Paper ID #35760

Structural Evaluation of the Ontario & Western Railway's Original Bridge at Fish's Eddy

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Dr. Mazurek joined the faculty of the Civil Engineering section in 1990. He was previously employed by General Dynamics' Electric Boat Division, where he provided submarine construction support and conducted engineering design and analysis associated with pressure hulls and other structures. He also conducted research in the area of structural noise and vibration transmission reduction in submarines. Just prior to joining the Academy faculty, he taught for one year at Lafayette College in the Department of Civil Engineering.

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Abstract

The New York, Ontario & Western Railway's three-span pin-connected truss experienced two collapses over its 15-year life. The second occurred in 1897 as an empty coal train was slowly crossing the bridge, when suddenly the middle span collapsed. While the cause was never established, a recent study demonstrated that the floor beam hangers were of unusual design and particularly vulnerable, and that failure of such a hanger could have been the cause of the bridge collapse. The present study considers the remaining members of the truss, and through a comparative analysis, examines whether these members shared vulnerabilities of the same degree as the floor beam hangers. Cooper ratings were completed based on both the original specifications that governed the design of the bridge, as well as the current recommended practices used by North American railroads. Using the same practices employed by the designers, the hangers were among the lowest rated members, but certain other members did display a somewhat lower rating. However, the designers overlooked a key provision which, when included, results in the hangers rating well below all other members. When applying current standards, an inflated rating occurs because the allowable stress criteria for hangers were inconsistently established relative to other criteria. Using previous standards from recent years instead, the hangers again displayed the lowest rating of all truss members. Regardless of the standard applied, the rated capacity of the hangers was found to be below the actual loading regularly operated over the bridge in the years immediately preceding the collapse. These results serve to validate the hypothesis that the hangers were among the most vulnerable members of the truss, and could indeed have been the trigger that led to the 1897 collapse.

I. Introduction

The New York, Ontario & Western Railway (hereafter referred to as the "O&W") was formed in 1880 from the bankrupt New York and Oswego Midland Railroad, and soon engaged in a program to renew and upgrade the right-of-way. In 1882, the Central Bridge Works of Buffalo, New York, was contracted to design a new bridge over the East Branch of the Delaware River, near Fish's Eddy, New York. The new structure was a three-span, wrought iron, pin-connected through truss, and accommodated a single track. The bridge functioned with no known incidents of significance until March 3, 1886, when the caboose of a southbound train derailed due to a broken rail and struck the end-post of the first truss, collapsing the span and crushing the car, killing all four of its occupants [6].

The bridge was restored and then, eleven years later, the middle span collapsed in the early morning hours of April 28, 1897, as a northbound train was crossing. Fig. 1 shows the collapsed structure. The 30-car train was under orders not to exceed 4 miles per hour while traversing the bridge, as it was to immediately take a siding to allow another train to pass. The locomotive had safely crossed the span prior to the collapse, and the train itself consisted of empty coal gondolas, eleven of which fell into the river with the failed span. Because the train was traveling so slowly, it was felt that derailment was not a likely cause, and the bridge "was considered so perfectly safe in every particular that the officials express the greatest surprise that an accident of this kind should happen" [5].

Fig. 1. Middle span collapse of the Fish's Eddy Bridge in 1897. (DeForest Douglas Diver Railroad Photographs #1948. Division of Rare and Manuscript Collections, Cornell University Library.).

A recent investigation by Mazurek and Tarhini [4] considered the possibility that the failure of a first-panel hanger triggered the collapse, as this could have resulted in the debris field exhibited by various photos taken after the incident, such as that shown in Fig. 1. Further, each floor beam hanger merely consisted of a single eyebar, with the floor beam riveted directly into its side. This unconventional design served to locally reduce the cross section of the eyebar, introduce large stress concentrations, and subjected the hanger to secondary bending stresses. The results of this study confirmed that the hangers were highly vulnerable and likely overstressed on a regular basis, especially during the final years leading up to the 1897 collapse.

