LOADING HISTORY OF HIGHWAY BRIDGES: COMPARISON OF STRESS-RANGE HISTOGRAMS

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This report presents the results of a loading history field test on a rural highway bridge in Maryland. Two ways of data reduction are compared: One technique notes only one stress event per truck passage, while the other technique produces several events for each vehicle. The composition and weight of the truck traffic are presented, along with a number of occurrences of multiple crossings. Several methods of estimating the fatigue life of the bridge are also attempted. Some conclusions from the study are that significant differences in the shape of stress-range histograms can result, depending on the inclusion or exclusion of the several secondary stress ranges, but that for stress ranges above 3.0 ksi no significant differences in the histograms are found. Higher average stress ranges were produced by multiple crossings than by single crossings. It was also concluded that the main load-carrying members of this bridge are not likely to suffer from traffic-induced fatigue distress.

•DURING the past several years, there has been under way an extensive program of field tests on highway bridges to determine the loading history of the main load-carrying members. The program is promoted on a national scale by the Federal Highway Administration and is guided by committees of the American Society of Civil Engineers and the Highway Research Board.

The actual field testing and data gathering are being done by various agencies and, therefore, some differences naturally result in the final data presentation. The main end product of each study is usually a series of stress-range histograms, where the magnitude of the stress range is plotted as the abscissa and the percentile of the total number of stress ranges is plotted as the ordinate. There can be a sizable variation in percentages based on the same number of truck passages, depending on how many vibrations caused by a single truck are recorded.

Because it is desirable to draw some common conclusions from all these tests and because the field test results are being adapted to laboratory fatigue tests, it is important that there be some standardization of recording, data reduction, and presentation.

It is the purpose of this report to describe the differences that may occur in the shape of a stress-range histogram and the importance of these differences. A suggestion for a common approach to the presentation of the data is then given. It is also intended to explore several methods for relating the field test results to an estimation of the fatigue life of the structure.

The comparison of results is based on a cooperative field test made in Maryland in July 1969. A crew from the Civil Engineering Department of the University of Maryland prepared the bridge site for testing and attached the strain gages. Separate, but simultaneous, data recording was made from 8 sets of 2 gages placed adjacent to each other by the University of Maryland crew and by a crew from the Structures and Applied Mechanics Division of the Federal Highway Administration.

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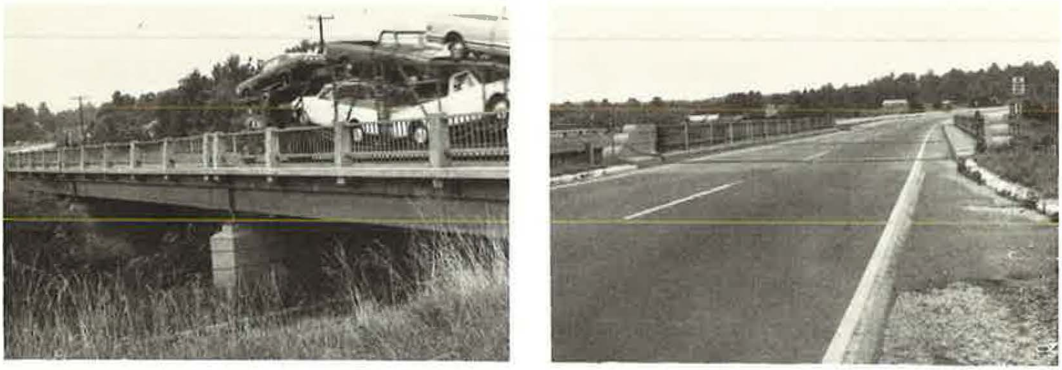

Figure 1. Bridge on US-301 southbound.

THE BRIDGE

The bridge is a 3-span continuous structure, built in 1949, and is located on US-301 near Md-4. The structure is built of steel WF sections with a 7-in. thick concrete deck and carries the southbound traffic across a small stream. Figures 1 and 2 show several views and details of the bridge.

Eight sets of strain gages were placed on 3 of the 7 girders at 3 cross sections, as shown in Figure 3. Most of the comparisons in this report will be based on readings obtained from gage position B1, which is on the second interior girder on the right side looking south and is at the middle of the first span. A more exact description of gage locations is given in another report (1).

The 2 gages at each location were oriented in the longitudinal direction on the bottom flange and were placed side by side as close to each other as physically possible. One gage served the Maryland recording equipment, and the other served the FHWA equipment. There should be no difference in strain readings of gages at each gage site.

