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MINNESOTA DEPARTMENT OF TRANSPORTATION

Research

Fatigue Evaluation of the Deck Truss of Bridge 9340

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16. Abstract (Limit: 200 words)

This research project resulted in a new, accurate way to assess fatigue cracking on Bridge 9340 on I-35, which crosses the Mississippi River near downtown Minneapolis.

The research involved installation on both the main trusses and the floor truss to measure the live-load stress ranges. Researchers monitored the strain gages while trucks with known axle weights crossed the bridge under normal traffic. Researchers then developed two-and three-dimensional finite-element models of the bridge, and used the models to calculate the stress ranges throughout the deck truss.

The bridge's deck truss has not experienced fatigue cracking, but it has many poor fatigue details on the main truss and floor truss system. The research helped determine that the fatigue cracking of the deck truss is not likely, which means that the bridge should not have any problems with fatigue cracking in the foreseeable future.

As a result, Mn/DOT does not need to prematurely replace this bridge because of fatigue cracking, avoiding the high costs associated with such a large project.

The research also has implications for other bridges. The project verified that the use of strain gages at key locations combined with detailed analysis help predict the bridge's behavior. In addition, the instrumentation plan can be used in other similar bridges.

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FATIGUE EVALUATION OF THE DECK TRUSS OF BRIDGE 9340

Final Report

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EXECUTIVE SUMMARY

Bridge 9340 is a deck truss with steel multi-girder approach spans built in 1967 across the Mississippi River just east of downtown Minneapolis. The approach spans have exhibited several fatigue problems; primarily due to unanticipated out-of-plane distortion of the girders. Although fatigue cracking has not occurred in the deck truss, it has many poor fatigue details on the main truss and floor truss systems. Concern about fatigue cracking in the deck truss is heightened by a lack of redundancy in the main truss system. The detailed fatigue assessment in this report shows that fatigue cracking of the deck truss is not likely. Therefore, replacement of this bridge, and the associated very high cost, may be deferred.

Strain gages were installed on both the main trusses and the floor truss to measure the live-load stress ranges. The strain gages were monitored while trucks with known axle weights crossed the bridge and under normal traffic. Two- and three-dimensional finite-element models of the bridge were developed and calibrated based on the measured stress ranges. These finite-element models were used to calculate the stress ranges throughout the deck truss.

The peak stress ranges are less than the fatigue thresholds at all details. Therefore, fatigue cracking is not expected during the remaining useful life of the bridge. The most critical details, i.e. the details with the greatest ratios of peak stress range to the fatigue threshold, were in the floor trusses. Therefore, if fatigue problems were to develop due to a future increase in loading, the cracking would manifest in a floor truss first. Cracks in the floor trusses should be readily detectable since the floor trusses are easy to inspect from the catwalk. In the event that the cracks propagate undetected, the bridge could most likely tolerate the loss of a floor truss without collapse, whereas the failure of one of the two main trusses would be more critical.

This research has implications for bridges other than 9340. The research verified that the behavior of this type of bridge can be deduced with a modest number of strain gages at key locations combined with detailed analyses. This instrumentation plan can be used in other similar bridges. Guidelines for service-load-level analyses of similar bridges are given to estimate typical fatigue stress ranges. Bridges may now be rated for fatigue in accordance with the new Load and Resistance Factor Rating procedures. Fatigue rating should be based on service-load-level analyses conducted according to these guidelines. If the results of preliminary assessment indicate that there is still concern about fatigue, the analyses should be calibrated with limited strain-gage testing.

CHAPTER 1

INTRODUCTION

PROBLEM STATEMENT

Bridge 9340 supports four lanes in each direction (eight lanes total) of I-35W across the Mississippi River just east of downtown Minneapolis. The Average Daily Traffic (ADT) is given as 15,000 in each direction, with ten percent trucks. Bridge 9340 consists of a deck truss and steel multi-girder approach spans built in 1967. The deck truss, shown in Figure 1, has a center span of 139 meters, north and south spans of 80.8 meters and cantilever spans of 11.6 and 10.9 meters. The bridge was designed using the 1961 American Association of State Highway Officials (AASHO) Standard Specifications [1]. At that time, unconservative fatigue design provisions were used. The American Association of State Highway and Transportation Officials (AASHTO) fatigue design rules were substantially improved as a result of research at Lehigh University in the 1970's [2,3].

The approach spans have exhibited several fatigue problems; primarily due to unanticipated out-of-plane distortion of the girders. Although fatigue cracking has not occurred in the deck truss, it has many poor fatigue details on the main truss and floor truss systems.

Stress ranges calculated using the lane load as live load are greater than fatigue thresholds for many of the details. The poor fatigue details in the deck truss include intermittent fillet welds, welded longitudinal stiffeners and welded attachments at diaphragms inside tension members. These details are classified as Category D and E with threshold stress ranges 48 and 31 MPa, respectively.

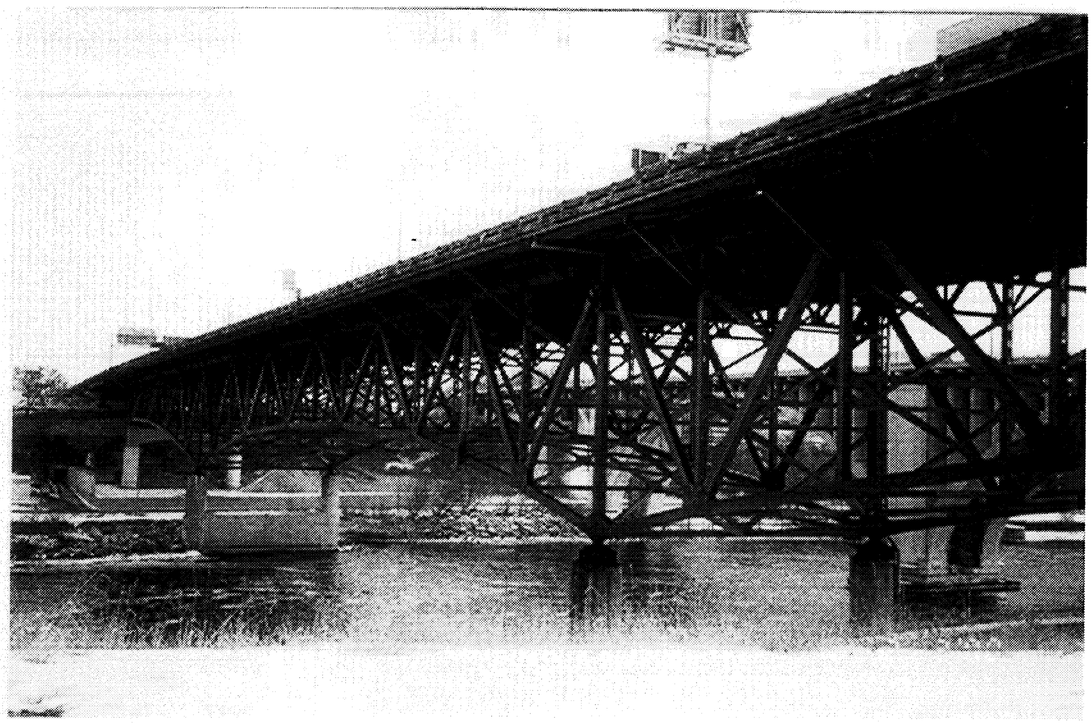


Figure 1: Bridge 9340

The design analysis, using the AASHTO lane load in all lanes, shows design-live-load stress ranges in the truss members much higher than these thresholds. Design-live-load stress ranges were greatest, up to 138 MPa, in members that experience load reversal as trucks pass from the outside spans onto the center span. The predicted average life at that stress range is between 20,000 and 40,000 cycles. With 15,000 trucks per day crossing the bridge in each direction, these details should have cracked soon after opening if the stress ranges were really this high.

