Strain Gage Installation on the I-75 Bridge at Covington, Kentucky and Subsequent Data Analysis

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STRAIN GAGE INSTALLATION ON THE I-75 BRIDGE AT COVINGTON, KENTUCKY AND SUBSEQUENT DATA ANALYSIS

by

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in cooperation with
Transportation Cabinet
Commonwealth of Kentucky

and

Federal Highway Administration
U. S. Department of Transportation

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January 31, 1985
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INTRODUCTION

In June 1984, the Kentucky Transportation Research Program (KTRP) was requested by the Kentucky Transportation Cabinet to install strain gages on structural members of the I-75 (Brent Spence) Bridge at Covington. This work was to be part of a renovation/modification project on that structure.

In response to this request, the Kentucky Transportation Research Program submitted a Federal Aid Task Order Proposal that was officially approved in July 1984. Tentative permission to proceed with the work had been granted in June. KTRP personnel contacted Dr. John M. Kulicki, of Modjeski and Masters Consulting Engineers of Mechanicsburg, PA., who were doing the engineering work on the bridge, and requested clarification as to which members needed to be gaged and as to what type of data analysis was required. Dr. Kulicki replied in late June (Appendix No. 1).

Dr. Kulicki stated that the strain gages should be placed on the upstream and downstream trusses on members U1-U2 (diagonals) and U5-U7 (upper chord members) at the Kentucky end of the bridge. A gage was to be placed parallel to the principle stress axis on each web outer face of the members. Those members were built-up, riveted box beams. Readings from the gages were to be averaged to minimize effects of eccentricity. For the data analysis, Dr. Kulicki requested the magnitude and frequency of average live loads on those members. That information can be used to predict the structural life of the members due to existing traffic loads (1).

A workable strain gage system was configured by KTRP personnel and unavailable components were ordered in late July. The equipment delivery was scheduled for early September.

On August 28, KTRP personnel met with KYDOH, District 6 (Covington), and City of Cincinnati personnel to coordinate plans to install the strain gages. At the meeting it was decided that the wires could be installed during the daytime as only one lane would need to be closed at a time for the upstream and downstream trusses. The strain gages were to be installed on Saturday night, September 22. The bridge was to be completely closed from midnight to 10:00 a.m. the next day. Traffic on I-75 was to be diverted over the US-25 bridge.

On September 18 and 19, the upper lanes of the bridge were closed and strain gage wires were placed by KYDOH and KTRP personnel. This was accomplished by placing one-inch plastic conduit through the box members (vertical posts and U1-U2 diagonals). The conduit ran from locations near intended strain gage sites, above the upper deck, down to the lower chord situated below the lower deck. The wires were run down the conduit and through the lower chord to a deck beam over a levee which runs under the bridge. The wires were secured at the deck beam pending installation of the gages.
On September 22, KTRP personnel began to prepare for the strain gage installation. An equipment-housing tent was placed on the levee under the deck beam where the strain gage wires were secured. The electronic equipment was connected and all functions were tested prior to work on the bridge. Electric power was furnished by a portable generator. Two lift-bucket trucks were employed for the installation. One was provided by KYDOH District 6 and the other was furnished by the University of Kentucky. A portable light plant was also used for the installation.

The bridge was made available at 1:30 a.m. KTRP personnel erected the light plant and attempted to set-up both bucket trucks on the upstream and downstream diagonals (U1-U2). Unfortunately, both units could not be installed side-by-side. Therefore, one bucket truck was moved to the downstream upper chord (U5-U7) for grinding work. The other was used to install the gages (transducer) on the upstream diagonal.

Several unforeseen problems became evident during the course of the work. First, the paint had been applied thickly by brushing and readily clogged the 20 grit abrasive discs used for preliminary grinding. Also, the steel surfaces were unusually coarse for uncorroded hot-rolled plate. Those factors caused more time to be consumed in steel preparation than originally anticipated. Also, the wiring and soldering of the gages proved to be very difficult due to wind-induced fluttering of the lift bucket. This fluttering complicated all the other strain gage placement operations to some degree.

At approximately 3:30 a.m. the gage installation on the upstream diagonal was completed and checked when it began to rain. The rain continued through the morning and thwarted further installation operations. The gage was calibrated and monitored for several days. However, the output signal from the signal conditioning unit was very weak and did not change significantly when the bridge was loaded. Thereafter, it was decided to dismantle field installation and plan another installation at the earliest possible date.

On Saturday, September 29, KYDOH and KTRP personnel made another attempt to install the remaining gages. The downstream upper chord (U5-U7) was gaged (Figure 1). The adjacent transducer on the upstream upper chord was almost completed when it again began to rain. Before the gages at that location could be sealed, rainwater shorted the installation and rendered it useless. The transducer on the upstream diagonal was found to be still functional. Both gages were monitored for about 30 hours, when at the request of KYDOH, the installation was again removed and a letter on the preliminary findings was sent to KYDOH(2).

Problems with system sensitivity and recording performance led to a laboratory experimentation and revision of the test set-up and strain-gage calibration technique.

On the weekend of October 20-21 another attempt was made to complete the gage installation. Rain also thwarted this attempt. On October 27, gages were successfully installed on the upstream upper chord and the downstream diagonal (Figure 2). Unfortunately, the upstream diagonal
transducer would not function even after new gages were installed. Presumably, a wire failure caused this problem. The remaining functional strain gage systems at the other three locations were monitored over a three-day period by KTRP personnel despite failures of four different portable generators.

Between November 7, 1984 and January 28, the field recorded analog strain gage data (on magnetic tape) was digitized and subsequently analyzed by computer. The findings of this analysis are contained in the RESULTS and Appendix sections of this report.

TEST SYSTEM

The test system consisted of strain gages, signal wires, a signal conditioner, an input amplifier, and a recorder (Figure 3).

Foil-type, bonded, 350-ohm strain gages were selected for use on the bridge. BLH No. FAE-25-35-S6-ET constantan encapsulated gages, were used. Permabond 910 adhesive was used as the bonding agent. Adhesive bonding was selected over epoxy bonding and welding due to KTRP familiarity with adhesives and the short time required to attach the gages to steel (approximately five minutes per gage). Vibration of the lift bucket would have made use of the other two bonding methods virtually impossible.

A four-gage full bridge (transducer) was selected with two active gages aligned along the structural members and two passive gages each mounted transversely to the active gages (Figure 4). One active and one passive gage were installed on each of the web outer faces and were interconnected with lead wires. Strain relief lead wires were connected from the strain gages to terminal strips (Figure 5). On completion of the gage installation, the transducers were covered with "Barrier E" (neoprene sheet) and steam tape to preclude the entrance of moisture (Figure 6).

A seven-wire cable configuration was selected for the strain gage installation (Figure 7). This arrangement was chosen because it allowed shunt-calibration of the full-bridge transducers and it also obviated lead-wire resistance errors. However, as will be explained, the shunt calibration was abandoned and a six-wire configuration was adopted (the lcal lead, shown in Figure 7, is not used). This system also eliminated lead-wire error.

The signal conditioning (i.e. strain gage excitation, sensitivity, and signal output) were accomplished using a Daytronics Model 9000 Signal Conditioner with: a Model 9530 Visual Indicator, a Model 9305A Channel Caller, and four-Model 9170 Signal Conditioners.

The analog signal output from each channel of the signal conditioner was fed into a four-channel Lockheed Model S289 recorder using magnetic tape (reel-to-reel). A custom X2 input amplifier was eventually patched in between the signal conditioner and the tape recorder to improve signal output from the recorder.
Shunt calibration was originally chosen for the full bridge transducers. In this method, a known resistance is shunted across the transducer and the span is adjusted to a pre-calculated shunt strain based on the relation:

\[
e(\text{span}) = \frac{R(\text{gage}) \times 10^6}{K \cdot R(\text{shunt})}
\]

Where:
- \( e(\text{span}) \) = pre-calculated shunt strain
- \( R(\text{gage}) \) = gage resistance
- \( K \) = gage factor
- \( R(\text{shunt}) \) = shunt resistance.