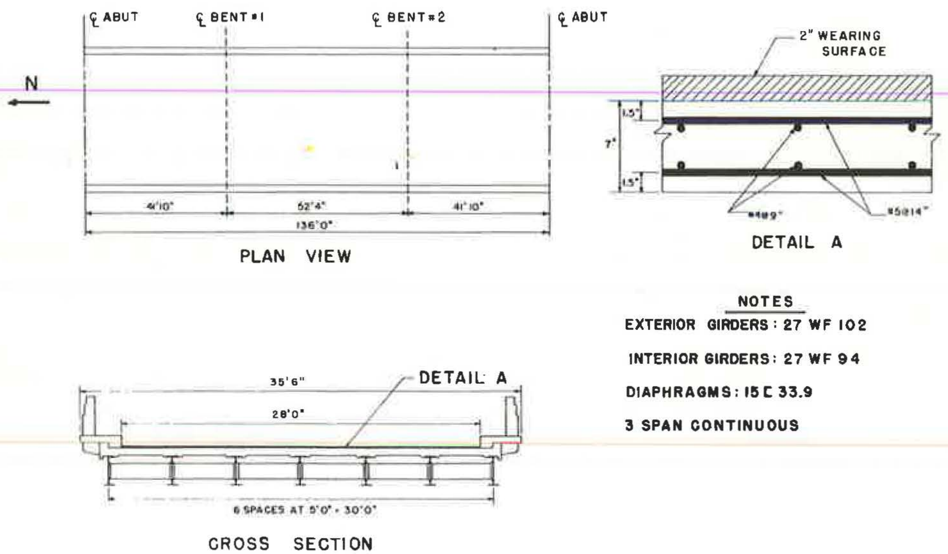

Figure 2. Structural details of bridge.

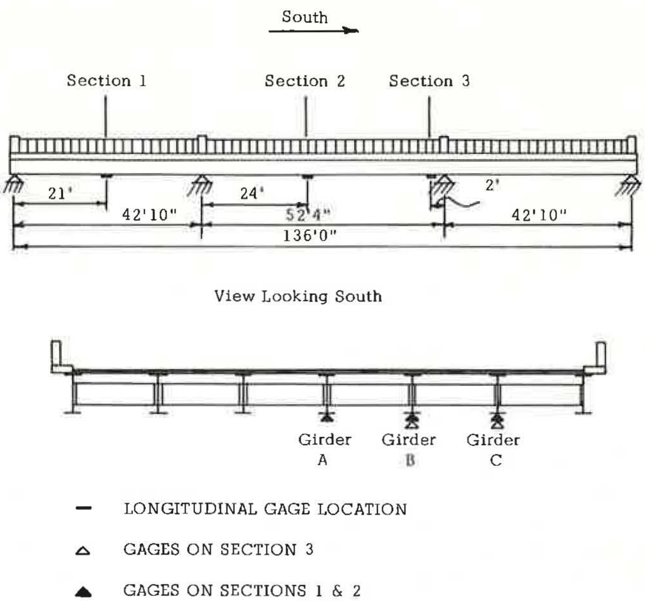

Figure 3. Strain gage locations.

The design stresses for a point corresponding to gage position B1 were as follows:

Load	ksi
Dead	5.9
Live, with impact	12.1

These calculated stresses were based on the 1944 AASHO Bridge Specifications.

DATA ACQUISITION AND REDUCTION

University of Maryland

When a truck crossed the structure, a strain trace from gage B1 resulted in a typical response curve as shown in Figure 4. A trace produced by a 3-axle dump truck is shown somewhat idealized on this figure. Other axle configurations produced slightly different response records.

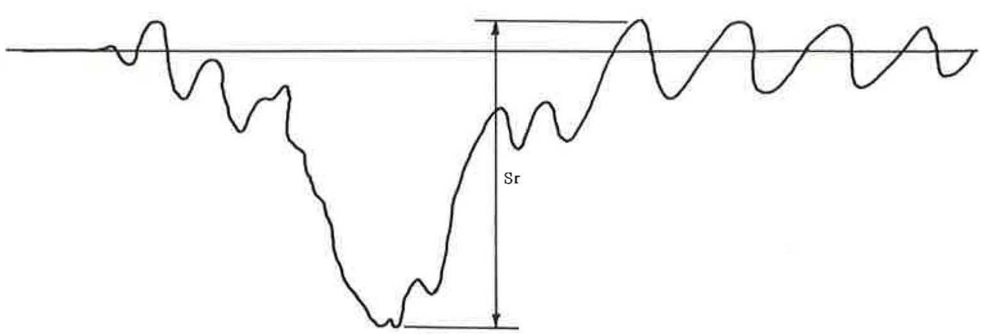

Figure 4. Typical strain trace caused by a 3-axle dump truck.

