COMPARISONS BETWEEN INDUCED GIRDER STRESSES AND CORRESPONDING VEHICLE WEIGHTS

Conrad P. Heins, Jr., and Ratten L. Khosa,
Department of Civil Engineering, University of Maryland

The report presents the partial results of 4 loading history field tests on 4 simple-span, cover-plated bridges in Maryland. The resulting stress-range data are compared to the stiffness of a typical girder and the corresponding vehicle gross weights. The vehicle gross-weight data are categorized according to 5 truck classifications. A linear regression analysis of the data resulted in a series of equations that will permit evaluation of field-induced stress ranges of a typical girder that occur at midspan, on cover-plate end, and off cover-plate end and are due to standard truck types of known gross weights. These equations were then applied, in conjunction with a damage theory and assumed truck gross weight distributions, to estimate the bridge lives.

•THE PASSAGE of vehicles across highway bridges will induce varying dynamic strains and stresses. Continual application of such loads, if of sufficient magnitude, will cause noticeable permanent distress in the bridge girders and create a limiting bridge fatigue life. It is the purpose of this report to present empirical equations that will permit evaluation of these induced girder stresses. These stresses can then be used, in conjunction with a damage theory, to determine estimated bridge fatigue lives.

The development of the empirical equations is based on extensive field data collected during July and August 1968 by the Civil Engineering Department of the University of Maryland (1, 2). These tests were confined to simply supported, composite girder slab structures, with welded cover plates. The data are limited to the induced stress ranges and corresponding vehicle gross weights.

Although other data such as dynamic variation and mean stress are available and have been tabulated in histogram form, it has been demonstrated (5) that the dominant stress variable for all steels, beam types, and weld details is the stress range. It has also been shown (5) that minimum stress is not significant for cover-plated beams with welded ends. Therefore, the examination and use of the stress-range data in the fatigue analysis of a bridge is relevant.

BRIDGE DESCRIPTIONS

Each bridge in the study was a simply supported composite structure. The locations, characteristics, and properties of a typical interior girder are given in Table 1. Each bridge structure has a tapered cover plate, fillet welded to each girder. The plate width was always less than the girder flange width and had welded ends. Information regarding the spans in each bridge is also given in Table 1; all spans are simply supported.

Bridges 1, 2, and 3 were monitored during a 24-hour period, and bridge 4 was monitored during 7 continuous days. The number of vehicle passages or events used in the data tabulation, which consisted of both strain and vehicle characteristics, are as follows:

Bridge	Events
1	217
2	200
3	92
4	2,565

INSTRUMENTATION AND DATA

Generally, 8 strain gauges were monitored during the field testing of each bridge. The gauges were mounted at 3 possible positions: on and off the cover plate end of the bottom girder flange, bottom flange of girder at longitudinal midspan, and bottom of concrete slab. The particular gauge positions for each bridge tested are given in Table 2. Only the gauge responses at the bottom flange at midspan and at the end of the cover plate will be examined here in detail; the responses of the other gauges are give elsewhere (1, 2).

The basic equipment that was employed to obtain the dynamic records was a Brush light-beam oscillograph and 2 -6 K. C. 4-channel carrier amplifiers. Incorporated into the oscillograph is a time-line generator and a 10-event marker system, which aid in identifying the vehicle records and the vehicle speeds and axle spacings. A telephone was also attached to the event marker and, upon passage of a given vehicle, a given number would be dialed. This signal induced lines on the light-sensitive paper, thus identifying the vehicle that was previously classified on a log sheet.

After passing over the bridge, the truck was directed to a portable weighing station. At the station the distance between axles, the load on each axle, and the identifying number were recorded. After all of the data were collected, the oscillograph paper was edited and then read on the Gerber digital data reduction system. This system translated points on the dynamic records to digital card output. These specified points were selected during the editing of each record.

The loadometer data were also punched on cards so that they could be entered with the corresponding strain data. The tabulation of the strain and loadometer data was then accomplished by a series of computer programs (1, 2). The resulting output for each vehicle passage consisted of record number, body type, axle spacing, gross weight, weight distribution to axles, velocity, number of vibrations, and strain data including maximum dynamic range, maximum dynamic increment, and maximum mean values. These results, in tabular form and card output, were then reprocessed for the development of histograms or regression analyses (4).

An examination of the resulting data (1, 2) indicates that the stress-range response, at various positions on the bridge girder, varies as given in Table 3. These data indicate that the field-induced girder stress ranges due to 70 percent of the truck traffic equal approximately 1.0 ksi.

REGRESSION ANALYSIS

As previously described, the load history data have been reported in detail by Heins and Sartwell (1, 2). Because of the voluminous nature of these data, it was desirable to relate the trends rather than specific data. The field data that were selected were the stress ranges and the corresponding vehicle gross weights and the bridge girder properties.