The actual stress ranges can be determined by instrumenting the bridge with strain gages and monitoring strains under both a known load and open traffic. Fortunately, the actual stress ranges are much lower than these design live-load stress ranges. Consequently, the fatigue life is far longer than would be predicted based on the design-live-load stress ranges. The difference between actual and predicted stress ranges is the result of conservative assumptions made in the design process. The primary reason is that the traffic on the bridge is 90 percent cars and weighs a lot less than the lane loading, (9.34 kN/m). The lane loading is approximately equivalent to maximum legal 356 kN trucks spaced at about 38 meters apart.

The lane load may be appropriate for a few occurrences during the life when there are bumper-to-bumper trucks in all lanes, and the bridge should be designed to have sufficient strength to withstand this load. However, a few occurrences of loading of this magnitude would not have a significant effect on fatigue cracking. In fact, it has been shown that essentially infinite fatigue life is achieved in tests when fewer than 0.01 percent of stress ranges exceed the fatigue threshold [4]. Therefore, only loads that occur more frequently than 0.01 percent of the time have an effect on fatigue. If there are 15,000 significant load cycles (trucks) per day, any load that happens less frequently than daily is irrelevant as far as fatigue is concerned. In observing this bridge closely over the period of more than a year, the authors have never seen a condition where there were closely spaced trucks in each lane.

Other reasons that the actual live-load stress ranges are lower than design stress ranges include unanticipated structural behavior at service load levels. This unanticipated behavior includes composite action of the slab and the floor trusses and unintended partial fixity at the piers due to bearings that do not respond to live loads.

Concern about fatigue cracking in the deck truss is heightened by a lack of redundancy in the main truss system. Only two planes of the main trusses support the eight lanes of traffic. The truss is determinate and the joints are theoretically pinned. Therefore, if one member were severed by a fatigue crack, that plane of the main truss would, theoretically, collapse.

However, it is possible that collapse may not occur if this happened. Loads may be redistributed and joints may resist rotation and develop bending moments. If the fractured main truss deflected significantly the slab could prevent the complete collapse through catenary action. In any event, a fracture in one of the main trusses would require prolonged closure of the bridge and a major disruption.

OBJECTIVE OF RESEARCH

This research was conducted to:

1) characterize the actual statistical distribution of the stress ranges;
2) evaluate the potential for fatigue cracking in the deck truss and, if there is the potential for cracking, to estimate the remaining life;
3) recommend increased inspection or retrofitting, if necessary.

SCOPE OF REPORT

This report covers a literature review, inspection of the deck truss, field-testing and analysis of the deck truss, and discussion of the results. There is a brief discussion of previous problems with the approach spans, otherwise the approach spans are not discussed in detail.

The bridge was instrumented with strain gages, load tested with dump trucks with known axle weights in early October of 1999, and monitored on and off from March to August of 2000 to characterize the statistical distribution of the stress ranges. The measured strains were used to calibrate two and three-dimensional finite-element models of the bridge. These finite-element models were used to calculate the stress ranges throughout the deck truss. These stress ranges were compared to the thresholds for the particular details at each critical location. The most critical details, i.e. the details with the greatest ratios of peak stress range to the fatigue threshold, were identified. Recommendations are made for focused visual inspection.

CHAPTER 2

BACKGROUND

FATIGUE RESISTANCE

The American Association of State Highway and Transportation Officials (AASHTO) bridge design specifications (both the Standard Specifications and the Load and Resistance Factor Design (LRFD) Specifications) contain similar provisions for the fatigue design of welded details on steel bridges [5,6]. Welded and bolted details are designed based on the nominal stress range rather than the local "concentrated" stress at the weld detail. The nominal stress is usually obtained from standard design equations for bending and axial stress and does not include the effect of stress concentrations of welds and attachments. Since fatigue is typically only a serviceability problem, fatigue design is carried out using service loads. Although cracks can form in structures cycled in compression, they arrest and are not structurally significant. Therefore, only members or connections for which the stress cycle is at least partially in tension need to be assessed.

Both AASHTO bridge specifications are based on the same set of fatigue-resistance curves (S-N curves). The relationship used to represent the S-N curve is an exponential equation of the form:

$$N = A\,S^{-3} \qquad \text{(Eq. 1)}$$

$$\text{or} \quad \log N = \log A - 3 * \log S$$

where: N = number of cycles to failure,

A = constant dependent on detail category

and S = applied constant amplitude stress range.

In design, the S-N curves give the allowable stress range for particular details for the specified life or number of cycles. In evaluation of existing bridges, these S-N curves can be used to estimate of the total number of cycles to fatigue failure for the actual measured stress range at a particular detail. The remaining life can be estimated by subtracting from the total cycles the cycles experienced in the past.

Each S-N curve represents a category of details. The AASHTO specifications present seven S-N curves for seven categories of weld details, Although E', in order of decreasing fatigue strength. Figure 2 shows the S-N curves for the detail categories C, D, E, and E'. (The categories A, B, and B' are usually not severe enough to cause cracking in service and therefore will not be discussed.) The S-N curves are based on a lower bound to a large number of full-scale fatigue test data with a 97.5 percent survival limit. Therefore, a detail optimally designed with these S-N curves and actually exposed to the stress ranges assumed in design has a 2.5 percent probability of cracking during the specified lifetime.

Figure 2 shows the fatigue threshold or constant amplitude fatigue limits (CAFL) for each category as horizontal dashed lines. When constant-amplitude tests are performed at stress ranges below the CAFL, noticeable cracking does not occur. For bridges in service, if almost all the stress ranges are below the CAFL, the fatigue life is considered essentially infinite. The CAFL for Category C, D and E is 69, 48, and 31 MPa, respectively.

