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Fatigue Evaluation of Stillwater Bridge (Bridge 4654)

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2.6 ksi

ksi

4.6

5

155

55155

5113

A vertical-lift bridge, the Stillwater Bridge, Bridge 4654, opened in 1931 across the St. Croix River between Minnesota and Wisconsin. To assess the remaining fatigue life of this bridge, strain gages were installed on an interior floorbeam and on a tension chord of a typical through truss span. The maximum stress range was 32 MPa at the centerline of the floorbeam. The measured data were rationalized by performing an analysis of the floor system and truss. The greatest ratio of the maximum expected stress range (18 MPa) to the fatigue strength (31 MPa) is at the centerline of the severely floorbeams located at the ends of the spans. Therefore, fatigue cracking is not expected in the steel members of a typical truss and taking the trucks off the bridge will have no significant effect on the fatigue life of the steel members in a typical through truss span.

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Fatigue Evaluation of Stillwater Bridge (Bridge 4654)

Final Report

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EXECUTIVE SUMMARY

The Stillwater Bridge, Bridge 4654, is a vertical-lift bridge opened in 1931 across the St. Croix River between Minnesota and Wisconsin. In order to assess the remaining fatigue life of this bridge, strain gages were installed on an interior floorbeam and on a tension chord of a typical through truss span. The strain gages were monitored while trucks with known axle weights crossed the bridge. The maximum stress range was 32 MPa 4.6ksi at the centerline of the floorbeam. The measured data were rationalized by performing an analysis of the floor system and truss. The measured stress ranges were typically about 85 percent of the predicted stress ranges, which is considered good agreement.

Based on these measurements and analyses, the maximum expected stress range at other locations in a typical bridge span can be estimated. The greatest ratio of the maximum expected stress range (18 MPa) 2.6ksi to the fatigue strength (31 MPa) 4.5ksi is at the centerline of the severely corroded floorbeams located at the ends of the spans. In contrast, the maximum expected stress range for the rivetted tension chord members is 12 MPa 1.7 ksi, which is 25 percent of the fatigue limit (48 MPa). Since the maximum ratio is only 8.41 ksi 58 percent, fatigue cracking is not expected in the steel members of a typical truss. However, since this ratio was the greatest at the severely corroded floorbeams at the ends of the spans, this location should be the focus of inspection and renewal efforts. (The lift span and the concrete slab approach spans on the west side were not included in this analysis and may possibly be more critical.) Therefore, taking the trucks off the bridge will have no significant effect on the fatigue life of the steel members in a typical through truss span.

CHAPTER 1

INTRODUCTION

The Stillwater Bridge, Bridge 4654, is a vertical-lift bridge across the St. Croix River between Minnesota and Wisconsin north of I-94. The Stillwater Bridge has been in use since 1931 and was posted with a weight limit in 1994 (Figure 1). This report describes the instrumentation and monitoring of this bridge to estimate the residual fatigue life of the bridge under present traffic and to assess the effect on the residual life of removing large trucks from the traffic.

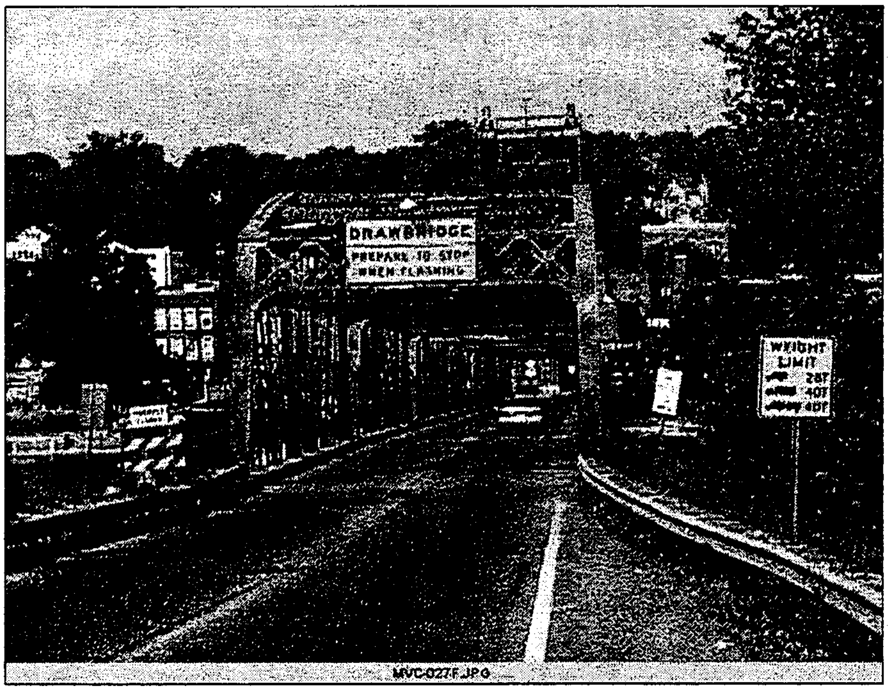


Figure 1: Bridge 4654 with Weight-limit Sign

The live-load stress ranges were expected to be highest in the floor system, with the stress in the truss dominated by dead loads. A visual inspection of the underside of the bridge was done noting the condition of the floorbeams, stringers and tension chords. The floorbeams and stringers are rolled shapes with high fatigue strength (Category A).