This paper reports on the continued investigation of this bridge, where the remaining primary members of the truss have been evaluated to determine whether they also possessed vulnerabilities of a degree similar to the first panel hangers. These assessments are based on the allowable stress criteria of both the original design specifications as well as those stipulated in the modern recommended rules of practice. Observations are then made regarding the likelihood of an alternative failure mechanism as compared to that previously demonstrated for the first panel hangers.

II. Background

The bridge consisted of three identical Pratt trusses skewed at 53.5°, each being 144 ft long and having nine panels. The compression members (upper chord, end posts, and vertical posts) consisted of double channels or built-up sections with lacing. All tension members (lower chord, diagonals, and first-panel hangers) used eyebars with the exception of the diagonal counters, which were rod members. The skew was accommodated within the first panel at the ends of each truss, resulting in a considerable difference in lower chord lengths on each side (approximately

22 ft versus 10 ft). Fig. 2 highlights the main structural details. The panel points were pinned, and all other connections were riveted. With the exception of the hangers (such as member *U*1-*L*1), all tension members were comprised of at least two eyebars. Each hanger, on the other hand, consisted of just a single eyebar, with the supported floor beam riveted directly into the body of the eyebar. A more common practice would have been to either connect the floor beam directly to the pin joint, or to rivet the floor beam into a hanger consisting of a much more robust, built-up section.

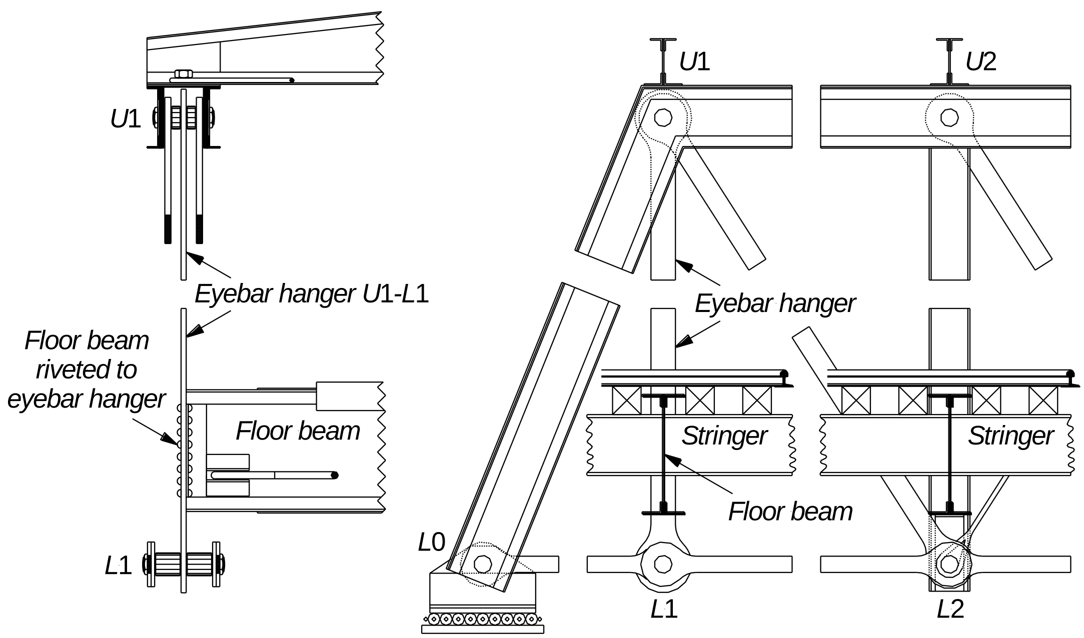

Fig. 2. Details of the Fish's Eddy trusses, showing a partial section (left) and elevation (right).

