where the wingwalls join the abutments. Apparently the upper chord and end posts had been functioning as an arch as well as restraining displacement of the heavy abutments and fill.

Figure 4. Severely corroded sections of the Twenty Mile Road bridge were replaced by welding new steel to sound original material.

Prior to lifting the six-panel Bauer Road bridge from its original abutments, the contractor removed railings, decking and stringers. Then a lifting sling was attached to the upper lateral struts at the third points of the span. Conventional truss analysis predicts that the bottom chord will be compressed when the bridge is lifted in this manner. Since the bottom chord consists of eyebars, which have negligible resistance to compression, it seemed likely that the trusses would collapse. The fact that the lift was accomplished without damage attests that the upper chord, hip joints and end posts possess significant bending strength.

Conventional truss analysis may underestimate the strength of a metal truss bridge. More comprehensive analysis techniques coupled with detailed modeling of connections may make it possible to quantify additional strength.

Inadequate Resistance to Wind Load

By modern design standards, the rehabilitated project bridges had inadequate resistance to wind load. It was necessary to employ a provision[1] that permits design wind speed to be adjusted from a nominal 100 MPH (45 m/s) to reflect favorable local conditions. The inland location of the Park and the low and sheltered sites of the project bridges justify a design wind velocity of 70 MPH (31 m/s). Despite the resulting 50% reduction of wind force, the original anchor bolts typically were inadequate, and each of the three bridges manifested other deficiencies.

Analysis of the 133rd Avenue bridge predicted that modern design wind loads would cause net axial compression of the windward lower chord eyebars. Since eyebars have negligible resistance to compression, they would buckle and the truss would become unstable. This was corrected by installing an unusually heavy deck to create enough tension in the lower chord to counteract the compression induced by wind. Alternatively, it may have been possible to rely on the deck or upper chord to stabilize the trusses as suggested in the preceding section.

The deck lateral ties of the Twenty Mile Road bridge were evaluated using the assumed strength of steel produced before 1905[3] and found to be inadequate. The ties, like other parts of this bridge (Figure 4) were too badly corroded to be salvaged. Replacing them with new steel, in the original sizes, was sufficient to provide the required wind resistance.

Structural analysis showed that the original portal braces of the Gale Road bridge were inadequate. Vertical struts had been arc welded to the lattice panels sometime after construction, apparently to correct perceived weak-

Figure 5. Forge-welded loop eyebars like these are obsolete.

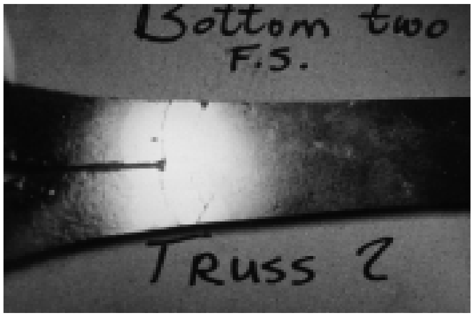

Figure 6. Dye penetrant inspection of a forge weld.

ness, and localized bending of horizontal members occurred after these reinforcements were installed. The original portal braces will be retained for display but not installed on the rehabilitated bridge. The replacement portal braces have larger connection gussets than the originals, and the lattice is steel angles of the same width as the original flat bars. The configuration and overall dimensions of the original portal braces are duplicated.

Conventional truss analysis may underestimate the strength of a bridge

Features Not Covered in Current Specifications

Pony trusses and loop eyebars (Figure 5) are obsolete, and there are no current standards to guide assessment of these features. Pony trusses are prone to lateral instability of the top chords. That is, the bridge tends to fold inward under heavy load. The two rehabilitated pony trusses were checked for stability by Holt's method[8] and both were found to have adequate factors of safety for pedestrian loading.

Single-load tests of seventeen wrought iron loop eyebars reported by Ellerby et al[6] demonstrated that fracture may occur at a forge weld rather than in the body of a bar, sometimes at a load significantly less than the design strength of the bar. As part of the same investigation, twenty-six wrought iron loop eyebars were repeatedly loaded to working stress level. The number of load cycles to failure suggests that the bars could have remained in highway service for many more decades. When fatigue fractures finally did occur, they were in the loops (except for two bars, which initially had large cracks at forge welds). The investigators speculated that repeated flexing of the loops was a critical factor and noted the deleterious effect of poor fit on the pin.

The usual practice for the project bridges is to inspect eyebar eyes and forge welds visually and by ultrasonic and dye penetrant methods (Figure 6). Cracks are ground out and bars are built back to original profile by arc welding. Testing has shown that careful arc welding restores full strength[6].

Conclusion

Selected historic metal truss bridges that are rehabilitated to near-original condition can satisfy modern safety standards for pedestrian service. This is demonstrated by the bridges on display in the Calhoun County Historic Bridge Park.

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