CHAPTER 2

DESCRIPTION OF BRIDGE AND TEST PROCEDURES

DESCRIPTION OF BRIDGE

The modern, long-span, high-rise vertical-lift bridge was first conceived and developed by John Alexander Low Waddell (1854-1938) in the last decade of the nineteenth century. In 1907, Waddell formed a partnership with John Lyle Harrington (1868-1942), who was largely responsible for reworking Waddell's invention into what is now known as the "Waddell and Harrington vertical lift". Six vertical-lift highway bridges were built in Minnesota and Wisconsin prior to World War II. At least Waddell and Harrington or successor firms designed five. All were of the Waddell and Harrington type. The Stillwater Bridge (Bridge 4654) is the only one that survives [2].

Bridge #4654 is 321 meters (1053 ft.) long and consists of ten spans. Spans #1 and #2 (counting from the west) are concrete slab spans and were not considered as part of this study. The river spans (#3 through #9) are "simple span" riveted steel through trusses. Span #4 lifts vertically. Span #10 is a short steel stringer span built into the east abutment [3].

Each of the truss spans have eight floorbeams which are designated "channel beams CB 30x115". The dimensions of these channel beams are essentially the same as the standard W30x116 rolled shape. Above the floorbeams are 11 stringers supporting the roadway. They are bolted to the top flange of the floorbeam [3].