The instrumentation used by the university recorded such traces from each gage on oscillograph paper for the passage of 1,275 trucks. The principal data reduction for this test consisted of obtaining the maximum stress range only for each record. The maximum stress range, Sr, is shown in Figure 4. Notes were made during testing so that each record could be related to a specific truck. Occasionally it happened that 2 or more trucks were on the bridge simultaneously, producing, however, only 1 record.

The dynamic field data were obtained with a Brush light-beam oscillograph and two 4-channel carrier amplifiers. A time line generator and event marker system permitted evaluation of the axle spacings and vehicle speeds.

The dynamic records were edited and read on a Gerber digital-data reduction system. The system translated selected points on the record to digital output that was punched on cards.

The data from the vehicle classification notes were also punched on cards. Several computer programs were subsequently used to further process the data and produce the desired output of strains and vehicle types.

Federal Highway Administration

The data-acquisition system used by FHWA is an automated computer controlled system, which takes the output from strain gages in the form of analog voltages, digitizes these voltages, and stores and tabulates strain ranges for specified periods of time. This equipment is described further in another report (2). There is no record of a visual strain trace; neither is it possible to relate individual strain ranges to specified trucks.

The usual period of recording was 1 hour. Four minutes of each hour were used for typing the results. No strains were recorded during the typing period. Exact correlation of data between the 2 recording systems for selected hours was possible by marking on the Maryland notes the exact time that sampling began and ended on the FHWA system. Strains were sampled for 63 hours by the FHWA system, during all hours of some days.

The definition of what is recorded as a stress range in the FHWA system is shown in Figure 5. The dashed line represents a level of strain below which no recordings are made. This is done to eliminate the counting of the many small vibrations caused by cars. This level was set at 5 microinches per inch of strain in the subject test.

Figure 5 shows that, in addition to the major stress range, several other stress ranges are recorded, as long as the trace goes beyond the dashed line and returns to the zero level. Thus, it is possible for 1 truck to produce a number of stress ranges at a point in a bridge. In fact, some of the secondary stress ranges due to 1 truck may be larger than the maximum stress range caused by another truck.

Traffic Description

The traffic across the bridge is rural high-speed traffic in a nearly flat or gently rolling terrain. The speed limit is 55 mph. No vehicle speed measurements were made in this study.

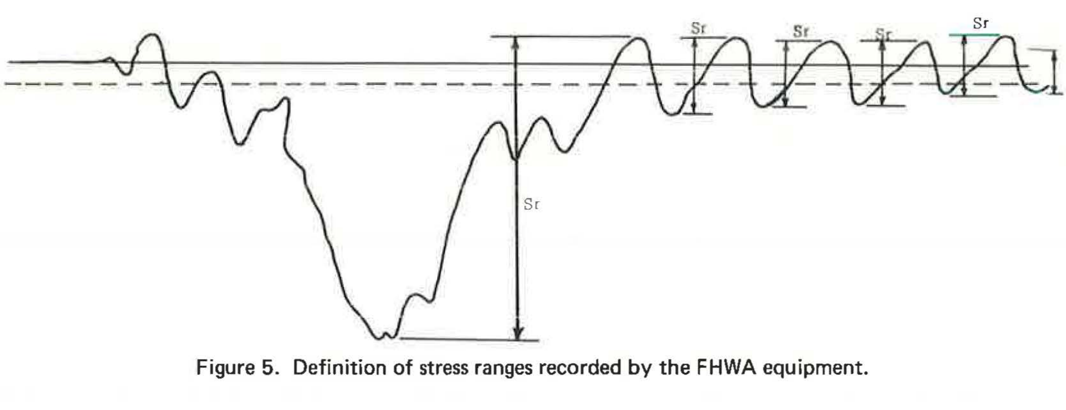

Figure 5. Definition of stress ranges recorded by the FHWA equipment.

The average daily traffic was 10,500 vehicles, with 17 percent trucks, based on the 1968 traffic count. The makeup of the truck traffic is described elsewhere in this report.

COMPARISON OF FIELD DATA

Stress-Range Histograms

The following comparisons are based on the data obtained from 7 selected hours of sampling. Both the Maryland data and the FHWA data were caused by the same 350 trucks. Those trucks missed by the FHWA system during typing periods were excluded from the Maryland data. Therefore, any apparent differences are due to the difference in the number of stress ranges only.

The 2 stress histograms resulting from the 350 trucks are shown in Figure 6. There is a marked difference in the 2 sets of data; the FHWA data show a very large percentage of small values (600 psi). The stress ranges always originate from zero. For example, the bars from 2.4 to 3.0 ksi represent a stress range from zero to somewhere between 2.4 and 3.0 ksi.

