FATIGUE LIFE OF BRIDGES UNDER REPEATED HIGHWAY LOADINGS

Terry R. Douglas, Department of Civil Engineering, University of Alabama

An investigation was made to determine cumulative effects of vehicle loadings on the useful life of steel-stringer highway bridges typical of those encountered on heavily traveled routes in Alabama. Three typical steel-stringer bridges were selected for strain-gauge instrumentation and data collection to obtain their in-service stress histories. Strain-event data were collected at critical points on the steel stringers of each bridge until a representative 24-hour around-the-clock sample was obtained. Truck counts and classifications were made during the 24 hours of sampling to estimate the annual volume of trucks producing significant stresses in the steel stringers and to establish truck-type frequency distributions at each bridge. Fatigue curves were developed for the steel stringers and were used in conjunction with the stress histories and currently accepted fatigue concepts to estimate the fatigue life of each bridge. Results indicated that the bridges were not in present danger of fatigue failure. However, this conclusion is limited to the 3 bridges investigated and the interpretation of the cumulative effects of the stress events recorded at each bridge.

•THIS REPORT is a study of cumulative effects of vehicle loadings on the useful life of steel-stringer highway bridges typical of those encountered on heavily traveled highways in Alabama. It is directed toward the experimental determination and interpretarion of the ranges and frequencies of the dynamic live-load stresses produced in the steel stringers of 3 such bridges under normal traffic conditions during a typical 24-hour period at each location.

The main objectives of this investigation were to (a) determine the frequencies of various levels of stress at several selected critical points on the steel stringers of each bridge; (b) relate the spectrum of truck types to the spectrum of dynamic live-load stresses produced at selected critical points on the steel stringers of each bridge; and (c) correlate the dynamic live-load stress events produced at selected critical points by normal truck loadings with accepted fatigue concepts for predicting the fatigue life of each bridge.

Each bridge is located on a 4-lane, divided highway carrying a traffic volume in each direction of 5,000 to 7,500 vehicles per day including 750 to 1,000 trucks other than panels and pickups. Results from loadometer studies on these routes indicated that approximately 10 percent of all trucks have loads heavier than those recommended by AASHO. Approximately 19 percent of the single-unit, 3-axle trucks were found to be overloaded; furthermore, about 6 percent of those were overloaded by 50 percent or more.

With continually increasing sizes, weights, and volumes of heavy trucks in highway traffic, the Bridge Bureau of the Alabama Highway Department has been increasingly concerned about the increased live-load dynamic stresses resulting from such traffic and the probable effects such stresses have on fatigue life of its bridges, particularly old bridges. This concern about the fatigue life of bridges is shared by all other highway departments and the Federal Highway Administration. This study was aimed at determining whether the 3 bridges investigated, which are typical of most steel girder designs, were subject to structural distress from fatigue stresses in the rolled steel girders.

TEST BRIDGES

The 3 bridges selected for investigation had steel-stringer spans with diaphragms or crossbeams connecting the individual stringers at the ends of the span and at intermediate points. Two of the bridges were simple-beam spans, and the third was the first span of a 3-span continuous bridge. Each bridge had a reinforced concrete deck and was designed for AASHO HS20-44 loading. The 2 simple spans were composite and the 3-span continuous was noncomposite construction. Each bridge had partial-length-tapered end cover plates with continuous fillet welds all around. The test bridge number, location, type, and span length are given in Table 1.

INSTRUMENTATION

At each test bridge electrical resistance strain gauges were attached to the 2 steel stringers in the primary traffic lane at the points of maximum moment and at points 4 in. from the ends of the cover plates. Previous study (1) has indicated that a distance of 4 in. from the end of the cover plate is far enough so that the strain measured is not at the point of maximum stress concentration and is near enough so that measurements reflect the strain response at the end of the cover plate. All strain gauges were located immediately below the web on the outer surface of the bottom flanges of the steel girders

DATA COLLECTION

In-service stress history for each of the 3 bridges was obtained by collecting strain-event data concurrently from the 4 individual gauge locations for given time intervals, ranging from 4 to 8 hours, until a representative 24-hour around-the-clock sample was obtained. Truck counts and classifications were made at each bridge during the 24 hours of sampling to estimate the annual volume of trucks producing significant stresses in the steel stringers and to establish truck-type frequency distributions at each bridge.