Shortly following the reorganization of the Midland into the O&W, the *General Specifications for Iron Bridges* [3] was developed, governing the design and construction of the Fish's Eddy truss spans in 1882. These specifications generally required that wrought iron be used for the superstructure elements. The iron was to have a minimum ultimate tensile strength of 50,000 psi and an elastic limit of not less than 25,000 psi. With regard to tension members, the allowable stress for counter rods and verticals was 8,000 psi, while that for bottom chords and main diagonals was 10,000 psi. Recognizing that hangers supporting floor beams are particularly vulnerable to impact loads, the allowable stress for these "and other similar members liable to sudden loading" was limited to 6,000 psi. The specification also required that in the case of tension members, "full allowance shall be made for reduction of section for rivet-holes, screw-threads, etc." In addition, any members subject to bending "from local loadings (such as distributed floors on deck bridges)" must be proportioned to support these bending effects in combination with the primary member stresses. For compression members with pinned-ends, the allowable stress employed a Rankine-Gordon formulation, given by:

$$f_{\text{all}} = \frac{8{,}000}{1 + \dfrac{L^2}{20{,}000\, r^2}} \tag{1}$$

Here f_{all} = allowable stress (psi), L = member length, and r = least radius of gyration.

The rules currently followed throughout North America for the design of steel railway bridges are the recommended practices given in Chapter 15 of the *Manual for Railway Engineering* (MRE), published by the American Railway Engineering and Maintenance-of-Way Association [2]. For tension members, the allowable stress is $0.55F_y$ (where F_y = yield strength) on the gross section and, in the case of eyebars, $0.45F_y$ on the net section through the pin hole. For floor beam hangers, the allowable tension stress, including bending, is reduced to $0.40F_y$ on the gross section, while a rather liberal limit of $0.50F_u$ (where F_u = ultimate strength) is currently permitted through effective net section of the riveted connection. For compression members with pinned ends, the allowable stress in the inelastic region (which generally applies for the subject truss) was adapted from the work of the Structural Stability Research Council, and given by:

$$f_{\text{all}} = 0.6F_y - \left(17{,}500\,\frac{F_y}{E}\right)^{3/2} \frac{(7/8)\,L}{r} \qquad (2)$$

Here E = modulus of elasticity.

O&W’s 1881 specification stipulated the dead and live loads to be used for bridge design. The dead load was to include the actual weight of the iron in the structure, as well as a floor load of 400 lb/ft of track to account for the weight of the rails, ties, and guard timbers. For the Fish’s Eddy Bridge, this resulted in a total dead load of 1350 lb/ft. The specified live load, consisting of two consolidation-class locomotives, one of which is shown in Fig. 3, and a uniformly-distributed trailing load of 2240 lb/ft representing the rest of the train, was remarkably similar to the Cooper system developed shortly thereafter and still in use today. For comparison, and to match the driver loads of the O&W specification, a Cooper E22 loading is shown in Fig. 4.

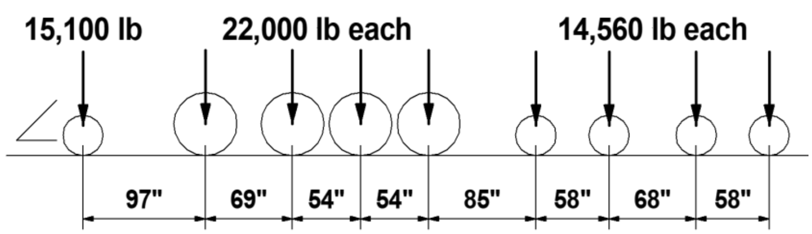

Fig. 3. Design locomotive axle loads as given by the 1881 O&W *General Specification for Iron Bridges.*

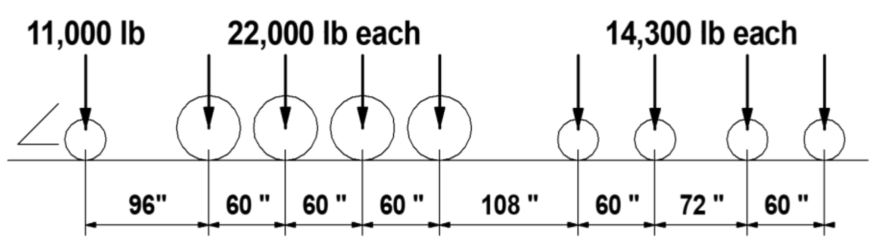

Fig. 4. Design locomotive axle loads representative of Cooper E22.

III. Assessment of Main Trusses

A. Analysis Methods

The main trusses were evaluated by applying both the O&W design specification for which the bridge was originally required to satisfy, as well as the current rules of AREMA. The dead load assumed in these assessments was the same as that used by the designers. When applying live loads in accordance with AREMA, these loads were modified to include impact and rocking effect as stipulated by the standard; the O&W specification, on the other hand, had no specific provision for impact.