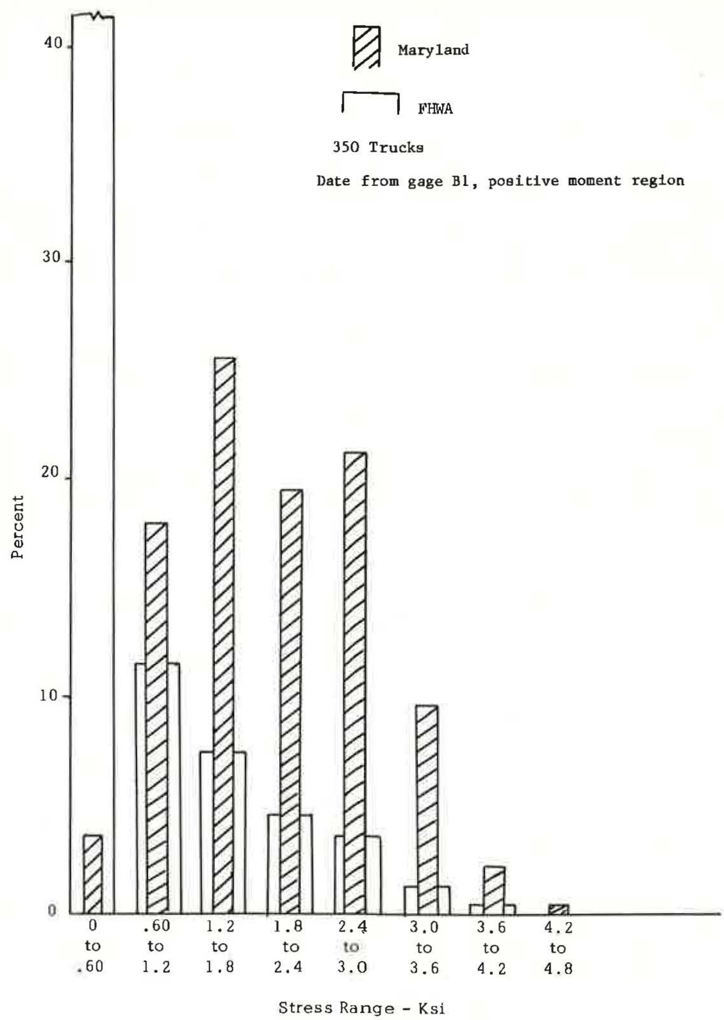

Figure 6. Comparison of stress-range histograms.

The actual numbers of stress ranges for the 2 sets of data are given in Table 1. It is seen in this table that, but for one exception, the FHWA column shows a greater number of stress ranges in each level. Only 334 stress ranges in the Maryland data result from the 350 trucks because there were several multiple crossings that produced only one record.

TABLE 1

COMPARISON OF STRESS RANGES RECORDED DURING 7 SELECTED HOURS

Stress Range (ksi)	Number of Occurrences	
	Maryland	FHWA
4.8		
4.2	1	
3.6	7	8
3.0	32	28
2.4	71	80
1.8	65	100
1.2	86	163
0.6	60	251
0	12	1,605
Total	334	2,235

Truck Traffic Composition

The composition of the truck traffic during the 7 selected hours is shown in Figure 7 for each of the hours, as well as for the total 7 hours.

No trucks were weighed during this test, but, during the previous year in July 1968, all trucks in the adjacent northbound lane were weighed during a 7-day period (3). There is no reason to believe that northbound traffic is different from southbound traffic, nor is it probable that the mean gross weight of the trucks has changed in one year. There-