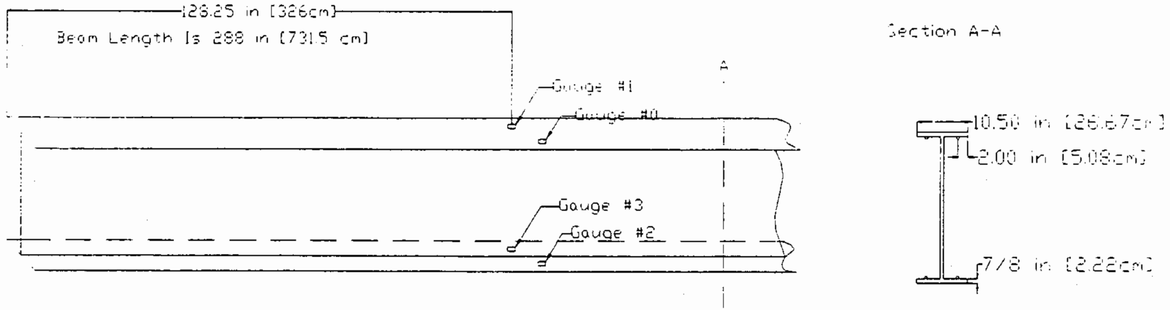


Figure 2a: Gauges 0-3 at center of floorbeam

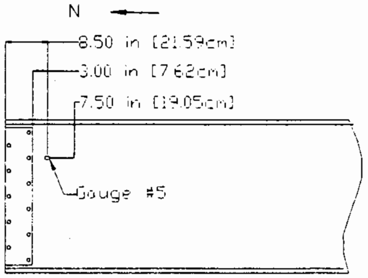


Figure 2b: Gauge 5 at end of floorbeam

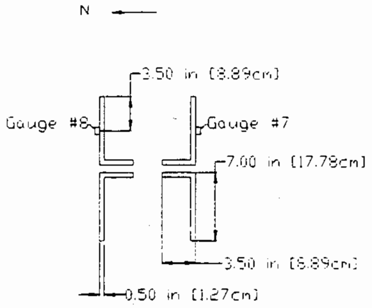


Figure 2c: Gauges 7 and 8; 70.5 in (179.07 cm) west of centerline of floorbeam

Figure 2: Strain-Gage Locations

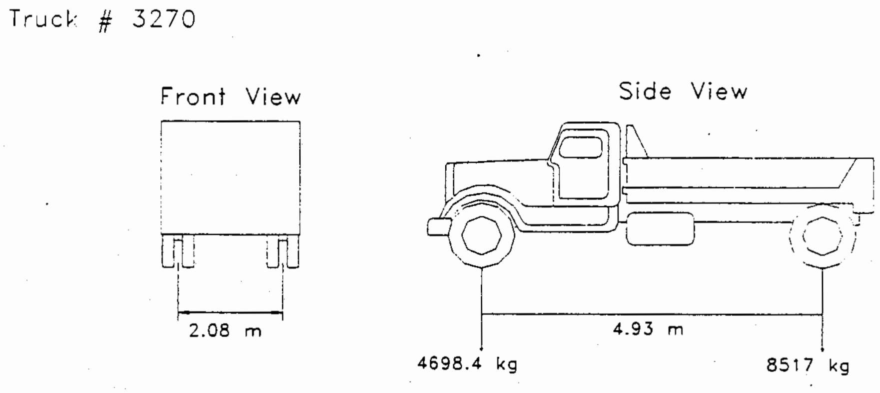


Figure 3a : MN/DOT truck #3270 unit weight = 13215.4 kg

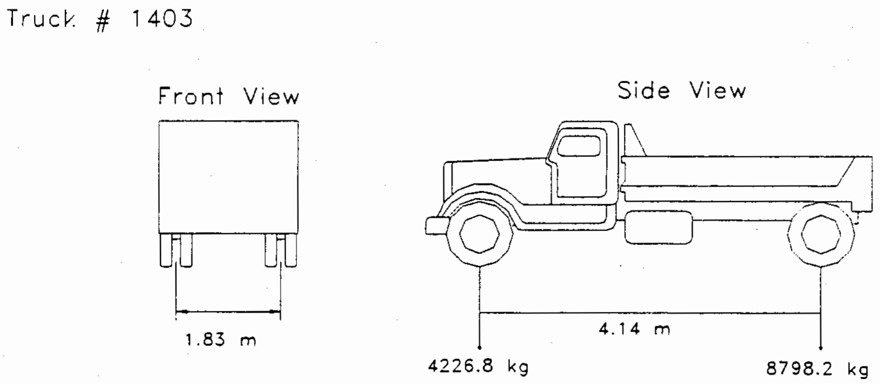


Figure 3b : MN/DOT truck #1403 unit weight = 13024.9 kg

Figure 3: Dimensions and Axle Weights of the Test Trucks

CHAPTER 3

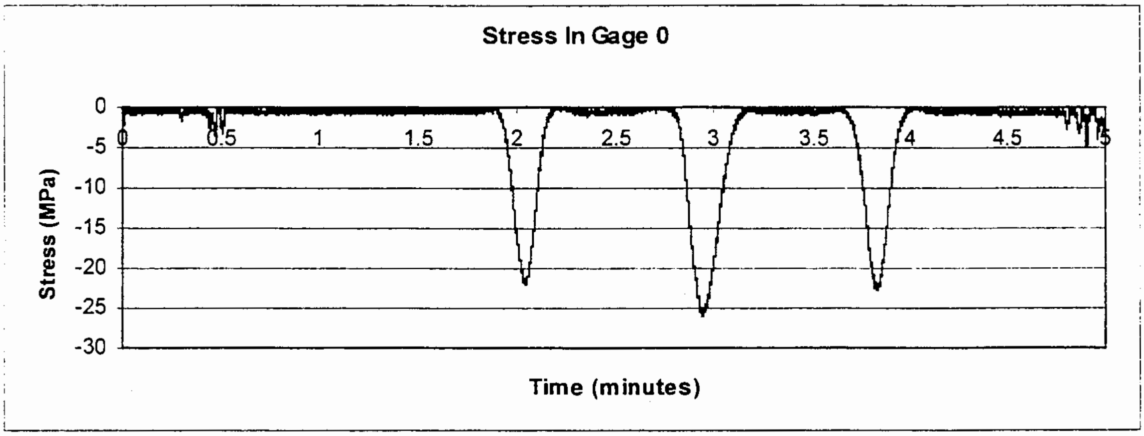
SUMMARY OF RESULTS

RESULTS OF STATIC TESTS

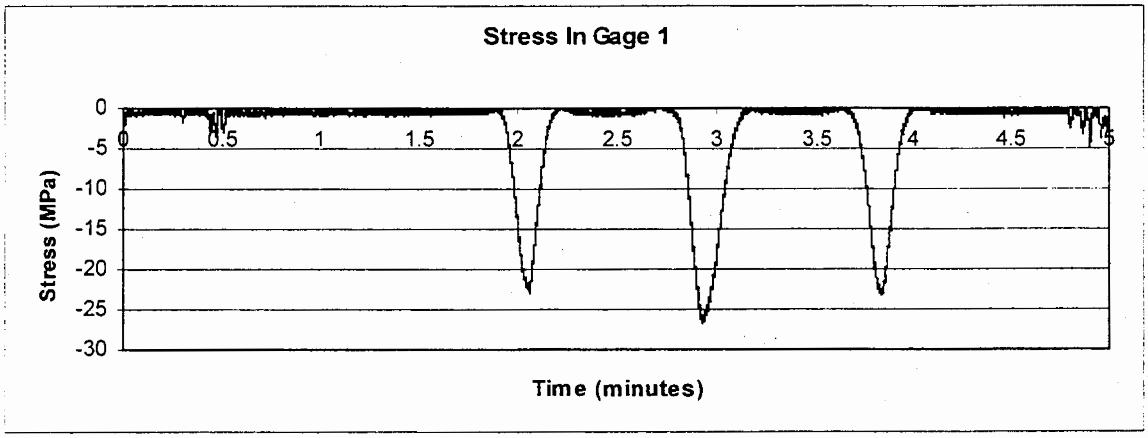
Figures 5a-i show five minute segments of the time histories of the strain gages during the static tests. The time scales are identical for all of these graphs in Figure 5. The peaks from each pass of the trucks line up on each page, indicating the peaks occurred at the same time. Gages 0 and 1 are on the top flange, while gages 2 and 3 are on the bottom flange. Since the pairs of gages on each flange give almost identical results, it is concluded that there is no weak axis bending of the floorbeam. Also, since the magnitudes of the stress peaks in compression in gages on the top flange are essentially equal to the magnitudes of the stress peaks in tension on the bottom flange, it can be concluded that the slab is not acting compositely with the beam for transverse bending.