Preliminary monitoring at the 4 selected critical points on each bridge indicated that panels, pickups, and automobiles caused no significant stresses at those selected points. Therefore, the representative 24-hour samples were restricted to the truck types shown in Figure 1, to buses, to tractors towing house trailers, and to any trucks not otherwise classified or excluded. Sampling of the strain events began in April 1969 and was concluded in June 1970. Samples were taken Monday through Thursday, excluding holidays, during the months of April, May, June, and July.

FREQUENCIES OF TRUCK TYPES

Frequency distributions of the 5 most common truck types as determined from the representative 24-hour sample for each bridge location are shown in Figure 2. In general, these distributions were quite similar except for some minor variations. Type III trucks were more frequent on bridge 1 than on the other bridges probably because those trucks were used in coal mine operations in the immediate vicinity. It was not unusual for this type of truck to cross the bridge 2 to 4 times daily between 8:00 and 3:00. Figure 2 shows that type 3S2 trucks were the most frequent on each bridge.

Figure 2 also shows the number of trucks counted during the representative 24-hour sample. This 24-hour count, when multiplied by 365, was used to estimate the annual truck volume causing possible significant stresses in the steel stringers.

STRAIN RESPONSE AND STRESS HISTORIES

A typical oscillograph record showing the strain response at the point of maximum moment and at the end-of-cover-plate locations of both instrumented stringers of bridge 1 is shown in Figure 3. This response was caused by the passing of a heavily loaded, type III truck traveling approximately 40 mph in the primary traffic lane of the bridge. The strain response is characterized by a single maximum value with no significant rebound strain. The oscillograph record shown in Figure 3 is typical of the several thousand strain events recorded for the test bridges. The stress range for a single event is defined as the algebraic difference between the maximum and minimum stresses.

Table 1. Description of test bridges.

Bridge	Span Length (ft)	Bridge Type	Location
1	80	Simple span, rolled beams with tapered end cover plates	Over Southern Railroad on road from Sayre to Alden
2	50	Simple span, rolled beams with tapered end cover plates	Over Warrior River on road from Kimberly to Blount County line
3	$80\frac{1}{2}$	First span of 3-span continuous, rolled beams with tapered end cover plates	Over Alabama River on road from Hunter Station to Prattmont

Figure 1. Truck identification and codes.

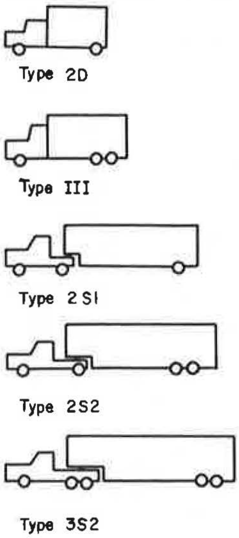


Figure 2. Distribution of 5 most common types of trucks.

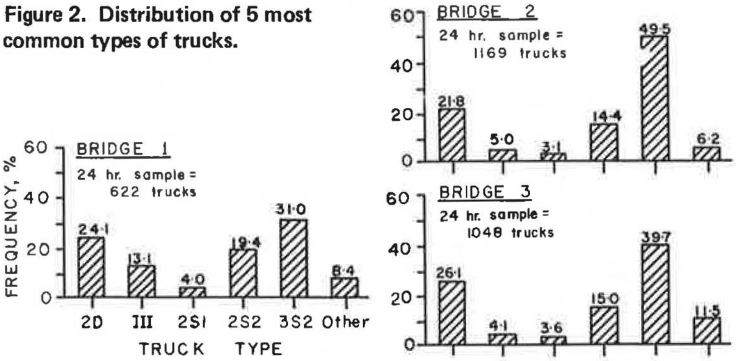


Figure 3. Typical strain or stress response of bridge 1.