The Daytronic Signal Conditioner had a built-in 59,000-ohm resistance intended for shunt calibration. When used with the 350-ohm BLH strain gages, this shunt required the use of a 5-volt gage excitation. After the second installation attempt, it was determined that this excitation was insufficient to produce suitable signal output voltages from the signal conditioner to the recorder. Therefore, the signal conditioner was adjusted to produce a 10-volt strain-gage excitation. However, with the 10-volt excitation, the span could not be adjusted to the correct calibration setting. Therefore, shunt calibration was abandoned.

To get maximum signal output response for stress changes, the signal conditioner span was adjusted to its maximum value on the signal conditioner. A full bridge transducer was placed on a steel bar, one-half inch thick by two-inches wide by 20-inches long. The bar was loaded in 5,000 psi increments to 20,000 psi on a Baldwin-Lima hydraulic tensile testing machine. Strain-gage (transducer) signal outputs were transmitted from the signal conditioner to the tape recorder for each stress level. Later, the recorder was replayed at each stress level and the corresponding voltage outputs were measured. Then, each channel was individually calibrated to insure continuity between readings. Thereafter, output voltages for each channel varied between each other by three percent. The stress in the steel varied with the average signal output voltage by bridge stresses from 5,930 psi per volt. This value was used to calibrate recorded voltages.

To make certain of the accuracy of this method, the following precautions were taken. First, a series of calibrated voltages were input to each channel of the recorder. The output voltages from those recordings were measured and each channel amplifier was adjusted to obtain equal output voltages from each channel for equivalent voltage inputs. The resulting inputs from equivalent loads on the test bar produced tape-recorded outputs within three percent for each channel.

To insure continuity between the different transducers, the resistance of each bridge was determined by measuring the resistance between the full bridge nodes both in the laboratory and the field. This was done by measuring the resistance between signal wires on the signal wire-to-signal connector pins. Those values varied by about 5 percent.
During the field tests, the transducers were monitored for 5-minute periods during each hour. The outputs of the three functional transducers were recorded on Scotch No. 177 magnetic tape. The input range of the recorder was set at 5 volts. The magnetic tape speed for all the tests was 2 inches per minute. One channel of the tape recorder had a voice override. This allowed for dubbing of recording times and for separation of the different data sets. In recording the data used in this report, 7 reels of 500-foot magnetic tape were employed. Taping began at 1:00 p.m., Sunday, October 28 and terminated at 8:00 a.m., Wednesday, October 31. Sixty-six consecutive hours were sampled.

During preliminary set-up under the bridge on Saturday, October 27, the signal conditioner output was fed into a storage oscilloscope. Adjustment of filters on the Daytronics 9170 Modules indicated a large fluctuation of the signal trace on the oscilloscope CRT which was magnified by use of the higher (10 volt) excitation. This signal appeared to be superimposed on a slower varying vertical movement of the trace (the vertical displacements of the trace being changes in the transducer voltage signal). The signal conditioner output could be filtered at: 2 Hz lowpass, 200 Hz lowpass, 2 KHz low pass, or no filtering. To investigate those fluctuations, all filtering was removed from the signal. Then, the filtering out of higher frequencies was increased until the high frequency fluctuation disappeared when 2 Hz lowpass filtering was employed.

The output signal filtering was raised to 200 Hz lowpass and the signal was frozen on the CRT screen using the oscilloscopes storage feature. The frozen image revealed that the fluctuations possessed a repeatable 60 Hz frequency. This indicated that the rapid fluctuation was electric noise probably due to the portable generator. An isolation transformer was used between the generator and the electrical equipment. However, it was not intended nor suited to remove 60 cycle noise. As no notch filter was available, the 2 Hz lowpass filter on the signal conditioner was employed. Similar tests at the higher frequencies revealed that the fluctuations present at those frequencies were all 60 Hz noise.

DATA PROCESSING

After the field data was recorded, the recorder and tapes were taken to the D. V. Terrell Civil Engineer Laboratory on the University of Kentucky campus and the data digitized using floppy disc storage and an IBM PC microcomputer with a Techmar "Lab Master 2009" analog-to-digital board. This system allowed the three active recorder channels to be digitized concurrently. IBM software was used to format the floppy discs.

The digitizing rate selected was 16 times per second or 8 times per cycle. This rate was based on field oscilloscope observations of signal variance rates and summary calculations that indicated a probable loading frequency of less than 2 Hz.
The digitizing operation was very slow due to our limited access of the analog-to-digital conversion system and limitations of the disc format in storing the discs completely. Eventually, about one disc was required for each five minutes of data (for the three functioning channels). This process was completed at the end of November.

Due to the limitations in processing the data on the IBM PC microcomputer, using data analysis program selected, the digitized data was transferred to the campus mainframe computer, an IBM 3081 K16. This was done via a telephone modem with a 1200 Baud Rate. About two hours was required to transfer each floppy disc data to main frame storage. This work was completed in late December.

Prior to processing the digitized data, one final alteration was made to the data sets. The signal conditioner output signals (at 2 Hz lowpass filtering) to the recorder were very "clean" and continuous when viewed through the oscilloscope CRT. However, when stored in the recorder and replayed through the oscilloscope, the resulting signals were sometimes infected with voltage spikes and other electrical "glitches" that would yield erroneous data if not eliminated. To counter this problem, a digital filtering system was devised to eliminate the voltage irregularities in the data caused by the recorder.

It was assumed that the digitizing intervals were very short in comparison to the rate of changes in the bridge loadings on the relevant members (one sampling every 60.25 milliseconds). By viewing the oscilloscope CRT and by using the visual output from the Model 9530 Visual Indicator in the field, it was discerned that the rate of stress change from transducers was less than 20,000 psi per Hz. Therefore, the change of rate of actual stress (not due to noise) would not exceed some value greater than:

\[
MSG \times MSC \times 20,000 \text{ psi/Hz} < 1,250 \text{ psi.}
\]

Where:
- MSG = maximum stress gradient
- MSC = maximum estimated stress per cycle
- NSC = number of samples per cycle

This value was determined to be conservative. The rise-time rate of the voltage spikes and glitches is known to be extremely high. To be conservative, two rise-time rates were considered; 2,000 psi per cycle and 3,200 psi per cycle. The 2,000 psi per cycle representing the 1,250 psi per cycle plus a safety factor of 750 psi per cycle. The 3,200 psi per cycle representing the maximum rate of change of a sinusoidally varying load over one Hz. Both of these values were assigned to the filtering inequality shown below:

\[
|S_{n+1} - S_n| < 2,000 \text{ psi (3,200 psi)}
\]

Where:
- Sn = the magnitude of the previous digitized stress
- Sn+1 = the magnitude of the next successive digitized stress.
The filtering process entailed comparing each digitized data point with the next successive point in the equality. If the inequality was not satisfied, it was presumed that a spike was encountered and the value of \( S_{n+1} \) was adjusted to equal \( S_n \). This process was repeated with successive data points until the inequality held. When the inequality was met by the successive data points, they remained unchanged. Very little valid data was lost by this process due to the short duration of the electrical interferences. Once the true output-signal voltage level was again encountered, it would not be significantly altered for more than a few load cycles. The digitized data was subjected to this filtering technique using each rise-time rate for one computer run.

The cyclic loadings imposed on bridge members by traffic, temperature, and wind, vary in an irregular pattern with time. It is difficult to count and group such loadings. One cyclic-load counting method currently in use is the “racetrack” or “ordered overall range method.” The method is used to condense long, complex cyclic loading histories or long, complex grouping of peaks and valleys to make it more useful. The condensation is a record of the most important features of the grouping and can act as the basis for calculations or predictions (3).

The method eliminates smaller ranges as shown in Figure 8. A racetrack of width “R” is defined, bounded by “fences” that have the same profile history. The only reversal points counted are those at which the “racer” would have to change from upward to downward or opposite as shown by points as A, B and C in Figure 8. The width “R” determines the number of reversals to be counted. In terms of stresses, “R” is the magnitude of stress below which no counting will occur (i.e. they are negligible).