All four gages show that the stress induced in the second peak is greater than in the first or third peak. The maximum stress range the floorbeam experiences during the second peak is 26 MPa. The trucks were backing up during the second peak, however there is no good explanation for the increase in stress range relative to the forward motion.

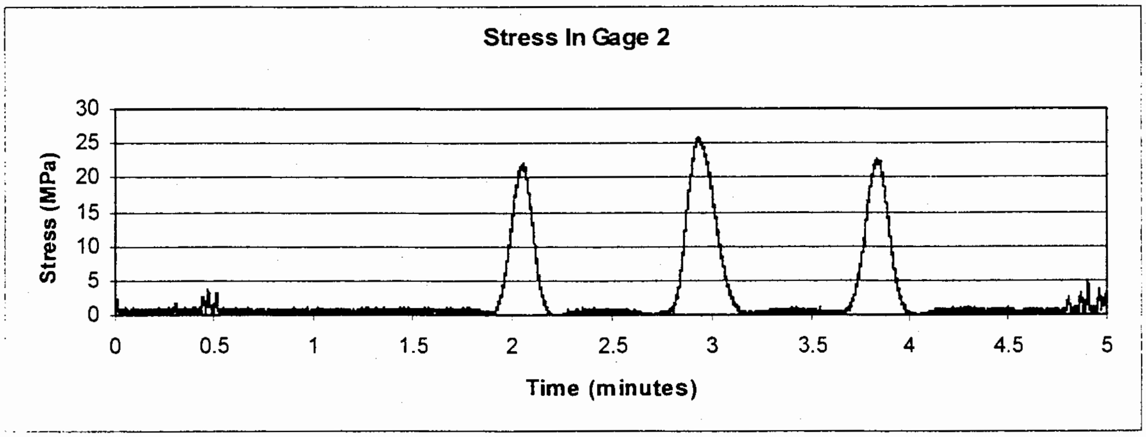
Because the floorbeam response is symmetric, the absolute values of gages 0-3 were averaged to determine a bending stress. This average bending stress will be the focus of the data reduction rather than describe all four gages individually. The average will further reduce noise and error. The average bending stress for the static test case is shown in Figure 5e.


Stress In Gage 0
Stress (MPa)
0
-5
-10
-15
-20
-25
-30
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Time (minutes)

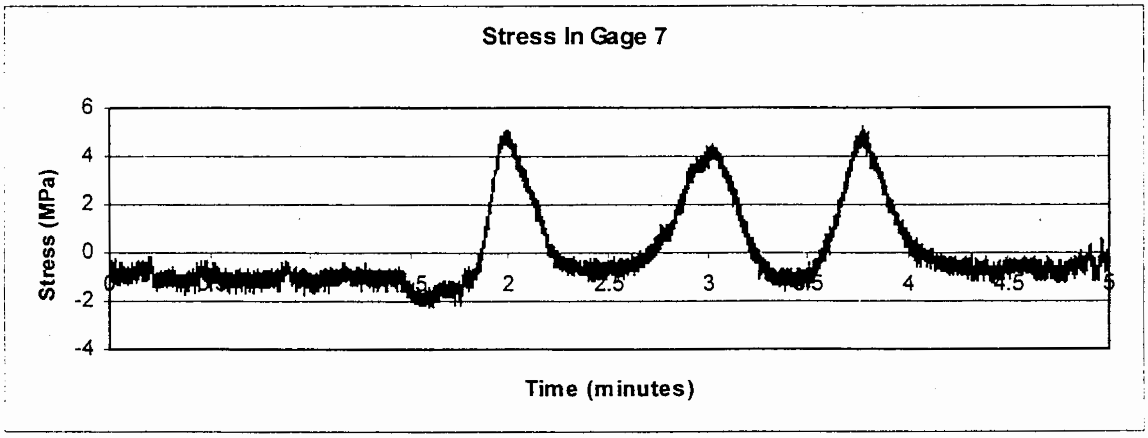
A


Stress In Gage 1
Stress (MPa)
0
-5
-10
-15
-20
-25
-30
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Time (minutes)