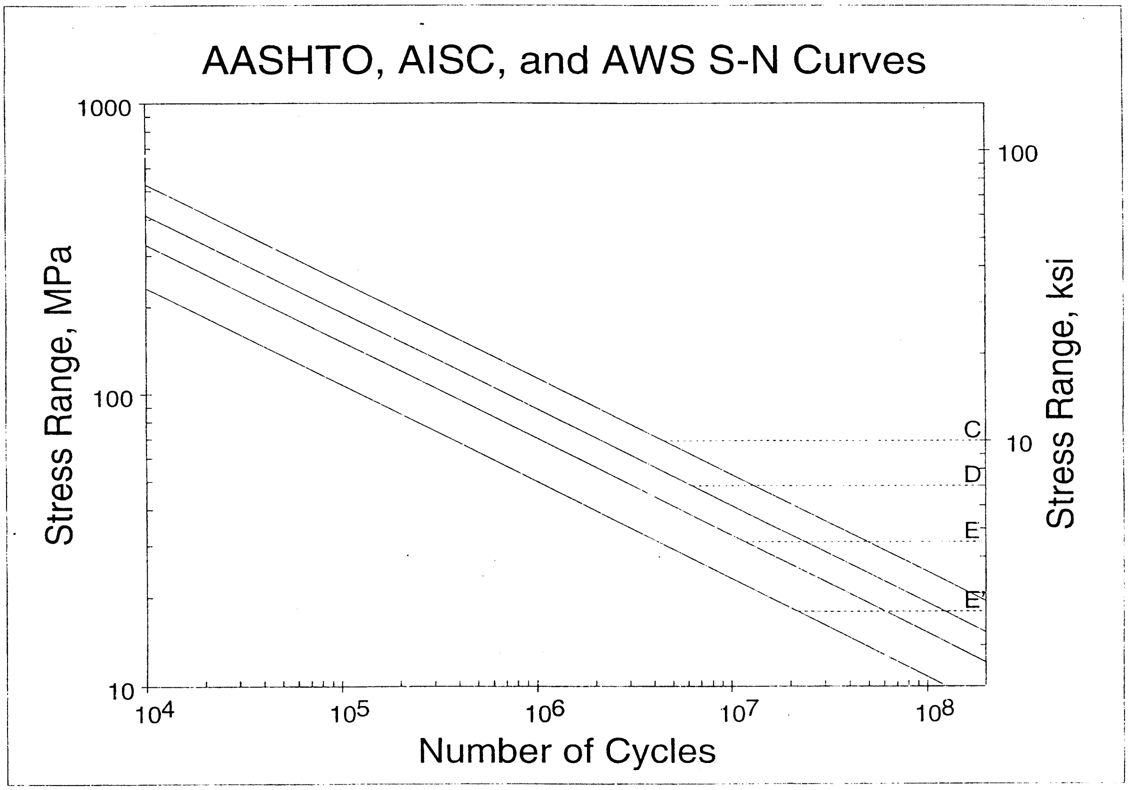


Figure 2: AASHTO Fatigue Resistance Curves

The critical details on Bridge 9340 are classified as non-load-bearing attachment details, i.e. attachments to structural members that do not carry significant load. With the exception of some special cases, these type of attachments are rated Category C if less than 51 mm long in the direction of the primary stress range, D if between 51 and 101 mm long, and E if greater than 101 mm long.

STRUCTURAL REDUNDANCY

In any structural system, loads are carried along a variety of simultaneous paths. The existence of these redundant load paths in a bridge ensures reliable structural behavior in instances of damage to some of the structural elements [7]. However, is there is no redundancy, failure of one member may cause the entire structure to collapse.

The Committee on Redundancy of Flexural Systems conducted a survey of steel highway and railroad bridges reported suffering distress in main load carrying members. Twenty-nine states and six railroad companies responded. A total of 96 structures were reported as suffering some distress. The survey found that most failures were related to connections, nearly all of which were welded. The data collected on bridges that suffered damage indicate that few steel bridges collapse if redundancy is present. The reported collapses involved trusses with essentially no redundancy [7].

In another study, Ressler and Daniels [8] found that the number of fatigue-sensitive details present in the structure significantly affected the system reliability of a nonredundant bridge. For example, the reliability of a span with 20 Category E' details was found to be substantially lower than the reliability associated with a single E' detail.

CALCULATED AND ACTUAL BRIDGE RESPONSE

Many studies have shown that the simplified calculations used to predict stresses in bridge members are inherently conservative [9,10,11,12,13,14,15,16]. As a result, the calculated stresses are often much higher than the actual service stresses and the fatigue assessment is unnecessarily pessimistic. From the form of Equation 1, it is clear that a small change in the estimate of the stress range results in a much larger change in the life, i.e. the effect is cubed. For example, if the stress range is conservative by only 20 percent, the computed life will be 42 percent too low.

The design calculations, load models, and the level of conservatism are appropriate for strength design where there is great uncertainty in the maximum lifetime loads. However, for fatigue evaluation of an existing bridge, an accurate estimate of the typical everyday stress ranges is required. Therefore, for fatigue evaluation of existing bridges, a more appropriate set of analysis assumptions is required and it is best if the analysis is "calibrated" relative to measured strain data.

In a large bridge, service live-load stress ranges typically do not exceed 20 MPa [10]. The stress ranges are small because the dimensions of the members of a large bridge are typically governed by dead loads and strength design considerations. Since the strength design must account for a single worst-case loading scenario over the life of the bridge, conservative load models are used (large factors of safety).

In addition to conservative load models, assumptions in analysis can also often lead to actual stresses being far lower than predicted stresses. An example of the effect of these assumptions is illustrated in a study of U.S. Highway 69 in Oklahoma crossing the South Canadian River [11]. Concerns of fatigue damage arose when poor welding techniques had been used in the widening of the bridge. Preliminary analyses had shown that stress ranges could exceed allowable stress ranges at over 100 locations on the bridge. However, when the bridge was instrumented with strain gages and monitored under known loads and normal traffic the largest measured stress range was found to be 27 percent of the allowable stress range, far below predicted.

In another study, fatigue concerns arose due to a considerable amount of corrosion on the floorbeams of Bridge 4654 in Minnesota [12]. The bridge was instrumented with strain gages and monitored under known loads and normal traffic. Here, measured stress ranges ranged from 65 to 85 percent of those predicted by analysis.

These disparities are due to the fact that analytical models often use assumptions that conservatively neglect ways in which the structure resists load. Sometimes the structural behavior could never have been predicted in design. For example, Dexter and Fisher [13] discuss the results of field tests on an adjacent pair of railroad bridges. It was found that ballast had fallen in the narrow space between the girders forcing the adjacent bridges to deflect together as if joined. This behavior distributed load from the bridge with the train on it to the other bridge, resulting is stress ranges less than half of predicted, especially in the exterior girder nearest the adjacent bridge.

In a study performed by Brudette et al. [14], more than 50 years of bridge test data were collected and examined to determine specific load-resisting mechanisms that are typically ignored in design or evaluation. The study revealed that lower stress ranges in a structure can be attributed to unintended composite action, contributions from non-structural elements such as parapets, unintended partial end fixity at abutments, and direct transfer of load through the slab to the supports.

- Composite Action: Bridges with shear connectors at the slab-girder interface typically display full composite action. However, some composite action is seen in the absence of shear connectors, resulting in lower stresses in the structure. At service load levels, composite action is even effective in resisting negative moment.