The logic of the method is that the distance from the highest peak to the lowest valley is the most important feature of the history. The distance from the second highest peak to the second lowest valley rates next in importance, provided that the second range (second peak to second valley) crosses the first range (maximum peak to maximum valley) or is outside of the time interval defined by the first two. By using this screening process for a succession of stress levels, “R_i”, the number of reversals corresponding to the range \( R_i \) to \( R_{i+1} \) can be determined by finding the difference between the number of reversals corresponding to \( R_i \) and \( R_{i+1} \), respectively.

Counting all the ranges and grouping them into lists of their magnitudes and frequencies by this method gives similar results as another counting technique called the “rainfall” method. A computer program utilizing the racetrack method described above was developed by modifying a program available in the literature (4). The complete computer program is contained in Appendix No. 2.

While cyclic stress counting is involved, a more difficult task is to assess the impact of the various stress ranges and frequencies on structural members. One useful consideration is the Palmgren-Miner...
cumulative damage law. This method assumes that if \( n_1 \) cycles of a given stress range \( S_1 \) were applied to a member, where \( N_1 \) was the fatigue limit for the member at that given stress range, then, failure or permanent damage could be expected when:

\[
\Sigma(n_1/N_1) = 1.
\]

An "effective" or resolved stress range can be derived using that relationship:

\[
Sr(Miner) = (\Sigma Sr_i^3)^{1/3}
\]

Where:
- \( Sr(Miner) \) = effective stress range (Miner's hypothesis)
- \( j \) = fraction of occurrence of \( Sr_i \)
- \( Sr_i \) = individual stress ranges

The RMS "effective" stress range for a variable stress spectrum can be defined as:

\[
Sr(RMS) = (\Sigma Sr_i^2)^{1/2}
\]

Where:
- \( Sr(RMS) \) = RMS stress range.

For the computer data analysis, the cyclic stress ranges were placed into 20 groups from 1,000 - 20,000 psi in 1,000 psi increments. The minimum stress range "R" was 100 psi. This was done to ensure that the same data was present on all recordings and that any apparent dead recording time was due to lack of traffic rather than to a failure of the recording system. The accumulated stress ranges totals of the 48-hour test for each channel and filter level contained in Appendix No. 3, Channel 1 is the upstream upper chord location (U5-U7). Channel 2 is the downstream diagonal location (U1-U2). Channel 3 is the downstream upper chord location (U5-U7).

Forty-eight hours of data (of the 66 hours of field data taken) were counted and incorporated in the "effective" stress range values. That data started at 6:00 a.m. Monday and ended at 6:00 a.m. Wednesday. Five minutes of data was recorded each hour on each functioning channel. The total cycles shown in the RESULTS section of the report were adjusted to account for the sampling time.
## RESULTS

<table>
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<tr>
<th>Channel</th>
<th>STRESS RANGE</th>
<th>CYCLES PER YEAR</th>
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<tr>
<td>Channel 1</td>
<td>Sr(Miner) 1,680 psi</td>
<td>1,153,783</td>
</tr>
<tr>
<td></td>
<td>Sr(RMS) 1,830 psi</td>
<td>&quot;</td>
</tr>
<tr>
<td>Channel 2</td>
<td>Sr(Miner) 1,150 psi</td>
<td>847,102</td>
</tr>
<tr>
<td></td>
<td>Sr(RMS) 1,220 psi</td>
<td>&quot;</td>
</tr>
<tr>
<td>Channel 3</td>
<td>Sr(Miner) 2,870 psi</td>
<td>4,878,908</td>
</tr>
<tr>
<td></td>
<td>Sr(RMS) 3,680 psi</td>
<td>&quot;</td>
</tr>
</tbody>
</table>

The results shown in the previous section indicate that the structure's overall stress ranges are significantly higher than expected. The higher stress ranges observed in the downstream sections (U5-U7) are attributed to the increased loading due to traffic congestion. Comparative analysis of the stress ranges suggests that the structure's capacity to withstand these loads is compromised.
CONCLUSIONS

The results shown in the previous section indicate that the structural members monitored on the I-75 bridge are not subject to excessively high cyclic stresses due to traffic loading. The highest "effective" or "resultant" stress range, detected on the downstream upper chord (U5-U7), was only 2,870 psi Sr(Miner) or 3,680 psi Sr(RMS). Those stresses are nearly equivalent as stress-life curves (S-N graphs) failure lines would be shifted to higher value for Sr(RMS) effective stresses compared to Sr(Miner) effective stresses.

If severe geometric defects (AASHTO Fatigue Category E) existed in the structural members (based on Category E, S-N graphs Reference 1, pp 20.), fatigue problems could be anticipated on that member after $2 \times 10^6$ cycles or 4 years of service, based on a yearly loading rate of $5 \times 10^5$ cycles. That such problems have not manifested themselves is an indication that the Category E assumption is too conservative for that structural member. Based on that same curve, the other two structure locations monitored would have fatigue lives in excess of 50 years. Built-up riveted beams, if properly designed and fabricated should not be high fatigue risks. It has been mentioned that tack welds were present on the beams and that those were scheduled for removal. That work is a prudent.

The stress ranges and yearly loading rates for that member (U5-U7 downstream) were much greater than for the similar location on the upstream truss. Possibly, the higher stress ranges encountered could be due to the greater quantity of heavy southbound truck traffic compared to a lighter weight of loaded trucks northbound.

Another possibility to explain the difference in the stress ranges was that some upward signal drift may have occurred in the analog recording during the field operation. The tape recorder was the weakest link in the operation. The unit was obtained on loan from another department at the University and was chosen due to its portability and availability. However, the recorder was subject to numerous electronic problems. Occasionally, the unit exhibited a tendency to drift a linear signal to higher values on one or more channels. As this tendency was very intermittent, and as no other suitable recorder was available, it was hoped that this proclivity would not be exhibited during the field recording operations. Due to the time limitations encountered in furnishing this report, no comprehensive review of the analog tapes was not possible.

The derived "effective" stress ranges used in this report can be magnified by a few high stress range readings. Drift and the use of the racetrack counting method would provide those additional readings.

The total number of recorded loadings (100-20,000 psi) was also the greatest for the downstream upper chord location (15,015). This was about 1.6 times as many loadings as for the upstream upper chord location. The diagonal on the downstream side had about 80 percent of the number of recorded stress cycles as the downstream upper chord location. This seems
logical since many of the impressed loads had less effect on the stresses in the diagonal. Therefore, some of the loadings which imparted stresses greater than 100 psi in the upper chord would impart lower stresses in the diagonal and those could not be considered.

The digital filter employed during the data processing functioned effectively. In some hourly data sets, no data was rejected. In others, one or two data events were rejected as being electrical spikes. In several cases 100-200 events were rejected. In one instance, some 3,000 events were rejected. In a majority of data sets, little or no data were rejected by the digital filtering process.

There was virtually no difference between the 2,000 psi and 3,200 psi filtered data, in terms of both the stress ranges accepted and the frequency of occurrence of those stress ranges. Therefore, the "effective" stresses were calculated using the most conservative filter (3,200 psi). The frequency of stresses and most of the stress ranges appear to be realistic (i.e. good data).

The accuracy of the derived values is probably within 10 percent stated "effective" stress ranges and their frequencies of occurrence. All counted stress ranges in a given category (2,000-3,000 psi for example) were considered to be the higher value (3,000 psi) in deriving the "effective" stress ranges. System errors include: accuracy of strain gage placement, variances in transducer response, minor non-linearity of the transducers over the strain range, and minor differences in signal strengths between the amplifier and the recorder for different channels. The first consideration stated in this paragraph should more than offset any possible reduction of recorded stresses due to the system errors.