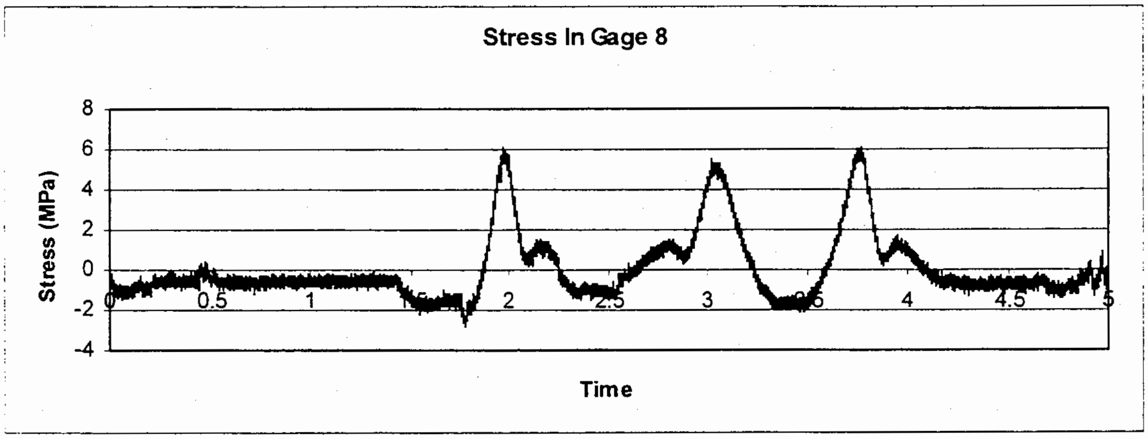
B


Stress In Gage 2
Stress (MPa)
30
25
20
15
10
5
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Time (minutes)