- Partial End Fixity: Often, bridges and bridge members are designed to behave as if they are simply supported. However, these supports usually do not behave as intended. Partial fixity in the end connections on beams causes a lower positive moment that would be obtained from the simply supported beam model. Bearings that are meant to be a roller boundary condition, or fixing the displacement in the vertical direction while allowing longitudinal movement, can become frozen due to corrosion, extremely cold weather or poor design. This can change the response of a bridge subjected to loading by introducing horizontal resistance where it was not intended.

- Transfer of Load Through Slab: Load distribution refers to the lateral distribution of load to longitudinal supporting elements. The slab typically does a much better job of spreading the load than anticipated in design. The lateral distribution is more favorable than assumed, and there is significant spreading of the load longitudinally, which is not even counted on in design. Often, part of the load is distributed directly to the supports bypassing the longitudinal stringers or girders.

In a similar study, the Ministry of Transportation of Ontario conducted a program of bridge testing that included more than 225 bridges over a period of many years [15]. The study revealed that in every bridge test there were surprising results that were not expected the most common of which was a bridge's ability to sustain much larger loads that their estimated capacities.

Specifically, the following observations were made in the testing of steel truss bridges.

- The stringers of the floor system sustained a large share of the tensile force thus reducing the strains felt by the chord in contact with the floor system.

- Again, composite action in non-composite systems was shown to exist. However, subsequent tests showed that this composite action breaks down completely as the failure limit state for the girder is approached [16].

Although these unintended structural behaviors are nearly impossible to model, they often combine to produce actual stresses well below those calculated by simplified design calculations or even finite-element analysis of the idealized structure [10].

To calibrate the analysis, the results are compared to the measured response and changes are made in the model until the results agree reasonably well with the measurements. Strain gage data are typically acquired on several bridge members where maximum stress ranges are expected to occur. Measurements are typically made while a truck or multiple trucks of known weight and configuration traverse the bridge in the absence of other traffic. The results from this test eliminate uncertainty in the load and isolate the part of the error due to the analysis. The analysis is linear, so once it is calibrated it can be used to predict the stress ranges from the maximum legal load, permit loads, or groups of trucks as appropriate for the fatigue analysis.

Often, some measurements are also made in open traffic for several days to characterize the statistical distribution of the topical stress ranges, which is proportional to the statistical distribution of the truck axle weights or total gross weights. Some members (e.g. floorbeams) are loaded by each truck axle. The members of a large trusses such as bridge 9340 do not respond to each axle load separately but rather respond with one cycle associated with the gross vehicle weight.) In highway bridges, a two or three day period seems to be satisfactory to capture a realistic representation of stress ranges and their respective frequencies [17]. It is best if the data collection system is left running continuously to capture both day and night traffic with both full and empty trucks. It may also be wise to capture seasonal changes in traffic and the response of the bridge by taking data in two or three day periods at various times of the year.

Once strain data at known locations has been accumulated, a finite element model of the bridge is generated. The model must be created with as much accuracy as possible before calibration begins. The model is then calibrated by adjusting: 1) the amount of composite action in members near the deck; 2) the fixity of the supports; and, 3) the distribution of loads on the deck; until calculated strains match measured strains. Once the model is calibrated by a limited number of measurements, it can be used to calculate strains throughout the bridge.

FATIGUE EVALUATION PROCEDURES

An actual service load history is likely to consist of cycles with a variety of different load ranges, i.e., variable-amplitude loading [4]. However, the S-N curves shown in Figure 2 are based on constant-amplitude loading. There is an accepted procedure for converting variable stress ranges to an equivalent constant-amplitude stress range with the same number of cycles. This procedure is based on the damage summation rule jointly credited to Palmgren and Miner (referred to as Miner's rule) [18]. If the slope of the S-N curve is equal to three, then the relative damage of stress ranges is proportional to the cube of stress range. Therefore, the effective stress range is equal to the cube root of the mean cube of the stress ranges [19].

$$S_{re} = (\Sigma\, p_i\, S_{ri}^{3})^{1/3} \qquad \text{(Eq. 2)}$$

The effective stress range is used the same way as the constant amplitude stress range, i.e. the S-N curve is entered with the value of the effective stress range and the intersection with the S-N curve defines the number of cycles in the total life, assuming that the effective stress range is relatively constant over the life. This procedure works fairly well in the shorter life regime where the effective stress range is much larger than the fatigue threshold.

When the effective stress range is on the order of the fatigue threshold or less, dealing with variable stress ranges becomes more complicated. Figure 3 shows the lower part of an S-N curve with three possible variable stress-range distributions superposed [20]. The effective stress range is shown as S_{re} in this figure and is used the same way as a constant-amplitude stress range with the S-N curves in the finite-life regime (Case 1 and Case 2).

For Case 3 in Figure 3, essentially all the stress ranges are less than the CAFL. In this case, long-life variable-amplitude fatigue tests on full-scale girders with welded details show that if less than one in 10,000 cycles exceed the CAFL, then essentially infinite life is obtained [4]. This phenomenon is the basis of what is called the "infinite-life" approach, which is incorporated in the AASHTO LRFD specifications [5].

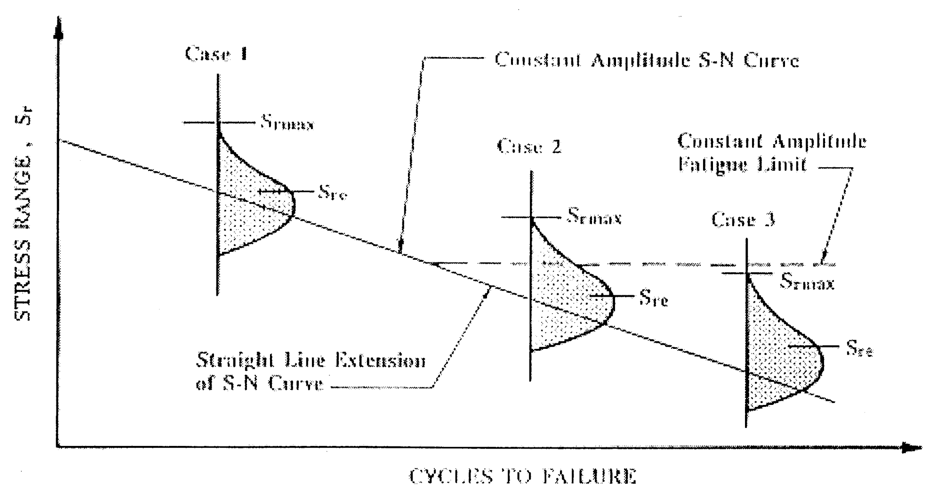


Figure 3: Possible Cases of S_{re} and S_{rmax} in Relation to the CAFL

Guide Specifications for the Fatigue Evaluation of Existing Bridges

Fatigue evaluation procedures for existing steel bridges were developed in a project sponsored by the National Cooperative Highway Research Program (NCHRP) that resulted in Report 299 [10]. This study was done to develop practical procedures that accurately reflect the actual fatigue conditions in steel bridges, which could be applied for evaluation of existing bridges or design of new bridges. The procedures utilized information gained from several years of research on variable-amplitude fatigue behavior, high-cycle, long-life fatigue behavior, actual traffic loadings, load distribution, and assessment of material properties and structural conditions.