We regret our delay in furnishing the Kentucky Transportation Cabinet with this report. However, a multitude of problems were encountered during this project which were not envisioned when we undertook it. We have been able to acquire a workable system, personnel experience and a data processing technology which should be useful to the Kentucky Transportation Cabinet in the future for similar projects, hopefully in a more timely manner.
REFERENCES


2. Letter to Mr. Tom Layman, Kentucky Transportation Research Cabinet from Theodore Hopwood II, Kentucky Transportation Research Program, dated October 10, 1984.


Figure 1. Strain Gage System on the Downstream Upper Chord (U5-U7)

Figure 2. Strain Gage Installation on the Downstream Diagonal (U1-U2). Note Conduit & Signal Wire at Left.
Figure 3. Strain Gage Test System

- FULL BRIDGE TRANSDUCER (4-SPAN GAGES)
- 8 CONDUCTOR 22 GA. WIRE
- DAYTRONICS MODEL 900 SIGNAL CONDITIONER
- X2 AMPLIFIER
- LOCKHEED MODEL 5289 4-CHANNEL FM RECORDER
Figure 4. Fall Bridge 7-Wire Strain Gage Transducer Layout
Figure 5. Strain Gage System Showing Lead Wires & Terminal Strips. Note the Rough Texture of the Steel Plate.

Figure 6. Protective Covering on Completed Transducer
**Figure 7. Seven-Wire Piezoelectric Cable**

**Figure 8. Racetrack Counting**
APPENDIX NO. 1

(Letter from Dr. Kulicki)
Mr. Ted Hopwood  
University of Kentucky  
Transportation Department  
Lexington, Kentucky 40506

RE: I-75 BRIDGE OVER THE OHIO RIVER - STRAIN GAGE INSTRUMENTATION

Dear Mr. Hopwood:

In response to your questions concerning installation of strain gages on truss members of the I-75 Bridge over the Ohio River, we are transmitting the following:

- A general elevation of the truss span indicating truss members where strain gage installation is required. The specific members are as follows:
  - U1-L2 (upstream and downstream trusses)
  - U5-U7 (upstream and downstream trusses)

Note that these members are on the south (Kentucky) half of the bridge only.

- A truss detail sheet indicating a typical minimum acceptable distance between a truss gusset and the installation point of a strain gage.

- Reproductions of Chapters 3 and 4 of the "BRIDGE FATIGUE GUIDE" (1977) by Dr. John W. Fisher. This is to aid you in reduction of field data for construction of a stress histogram and effective stress range computation.

If you should have any questions on the transmitted material or if we can be of further assistance, please do not hesitate to call.

Very truly yours,

[Signature]

JOHN M. KULICKI, Ph.D.,
Partner

encl/as

cc: Mr. Warren Miller, H. W. Lochner, Inc.
CHAPTER 3
STRESS CYCLES FOR DESIGN

Overall, the history of both highway and railroad bridges has been quite satisfactory. The failures that have occurred pointed out the importance of properly considering in design and fabrication the factors that influence the fatigue strength of steel bridge structures. Some fatigue crack growth has occurred in a few bridge structures and components. The possibility of fatigue cracking under relatively high stress range conditions was demonstrated by the coverplated steel beam bridges of the AASHO Road Test. More recently, cracks were observed in a coverplated bridge located on an interstate highway which carried an unusually high volume of heavy truck traffic, causing large numbers of cyclic stress.

Fatigue cracks have been observed in other structures and their occurrence usually resulted from conditions that were not accounted for in design. These conditions have included: tank welds that were not incorporated into final welds, but were used during fabrication as means of temporary attachment; the addition of welded plates or attachments without considering their reduction in fatigue strength; unaccounted for out-of-plane displacement, induced stresses; and details which changed the structures' behavior, such as connections which provided fixity when simple supports were assumed in the design. Many of these latter types of failures have been due to oversights in either design or fabrication and account for most of the adverse behavior experienced.

Fatigue specifications in the United States originated from railway bridge design, which required reductions in allowable stress when members were subjected to load reversals. During the 1940's both AREA and AASHO developed the AWS bridge specifications for welded structures. These provided for three load cycle conditions: 100,000; 600,000; and 2,000,000. Allowable stresses were expressed in terms of the maximum stress and varied with the stress ratio \( R \), defined as the ratio of minimum to maximum stress. These provisions were based on available test data, mainly on small plate specimens, and 2,000,000 cycles was generally assumed to be the run-out or infinite life condition.

Little change in these provisions occurred until 1965, when the steel bridge fatigue provisions were adopted by AASHTO. These provisions were developed from accumulated data from a variety of sources and a reexamination of older test data. Various types of conditions and details were divided into nine different classifications for fatigue lives of 100,000, 500,000; and 2,000,000 cycles. The allowable fatigue stress was still expressed in terms of the maximum stress, with provisions for stress ratio and steel strength. In the 1965 provisions, some details and members were permitted higher allowable stresses for high strength steels, whereas other details were not permitted such increases.

Minor changes were introduced as further data became available and the data base increased. Many of the early fatigue studies were carried out on A7 and A36 steels, while more recent studies were concentrated on higher strength steels. Because of this, some differences attributed to steel strength were more likely due to changes in welding techniques and improved experimental procedures, rather than the yield point of the material. Many past studies did not provide for an experiment design that would permit a statistical evaluation. Hence, it was not possible to provide a statistical analysis of the design factors that influence fatigue strength and determine their significance. Duplication was rare, critical variables were not controlled systematically, and the experimental error was not defined.

In order to overcome these limitations, the National Cooperative Highway Research Program supported a comprehensive study on "The Effect of Weldments on the Fatigue Strength of Steel Beams" at Lehigh University. These studies used statistically designed experimental programs under controlled conditions, so that analysis of the data could reveal the significance of the parameters believed to be important in fatigue behavior.

These studies and other work available in the literature permitted a comprehensive specification to be developed. These provisions were first adopted by AASHTO in 1973 and issued as Interim Specifications—1974. Revisions have been made in 1975, 1976, and 1977. Following is a brief description of the laboratory studies and criteria used to establish the current AASHTO Fatigue Tables 1.7.2A1, 1.7.2A2, and 1.7.2B shown in Appendix B.

Experience with actual highway bridge structures in the United States has demonstrated that fatigue crack growth can occur when a bridge is subject to extremely high volumes of truck traffic. This behavior is related to the fact that 2,000,000 cycles of loading does not correspond to a fatigue limit or crack growth threshold for some structural details, as was previously assumed in various specifications. Fatigue damage in some cases can occur from many cycles of low stress range.
A reevaluation of the design life provisions was necessary to prevent occurrences on other bridges located on extremely heavily traveled arteries. Furthermore, studies on some transverse members subjected to wheel loadings suggested that higher stress ranges occur a larger number of times than was observed in main longitudinal members.

In order to develop a relationship between the design stress range and the actual truck traffic for the extremely heavily traveled artery, bridge lives were estimated from laboratory tests, assuming that damage accumulated in a linear fashion as suggested by Miner. The applicability of this procedure was subsequently verified by extensive studies of beams under random variable stress cycles. The fatigue studies used to develop design stress range values have shown that the fatigue life, $N'$, is related to the applied stress range $S_n$, as follows:

$$N' = A S_n^{-3}$$  \hspace{1cm} (1)

where $A$ is a function of the fatigue behavior of a detail. The design stress ranges are represented by nearly parallel stress-life curves. Throughout the nation, load-stress history measurements indicate that the measured stress ranges are almost always less than the design stress range, due to such factors as differences in load distribution, impact, actual truck loadings, etc. Consequently, for fatigue design the actual stress range produced by vehicles similar to the design truck is a factor $\alpha$ (less than one) times the design stress range.