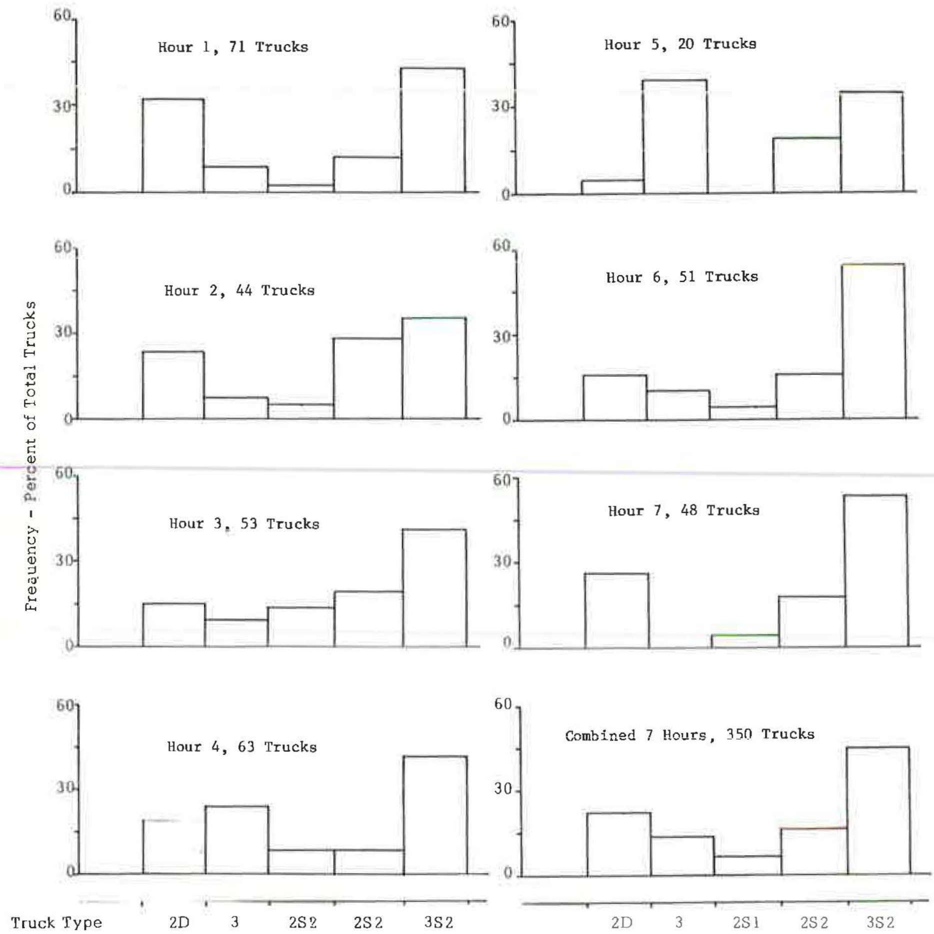

Figure 7. Distribution of truck types for each of the 7 selected hours.

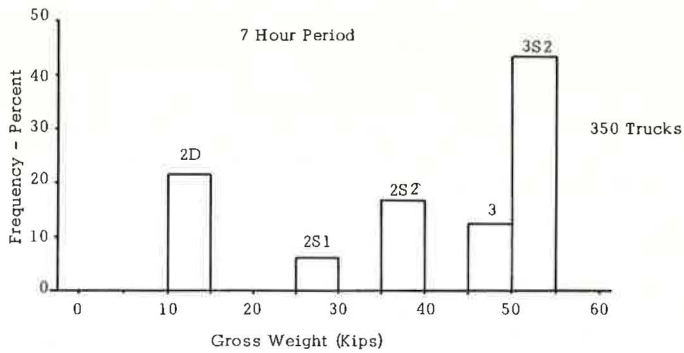

Figure 8. Frequency of mean gross weight of trucks for 7 selected hours.

fore, it will be assumed in this report that the mean gross weights of the 5 truck types are as follows:

Truck Type	Gross Weight (kips)
2D	10 to 15
3	45 to 50
2S1	25 to 30
2S2	35 to 40
3S2	50 to 55

For the 7-hour comparison, the frequency of assumed mean gross weight per truck type is as shown in Figure 8.

Stresses Above 3 ksi

It can be argued, based on past laboratory fatigue tests, that stress ranges below the fatigue limit of the material have no effect on the life expectancy of the structure and can, therefore, be ignored. However, recent tests at Lehigh University (4) on the fatigue of weldments tend to show that there may not be a material fatigue limit. There is, however, a practical fatigue limit in that it would take almost forever for small stresses to cause detectable damage.

What are "small" stresses? If all the stress ranges below 3.0 ksi are dropped from both columns in Table 1, very few numbers remain, and it becomes difficult to make statistically meaningful comparisons of the data. Even so, by using the t-test, it was shown that at the 95 percent confidence level there is no significant difference in the means of the 2 sets of data above 3.0 ksi.

A more meaningful comparison of the 2 sets of data can be made if all values above 3.0 ksi are compared for the entire test period as given in Table 2. The Maryland data were recorded during the daytime hours, while the FHWA data include some nighttime traffic. The previously selected 7-hour data (Table 1) are included with data given in Table 2. The Maryland data were collected during passage of 1,275 trucks, for 27 random hours during 4 consecutive days, and the FHWA data were collected in 61 hours and involve an estimated 2,500 trucks.