In NCHRP 299, it is stated that fatigue checks should be based on typical conditions that occur in the structure, rather then the worst conditions expected to occur as in a strength design. The procedure begins with determination of a nominal stress range for the truck traffic crossing the bridge. This stress range is then compared to the S-N curve for the type of detail found on the structure to determine the number of cycles to failure. Then the life of the detail can be calculated using current estimated truck volume, the present age of the bridge, and the number of load cycles for each truck passage.

NCHRP report 299 provides the following equation to calculate fatigue life for an estimated lifetime average daily truck volume based on stress range measurements taken at the bridge site.

$$Y_f = [(f\,K \times 10^6) / (T_a\,C\,(R_s\,S_{re})^b)] - a \qquad \text{(Eq. 3)}$$

where,

Y_f = remaining fatigue life in years

S_{re} = effective stress range

R_s = reliability factor

C = stress cycles per truck passage

K, b, and f = fatigue curve constants

T_a = estimated lifetime average daily truck volume

a = present age of bridge in years

Further discussion of these variables follows.

Effective Stress Range

The effective stress range is calculated from Equation 2 using stress-range histograms obtained from field measurements on the bridge under normal traffic. The stress range may be computed from an analysis where the loading is the cube root of the mean cube of the gross-vehicle-weight histogram. Alternatively, an HS-15 truck (HS-20 loading multiplied by 0.75) may be used to calculate the effective stress range if measurements are not available.

Reliability Factor (R_s)

The reliability is used when calculating the remaining safe life. It is used to ensure that the actual life will exceed the safe life to a desired probability. When calculating the remaining mean life, the reliability factor is 1.0. When calculating the remaining safe life, multiply the computed stress range S_{re} by a reliability factor:

$$R_s = R_{so}\,(F_{s1})\,(F_{s2})\,(F_{s3}) \qquad \text{(Eq. 4)}$$

where,

R_s = reliability factor associated with calculation of stress range

R_{so} = basic reliability factor

= 1.35 for redundant members

= 1.75 for nonredundant members

F_{s1} = 0.85 if effective stress range calculated from stress range histograms obtained from field measurements

= 1.0 if effective stress range calculated by other methods

F_{s2} = 0.95 if loads used in computations are for site-specific weigh-in-motion measurements

= 1.0 if the AASHTO fatigue truck is used

F_{s3} = 0.96 if rigorous analytical method is used to determine load distribution

= 1.0 if approximate method based on parametric studies is used

Stress Cycles Per Truck Passage (C)

A single truck traveling over a bridge can often have a complex response resulting in more than one stress cycle per truck passage. Whereas most main members feel just one cycle per truck, transverse members near the deck may feel each axle load as it passes. The number of stress cycles per truck passage, C, has been determined for various types of bridge members. The number of stress cycles per passage for Bridge 9340, a deck truss bridge, is 1.0.

Fatigue Curve Constants (K, b and f)

The equation for the S-N curves was given in Equation 1. The parameter b is the exponent and is 3.0 for the AASHTO S-N curves. For convenience in calculating the remaining life in years, the detail constant K is used (Eq. 5).

$$K = A / [365 \times 10^6] \qquad \text{(Eq. 5)}$$

Where A was defined for Equation 1. There is considerable scatter in the fatigue data on which Eq. 4 is based. It is normally assumed that the scatter in stress range values follows a log-normal statistical distribution for a given N. Consequently, allowable nominal stress ranges are usually defined two-standard deviations below the mean stress ranges. Since the mean and allowable S-N curves for a given detail are assumed to be parallel on a log-log plot, the ratio of stress ranges for the two curves is the same at all cyclic lives [10].

The constant f is used to modify the constant K to reflect the mean remaining life rather that the safe remaining life. The constant f equals the ratio of the mean-life curve intercept, A', to the safe-life curve intercept, A. For categories B through E', the ratio of mean to allowable stress range does not vary greatly and averages 1.243. Because of the power of 3 in the S-N curve, the corresponding ratio of mean to safe lives is equal to 1.243 cubed, or 1.92. Thus, the value of f is taken as 2.0 while calculating mean life. If the safe life is being calculated, f equals 1.0 [10].

Lifetime Average Daily Truck Volume (T_a)

The present average daily truck volume in the outer lane, T, can be calculated from the ADT at the site as follows:

$$T = (ADT)\, F_T\, F_L \qquad \text{(Eq. 6)}$$

where

ADT = present average daily traffic volume in both directions

F_T = fraction of trucks in the traffic

F_L = fraction of trucks in the outer lane

The ADT can be determined by doing a traffic count or may be obtained from Department of Transportation data for the location of interest. The fraction of trucks in the traffic is suggested to be 0.20 for rural interstate highways, 0.15 for rural highways and urban interstate highways, and 0.10 for urban highways. The fraction of trucks in the outer lane may be determined from Table 1.

Table 1: Fraction of Trucks in Outer Lane [10]

Number of Lanes	2-Way Traffic	1-Way Traffic
1	-	1.00
2	0.60	0.85
3	0.50	0.80
4	0.45	0.80
5	0.45	0.80
6 or more	0.40	0.80

Using the calculated present average daily truck volume in the outer lane, T, the annual growth rate, g, the present age of the bridge, a, and Figure 4, the lifetime average daily truck volume in the outer lane can be determined. The annual growth rate can be determined from Table 2. This table lists annual growth rates estimated from Annual Average Daily Traffic (AADT) data taken at counting stations throughout the United States between the years 1938 and 1985.

Table 2: Observed Average Daily Traffic Growth Rates [10]

Type of Highway	Rural or Urban	Growth Rate %
Interstate	Rural	4.45
	Urban	4.98
U.S Route	Rural	2.87
	Urban	4.19
State Route	Rural	3.77
	Urban	3.27

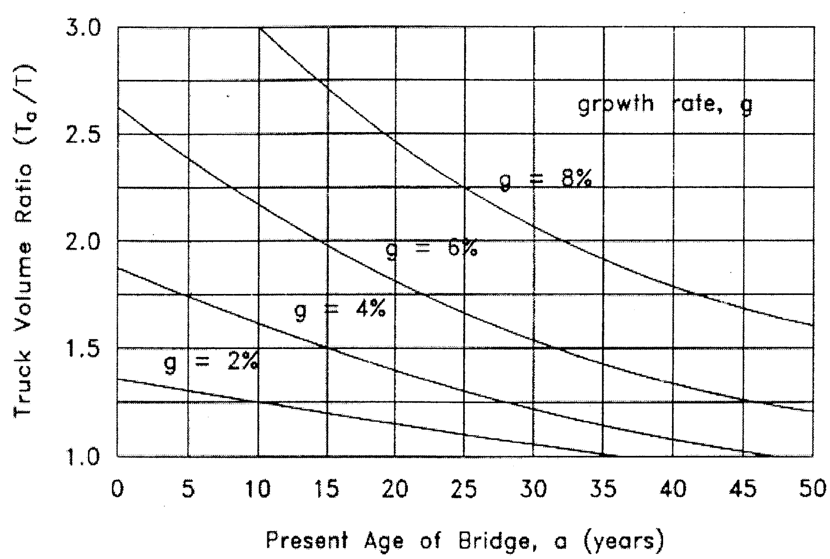


Figure 4: Truck Volume Ratio (T_a / T) [10]