The relationship between gross vehicle weight ($GVW$) and stress range can be considered linear, and is usually constant for similar vehicles. Hence, the relationship between actual stress range and ($GVW$) can be expressed as:

$$S_n = \alpha \beta (GVW)$$  \hspace{1cm} (2)

where $\beta$ is the elastic constant relating load and stress to a particular location on the structure. Miner's linear fatigue damage equation, $\Sigma n_i / N_i = 1$, yields the following relationship when expressed in terms of Eqs. (1) and (2):

$$\frac{(\alpha \beta)^3}{\alpha} \sum n_i / (GVW)^3 = 1$$  \hspace{1cm} (3)

where $n_i$ is the number of occurrences of $(GVW)$. When expressed in terms of frequency of occurrence of $(GVW)$, (see Fig. 25), Eq. (3) yields:

$$\frac{(\alpha \beta)^3}{\alpha} (GVW)^3 (ADTT)^3 D_L \Sigma \gamma_i \phi_i^3 = 1$$  \hspace{1cm} (4)

where

$$ADTT = \text{average daily truck traffic}$$

$$D_L = \text{design life in days}$$

$$\phi_i = \text{ratio of actual vehicle weight to design vehicle weight, } (GVW)_a / (GVW)_d$$

$$\gamma_i = \text{fraction of } (ADTT) \text{ for } (GVW)_a$$

The summation in Eq. (4) is a function of the vehicle weight distribution and was determined from the 1970 FHWA loadmeter survey (see Fig. 25). Figure 26 shows $\gamma, \phi, \phi^3$ plotted as a function of $GVW$. The sharp rise of the curve as the design load is approached indicates that most fatigue damage is likely to result from vehicles near the design vehicle weight.

The summation of $\gamma, \phi^3$ in Eq. (4) for all vehicles in the loadmeter survey is about 0.35. If all vehicles in excess of 20 kips are assumed to cause damage, Eq. (4) can be conservatively expressed as:

$$\frac{\alpha^3}{\alpha} (\beta (GVW)_a)^3 (ADTT)^3 D_L (0.35) = 1$$  \hspace{1cm} (5)

The term $\beta (GVW)_a$ is the design stress range. Since design stress range can be determined from Eq. (1) for a specified number of constant stress cycles, $N$, the following ratio between the total number of trucks and constant stress cycles results:

$$\frac{(ADTT)^3 D_L}{N} = \frac{2.85}{\alpha^3}$$  \hspace{1cm} (6)

The factor $\alpha$ is the ratio of the actual stress range due to the passage of a design vehicle and the design stress range, and is less than one. Conservative values of $\alpha$ of about 0.8 for transverse members and 0.7 for longitudinal members were determined from field tests and used to derive the $(ADTT)$ found in Table 17.2B of the AASHTO Specifications.

![Fig. 25. Gross vehicle weight distribution from 1970 FHWA Nationwide Loadmeter Survey](image)

![Fig. 26. Probable damage caused by various truck weights](image)
All available studies indicate that most of the stress cycles caused by vehicle traffic are below the fatigue crack growth threshold (i.e., the actual stress range is less than the stress range which will propagate a crack from an initial discontinuity for the category corresponding to more than 2,000,000 stress cycles). No damage is believed to be caused by stresses below the fatigue crack growth threshold unless the maximum stress range in the variable stress spectrum exceeds the fatigue limit. Hence, the actual $\Sigma \gamma / \phi$, is less than the value 0.35 recommended. The differences between actual stress cycles and the design condition also indicates that a transverse lateral wheel load distribution factor of $S/7$ is reasonable, especially for fatigue design of longer span steel I-beam bridges with a concrete floor, when both lane and truck loading must be examined. It reflects the fact that traffic induced stresses are caused primarily by single traffic lane loading.

When the few known fatigue cracks in bridges are compared with the (ADTT) and observed stress history measurements, most of the damage appears to be caused by the heavier trucks. Only 10% to 15% of the (ADTT) appears to result in stresses causing crack growth. This condition is only true for the most severe design details, such as coverplated beams and attachments which have terminating weld toes. Most other details have much higher fatigue crack growth thresholds and no crack growth is likely under any loading condition, unless some unusual condition exists. Transverse members which receive loads directly from individual wheels experience proportionately more cycles of loading.

The stress cycle tables recognize the increased stress cycles to which transverse members will be subjected. Experience with a few bridges indicated that a greater possibility for fatigue cracking existed, and conservative provisions were developed pending further studies which could provide more rational values.

The minimum life expectancy under the worst possible combination of loading cycles and the resulting stress range is between 60 and 70 years if all stress cycles are assumed to cause damage. Obviously, the minimum life is even greater, since many stress cycles are below the fatigue crack growth threshold and cause no damage at all. Since highway bridges are subjected to both deterioration and obsolescence, 60 to 70 years seems a reasonable life to anticipate should fatigue be a controlling factor. For the vast majority of bridges and their components, no crack growth is expected at all.

Experience with existing structures indicated that the design conditions used for Cases II and III were satisfactory. No fatigue problems have been experienced with bridges in these categories. Hence, the previously used stress cycle table was retained for longitudinal bridge members unless extreme numbers of truck passages were expected. Further load history studies will no doubt lead to refinement and better estimates of the ratio $\alpha$ of actual stress range to the design stress range, including the transverse distribution effect and its relationship to the vehicle weight distribution. Most highway structures are not subjected to the extreme volumes of truck traffic indicated by Case I. Therefore, the designer should not unduly penalize the fatigue design of a structure by using Case I, unless it appears to be warranted by traffic projections.

This section has described the assumptions used to develop the AASHTO stress cycle table for the design of highway bridge structures (see Table 1.7.2B). It is apparent that average conditions were used and assumed to apply to all highway bridges. If well defined traffic conditions are known, these can be used to determine a suitable design life using the method developed. For example, if an analysis indicates that the ratio $\alpha$ of actual stress range due to the passage of a design vehicle to the design stress range is 0.5 and an (ADTT) of 3,000 is expected with the same vehicle weight distribution shown in Fig. 26, Eq. (6) could be used to estimate the required constant stress cycles. For a 60-year life this would yield:

$$N = \frac{(ADTT) (D_f) \alpha^3}{2.85}$$

$$= \frac{3,000 (365) (60) (0.5)^3}{2.85}$$

$$= 2,882,000 \text{ cycles}$$

Hence, fatigue design could be based on the stress ranges corresponding to this life, using the plots given in Fig. 30 (see Chapter 4). This results in stress ranges of 7.1 ksi for Category E, 8.9 ksi for Category D, 12 ksi for Category C, 16 ksi for Category B, and 24 ksi for Category A.

It is also apparent that the stress cycles for design will be substantially different for railroad and mass transit bridge structures. Comparable design cycles can be developed based on span length, stress cycles per train, frequency of trains, type of member, and other conditions. These lead to design conditions that can be placed into a table analogous to Table 1.7.2B of the AASHTO Specifications. Such a table has been developed for railroad bridges in the AREA Specifications (see Table 1.3.13A in Appendix B).

AASHTO also adopted material toughness provisions in 1974 which insure adequate performance providing fatigue crack growth does not occur.31

Three primary factors control the susceptibility of a structure to brittle fracture. These are material toughness, flaw size, and stress level.22,23

Concern with nonredundant members, i.e., single box girder, two plate girder, or truss systems, etc., where failure of a single element could cause collapse of the structure, resulted in the adoption of a greater factor of safety for these types of structures in 1977, i.e., to further minimize the possibility of fatigue crack growth, the allowable stress range has been reduced for nonredundant members. This was accomplished by shifting one range of loading cycles for fatigue design, which results is a reduction in allowable stress range. Although a completely rational explanation cannot be supplied, the very restrictive stress range that results for certain categories will require the designer to investigate details that provide less reduction in fatigue strength.48
CHAPTER 4
STRESS RANGE CONCEPT

The fatigue strength of a particular structural joint has been evaluated in the past by tests on specimens that simulated the prototype connection, or on smaller connections which were similar. Only approximate design relationships were developed, because of the limitations of the test data. Often many variables were introduced into the experiment with a limited number of specimens, which made it impossible to clearly establish the significance of stress conditions, details, type of steels, and quality of fabrication.