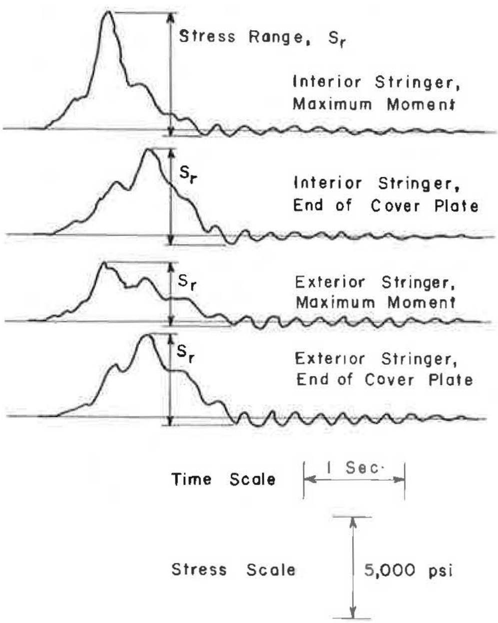
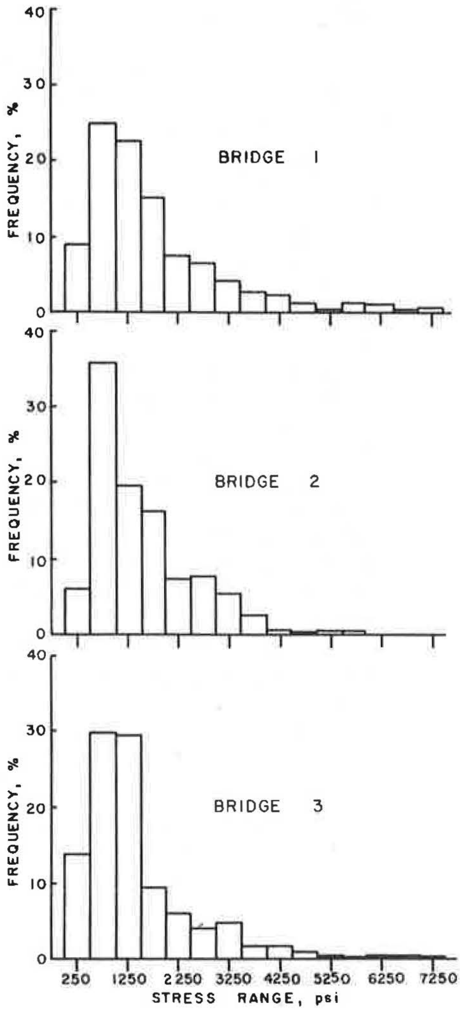


Figure 4. Distribution of stress range at end-of-cover-plate point on interior stringer.



Summaries of the stress-range frequencies for the most critical point, insofar as fatigue is concerned, on each bridge are shown in Figure 4. These stress-range frequency distributions were determined from the strain events recorded during the 24-hour sampling period for each bridge.

FATIGUE CURVES

Fatigue curves presented in this report for steel beams with partial-length cover plates were developed from the Munse-Stallmeyer data (2) and the modified Goodman law (3) to establish stress-range values and their corresponsing number of cycles to failure. Munse and Stallmeyer presented test results demonstrating the effects of structural details on the fatigue behavior of welded flexural members. Included in the data for partial-length-tapered end cover plates welded all around were the stress range values at 2×10^6 cycles and 1×10^5 cycles for constant cycle zero-to-tension loading. The stress-range values at 2×10^6 cycles and 1×10^5 cycles were reported as 11,400 and 34,000 psi respectively. These data were obtained from tests conducted on A-373 steel rolled beams with partial-length-tapered end cover plates welded all around by using E7016 electrodes and manual arc welding.