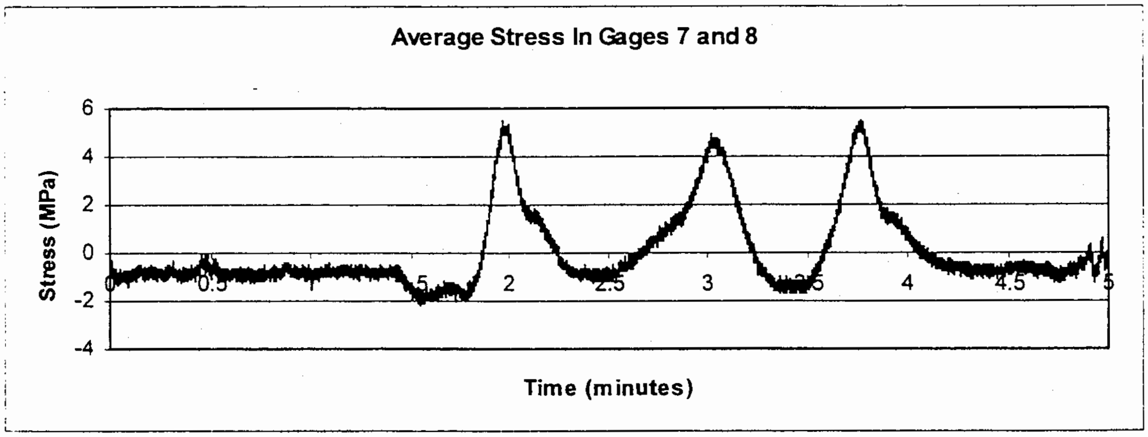
C


Stress In Gage 7

G

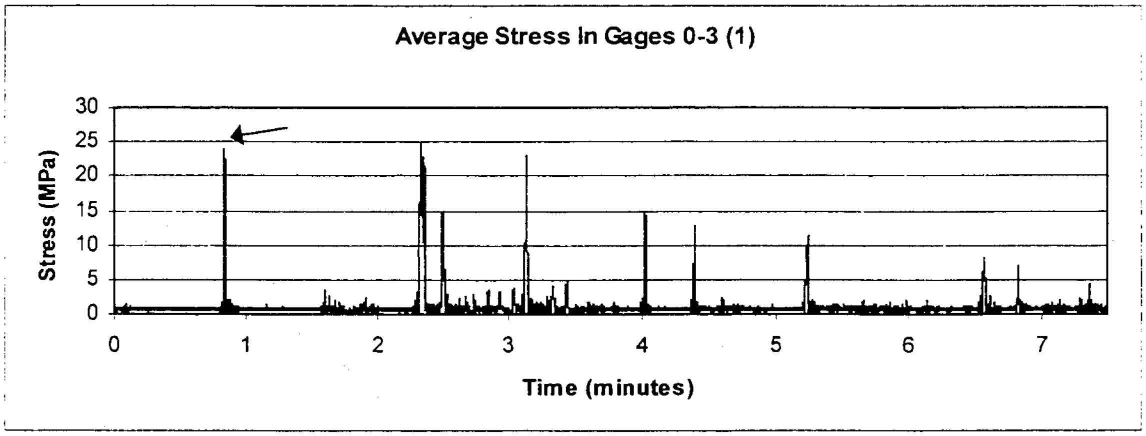

Stress In Gage 8

H

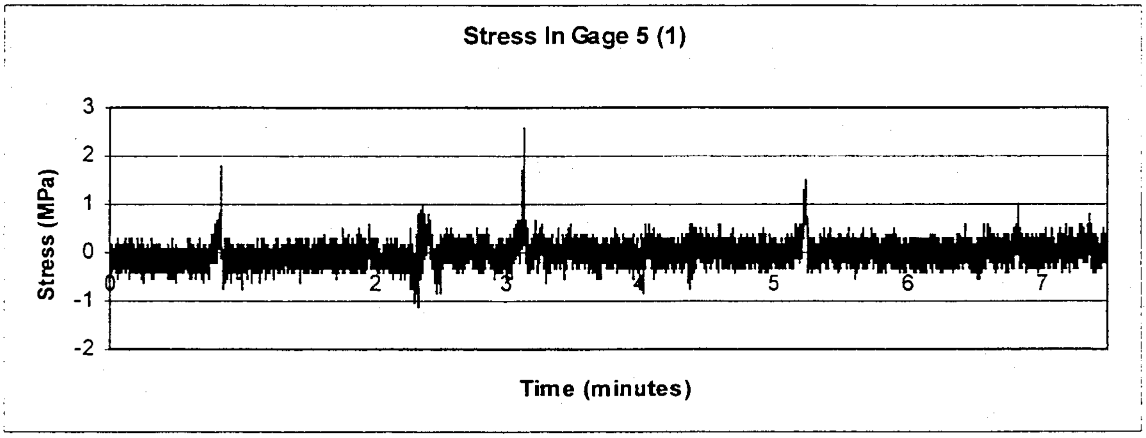

Average Stress In Gages 7 and 8

I

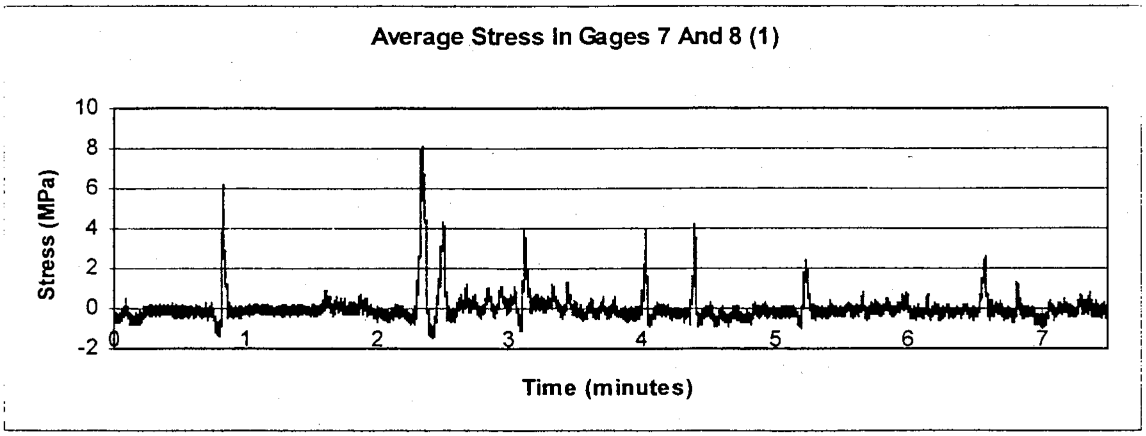
Figure 5: Time Histories of the Response during the Static Test


Average Stress In Gages 0-3 (1)
Stress (MPa)
30
25
20
15
10
5
0
0
1
2
3
4
5
6
7
Time (minutes)

A


Stress In Gage 5 (1)
Stress (MPa)
3
2
1
0
-1
-2
0
1
2
3
4
5
6
7
Time (minutes)

B


Average Stress In Gages 7 And 8 (1)
Stress (MPa)
10
8
6
4
2
0
-2
0
1
2
3
4
5
6
7
Time (minutes)

C

The five-minute stress histories for the single-truck dynamic test are shown in Figures 7a-7c. The two trucks passed over the floorbeam at about 4.0 and 4.6 minutes, respectively, as indicated with arrows on Figure 7a. The peak stress range recorded for gages 0-3 near the middle of the floorbeam is 18.4 MPa for the second truck.

Theoretically, the peak stress range from two trucks (26 MPa) should have been about twice this 18.4 MPa peak stress range from one truck. This discrepancy is discussed again with respect to the analysis in Chapter 4, where it is noted that the measured stress range for one truck is relatively close to the predicted stress range, whereas the measured stress range for two trucks is much less than predicted. Thus, the fact that the measured response for two trucks is less than twice the measured response for one truck indicates nonlinearity in the strain response to increasing load. This nonlinearity probably has to do with increasing significance of alternate load paths at the higher load levels. The increasing difference between the measured and predicted stress ranges indicate that these alternate load paths are not considered in the analysis.

The peak stress range in gage 5 near the end of the floorbeam was 2.5 MPa from the second truck. Note that the first truck caused no noticeable response at all in gage 5. This may indicate that the partial fixity in the end of the floorbeam may have slipped as the first truck passed over.

The peak stress range in the tension chord (gages 7 and 8) was 4 MPa, as compared to 6.2 MPa for the two-truck dynamic test. The ratio of the response to one truck to the response to two trucks was expected to be about 71 percent due to the load distribution between the two trusses.

Figures 8a-8c represent the stress histories for another single-truck dynamic test for which a two-by-four piece of lumber was placed transversely in the road to increase the effect of impact. The peak measured stress range near the middle of the floorbeam (gages 0-3) was 23.5 MPa which was 28 percent greater than the 18.4 MPa stress range measured for the single-truck dynamic test without the two-by-four. However, most of this increase can be attributed to the fact that the truck was driving down the centerline of the bridge for this test, not in one lane as in the single-truck dynamic test without the two-by-four.

The stress range in gage 5 near the end of the floorbeam was 1.3 MPa, less than the case without the two-by-four. Similarly, the 3.3 MPa stress range in the tension chord (gages 7 and 8) is also less than the case without the two-by-four. The stress ranges at these locations would be expected to be lower as a truck moved away from the instrumented side of the bridge toward the centerline, as opposed to the stress range near the middle of the floorbeam, which would increase.