Figure 5 shows the general arrangement of the truss and the nomenclature used for each panel point. When analyzing the hangers, the significant skew of the trusses was accounted for in the loads on each rail, and since the hangers are only affected by first-panel loads, the skew did result in a small difference in hanger force. When analyzing the rest of the truss, however, the skew's influence on rail loads was not included, as its effect is negligible. This was because the bulk of the truss members were affected by loads applied to all panels, and that the effects of the skew at one end of the span tended to balance out the effects at the other end.

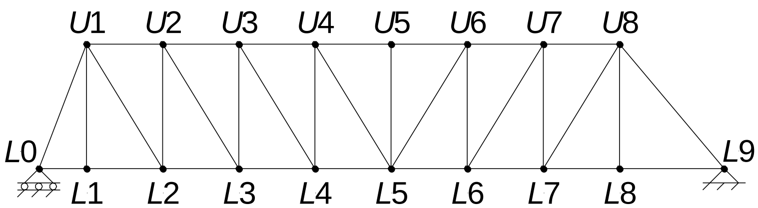

Fig. 5. Truss configuration.

Influence lines were generated for each member of the truss, and these lines were used both to apply dead loads as well as to support the evaluation of live loads. While two-dimensional structural analysis software was employed to support various aspects of the truss analyses, the primary tool used for live load assessment was spreadsheet analysis, where modeled trains were incrementally moved across the structure. For example, Fig. 6 illustrates the force developed in member *L*7-*L*8 as a Cooper E10 (with no impact) train crosses the bridge. The length of all trains was established to ensure that the maximum possible member force due to each particular train was obtained. Trains were also operated in both directions to establish which direction of travel created the maximum member force.

Certain members of the truss, namely the vertical posts, incorporated channel shapes that have long since been discontinued. A review of various historical databases, including shape data published by the Pottsville Iron and Steel Co. in 1885, proved fruitless in locating the specific channels used in the bridge. Thus, where necessary, modern structural steel shapes were identified that closely approximated the original channels, and the data for these modern shapes used instead.

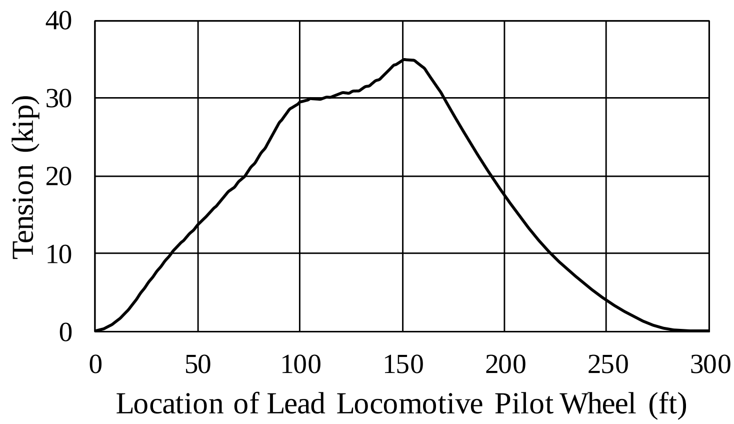

Fig. 6. Force in member *L*7-*L*8 due to a Cooper E10 load.

B. Validation of Designer's Structural Analysis

In order to confirm the methodologies employed by the engineers of the Central Bridge Works, and more specifically, the loads they used, structural analyses were performed to validate the results documented by these designers. As stated earlier, the O&W specification stipulated a design dead load of 1350 lb/ft for the bridge, and a design live load that closely resembled a Cooper E22 loading (with no provision for impact). Using these same loads, Table I shows the results of the structural analysis for the members of panels 8 and 9; also included are the member forces as documented by the original designers. As this table reflects, the outcome of the present analysis agrees rather closely with that of the bridge's designers, with deviations well under 5%.

Table I. Member Forces Due to O&W Specification Loads.