The Munse-Stallmeyer fatigue data on partial-length-tapered end cover plates were obtained by constant cycle zero-to-tension tests. The minimum stress during these cyclic loading tests wes zero. In actuality, because of the weight of the bridge structure itself, there exists in the steel bridge stringers a minimum or dead-load stress of other than zero. A loading test that includes a dead load or minimum stress is referred to as a constant cycle dead load-to-tension loading test. The effect of this dead load or minimum stress for a constant fatigue life is to reduce the corresponding stress range value.

One procedure for determining the effect of a dead load or minimum stress for a constant fatigue life on the stress-range value obtained from zero-to-tension loading tests is shown in Figure 5. This procedure is known as the modified Goodman law or modified Goodman diagram. These diagrams were constructed from the basic Munse-Stallmeyer data by assuming an ultimate tensile strength of 60,000 psi. The stress-range value (3,800 psi) at 2×10^8 cycles for zero-to-tension loading ($S_{min} = 0$) was calculated as $\frac{1}{3}$ the stress-range value at 2×10^6 cycles ($S_{min} = 0$) according to House Document 354 (4). The diagrams shown in Figure 5 are plotted as straight lines in such a way that they converge on the ultimate tensile strength. As demonstrated by Grover, Gordon, and Jackson (3), this straight-line approximation of fatigue behavior at various lifetimes gives conservative values of stress range for minimum stresses below the ultimate tensile strength.

Fatigue curves shown in Figures 6 and 7 were developed for the end-of-cover-plate gauge locations only because the stresses determined at other gauge locations (at points of maximum bending moment) were well below the endurance limit of the material. These fatigue curves were developed by using 2 methods in which dead-load stresses were both neglected and considered. This procedure resulted in a total of 4 different curves describing the fatigue behavior at the end-of-cover-plate gauge locations. Dead-load stresses at the end of cover plates were estimated from bridge plans, and an average value of 7,500 psi was selected as being representative and was used in preparing the fatigue curves.

Method 1 fatigue curves (Fig. 6) were developed by assuming a linear log-log relation between stress range and cycles to failure. Method 2 fatigue curves (Fig. 7) were developed by assuming a linear relation between stress range and the logarithm of cycles to failure.

FATIGUE LIFE

The service conditions of bridges require that the steel stringers undergo many cycles of stress having many different magnitudes applied in a random manner. Miner's cumulative fatigue damage theory allows these factors to be considered when fatigue data are analyzed (5).

Figure 5. Modified Goodman diagram prepared from Munse-Stallmeyer data for partial-length-tapered end cover plates welded all around.

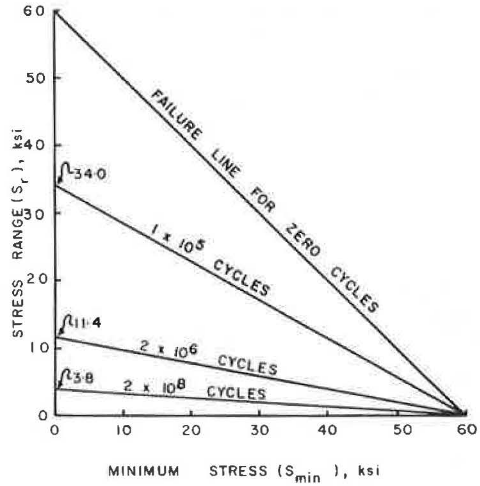


Figure 6. Method 1 fatigue curves.