A substantial amount of experimental data has been developed on steel beams since 1967, under the sponsorship of the National Cooperative Highway Research Program (Project 12-7), which has shown that the most important factors that govern the fatigue strength are the stress range and the type of detail. Stress range means that only the live load and impact stresses need to be considered when designing for fatigue. These findings were observed to be applicable to every beam and detail examined. Beam specimens were used to overcome some of the limitations of smaller simulated specimens. These beam tests and other work available in the literature were used to develop a comprehensive specification based on stress range alone.

A brief summary of some of the test data is given here to demonstrate that stress range and type of detail are the two factors which are most likely to govern the fatigue strength.

INITIAL DISCONTINUITIES

Two types of welded plate girder details examined in the laboratory are reviewed in this brief summary: (1) the welded plate girder without attachments and (2) beams with welded cover plates. Test data has demonstrated that all fatigue cracks commence at some initial discontinuity in the weldment, or at the weld periphery, and grow perpendicular to the applied stresses. In the welded plate girder without attachments, most laboratory fatigue cracks were observed to originate in the web-to-flange fillet welds at internal discontinuities such as porosity (gas pockets), incomplete fusion, or trapped slag. It should be noted that these discontinuities are always present, independent of the welding process and techniques used during fabrication. Identical behavior has been observed in the laboratory for longitudinal groove welds with either incomplete or complete fusion.

The coverplated beam provides a structural detail in which crack growth starts at the weld periphery, where small sharp discontinuities exist at the toes of fillet and groove welds made by conventional welding processes. The fatigue crack in a coverplated beam, with or without transverse fillet welds, forms from these micro-discontinuities perpendicular to the applied stress.

References 2 and 10 contain a number of photographs of fatigue cracks. These photographs illustrate the various types of discontinuities that exist in structural joints. Under large cyclic stresses these discontinuities grow and eventually result in failure. The test data are described in the following discussion of fatigue strength.

FATIGUE STRENGTH

The test data for the welded plate girder without attachments and coverplated beams are summarized in Fig. 27. Stress range is plotted as a function of cyclic life for several different levels of minimum stress on a log-log scale. It is visually apparent that stress range accounted for the fatigue strength for both structural details, i.e., minimum or maximum stress did not have a significant influence on the fatigue behavior. The ratio of minimum to maximum stress, R, did not affect the stress range to cycle life relationship. The coverplated beam results included wide cover plates, thick cover plates, and cover plates on both rolled and welded beams.

No significant difference was observed for either the welded girder or coverplated beam that could be attributed to the type of steel when a given detail was subjected to the same stress range conditions. This is readily demonstrated in Fig. 28, where the results are plotted for three grades of structural steel with yield stress ranging from 36 ksi to 100 ksi, representing the extremes generally used in bridge construction.

The data plotted in Figs. 27 and 28 show clearly that stress range is the critical stress variable for all structural steels. The results also confirm the significance of the type of detail. The coverplated beam only provided about 45% of the fatigue strength of the welded plate girder without attachments.

Studies on other details have also confirmed that stress range alone is the only significant factor for designing a given detail against fatigue. Results on beams with transverse stiffeners, attachments, and transverse groove welds have also demon-
illustrated that minimum stress and type of steel are not critical factors. Groove welded splices at flange width transitions in A514 steel were more severely affected by the straight tapered transition. This led to the requirement for a curved transition for A514/A517 steel.

In a transverse groove weld with the reinforcement left in place, the stress concentration at the weld toe, with its associated small micro-discontinuities, is usually more severe than nominal internal discontinuities. However, if lack of penetration, slag inclusions, or other internal discontinuities are large in size, crack growth will become more critical at the internal location. In bridge construction, transverse groove welds that are subjected to tension or reversal of stress are generally nondestructively tested to prevent large internal discontinuities from occurring. Also, the weld reinforcement is often removed, so that the weld toe is not critical and a high fatigue strength results.

All evidence indicates that the termination of groove and fillet welds provides a more critical crack growth condition than internal discontinuities in the weld. This is illustrated in Fig. 29 where the test data for three typical welded details are summarized. The welded detail with the highest fatigue strength is the welded beam without attachments. The same strength was observed in groove welded flange splices. In these flange splice details, cracks normally grow from internal discontinuities that are perpendicular to the applied stresses. The other two details shown in Fig. 29 are a short attachment (4 in. long) and the coverplated beam. Both fatigue strength relationships were defined by cracks that formed at the end of the attachment at their weld toes. When the attachments were very short, as with a transverse stiffener, the fatigue strength approached the strength of a welded plate girder. For an attachment 4 in. long, Fig. 29 shows that the fatigue strength is about mid-way between the upper bound (welded beam) and the lower bound (coverplated beam). Attachments longer than 4 in. quickly approach the lower bound condition given by the coverplated beam.

The stress range values given in Table 1.7.2A1 were derived from the 95% confidence limits for 95% survival. Rolled beams were used for Category A, welded plate girders for Category B, stiffeners and short 2-in. attachments for Category C, 4-in. attachments for Category D, and coverplated beams for Category E. The stress range cycle life relationships are plotted in Fig. 30 for each design category. After 2,000,000 cycles, the stress range approaches the crack growth threshold level for the various details and becomes a constant value. For more than 2,000,000 cycles, the fact that transverse stiffeners are less severe than a 2-in. attachment is reflected by an allowable stress range of 12 ksi, which appears to be representative of the threshold level for this design condition.

Fig. 27. Effect of minimum stress and stress range on the cycle life for the welded end of coverplated beams and plain welded beams.

Fig. 28. Effect of stress range and type of steel on the cycle life of coverplated and plain welded beams.

Fig. 29. Comparison of short welded attachments with coverplated and plain welded beams.
RESIDUAL STRESSES

All welding processes result in high tensile residual stresses, which are at or near the yield point in the weldment and base metal adjacent to it. These occur as the weld shrinks upon cooling. Thus, in the initial stages of fatigue crack growth in an as-welded structure, most of the fatigue life occurs in regions of high tensile residual stress. Under cyclic loading, the material at or near the initial discontinuity will be subjected to a fully effective cyclic stress, even in cases of stress reversal. This is the main reason why stress range alone is the variable describing the fatigue behavior of welded joints. As a result, the effect ratio, R, does not play a significant role when describing the fatigue strength of welded details, because the maximum stress at the point of fatigue crack initiation and growth is always at the yield point. Most of the fatigue life is exhausted by the time the fatigue crack propagates out of this high tensile residual zone.

An examination of the available data has shown that cracks have grown in the tensile residual stress zones of beam flanges subjected to cyclic compression alone. However, these studies also showed that the crack arrested as it grew into adjacent compressive residual stress regions. No beams lost load carrying capability as a result of compression flange cracks unless out-of-plane bending stresses were introduced.

The existence of small cracks confined to the tensile residual stress regions of components subjected to compression alone is analogous to the tension splice proportioned to carry only part of the member's strength, with the balance of this force resisted in bearing.

As a result of this behavior, the fatigue design criteria is limited to regions subjected to tension or stress reversal. If the member is subjected to stress reversal, fatigue must be considered no matter how small the tension component of stress range is, since the crack generated in a tensile residual stress zone could be propagated to failure with very small components of the tension portion of the stress cycle. It is apparent that residual stresses play an important role in both the formation of cracks from discontinuities that reside in the tensile residual stress zone and the arrest of cracks as they grow into a compressive residual stress zone of a member subjected to compression alone.

VARIABLE STRESS CYCLES

The most widely used method to account for cumulative damage is the Miner hypothesis. Variable stress cycle damage is accumulated in proportion to the relative frequency of occurrence of each level of stress range. Other methods have been proposed, but Miner's hypothesis is among the simplest.