TABLE 2

STRESS RANGES ABOVE 3.0 ksi

Stress Range (ksi)	Number of Occurrences	
	Maryland	FHWA
6.0		
5.4	1	2
4.8	1	3
4.2	7	10
3.6	36	51
3.0	107	230
Total	152	296

No significant difference between the means of the 2 sets of numbers given in Table 2 was found at the 95 percent confidence level. The stress-range histograms, shown in Figure 9, will result when these numbers are converted to percentages. This figure represents the best estimate of the stress ranges above 3.0 ksi to which this bridge is subjected at the maximum positive moment section in the end spans during present-day traffic. The corresponding truck traffic distribution with the assumed mean gross weights are as shown in Figure 10.

Multiple Crossings

The bridge under investigation is a 2-lane structure and, therefore, occasionally 2 or more trucks may cross the bridge at the same time. It is also possible to have more than 1 truck in the same lane at the same time because the 3 spans add up to a total length of 136 ft.

Multiple crossings were noted and recorded during some of the sampling periods. Table 3 gives a comparison of the stress ranges produced by 53 multiple crossings and by 1,170 single crossings. The mean value of the stress ranges for multiple crossings is approximately 2.3 ksi, while the mean value for the single crossings is approximately 1.9 ksi. This results in a significant difference at the 95 percent confidence

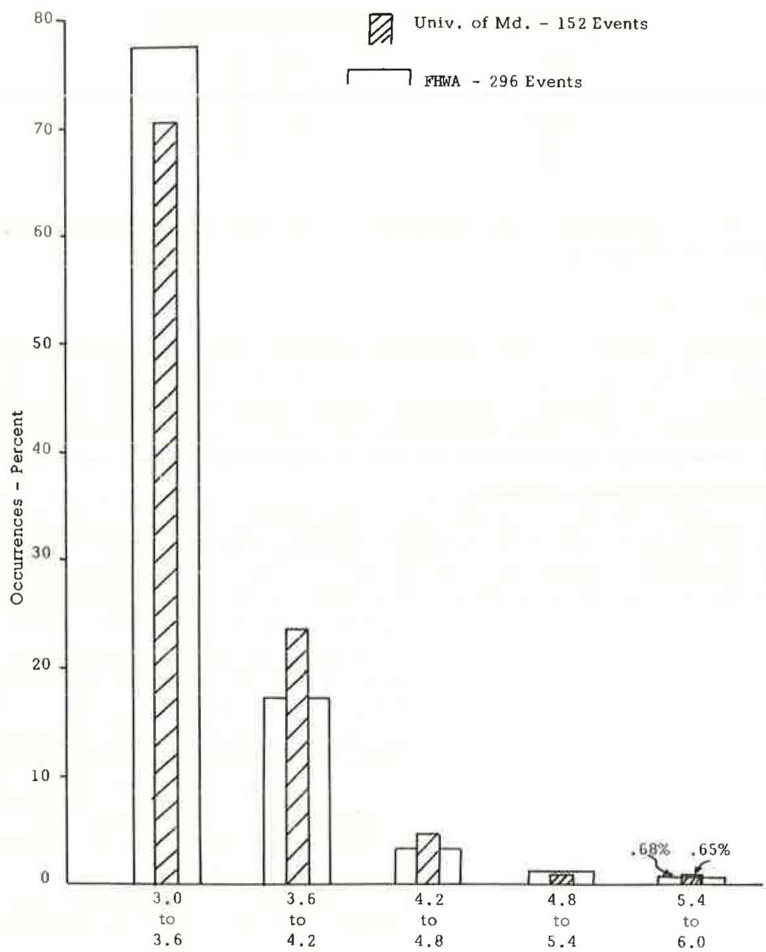

Figure 9. Stress ranges above 3 ksi.

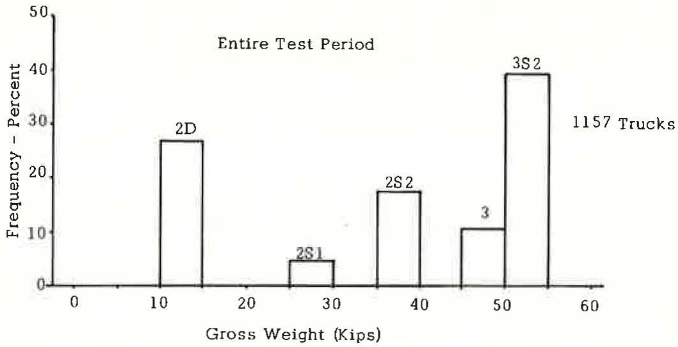

Figure 10. Frequency of mean gross weight of trucks for entire test period.

level, although it is seen in the table that none of the 53 multiple crossings produced a stress range greater than 4.2 ksi.