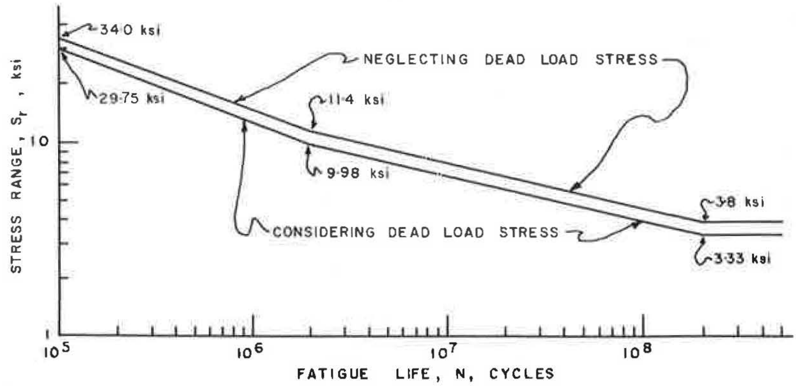


Figure 7. Method 2 fatigue curves.

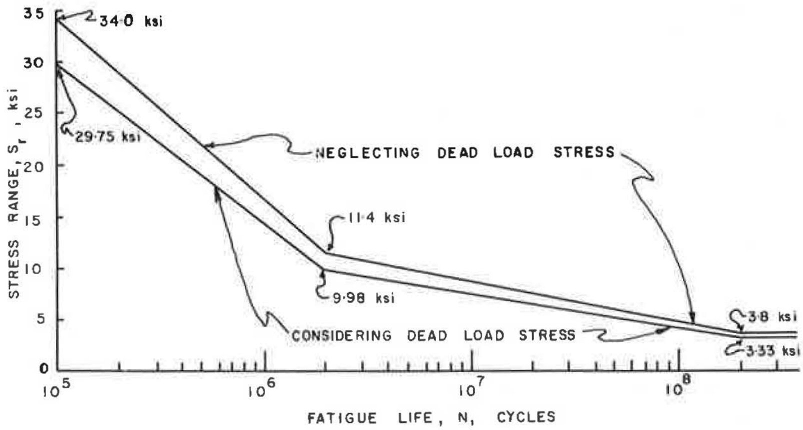


Table 2. Estimated fatigue lives and annual truck volume for test bridges.

Bridge	Annual Truck Volume	Fatigue Life Considering Dead-Load Stress (year)		Fatigue Life Neglecting Dead-Load Stress (year)	
		Method 1	Method 2	Method 1	Method 2
1	227,030	1,329	2,118	2,160	3,516
2	382,520	2,599	4,198	4,516	6,640
3	426,685	6,181	8,536	10,742	12,226

In Miner's theory it is hypothesized that, if a material is subjected to n_i cycles at a stress-range level S_i where its expected fatigue life is N_i cycles, the fractional fatigue life used up is n_i/N_i. It is further assumed that, if a material is subjected to a stress-range spectrum $S_1, S_2, \ldots, S_i, \ldots$ and if the cycles are applied $n_1, n_2, \ldots, n_i, \ldots$ times at the respective stress-range level, then fatigue failure will occur when

$$\sum_{i=1}^{k} n_i/N_i = 1$$

Miner's cumulative damage theory was used in conjunction with each of the 4 fatigue curves for determining the fatigue life of each bridge based on the stress-range frequency distribution at the end-of-cover-plate gauge location on the most highly stressed stringer. For each bridge the interior stringer was found to be most highly stressed.

Table 2 gives the predicted fatigue life of each bridge and the estimated annual truck volume causing possible significant stresses in the steel stringers (details of procedure used to determine fatigue lives and given in the Appendix.) The estimated annual truck volume, determined by multiplying the 24-hour count by 365, and the stress-range frequency distributions were assumed to be constant for the entire life of each bridge in determining the fatigue lives.

With both methods, the fatigue lives determined by considering dead-load or minimum stress were significantly less than fatigue lives determined by neglecting dead-load stress. Fatigue lives based on method 1 were less than corresponding ones based on method 2. The several fatigue lives were calculated for each bridge to emphasize the wide variations that exist depending on the particular relation assumed between stress range and cycles to failure and whether minimum stresses are considered. The large variations in fatigue life predicted by these several procedures would indicate that additional experimental work is needed for determining more rational estimates of fatigue life, thereby narrowing the range of uncertainty.