It was assumed that a linear relationship exists among the following parameters:

$$\sigma_r\ (S/L) = A + B(G) \tag{1}$$

where

σ_r = induced dynamic stress range on girder, ksi;
G = gross weight of vehicle that induces stress range, kips;
S = elastic section modulus on bottom flange of girder, in.^{3};
L = girder span length, in.; and
A, B = coefficients obtained from a regression analysis of data.

It should be noted that Eq. 1 will reflect the position of the field strain gauge and corresponding girder property.

The regression analysis represents a linear least square fit of the plotted data, $\sigma_r\,(S/L)$ versus G, along the ordinate and abscissa respectively. The standard deviation or dispersion of the data about this regression line will provide a guide as to the confidence of the data. The standard deviation, ±S, will be measured with reference to the ordinate, σ_r S/L, throughout this study. The linear relationship was selected primarily because of the simplicity of the equation.

The relationships that were developed according to Eq. 1 will have 5 categories for the 3 gauge locations. The 5 categories represent the truck classifications designated as 2D, 3, 2S-1, 2S-2, and 3S-2. These identifications, as shown in Figure 1, generally represent those vehicles that travel through Maryland and can be so classified.

RESULTS

The regression analysis of the modified field data and bridge characteristics resulted in the evaluation of the coefficients A and B for each truck type and gauge location, as given in Table 4. The data that were used to establish these constants comprised the composite data collected during the monitoring of all 4 bridge structures. The modulus of elasticity of steel was assumed equal to 29×10^3 ksi.

Plots of the regression lines for the 5 truck classifications and girder positions are given in Figures 2, 3, and 4. Figure 2 shows the regression line of the data observed on the cover plate end; Figure 3, the plot of data observed off the cover plate end; and Figure 4, the trends at the midspan of the girder.

The standard deviations about the regression equations are also given in Table 4. Generally, the dispersions about the mean line, for 95 percent of the population, is ± 0.20 kip.

Figure 3 shows that the curves for truck types 2D and 3 and those for types 2S-1, 2S-2, and 3S-2 can be combined into 2 curves as shown in Figure 5. These combinations will yield the following general equations:

$$\sigma_r\ (S/L) = 0.0715 + 0.0245\ (G) \tag{2}$$

for truck types 2D and 3, and

$$\sigma_r\ (S/L) = 0.1211 + 0.0153\ (G) \tag{3}$$

for truck types 2S-1, 2S-2, and 3S-2.

These equations are important for cover-plated beams, for they represent the response of the beam at that location that governs the fatigue life (3, 5). These equations are only applicable for those bridges examined in this study.

BRIDGE LIFE

Damage Theories

The usefulness of the equations just described can be demonstrated by examining the probable fatigue life of a given bridge. The probable fatigue life of a given bridge may be referenced to the behavior of a single member of that system. Because the vehicles and thus loads that cross the structure are random, some cumulative damage criteria should be applied. The most common damage criterion that is currently being applied is Miner's hypothesis (7).

The evaluation of stress ranges for the many vehicles crossing a bridge would be a tremendous task. However, by the application of Eqs. 2 and 3 and the use of the gross-weight data for the various vehicles crossing a given bridge, the induced stress ranges can be readily computed.

The damage criterion is expressed as

$$\sum (n/N_f) = 1 \tag{4}$$

where n and N_f are as defined previously (7).

Estimated Bridge Life

The linear damage criterion will now be applied in estimating the fatigue life of the 4 bridges under study. To apply Miner's equation (4) requires the number of load applications, n, at a given stress range and the corresponding failure life. The traffic pattern, thus vehicle classifications and weights, for the respective 4 bridges must be determined. A statistical technique and computer program have been developed (5)

Table 1. Location and characteristics of test bridges.

Characteristic	Bridge 1	Bridge 2	Bridge 3	Bridge 4
Location	I-83S over Bunker Hill Road near Hereford	US-301 over MD-5 near Waldorf	US-1 over I-495	US-301 over Western Branch Creek
Slab thickness, in.	7	7	7	7
Girder				
Number	8	8	7	7
Spacing	5 ft 11 in.	5 ft 3 in.	7 ft 2 in.	7 ft
Span				
Number	3	2	5	3
Length	27 ft 7 in., 47 ft[a], 22 ft	76 ft	38[a], 42, 77, 84, 36 ft	42[a], 52, 42 ft
Roadway width, ft	39	30	39	40
Size	27WF76	36WF194	27WF84	27WF97
Cover plate	12 in. × $^{11}/_{16}$ in. × 33 ft	10 in. × $^{7}/_{8}$ in. × 43 ft	7 in. × $^{1}/_{2}$ in. × 24 ft	6 in. × $^{5}/_{8}$ in. × 25 ft
Section modulus, s, at bottom, $in.^3$				
With cover plate	471.5	1,158.0	411.0	464.0
Without cover plate	264.0	835.0	318.0	358.0
s/L, in.				
With cover plate	0.835	1.260	0.986	0.920
Without cover plate	0.468	0.908	0.765	0.710

[a]Test span length.