CHAPTER 3

DESCRIPTION, DESIGN, AND HISTORY OF BRIDGE 9340

DESCRIPTION OF BRIDGE

Bridge 9340 carries I-35W over the Mississippi River just east of downtown Minneapolis. Constructed in 1967, the 581 meter long bridge has 14 spans. The south approach spans (Spans #1-#5) are steel multi-beam. The main spans (Spans #6-8) consist of a steel deck truss. The north approach spans include both steel multi-beam (Spans #9-#11) and concrete slab span (Spans #12-14).

There are two steel deck trusses. Most of the truss members are comprised of built-up plates (riveted) while some of the diagonal and vertical members are rolled I-beams. The connections include both rivets and bolts. The truss members have numerous poor welding details. Recent inspection reports have noted corrosion at the floorbeam and sway brace connections, and pack rust forming between connection plates [21].

The bridge deck above the deck truss is 32.9 meters wide from gutter to gutter. Three continuous spans cross the river, the north and south span measuring 80.8 meters and central span measuring 139 meters. Three of the four piers supporting the river crossing have two huge geared rollernest bearing assemblies while the second pier from the north is a fixed connection. These truss bearings have moderate corrosion [21].

The two main trusses have an 11.6-meter cantilever at the north and south ends. There are also 27 floor trusses, spaced at 11.6 meters. These floor trusses frame into the vertical members of the main truss. The floor trusses consist of WF-shape members and have a 4.97-meter cantilever at each end.

The built-up box sections have attachments measuring 8.9 cm square welded to diaphragms at the interior of all tension members (Figure 5). There are also intermittent fillet welds at the interior of all box sections. These are both Category D details. The floor truss members have longitudinal stiffeners measuring 30.5 cm, which would be considered a Category E detail (Figure 6).

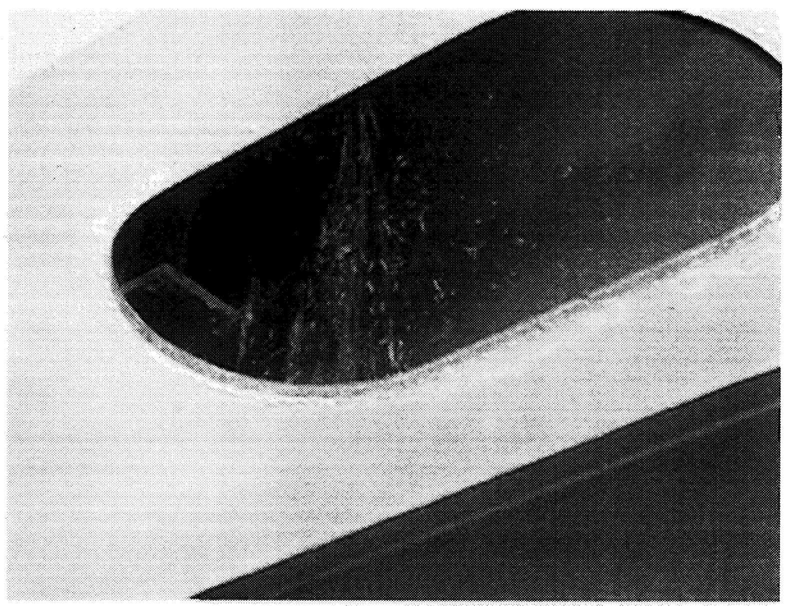


Figure 5: Welded Attachment at Interior of Box Section of Main Truss

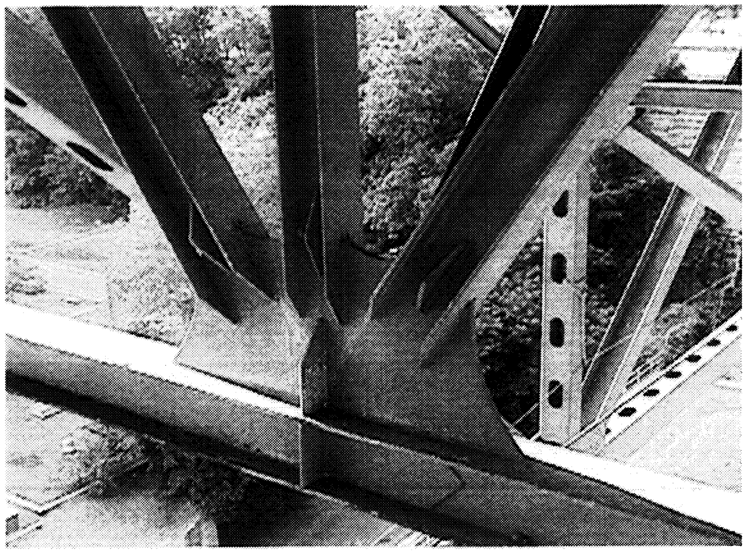


Figure 6: Longitudinal Stiffeners at Floor Truss Connections

BRIDGE DESIGN

Bridge 9340 was designed using the 1961 AASHO specifications [1]. This code utilizes a uniform lane load and a truck for live load. The uniform live load consists of a 9.34 kN per linear meter of load lane and a concentrated load of 11.6 kN for shear. The truck load uses HS-20 truck which has a front axle load of 35.6 kN followed 4.27 meters behind by a 142.3 kN axle followed anywhere from 4.27 to 9.14 meters behind by another 142.3 kN axle. The wheels of the HS-20 truck are spaced 1.83 meters apart. All loads are patterned for maximum effect. Resulting load effects are reduced by ten percent if the maximum load effect is produced by loading three lanes, and by 25 percent if four or more lanes are loaded.

The design of the main trusses utilized the uniform lane loads. All four lanes above the truss being designed and the three nearest lanes opposite the centerline were loaded. Using a tributary length of 11.6 meters for each panel point of the truss, this loading results in a concentrated load of 367 kN and a uniform load of 343.8 kN. The south cantilever of the main truss has a tributary length of 16.6 meters and thus a uniform design load of 489.3 kN. The north cantilever of the main truss is designed using four loaded lanes and a tributary length of 25.5 meters and does not consider the effect of the floor truss cantilever as most of the tributary length is outside of the truss region. This results in a uniform design load for the north cantilever of 716.2 kN.