The 53 multiple crossings compared with the 1,170 single crossings should not be taken as a true indication of the frequency of occurrence of multiple crossings. The definition of "multiple" crossing in this test is very loose, in that what it means is that there was not time to make 2 clearly separate strain records when 2 or more trucks approached the bridge close to each other. The trucks could have been separated by as many as 150 ft as they crossed the bridge. Also, the 53 and the 1,170 crossings occurred in about 32 daylight hours in a total time of 4 days.

PROBABLE FATIGUE LIFE

Given certain material behavior characteristics under cyclic loading, and with certain other assumptions, an attempt can be made to determine the probable fatigue life of this structure.

Because there are no stress raisers, as in partial-length cover plates, it will be assumed that the governing material behavior will be that pertaining to plain rolled beams. The most critical section is the positive moment section in the side spans. Gage B1 was located in this section. The bridge is made continuous by riveted splices, which, of course, are not in places of high moment. The negative stresses, both calculated and measured, were not as high as the positive moment stresses.

Several different approaches to estimate the fatigue life of this structure will now be explored. The several methods are each somewhat related, and the assumptions associated with them will be presented for each use.

TABLE 3

STRESS RANGES CAUSED BY MULTIPLE AND SINGLE CROSSINGS

Stress Range (ksi)	Number of Occurrences	
	Multiple Crossings	Single Crossings
6.0		
5.4		1
4.8		1
4.2		7
3.6	4	32
3.0	7	100
2.4	12	201
1.8	15	246
1.2	10	297
0.6	5	251
0	—	34
Total	53	1,170

Root Mean Square Method

There are some limited laboratory fatigue data (4) that tend to indicate that one can relate random, variable load stress ranges to constant stress ranges by calculating the root mean square of the variable ranges, which is then assumed to produce the same damage as the constant stress range of the same value. Using the measured stress ranges and their frequencies above 3.0 ksi, one obtains a root mean square stress of 3.6 ksi.

In the recently completed and extensive tests of constant cycle fatigue made at Lehigh and Drexel Universities, the following relationship between constant cycle life and stress range for A-36 plain rolled sections was developed:

$$\text{Log } N = 10.637 - 2.943 \log Sr \quad (1)$$

where Sr = stress range.

The laboratory investigation did not go beyond 10 million cycles, and it is, therefore, assumed that Eq. 1 holds true beyond this value. Substituting 3.6 ksi for Sr, one obtains a life of 1×10^9 cycles.

The present ADT across the bridge is about 10,000 vehicles. Assuming that 20 percent of this is trucks, one gets about 730,000 trips per year. If a further assumption is made that 12.5 percent of the trucks produce the 25 percent stress ranges above 3.0 ksi, there remain about 182,500 damage-producing stress ranges per year. The bridge was constructed in 1949 and, if whatever the traffic lacked in weight and frequency in the last 20 years will be made up in the next 20 years, one can assume that in 40 years there will be $40 \times 182{,}500 = 7{,}300{,}000$ cycles of damage-producing stress. This is still far below the 1×10^9 cycles to failure found earlier.

Another way of looking at this matter is to calculate backward in Eq. 1, substituting 7.3 million cycles for N. One gets a stress range of 18.7 ksi, which is above the combined dead and live load allowable stress.

Miner's Method

A more common procedure for estimating cumulative fatigue damage is to use Miner's hypothesis (5), which says that damage is proportional to the number of applied cycles divided by the total number necessary to produce failure at a certain stress range. The summation of all the fractions at the various stress ranges is equal to unity at failure.

To use this method, one must have an appropriate S-N curve, and one must know or properly estimate the number of cycles at the various stress ranges. Instead of an S-N curve, Eq. 1 will be used, again with the assumption that it holds true beyond 10 million cycles.

Based on the field test, it is estimated that the 182,500 yearly stress ranges above 3.0 ksi are distributed as follows:

ksi	Stress
3.3	127,750
3.9	30,500
4.5	9,125
5.1	5,475
5.7	1,825
6.3	1,825

It is also assumed that both the number and the distribution of stress ranges can be held constant for 40 years.

The summation of the fractions is less than 0.0005, and it is evident that this bridge does not have a fatigue problem.

Extreme Load Method

It is often argued that, although no danger of fatigue distress in highway bridges appear to exist at present, the weight and number of trucks are ever increasing and we must look to the future. Such an argument is not entirely valid because neither the weight nor the number of trucks on any road can increase upward without limits.

It is very rare to have truck volumes greater than 200 per hour on a highway. As the percentage of trucks increases, the total traffic volume decreases; thus, the average speed will also decrease, and the level of service generally deteriorates. However, to illustrate the effect of an "extreme" load condition, an attempt will be made here to produce an upper limit of truck traffic capacity across this bridge.