Member	Orig. Design (kips)	Present Analysis (kips)	Deviation (%)
*U*7-*U*8	172.9 (comp.)	174.8 (comp.)	+1.10
*L*7-*U*8	120.0 (tens.)	116.3 (tens.)	−3.08
*L*7-*L*8	116.2 (tens.)	118.1 (tens.)	+1.64
*L*8-*L*9	116.2 (tens.)	118.1 (tens.)	+1.64
*U*8-*L*8	43.2 (tens.)	44.1 (tens.)	+2.08
*U*8-*L*9	178.1 (comp.)	179.3 (comp.)	+0.67

C. Capacity Assessments

Although first introduced in the late 1800's, the Cooper load continues to be the universal standard followed throughout North America for both the design and rating of railroad bridges. Thus, in order to perform a comparative analysis of the Fish's Eddy Bridge, the Cooper system was employed to rate each member of the truss based on both the original 1881 design specifications as well as the current provisions of AREMA. The results of this analysis are shown in Table II.

Table II. Cooper Ratings.

Chord Members			Diagonals, Hangers, and Posts		
Member	O&W Specification	AREMA	Member	O&W Specification	AREMA
*U*1-*U*2	E37.5	E53.9	*U*1-*L*0	E30.3	E43.3
*U*2-*U*3	E24.7	E38.0	*U*1-*L*1	E25.3	E30.0
*U*3-*U*4	E24.6	E38.4	*U*1-*L*2	E29.4	E39.5
*U*4-*U*5	E24.5	E40.6	*U*2-*L*2	E26.0	E29.8
*U*5-*U*6	E24.7	E36.7	*U*2-*L*3	E23.2	E38.1
*U*6-*U*7	E25.8	E39.6	*U*3-*L*3	E32.2	E33.3
*U*7-*U*8	E26.9	E40.5	*U*3-*L*4	E24.9	E27.3
*L*0-*L*1	E29.0	E35.9	*U*4-*L*4	E47.5	E48.9
*L*1-*L*2	E29.0	E35.9	*U*4-*L*5	E27.9	E28.4
*L*2-*L*3	E23.6	E28.5	*U*5-*L*5	E57.3	E71.6
*L*3-*L*4	E22.7	E28.8	*U*6-*L*5	E28.6	E41.7
*L*4-*L*5	E24.5	E30.4	*U*6-*L*6	E40.1	E42.2
*L*5-*L*6	E23.1	E38.1	*U*7-*L*6	E24.7	E38.5
*L*6-*L*7	E22.8	E38.3	*U*7-*L*7	E22.8	E25.7
*L*7-*L*8	E22.6	E25.9	*U*8-*L*7	E25.7	E28.6
*L*8-*L*9	E22.6	E25.9	*U*8-*L*8	E24.4	E26.8
			*U*8-*L*9	E24.9	E37.2

In all cases the O&W specification provides a lower rating than that reflected by the current AREMA standards, even though AREMA requires a significant increase in loading due to impact and rocking effect that the original design specifications do not. Nonetheless, the fundamental reason why the O&W specification results in a lower rating is that the allowable stresses were far more conservative. This is especially true for the compression members (i.e., the upper chord, end posts, and vertical posts), where a noticeably higher degree of conservatism is evident. In addition, while provisions for impact were not explicitly included in the O&W specification, it could be argued that one reason the allowable stresses were so very conservative was to indirectly accommodate the effects of impact. The overall rating for a bridge is governed by the lowest rating among all members; with the focus of this paper limited to the trusses, the rating based on the O&W specification is Cooper E22.6 (members *L*7-*L*8 and *L*8-*L*9), and for the AREMA provisions is Cooper E25.7 (member *U*7-*L*7).

As noted earlier (and as reflected by Fig.'s 3 and 4), the design live load of the O&W specification is somewhat close to a Cooper E22, and was therefore almost at capacity with an overall rating of E22.6 (based on this specification). After the bridge went into service in 1882, the actual train loads gradually increased over the structure's 15-year life, and by the 1890's the bridge was routinely subjected to loadings in excess of Cooper E29. This load level exceeds even the overall rating based on the current AREMA standards of Cooper E25.7. Thus, based on either measure, the bridge was regularly subjected to overloads in the years leading up to its 1897 collapse.