Load is distributed from the floor system to the floor truss through the stringers. The stringers are continuous over four spans from panel points 0 to 8 and 8' to 0' and continuous over six spans from panel points 8 to 8'. The internal reactions of the four span continuous stringers were found under a HS-20 truck loading and applied to the floor truss in design. Each axle is spaced at 4.27 meters in the design. The HS-20 trucks were then placed in the lanes either shifted toward the curb or the centerline of the roadway to get the maximum load possible on each stringer and to each node in the floor truss. An impact factor of 30 percent was included in the design.

HISTORY OF BRIDGE

Bridge 9340 was built in 1967. While there have been no structural problems with the deck truss, there have been recent problems with the approach spans on both ends of the bridge. In 1997, cracks were discovered in the cross girder at the end of the approach spans. A small section of the end of each main truss is attached to bearings at reinforced openings in the cross girder. It appeared that resistance to movement of the bearings was causing significant out-of-plane forces and associated distortion on the cross girder, leading to cracks forming at the termination of the stiffeners reinforcing the opening. The cross-girder was retrofit by drilling holes at the tips of the cracks and adding struts from the reinforcing stiffeners back to the girders to reduce the distortion. This retrofit has been successful so far in preventing further crack propagation.

One year later, web gap cracking was discovered at the top of diaphragm attachment plates where they were not welded to the top flange in negative moment areas of the continuous girders. One crack had grown nearly the full depth of one of the girders. This girder was retrofit by drilling a large hole at the crack tips and bolting large web doubler plates to reinforce the cracked area. Other smaller cracks discovered at that time had holes drilled at their ends. Additional holes were drilled in the connection plates and the diaphragms in the negative moment areas were placed much lower to increase the flexibility. The bolts were replaced with the next size lower and were only tightened to a snug condition to allow some slip. Strain gages were placed in the web gap regions of the girder webs to read the values of strain before and after the retrofit. Before the retrofit, stress ranges were large enough to explain the cracking. These stress ranges were reduced by more than 50 percent by the retrofit to levels that would not be expected to cause further cracking [22].

The presence of birds has caused some concern for the deck truss. The main truss is constructed of built-up box sections that in the past have housed many pigeons. It is known that guano can have highly corrosive effects on steel and that extreme corrosion can lead to fatigue problems. Therefore, in the summer of 1999 when the bridge was painted, the access holes of the box sections were fitted with covers to prevent birds from entering the truss members.

CHAPTER 4

FIELD TEST PROCEDURES

LOCATION OF STRAIN GAGES

Due to the ease of access provided by the transverse catwalk, panel point 10 was chosen for the placement of strain gages. This is located in the negative moment region of the continuous three span truss, therefore the lower chord would be expected to be in compression and the upper chord would be in tension under loading.

Six gages were put on each of the east and west main trusses and the floor truss. On the main trusses, a gage was placed on the interior and exterior of the members at mid-depth, to avoid any bending effects. An upper chord (U8-U10), a diagonal (L9-U10), and a lower chord (L9-L11) were instrumented. These members are identified in Figure 7 as the bold members next to panel point 10. The gages were placed at least one section depth away from the connection to avoid stress concentrations.

The floor truss has gages on the east side of the centerline. A gage was placed on the upper and lower flanges of an upper chord (U5-U6), a diagonal (U5-L7), and a lower chord (L4-L7) (Figure 8). These gages were also placed at least one section depth away from the connection to avoid stress concentrations. Figure 9 shows the gages in place on the exterior of the east truss on the upper chord and the diagonal.

North

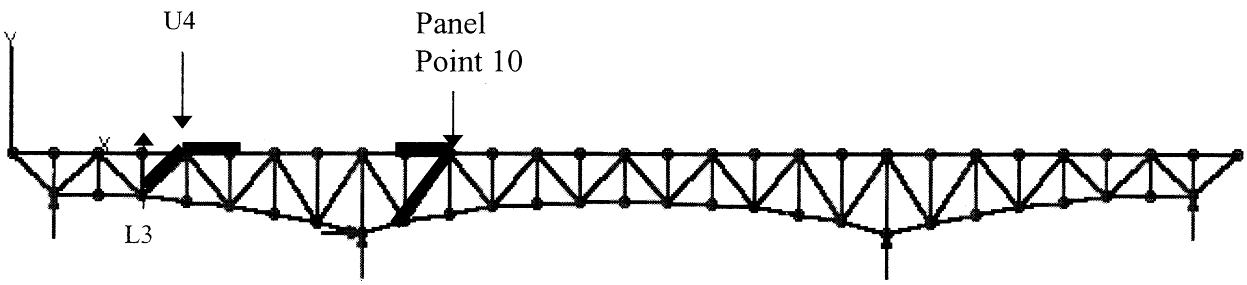


Figure 7: Gaged Locations on the Main Truss

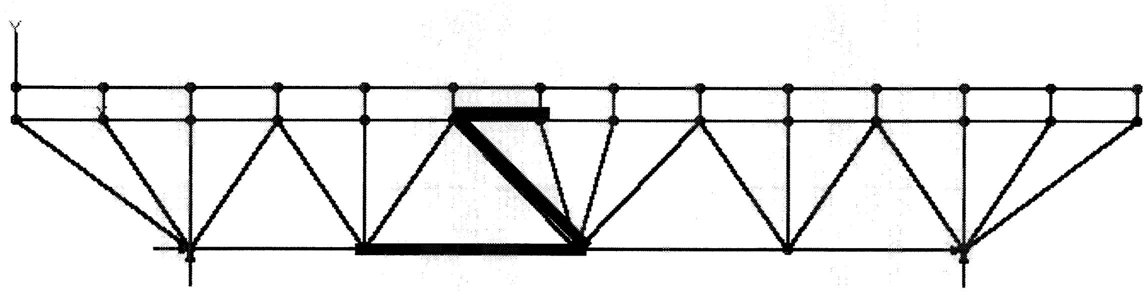


Figure 8: Gaged Locations on the Floor Truss

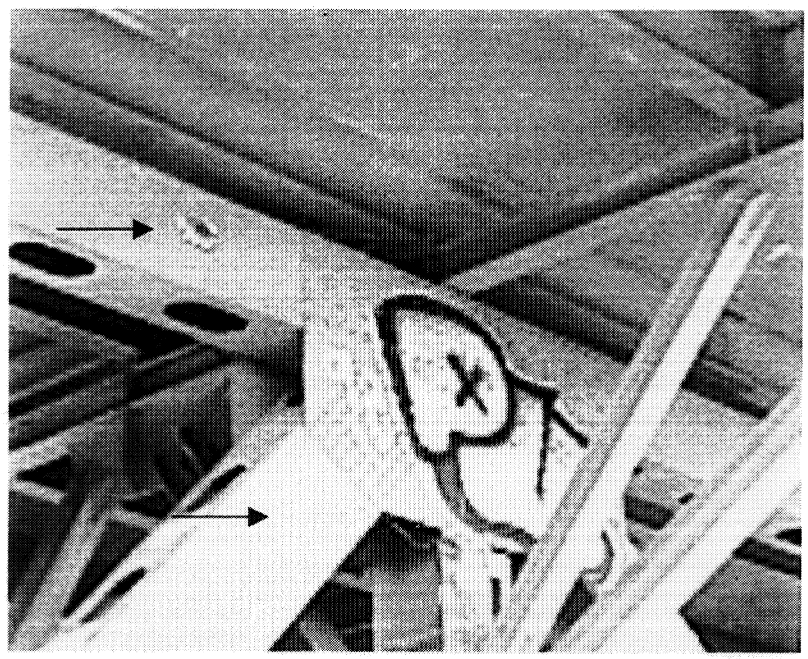


Figure 9: Gaged Upper Chord and Diagonal on Exterior of East Truss

A reversal member (U4-U6) was instrumented, i.e. a member that experiences stress in one direction from approaching trucks and stress in the other direction when the trucks pass over the pier. A member with very high design stress ranges in tension (L3-U4) was also instrumented. These members were located on the south side of the west truss and are designated in bold in Figure 7. Gages were attached to the interior and exterior of these members at mid-depth, also at least one section depth away from the connection.