In order to evaluate the significance of random variable stress cycles and assess the applicability of cumulative damage criteria such as Miner's Rule or the RMS (root-mean-square) procedure, a program of study was undertaken in 1971 under the sponsorship of the National Cooperative Highway Research Program (Project 12-12). The study was carried out at the Research Laboratory of U. S. Steel Corporation on beams identical in geometry to those tested on Project 12-7 at Lehigh University.
The results of this study indicated that Miner’s linear damage hypothesis and the RMS stress range both provided methods of relating random variable stress cycles to constant cycle data. An effective stress range can be developed using Miner’s linear fatigue damage relationship \( \sum n_i/N_i = 1 \) together with Eq. (1) (see Chapter 3) as:

\[
S_{i,miner} = \sum \gamma n_i S_n^{21/13}
\]  
(8)

where \( \gamma \) is the frequency of occurrence of stress range \( S_n \).

The RMS stress range for a variable stress spectrum can be defined as:

\[
S_{i,RMS} = \left| \sum \gamma n_i S_n^{41/132} \right|
\]  
(9)

The results of coverplated beams tested under variable cyclic loading are plotted in Figs. 31 and 32 and compared with the mean and lower confidence limit given in Fig. 29 for constant cycle loading. Equation (8) was used to determine an effective Miner’s stress range for the variable stress spectrum for the points plotted in Fig. 31, and Eq. (9) was used to determine an effective RMS stress range for the test points plotted in Fig. 32. The variable stress spectrums conformed to a Rayleigh distribution as shown schematically in Figs. 31 and 32. It is apparent that Miner’s linear damage relationship and the RMS stress range both provided good methods of transforming the variable stress spectrum into an equivalent effective stress range. A second factor is also apparent at the lower levels of effective stress range. Several tests were conducted with an effective stress range below the constant cycle fatigue limit. Some cycles in the stress spectrum exceeded the constant cycle fatigue limit and this apparently caused all stress cycles to contribute to fatigue damage. The plotted points are seen to fall between the confidence limits. Hence, if no crack growth can be tolerated and extreme life is required, all stress cycles should be less than the fatigue limit.

**CURRENT RESEARCH**

Considerable research is underway in the United States and abroad on structural fatigue. Studies are continuing on the high cycle fatigue behavior of the lower fatigue strength details, variable stress cycles, curved girder details, methods to retrofit or repair fatigue-damaged members, the effect of environmental conditions, and other related problem areas.

Studies on full scale welded bridge details, completed in 1976, indicated that full sized coverplated beams have less fatigue strength than implied by Category E\(^{24}\). A comparison of this test data with results of studies on several bridges that experienced fatigue cracking shows reasonable agreement with the laboratory findings and field experience\(^{25}\).

Work currently underway on NCHRP Project 12-15(2) will provide a more comprehensive data base on full scale beams, so that an appropriate design category can be provided in the near future.

Stress history studies are continuing or are planned, so that the stress spectrum can be better defined for both highway and railroad structures. Most of the studies have focused on bridges of short or medium span length. The behavior of larger span bridges is now under study. Field measurements are also being made to help evaluate methods of retrofitting and upgrading older bridges.
APPENDIX NO. 2

(Fatigue Computer Program)
***** ORDERED, OVERALL RANGE (ALSO KNOWN AS RAINFALL) METHOD *****

***** BY JESSE G. MAYES, KTRP, UNIVERSITY OF KENTUCKY ******

January 31, 1985

This program was adapted from a program presented in a paper by

This program determines the number of stress cycles for NR stress ranges, range(1)-range(NR), summed over several consecutive hours.

Each data record (NTH) contains stress values P1(N,K) for K=1,NC channels.
P1 values should be on unit 11—see format 1000

Dimensions are shown here

Real P1(NSAMP+1), T1(NSAMP+1), RP1(NSAMP+1)
Integer P2(NSAMP+1), T2(NSAMP+1), RP2(NSAMP+1)
Dimension CHOLD(NC), CH(NC)
Dimension A(NSAMP,NC)
Dimension NSUM(NR,NC)
Dimension NSPIK(NC)
Dimension RANGE(NR)

Real P1(4801), T1(4801), RP1(4801)
Integer P2(4801), T2(4801), RP2(4801)
Real HI, LO, NXT
Integer FRST, TTL, FST
Dimension CHOLD(3), CH(3)
Dimension A(4800,3)
Dimension NSUM(30,3)
Dimension NSPIK(3)
Dimension RANGE(30)

Read miscellaneous data
C READ THE STARTING HOUR AND ENDING HOUR (NUMBERED SEQUENTIALLY)
READ(5,5000) IHST,IHEND
5000 FORMAT(2I5)
C READ THE STARTING DAY (NUMBERED SEQUENTIALLY)
READ(5,5000) IDAY
C READ THE STARTING TIME (MILITARY)
READ(5,5000) ITIME
C READ CONVERSION FACTOR (VOLTS TO STRESS IN KSI)
FACTOR = 5.934
READ(5,5100) FACTOR
5100 FORMAT(F5.0)
C READ THE KSI DIFFERENCE LIMIT TO INDICATE A VOLTAGE SPIKE
READ(5,5100) SPIK
C READ THE NUMBER OF SAMPLE POINTS IN SAMPLE INTERVAL
READ(5,5000) NSAMP
C READ THE NUMBER CHANNELS, NC
READ(5,5000) NC
C READ THE NUMBER OF STRESS RANGES, NR
READ(5,5000) NR
C READ THE STRESS RANGES, RANGE(IR)
DO 100 IR=1,NR
READ(5,5100) RANGE(IR)
DO 110 K=1,NC
NSUM(IR,K)=0
110 CONTINUE
100 CONTINUE
C DO LOOP 115 (BOTTOM OF PROGRAM) FOR EACH HOUR
C DO 115 IHOUR=IHST,IHEND
C READ P1 ARRAY AND AND DETERMINE NUMBER OF POINTS, NDATA
C NDATA <= NSAMP
DO 120 I=1,NC
NSPIK(I)=0
120 CONTINUE
C NSAMP<=100000
DO 130 NDATA=1,100000
READ(11,1000,END=9999) (CH(I),I=1,NC)
1000 FORMAT(E11.4,5X,E11.4,5X,E11.4)
   DO 140 I=1,NC
   140 CH(I)=FACTOR*CH(I)
C
C CHECK FOR VOLTAGE SPIKE--IF SO SET EQUAL TO PREVIOUS VALUE
C START CHECK WITH SECOND RECORD
   IF(NDATA.EQ.1) GO TO 150
   DO 160 I=1,NC
      DIFF=CH(I)-CHOLD(I)
      IF(ABS(DIFF).GE.SP1K) CH(I)=CHOLD(I)
      IF(ABS(DIFF).GE.SP1K) NSPIK(I)=NSPIK(I)+1
   160 CONTINUE
C
   150 CONTINUE
   DO 170 I=1,NC
      CHOLD(I)=CH(I)
   170 A(NDATA,I)=CH(I)
   130 CONTINUE
9999 CONTINUE
   NDATA=NDATA-1
   ADJ = 1.*NSAMP/NDATA
C DEFINE CHANNEL K
   DO 200 K=1,NC
C START NEW PAGE AND WRITE HEADINGS
   WRITE(6,6000)
6000 FORMAT(1X)
   WRITE(6,6010) IHOUR,IDAY,ITIME
6010 FORMAT(///////////,1OX,HOUR = ´,I2,´ DAY = ´,I2,´ TIME = ´,I4)
   WRITE(6,6020)
6020 FORMAT(1OX,SAMPLE TIME = 5 MINUTES (4800 READINGS) - ADJUSTED, &" IF NECESSARY")
   WRITE(6,6030) SPIK,NSPIK(K)
6030 FORMAT(1OX,NUMBER OF VOLTAGE SPIKES > ´,F5.2,´ = ´,I5)
C ******************************************************
C
   WRITE (6,6040) NDATA,ADJ
6040 FORMAT (10X,THE NUMBER OF DATA POINTS = ´,I5/10X, &
      ADJUSTMENT FACTOR = ´,F5.2/)
   WRITE (6,6050) K
6050 FORMAT (10X,CHANNEL NUMBER ´,I2/)
C SET P1 ARRAY = ARRAY FOR CHANNEL K
   J=0
   DIFN=0
   DO 210 I=1,NDATA
      J=J+1
      P1(J)=A(I,K)
      IF(J.EQ.1) GO TO 210
C
C MAKE SURE PREVIOUS VALUE WAS A PEAK; IF NOT THROW OUT
   DIFO=DIFN
   DIFN=P1(J)-P1(J-1)
   IF(J.EQ.2) GO TO 210
C
   IF(DIFO*DIFN.LT.0) PREVIOUS VALUE WAS A PEAK
IF (DIFO * DIFN.LT. 0) GO TO 210
P1(J-1) = P1(J)
J = J - 1