CONCLUSIONS

Only a casual inspection of the estimated fatigue lives given in Table 1 for the 3 bridges is required to see that in no case is the fatigue life less than 1,329 years, and in most cases it is considerably longer. If it is assumed that either of the procedures used for determining fatigue lives is reasonably valid, one would conclude that currently employed bridge design specifications lean heavily toward the conservative side. This conclusion, however, is necessarily limited to the 3 bridges investigated and to the critical points considered on the stringers of these bridges.

The validity of the fatigue lives given in Table 1 is limited to the applicability of the Munse-Stallmeyer data, Goodman's modified law, Miner's cumulative damage theory and the interpretations explained in the body of this report.

The Munse-Stallmeyer data were extrapolated by assuming the stress range value 2×10^8 cycles to be $\frac{1}{3}$ the stress range value at 2×10^6 cycles according to House Document 354 (4). This extrapolation was necessary because no fatigue data in this range were available.

The Munse-Stallmeyer data were restricted to stress-range values obtained from constant cycle zero-to-tension loading tests. To incorporate the effect of a constant dead-load or minimum stress on the fatigue lives, the modified Goodman law was used to modify the stress-range values obtained from constant cycle zero-to-tension loading tests.

Miner's cumulative damage theory assumes that fatigue damage is independent of the order of application of the various stress levels. In reality, bridges are subject to random loadings and hence random stresses, and the use of a fatigue damage concept neglecting this fact may not be entirely valid.

Other factors that were not considered in this study and that could significantly affect the fatigue life of the steel-stringer bridges include the effect of combined stresses, surface roughness, residual stresses, temperature ranges, creep, and corrosion. Each of these factors would tend to alter the fatigue life if their effects could be incorporated

ACKNOWLEDGMENTS

This investigation was performed in the Civil Engineering Department of the University of Alabama at Tuscaloosa under the sponsorship of the Alabama Highway Department and the Federal Highway Administration. The support, cooperation, and assistance given by these organizations is gratefully acknowledged. However, the opinions, findings, and conclusions expressed in this paper are those of the author and not necessarily those of the Alabama Highway Department or the Federal Highway Administration.

REFERENCES

1. Hulsbos, C. L., and Linger, D. A. Dynamic Tests of a Three-Span Continuous I-Beam Highway Bridge. HRB Bull. 279, 1961, pp. 18-46.
2. Munse, W. H., and Stallmeyer, J. E. Fatigue in Welded Beams and Girders. HRB Bull. 315, 1962, pp. 45-62.
3. Grover, H. J., Gordon, S. A., and Jackson, L. R. Fatigue of Metals and Structures. U.S. Govt. Printing Office, Washington, D.C., 1954.
4. Maximum Desirable Dimensions and Weights of Vehicles Operated on the Federal Aid System. 88th Congress, 2nd Session, August 19, 1964, House Doc. 354, U.S. Govt. Printing Office, Washington, D.C., pp. 143-149.
5. Miner, M. A. Cumulative Damage in Fatigue. Jour. of Applied Mechanics, Vol. 12, Sept. 1945, pp. A-159.
6. Cudney, G. R. The Effects of Loadings on Bridge Life. Highway Research Record 253, 1968, pp. 35-71.
7. Richart, F. E., Jr., and Newmark, N. M. An Hypothesis for the Determination of Cumulative Damage in Fatigue. Proc. ASTM, Vol. 48, 1949.
8. Dolan, T. J., Richart, F. E., Jr., and Work, C. E. The Influence of Fluctuations in Stress Amplitude on the Fatigue of Metals. Proc. ASTM, Vol. 49, 1949, pp. 646-682.
9. AASHO Road Test: Report 4—Bridge Research. HRB Spec. Rept. 61D, 1962.
10. Daniels, J. H., and Fisher, J. W. Fatigue Behavior of Continuous Composite Beams. Highway Research Record 253, 1968, pp. 1-20.
11. Stephenson, H. K., Noel, J. S., and Mayfield, A. D. Truck Weight Trends Related to Highway Structures. Texas Transportation Institute, Bull. 19, July 1962.
12. Galambos, C. F., and Armstrong, W. L. Loading History of Highway Bridges. Federal Highway Administration, May 1968.