The wires leading from the gages ran to a central point on the transverse catwalk where they were wired into a data acquisition system housed in a locked electrical box. The box was attached to the catwalk railing using U-bolts. This set up is shown in Figure10.

Figure 10: Data Recording Station on Catwalk of Bridge

TEST DESCRIPTIONS

Controlled Load Tests

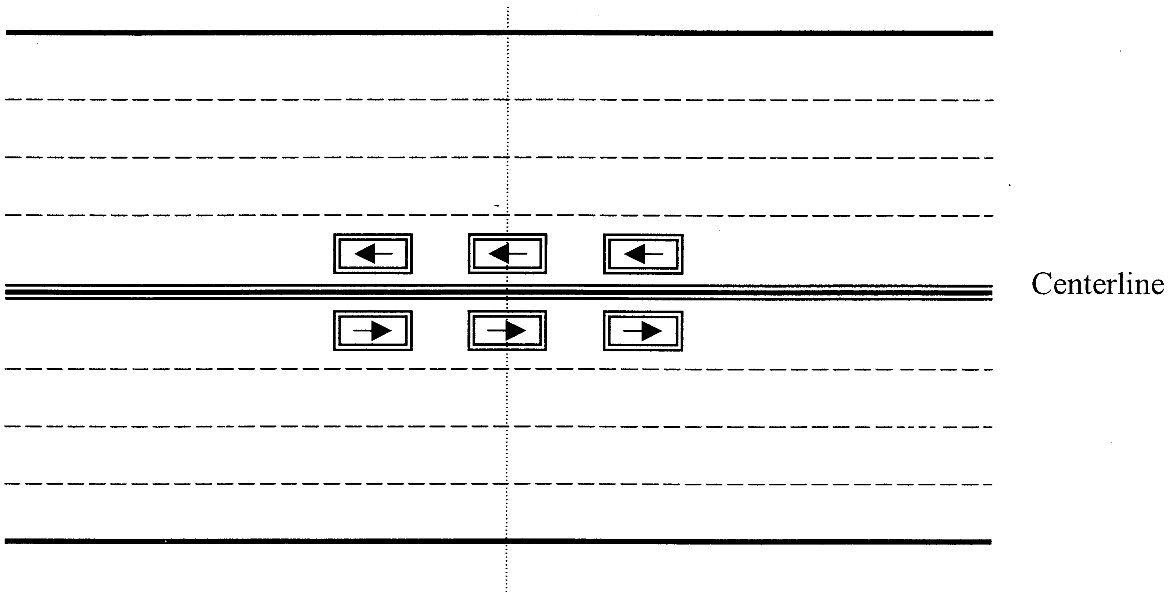
Over the course of two days, four types of tests were conducted. All tests took place after midnight to minimize interference with traffic. Nine Minnesota Department of Transportation (Mn/DOT) tandem-axle dump trucks, each with a gross vehicle weight 227 kN, were used. Strains for this test were recorded for the gages at panel point 10 only, not at the reversal and high-tension-stress members.

Test 1 consisted of two groups of three trucks, with each set driving in a single file line in the left lane in each direction of traffic. This test required that the left lanes were closed. This was done with signing and traffic control provided by Mn/DOT. To represent static conditions each line of trucks were traveling at a crawling speed. The trucks were to follow each other as closely as possible. Optimally, the middle trucks in each group were to meet simultaneously at panel point 10, directly above the instrumented floor truss (Figure 11a).

Test 2 consisted of running all nine trucks in a 3 x 3 formation. The trucks were to travel as close as possible to each other while maintaining highway speeds. Three round trips were made, i.e. three trips in the southbound direction and three in the northbound direction. No lane closures were required for this test. This test set up is shown in Figure11b.

Test 3 consisted of using all nine trucks and running them in a single file line as close as possible to each other (Figure 11c). This was done in the third lane from the centerline as it was the lane most directly over the main truss. The test was run at highway speeds with no lane closures.

In Test 4, the trucks ran side-by-side in groups of three. All nine trucks were used with each group of three following the preceding group by no less than one-half mile. This was done to ensure that only one group of three would be on the bridge at a time. This test was also run at highway speeds. No lane closures were required for this test. The set-up is shown in Figure 11d.



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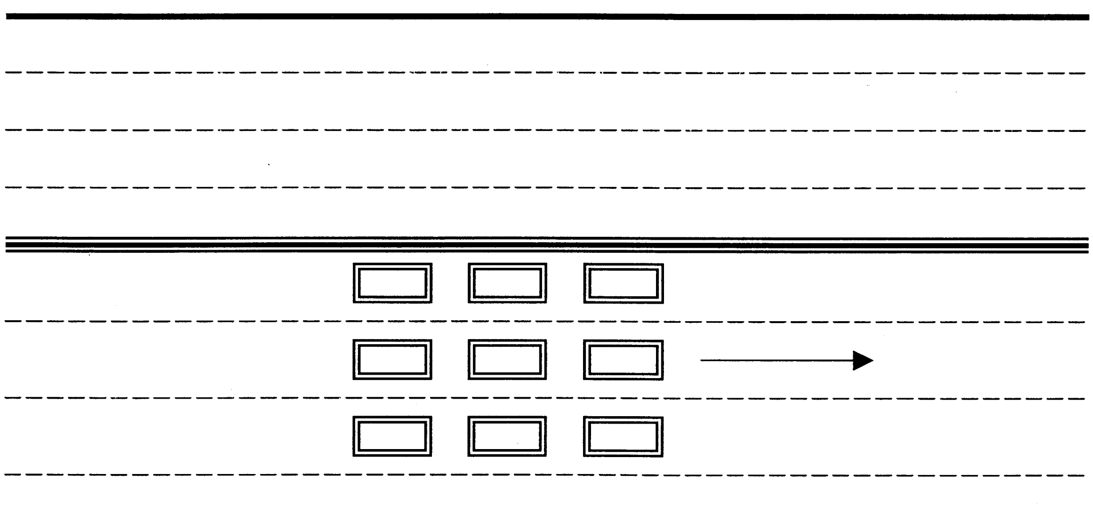

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