From the Highway Capacity Manual (6), it appears to be possible to have 420 trucks per hour on a 2-lane, 1-way, level highway, along with about 1,200 cars all traveling at 35 mph under ideal conditions. This adds up to 10,080 trucks per day and 3,679,200 trucks per year. If the bridge is in service for 30 years more, it will supposedly receive about 100 million truck crossings.

It is not too unreasonable to assume that the gross weight limit is 100 kips. It will be assumed for this calculation that at least half of the trucks are fully loaded to 100 kips. This is, of course, not substantiated by the present field tests.

An approximate relationship between gross weight and live load stress range was developed in an earlier investigation (7) for simple spans. This relationship adapted to the present case leads to the following expression:

$$Sr = 1.3 + 0.053\ G_W \qquad (2)$$

where G_W is the gross weight in kips. For 100 kips, this results in a stress range of 6.6 ksi. Assuming that this is the root mean square stress range introduced earlier and substituting Sr = 6.6 ksi into Eq. 1, we obtain a total of 210 million cycles necessary to produce failure.

Again, it is evident that, even under such abnormal traffic conditions, no fatigue distress will result in the plain rolled section under study. If the bridge had been constructed with beams that had partial length, end-welded cover plates, it could only withstand about 4 million cycles of a 6.6 ksi stress range. However, it could withstand the more realistic root mean square stress range of 3.6 ksi about 24 million times. This again assumes that the appropriate log N versus log Sr relationship is valid beyond 10 million cycles.

SUMMARY

The results of a loading-history field test on a rural highway bridge in Maryland were presented. Two ways of data reduction were compared: One technique noted only one stress event per truck passage, while the other technique produced several events for each vehicle. It was shown that the 2 methods could lead to widely different fatigue life assumptions, but that, for stress ranges above 3.0 ksi, practically identical stress-range distributions and frequencies resulted.

The composition of the present-day truck traffic was presented, with the weight data being adapted from the adjacent northbound lanes.

Several methods of estimating the remaining life of the bridge were presented. Some used conventional theories and constant cycle laboratory test data, while others relied on more recent test results and theories. One extreme case of traffic condition was also presented.

It is believed that the stress-range histogram as presented for the stresses above 3.0 ksi (Fig. 9) is meaningful and is representative of the present conditions on this structure.

The truck traffic classification is also believed to be representative of true conditions. A somewhat lower reliability should be placed on the truck weight data because it was obtained in the previous summer and borrowed from the adjacent roadway.

The dual truck crossing data should be regarded with the caution as previously described. It is believed that much fewer meaningful dual crossings occur than the data indicate.

Of the several methods of estimating the fatigue life of the structure, the root mean square method appears most promising, although whether this approximation of a constant stress cycle is always valid must be further tested in the laboratory.

The extreme loading method is somewhat extreme, however, it does serve the purpose of pointing out that structures with plain rolled beams are not likely to suffer from fatigue damage.

The assumption that the several log N versus log Sr relationships remain linear beyond 10 million cycles is probably not correct, but this assumption yields conservative results.

CONCLUSIONS

Several conclusions and recommendations can be made, based on the field test results and the accompanying fatigue analysis.

1. Significant differences in the shape of stress-range histograms can result, depending on the inclusion or exclusion of the several secondary stress ranges produced by a single vehicle.
2. No significant difference in the shape of the stress-range histograms resulted when only stress ranges above 3.0 ksi were considered. This conclusion may not be universally applicable to other bridges, and it is recommended that in other field tests, in addition to recording the major stress range produced by each vehicle, any secondary stress ranges above 3.0 ksi be recorded as well.
3. There was a significant difference between the means of stress ranges caused by dual crossings and those caused by single crossings. More experimental evidence on the nature and frequency of dual or multiple crossings needs to be gathered, including a variety of traffic situations.
4. It appears that the main load-carrying members of this bridge do not now, did not in the past, and likely will not ever in the future suffer fatigue distress caused by traffic-induced stresses. However, this conclusion has not been verified in this bridge for the deck reinforcing steel and secondary members, such as the diaphragms.
5. It also appears that some adjustments are needed in the main load member fatigue provisions of the 1969 AASHO Bridge Specifications. Both the allowable stress range and the number of design cycles should be modified. Field test results from this study showed that there was never a recorded live load stress range that exceeded one-half of the design live load with impact stress range.
6. Further laboratory fatigue work is recommended, both to extend S-N curves to as many as 200 million cycles, and especially to determine the effects of variable loadings.

ACKNOWLEDGMENTS

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