It is concluded that the effect of the two-by-four was minimal.

RESULTS OF THE UNCONTROLLED (OPEN TRAFFIC) DYNAMIC TESTS

Since significant stress ranges occur primarily in the floorbeam, the data reduction was focussed on the bending stress in the floorbeam at midspan. In most of the recorded time histories of the bending stress during open traffic, there was none or just one stress cycle that exceeded 18 MPa. Figure 9a shows a time history of the bending stress including a big stress cycle caused by a five-axle semi-truck rolling over the floorbeam.

In the time history shown in Figure 9a, the maximum stress range was 24.6 MPa, which is one of the ten biggest stress cycles to be recorded during 55 hours of monitoring. (The largest stress range recorded during that period was 29 MPa.) The 24.6 MPa stress cycle shown in Figure 9a is also shown on an expanded time scale in Figure 9b. The two separate peaks in the graph show the effect of the front and rear axle groups going over the floorbeam. Also, one can count the peaks and valleys leading up to the first major peak to find that the natural frequency of the floorbeam is about 12 Hz. The amplitude of the bouncing at 12 Hz is relatively small compared to the magnitude of the total stress cycle, which is another indication that the effect of impact is minimal in the floorbeam.

Data files were imported into Excel spreadsheet and cycles were counted using an algorithm programmed in Visual Basic in Excel. The algorithm is based on the "mean-crossing" cycle counting method. This method counts a new cycle every time the stress crosses from below to above the mean. In the case of the live-load bending stress range in the floorbeam, the mean is essentially zero and the cycles are always in one direction. This simplifies the application of the algorithm.

To avoid counting thousands of small insignificant fluctuations in the stress as cycles, cycles were not counted until the stress increased above a cut-off stress, which was set at 2.1 MPa, which is less than 10 percent of the smallest fatigue limit (18 MPa for Category E'). It was found that this cut-off stress discriminated between cars, pick-up trucks, and sport utility vehicles, which result in stress ranges less than 2.1 MPa, and delivery vans, which resulted in stress ranges greater than 2.1 MPa.

The stress range associated with a cycle is the algebraic difference between the maximum peak of the stress value between incidents of crossing the cut-off stress and the minimum stress. Since the stress cycles were all in one direction, the minimum is essentially zero, and the stress range is also equal to the peak stress.

This method ignores the fluctuations that occur in a cycle. For example, the cycle in Figure 9b would be counted as one cycle with a range of 24.6 MPa. Note that after the peak, the stress declined to about 12 MPa and then increased again to about 20 MPa. This intermediate stress range of 8 MPa (from 12 to 20 MPa) is ignored. Other cycle counting methods, such as the rainflow algorithm, would count these intermediate cycles. However, the mean-crossing method gives a better one-to-one correspondence between stress cycles and trucks.

Altogether, 55 hours of data were analyzed and each stress range over the cut-off stress range of 2.1 MPa was tabulated. These stress ranges were sorted into discrete bins of 3.5 MPa intervals for each gage and for the average bending stress (average of gages 0-3). The counts of the stress ranges are shown in Tables 1a and 1b.

While the number of events at the high stress range levels for the average of gages 0-3 correlates strongly with those of the individual gages, we see a large disparity at the lower stress range levels. This is due to the significance of noise at that low stress range level. Therefore, it is believed that the values shown for the averages of gages 0-3 are the least affected by noise or error.

The total number of stress ranges exceeding 2.1 MPa during the 55 hour period was 11,888. This is an average rate of 5190 trucks per day in both directions. Typical traffic count data supplied by MnDOT indicate an average daily truck traffic (ADTT) of 2480 in one direction. Assuming similar traffic in the other direction the MnDOT data would suggest an ADTT of 4960 in both directions. The good agreement between the rate of stress cycles and the ADTT shows the cut-off is discriminating well between cars and trucks.

The distribution of trucks by vehicle type in the traffic count data is also qualitatively in agreement with the stress range distribution. For example, the traffic count data shows about 3.5 percent of the trucks are semi-trucks, and 2.3 percent have five or more axles. From Table 1b, it can be seen that only 490 stress ranges out of the 11,888 (4.1 percent) exceeded 7 MPa. It can be concluded that semi-trucks caused most of these stress ranges. Furthermore, only 2.0 percent of the measured stress cycles exceeded 10.5 MPa; it can be concluded that most of these stress cycles were caused by trucks with five or more axles. Only 1.1 percent of the stress cycles the ratio of the peak stress range to the rmc is sensitive to the cut-off stress range used in the cycle counting procedure.

The beam was assumed to be simply supported. The low stress range levels experienced at gage 5 verified that the fixity was about 13 percent of what would be expected for a rigid connection, as discussed in Section 3.1.

