Rehabilitation of Dearborn County Bridge 159

On George Street, Aurora in Dearborn County

Des. No.: 0401194

Project No.: BRO-9915( )

Design Computations

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Overview of Truss Analysis

The bridge has quite a history. It's made from wrought iron, not steel. In its 123-year life it has had several renovations, and has served different requirements. For many years it carried US 50, which now crosses Hogan Creek a few hundred feet upstream.

The most significant repair project was the 1987 rehabilitation project that replaced the entire deck system with members made from A588 steel. For historical accuracy, the depth and length and flange width of the floorbeams was made identical to the originals' dimensions. But the new deck system was designed for the current HS-20-44 design load. Most, but not all, of the truss deterioration was repaired, and the overall truss was brought up to an HS-20-44 postable load limit. Also, the deteriorated approach span, a steel truss of much different and less durable construction than the main span, was replaced with the present beam span. A588 steel was used for this span, also, and it was designed by the undersigned engineer for HS-20 loading.

Coupon samples were taken from the main span truss, and were tested at a testing lab to determine the safe allowable stress. These calculations are shown on a subsequent page. It might be noted that an additional 12.5% reduction was made to the allowable stress, in addition to the safety factors mandated by AASHTO.

In the course of our field measurements we checked the members for section loss due to rust. We were pleased that there has been only modest section loss. Wrought iron is resistant to rust (though not immune to it), and this bridge has been kept painted and cared for better than those in rural locations. We estimate that the section loss has been less than two percent on the eyebar tension members, and less than five percent on the major compression members. For this analysis and load rating we assumed 5% section loss for every member. We think this overestimates the loss.

The truss was modeled using a finite element stress analysis program. Many runs were made. We found that the new sidewalk, with its AASHTO mandated loading, was heavier on the upstream truss than the utility pipes were on the other end of each floorbeam. Subsequent runs used the sidewalk loads. Two details of the structure gave particular worry when setting up the models. These details, and our method of dealing with them, are described in the following three paragraphs.

The capacity of the truss verticals is determined by buckling, not by the material strength. The builders went to considerable trouble to install horizontal tie rods at mid-height of the verticals to brace them against buckling in their weakest direction. These rods are present and functioning, but this engineer was not totally certain they can be relied on to provide 100% of the desired support, and chose to be cautious on the matter. Therefore, we analyzed the truss twice; once

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assuming the rods provided support at mid-height, and again assuming the rods were not there at all. Then, we used a load limit in between, but closer to the "no bracing rods" results, which were always lower. We used a value 25% of the way from the "no bracing rods" answers to the "full support" answers.

Traditional truss analysis assumes that members are pin-connected at each joint. The upper chord is indeed made of pieces only one panel long. However, at each panel point, thick and robust splice plates connect the pieces. The upper chord is probably closer to a continuous member than to a string of pin-connected segments. It is difficult to tell for sure. But adding continuity adds redundancy, and that increases the strength of the bridge. It also affects stresses in other truss members in ways that are difficult to predict. Better to run the truss analysis both ways and compare. We did that, and used member load values 25% of the way from the pin-connected assumption (generally weaker) to the continuous top chord assumption (generally stronger). We view this as a conservative approach to a complex detail.

So there were four design conditions: upper chord pinned with verticals braced, upper chord pinned with verticals unbraced, upper chord continuous with verticals braced, and upper chord continuous with verticals unbraced. Interpolations were made between these results. Results are noted on a side view of truss on a following page. The truss easily rates 20 tons (plus) for a posted load limit. However the new floorbeams, stringers, and approach span girders were designed for only 20 tons. We are recommending a posted load limit of 15 tons. We simply choose not to work the bridge to its safe limit, but to reduce the fatigue factor so this historic and valuable structure can continue to serve many decades into the future. With proper maintenance, another century or two is not an unreasonable expectation.

James Barker, P.E.
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Determination of Allowable Stresses

In 1983, during design of the 1987-8 rehabilitation project, I had five samples taken from Bridge #159 and tested. Three were pieces from vertical members and two were from floorbeam webs. These were tested to ultimate failure by Alt & Witzig Engineering of Indianapolis. The work was their File Number T3205-1 of December, 1983. Their results were as follows.

Sample	Ultimate Stress (ksi)	Yield Stress (ksi)
L1	50.7	short plateau at approx. 36 ksi
L2	51.2	short plateau at approx. 36 ksi
V3	51.2	not recorded
F4	52.8	not recorded
F5	53.0	not recorded

The 36 ksi yield strength is right in the middle of the range of other, more recently tested wrought iron samples that had ultimate strengths around 50 ksi. However, in 1983 the testing lab personnel did not observe the yield point as carefully as I would now wish.

The mean of the ultimate strengths is 51.78 ksi, and the standard deviation for the five values is

$$\sqrt{\frac{\text{sum of the squares of (mean minus each value)}}{\text{(number of samples minus one)}}} = 1.05 \text{ ksi}$$

And 1.65 standard deviations is 1.72 ksi. Taking 36 ksi as the average yield stress, use 36 – 1.72 = 34.7 ksi as the usable yield stress.

Set Inventory allowable stress to be 55% x 34.7 ksi = 19.1 ksi
my own personal 12½ % extra safety factor → x 0.875
16.7 → round down to 16.5 ksi

Set Operating allowable stress to be 75% x 34.7 ksi = 26.0 ksi
my own personal 12½ % extra safety factor → x 0.875
22.8 → round down to 22.5 ksi

Thus, the allowable stresses used here for setting the load limit are 1/8 = 12½ % lower than the values that are recommended by the National Bridge Inspection Standards. However, they are above the default values recommended when coupon testing is not done (14.6 ksi for Inventory, 20 ksi Operating).

James Barker, P.E.
August 12, 2004