210 CONTINUE
C REDEFINE THE NUMBER OF DATA POINTS, N, TO BE THE NUMBER OF PEAKS, J
N = J
NP1 = N + 1
DO 220 I = 1, N
  P2(I) = I
220 CONTINUE
CALL HLO(P1, P2, N, HI, LO, FRST, DMIN)
CALL RESEC(P1, P2, T1, T2, N, FRST)
WRITE (6, 6060) HI, LO
6060 FORMAT (1X, 'MAX STRESS = ', F7.2, ' KSI/',
         * 10X, 'MIN STRESS = ', F7.2, ' KSI/',
         * 10X, 'STRESS RANGE   # REVERSALS',
         * '# CYCLES/')
P1(NP1) = P1(1)
P2(NP1) = NP1
NORD = 0
DOLD = 100000
C
C DO LOOP 230 FINDS THE NUMBER OF REVERSALS FOR EACH RANGE, 1 - NR
C
DO 230 IR = 1, NR
  DMIN = RANGE(IR)
  IF (DMIN .GT. ABS(HI - LO)) GO TO 32
  RPI(1) = P1(1)
  NXT = P1(2)
  RP2(1) = FRST
  TTL = 2
  NOR = 1
  XP = RP1(1)
30 IF (ABS(XP - NXT) - DMIN) 31, 31, 35
31 IF (TTL .LE. (N - 2)) GO TO 81
32 CONTINUE
  NORD = 0
 NCYCLE = 0
  IF (IR .GT. 1) WRITE (6, 6070) DOLD, DMIN, NORD, NCYCLE
6070 FORMAT (10X, "#5.1", "KSI TO ", #5.1, "KSI ", #17,8X,17)
  DOLD = DMIN
  NOR = 0
  GO TO 230
81 TTL = TTL + 2
  NXT = P1(TTL)
  GO TO 30
35 TRP = NXT
  RPT = P2(TTL)
37 TTL = TTL + 1
  IF (TTL .GT. NP1) GO TO 45
38 NXT = P1(TTL)
  IF (ABS(NXT - TRP) - DMIN) 39, 39, 40
39 TTL = TTL + 1
IF (TTL.GT.NPl) GO TO 45
NXT=P1(TTL)
XP=RP1(NOR)
IF (ABS(XP-NXT)-ABS(XP-TRP)) 37,37,35
40 NOR=NOR+1
RP1(NOR)=TRP
RP2(NOR)=RPT
GO TO 35
45 CONTINUE
IF (RP2(NOR).GE.FRST) GO TO 56
DO 50 I=1,NOR
IF (RP2(I)-FRST) 49,50,50
49 FST=I
GO TO 55
50 CONTINUE
55 CONTINUE
CALL RESEC (RP1,RP2,T1,T2,NOR,FST)
56 CONTINUE
C ADJUST TO NSAMP READINGS, IF NECESSARY
NOR=ADJ*NOR + .5
C NOR SHOULD BE ODD
NOR=((NOR-1)/2)*2 + 1
C NUMBER OF REVERSALS BETWEEN RANGE(IR-1) AND RANGE(IR) IS THE
C DIFFERENCE BETWEEN THE NUMBER OF REVERSALS ABOVE RANGE(IR-1)
C AND THOSE ABOVE RANGE(IR)
NORDIF=NOROLD-NOR
C NUMBER OF CYCLES = HALF THE REVERSALS
NCYCLE=NORDIF/2
NSUM(IR,K)=NSUM(IR,K)+NCYCLE
IF(IR.GT.1) WRITE(6,6070) DOLD,DMIN,NORDIF,NCYCLE
C6070 FORMAT(10X,¨ ,F5.1,¨ KSI TO ,F5.1,¨ KSI ,I7,8X,17)
C
C REINITIALIZE AND DO AGAIN
DOLD=DMIN
NOROLD=NOR
230 CONTINUE
200 CONTINUE
ITIME=ITIME+100
IF(ITIME.GE.2400) IDAY=IDAY+1
IF(ITIME.GE.2400) ITIME=0
115 CONTINUE
999 CONTINUE
DO 240 K=1,NC
WRITE(6,6000)
WRITE (6,6080) IHSTR,IHEND
6080 FORMAT(//////////,10X,¨ TOTALS FOR HOUR ´ ,I3,¨ THROUGH HOUR ´ ,I3/) WRITE (6,6090) K
6090 FORMAT (10X,¨ CHANNEL NUMBER ´ ,I2/) WRITE (6,6100)
6100 FORMAT( 10X,¨ STRESS RANGE # CYCLES¨/) DO 250 IR=1,NR
IF(IR.GT.1) WRITE(6,6110) RANGE(IR-1),RANGE(IR),NSUM(IR,K)
6110 FORMAT(10X,¨ ,F5.1,¨ KSI TO ,F5.1,¨ KSI ,I7) 250 CONTINUE
240 CONTINUE
   WRITE(6,6000)
   STOP
END
SUBROUTINE RESEC(P1,P2,T1,T2,N,FRST)
C THIS SUBROUTINE RESEQUENCES DATA SET
C FOR PROPER EXECUTION THE ARRAY P MUST BE DIMENSIONED TO FIT THE NUMBER
C (N) OF DATA POINTS + 1
   INTEGER FRST
   REAL P1(N),T1(N)
   INTEGER P2(N),T2(N)
   DO 15 I=FRST, N
      J=I-FRST+1
      T1(J)=P1(I)
      T2(J)=P2(I)
   15 CONTINUE
   L=FRST-1
   IF(L.EQ.0) GO TO 21
   DO 20 I=1,L
      J=I+N-FRST+1
      T1(J)=P1(I)
      T2(J)=P2(I)
   20 CONTINUE
   21 CONTINUE
   DO 25 I=1,N
      P1(I)=T1(I)
      P2(I)=T2(I)
   25 CONTINUE
   RETURN
END
SUBROUTINE HILO(P1,P2,N,HI,LO,FRST,DMIN)
C THIS SUBROUTINE FINDS HIGH AND LOW OF DATA SET AND DETERMINES DMIN
   REAL P1(N)
   INTEGER P2(N)
C FOR PROPER EXECUTION THE ARRAY P MUST BE DIMENSIONED TO FIT THE NUMBER
C (N) OF DATA POINTS + 1
   INTEGER FRST
   REAL HI,LO
   HI=P1(N)
   LO =HI
   DO 10 J=2,N
      I=N+1-J
      IF (P1(I).LE.HI) GO TO 5
      HI=P1(I)
      FRST=I
   10 CONTINUE
   GO TO 10
   5 IF (P1(I).GE.LO) GO TO 10
      LO=P1(I)
      FRST=I
   10 CONTINUE
   C DMIN=DMIN*(HI-LO)
   RETURN
APPENDIX NO. 3

(Recorded Data-Totals)
<table>
<thead>
<tr>
<th>Stress Range</th>
<th># Cycles</th>
</tr>
</thead>
<tbody>
<tr>
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<td>8463</td>
</tr>
<tr>
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<tr>
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<tr>
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<tr>
<td>7.0 KSI To 8.0 KSI</td>
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<tr>
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</tr>
<tr>
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</tr>
<tr>
<td>10.0 KSI To 11.0 KSI</td>
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<tr>
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<td># CYCLES</td>
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