The moment diagrams were then constructed and were used along with Equation 1 to get the resulting stress range in the floorbeam at midspan and at the ends. The moment of inertia is represented in the equation by *I* and *y* is the gage's distance from the neutral axis of the beam.

$$\sigma = \frac{My}{I} \tag{1}$$

The calculations yielded stress ranges in the center of the floorbeam much higher than those measured in the field. From the comparable stress ranges shown in Table 2, it is seen that the ratio of the measured to the calculated stress ranges was about 65 percent for the static and dynamic stresses for two trucks; i.e. the measured stress ranges are about 35 percent less than the stress ranges expected by analysis.

This disparity can be explained in a number of ways. First our placement of the truck wheels actually became over-conservative. Depending on the distance of the inside tires from each truck to the centerline, 10 to 25 percent of the difference between the measured stress range and the stress range predicted by analysis can be accounted for.

Note that for a single truck, the ratio of the measured to the predicted stress range was much closer, about 85 percent. It can be concluded that each of the factors decreasing the response relative to the prediction are only half as effective when there is only one truck. In other words, the factors causing the deviation from the analysis are nonlinear. For example, the transfer of load from the floor slab to the truss without bending the floorbeam could be due to the slab contacting the floorbeam more towards the ends than in the middle, which could be made worse by curvature of the floorbeam.

The calculated stress ranges in Table 2 also show that the stress range at the centerline of the floorbeam is about 10 percent greater than the calculated stress range at the measurement location. Therefore, the maximum measured stress ranges should be increased about 10 percent to consider the maximum bending stress at the centerline of the floorbeam.

The analysis can be used to estimate the stress range due to other loads. For example, the fatigue rating procedure described in NCHRP Report 299 [4] suggests that an HS15 truck should be examined (240 kN gross vehicle weight (GVW) 53 kN axle load). The resulting calculated stress range at the centerline of the floorbeam would be 25 MPa, however the estimate of the actual stress range would be 85 percent of 25 MPa or 21 MPa. This stress range could be scaled for any other HS loading, for example 28 MPa would be estimated for an HS20 truck (320 kN GVW).

ANALYSIS OF THE TRUSS

Analysis for the tension chords was done with a simple truss analysis. Loads were applied along the length of the tension chord as calculated from the influence line. For the load case of both trucks, rear axles aligned, the calculated stress range in the chord was found to be 6.6 MPa. The maximum measured stress range of 5.5 MPa during the static test is 83 percent of this calculated stress range. The measured stress range was 6.2 MPa during the two-truck dynamic test, but this reflects about a 12 percent increase due to impact. For the case of one vehicle in the lane closest to the truss, the calculated stress range in the chord is 4.7 MPa, which agrees fairly well with the measured stress range of 3.3 MPa for the single-truck dynamic test.

For a hypothetical worst-case axle group (four axles) with a total weight of 222 kN, the maximum calculated stress range in the tension chord of the truss is about 10 MPa. Increasing this estimate by 12 percent to account for potential impact effects gives an estimate of the stress range in the tension chord as high as 11 MPa.

FATIGUE ASSESSMENT OF THE FLOORBEAM

In order for fatigue cracking to occur, the stress range in the floorbeam would have to exceed the fatigue limit. The fatigue limit for rolled sections without severe corrosion (Category A) is 165 MPa. Since the maximum calculated stress range is only 36 MPa, it can be concluded that the stress ranges do not exceed the fatigue limit for the floorbeams without severe corrosion.

The fatigue limit for severely corroded members (Category E) is only 31 MPa [1]. However, the only floorbeams with severe corrosion are at the bridge piers (under the open expansion joint) where there are two beams. Therefore, the maximum calculated stress range at this location would be only 18 MPa, less than 60 percent of the fatigue limit.

Occasionally there was a floorbeam with about 10 percent section loss due to corrosion, but the section loss occurred at the ends of the beam where the stress range is very small. Since the maximum calculated stress range remains much less than the fatigue limit for all cases for the floorbeam, fatigue cracking is not expected in the floorbeam.

5. The maximum allowable stress range (fatigue limit) for rolled sections without severe corrosion is 165 MPa. Therefore, the maximum expected stress range (36 MPa) is less than 22 percent of the fatigue limit.

6. The fatigue limit for severely corroded beams is 31 MPa. However, the only beams that were severely corroded were at the ends of the span where there were two floorbeams to share the load. Therefore, the maximum expected stress range would be 18 MPa, which is 58 percent of the fatigue limit. Since this is the greatest ratio of the fatigue strength, the focus of inspection and renewal efforts should be on these severely corroded floorbeams.

7. The fatigue limit for the rivetted tension chord members is 48 MPa. The maximum expected stress range is 11 MPa, which is less than 23 percent of the fatigue limit.

8. The distance between each floorbeam is only 6.1 meters. Therefore, a truck that would have a gross weight large enough to have a great affect on the floorbeam would also have a length longer than 6.1 meters and would disperse it's load among more than one floorbeam. Therefore, it is not likely that the fatigue limits will be exceeded.

9. Since the stress ranges do not exceed the fatigue limits in any of these members, it is concluded that the fatigue cracking should not occur during the remaining life of the bridge. Therefore, taking the trucks off the bridge will have no significant effect on prolonging the fatigue life of the structural steel. Note that the lift span and the concrete slab approach spans on the west side were not included in this analysis and could possibly be more critical.