IV. Reconsideration of Hangers

In the study by Mazurek and Tarhini [4], the failure of a hanger was considered as a possible cause of the 1897 collapse. Such a scenario would support the resulting debris field illustrated in Fig. 1, since the failure of this component would not have necessarily led to immediate collapse, but instead might have allowed the locomotive and forward part of the train to safely exit the span while the rear portion proceeded to pile up at the site of the failed hanger. With derailed cars from the rear eventually colliding with other members of the truss, the span might have then collapsed, leaving all the cars heaped at one end as was observed.

It was previously noted that the hanger design was quite unusual as well, where each consisted of just a single eyebar, and the supported floor beam was riveted directly into the body of the eyebar. This unusual connection served to reduce the capacity of the hanger for three reasons. First was the rather significant loss of area, where the cross section through each line of rivets was almost a third less than the overall section. Second, the otherwise uniform stress field in the body of the eyebar was altered to one with very high stress concentrations. And third, there was a measure of secondary bending introduced into the eyebar by having the floor beam attached in this manner. For these reasons, it was speculated in the Mazurek and Tarhini study that the hanger might have been the most vulnerable component of the truss.

Based on the capacity assessment and comparative analyses of the present study, hanger *U*8-*L*8 in particular does display a rather low rating of Cooper E24.4 based on the O&W specification, although there are a number of members that do exhibit a slightly lesser rating. Consistent with the apparent practice of the designers, however, the rating of the hanger was based on the gross section only, even though the O&W specification clearly indicates that the loss of section through the riveted connection should be taken into account. When applying this condition, member *U*8-*L*8 only rates at Cooper E14.4, far below the overall rating for the truss at E22.6.

When considering the present-day criteria of AREMA, the capacity assessment resulted in a Cooper E26.8 rating for hanger *U*8-*L*8. In the application of this criteria, it should be noted that the allowable stress provisions for hangers in particular are less conservative and inconsistently established relative to other criteria, and for this reason, the hanger criteria are currently under review by committee. Interestingly, if the allowable stress rules for hangers as promulgated in the 1995 edition of the MRE are used, member *U*8-*L*8 would rate at only E24.8, which would make this the lowest rating among all the members of the truss.

An additional important issue regarding the hangers is that they were highly susceptible to cyclic loading and fatigue failure, a concern which does not apply to the same extent for the other members of the truss, partly because these members would not have been subjected to nearly the same number of load cycles that the hangers would have experienced. The effects of fatigue are also not reflected by the Cooper rating analyses. Thus, in all, these results support the hypothesis that the hangers were indeed among the most vulnerable of members in the Fish's Eddy Bridge.

V. Conclusions

Historical background has been presented regarding the O&W's three-span pin-connected through truss bridge near Fish's Eddy, New York, erected in 1882. The bridge experienced two major failures, the first taking place in 1886 when a derailed train struck an end post and collapsed the northernmost span. The second failure occurred in 1897, where empty coal cars were traversing the bridge when the center span suddenly collapsed for no apparent reason. A recent study demonstrated that failure of a floor beam hanger could have been the cause.

The present study examined the rest of the truss members to determine if any shared vulnerabilities to failure of the same degree as demonstrated for the hangers. Considering both the original specifications from 1881 as implemented by the design engineers as well as the current rules of AREMA, a rating analysis was performed for the truss. The results showed that under both standards, there were some members of the truss that displayed a Cooper rating slightly less than that for the hangers. However, the designers overlooked the application of a net section criterion mandated by the governing specifications, and the current rules of AREMA employ allowable stress criteria that are less conservative than those previously contained in recent editions of the MRE. When taking these factors into account, the hangers clearly display the lowest rating among the members of the truss. Furthermore, regardless of the standard and methodology used, the rated capacity of the hanger was regularly exceeded by the actual loads operating over the bridge in the years immediately preceding its collapse. Thus, this work serves to validate the hypothesis of the previous study that the hangers were indeed among the most vulnerable members of the bridge.

Acknowledgment

The authors are most grateful for the assistance provided by the Ontario & Western Railway Historical Society and its archivists, without whom this work would not have been possible.

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