APPENDIX

EXAMPLE OF PROCEDURE USED TO DETERMINE FATIGUE LIVES

This example illustrates the calculations for determining the fatigue life of bridge 1 based on the stress ranges and frequencies at the end-of-cover-plate point on the most highly stressed stringer; method 1 considering dead-load or minimum stress is used. (Method 1 provides a more conservative estimate of fatigue life than method 2.) Other fatigue lives given in Table 1 were determined in a similar manner.

1. From the proper stress-range frequency distribution (Fig. 4), determine the stress ranges and their corresponding frequencies as given in Table 3, columns 1 and 2 respectively. The estimated annual truck volume causing possible significant stresses was estimated from the 24-hour count to be 22,030.
2. Multiply each of the stress-range frequencies by the annual truck volume to determine the annual number of damage cycles at each of the stress-range levels as given in column 3 of Table 3.
3. From the proper fatigue curve (Fig. 6), determine the fatigue life at each of the stress-range values given in column 4 of Table 3.

Table 3. Fatigue life factors for bridge 1.

Stress Range (ksi)	Stress-Range Frequency (percent)	Annual Damage Cycles	Fatigue Life (10^7)	Annual Damage (10^{-3})
7.25	0.645	1,464.3	0.761	0.192
6.75	0.483	1,096.6	1.027	0.106
6.25	0.967	2,195.4	1.418	0.155
5.75	1.130	2,565.4	2.011	0.128
5.25	0.483	1,096.6	2.945	0.037
4.75	1.130	2,565.4	4.480	0.057
4.25	2.410	5,471.4	7.142	0.077
Total				0.752

4. Divide each of the annual damage cycles by the corresponding fatigue life as given in column 5 to obtain the annual damage factor at each of the stress-range values.

5. Add all the annual damage factors in column 5 to obtain the total annual damage.

The fatigue life is determined by taking the reciprocal of the total annual fatigue damage that corresponds to the fatigue life given in Table 2. For this example the total annual fatigue damage was found to be 0.752×10^{-3} or 0.000752. This means that 0.000752 represents the fractional part of the fatigue life of bridge 1 that is used up annually. Therefore, the estimated fatigue life of bridge 1 is as follows:

$$\text{Fatigue life} = 1/\text{total annual fatigue damage}$$
$$= 1/(0.752 \times 10^{-3}) = 1{,}329 \text{ years}$$

The fatigue life calculated in this example and those given in Table 2 were determined by assuming a constant annual volume of trucks causing possible significant stresses. If the annual truck volumes were to increase, there would be a corresponding decrease in the bridge fatigue lives.

If it is assumed that the annual truck traffic at the end of the bridge fatigue life is 10 times the present 227,030 and if it is further assumed that the increase is linear with time, it would mean that the average annual truck volume would be 5.5(227,030). The average total annual fatigue damage would be 5.5 times the present 0.752×10^{-3}. Based on these assumptions, therefore, the estimated fatigue life of bridge 1 would be $1/5(0.752 \times 10^{-3}) = 241$ years.

If the annual volume of truck traffic at bridge 1 were to remain constant from year to year, its estimated fatigue life by method 1 considering minimum (dead-load) stress would be 1,329 years. But if it is assumed that the annual truck traffic increases linearly with time and reaches 10 times the present volume at the end of the bridge fatigue life, the estimated fatigue life would be reduced to 241 years.

Because a tenfold increase in the annual truck traffic is not likely to be reached, it would appear that the estimated fatigue life of bridge 1 would be somewhere between 241 and 1,329 years. If perfect maintenance, no change in vehicle weights and composition of truck traffic, and a more moderate increase in such traffic are assumed, the theoretical fatigue life of bridge 1 would probably be between, say, 500 to 600 years.