Table 2. Gauge locations on bridges.

Bridge	Number of Gauges	Location
1[a]	2	Off end cover plate at one end of girder 4
	2	On end cover plate at one end of girder 4
	2	Off end cover plate at one end of girder 5
	2	On end cover plate at one end of girder 5
2[a]	1	Off end cover plate at one end of girder 5
	1	On end cover plate at one end of girder 5
	1	Off end cover plate at one end of girder 6
	1	On end cover plate at one end of girder 6
	1	Midspan of girder 3
	1	Midspan of girder 4
	1	Midspan of girder 5
	1	Midspan of girder 6
3	3	Midspan
	5	Bottom of slab
4[b]	2	Off end cover plate at each end of girder 4
	2	On end cover plate at each end of girder 4
	1	Midspan of girder 3
	1	Midspan of girder 4
	1	Midspan of girder 5
	1	Midspan of girder 6

[a]8-girder system. [b]7-girder system.

Figure 1. Truck classifications.

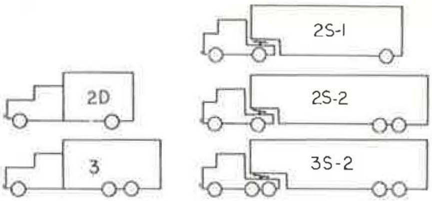

Table 3. Stress-range responses at various gauge locations.

Bridge	Gauge Location	Variation (ksi)	70 Percent (ksi)	1 Percent (ksi)
1	Off cover plate	0.1 to 2.0	0.7	1.8
	On cover plate	0.1 to 0.9	0.3	0.7
2	Midspan	0.1 to 6.0	0.8	6.0
3	Off cover plate	0.2 to 2.6	0.8	2.5
	On cover plate	0.1 to 1.6	0.6	1.5
	Midspan	0.2 to 2.6	0.8	2.5
4	Off cover plate	0.1 to 4.3	1.0	3.0
	On cover plate	0.1 to 3.6	0.5	2.0
	Midspan	0.1 to 5.6	1.2	3.4

Table 4. Equation coefficients and standard deviations.

	On Cover Plate			Off Cover Plate			Midspan		
Truck Type	Coefficient A	Coefficient B	Standard Deviation (kip)	Coefficient A	Coefficient B	Standard Deviation (kip)	Coefficient A	Coefficient B	Standard Deviation (kip)
2D	0.0625	0.0198	0.115	0.0254	0.0257	0.055	0.1122	0.0330	0.080
3	0.1205	0.0175	0.140	0.0840	0.0236	0.080	0.2547	0.0326	0.106
2S-1	0.1808	0.0105	0.130	0.1464	0.0136	0.070	0.2746	0.0178	0.101
2S-2	0.0840	0.0125	0.180	0.0699	0.0150	0.100	0.1740	0.0227	0.123
3S-2	0.1740	0.0110	0.210	0.1341	0.0139	0.107	0.5530	0.0147	0.121

Figure 2. Stress range on cover plate, Eq. 1.

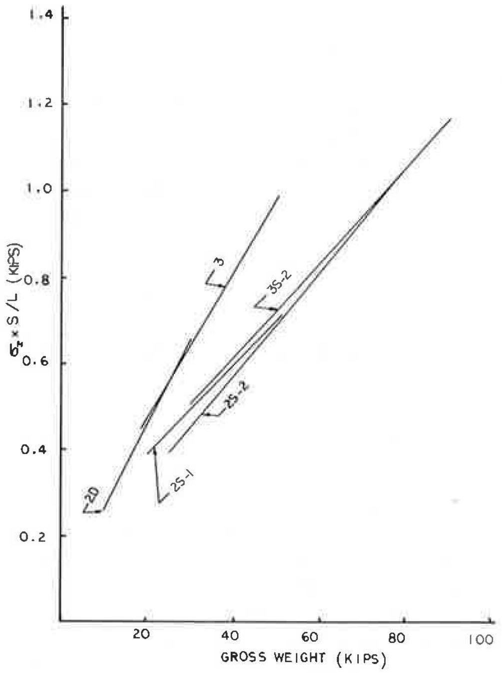

Figure 3. Stress range off cover plate, Eq. 1.

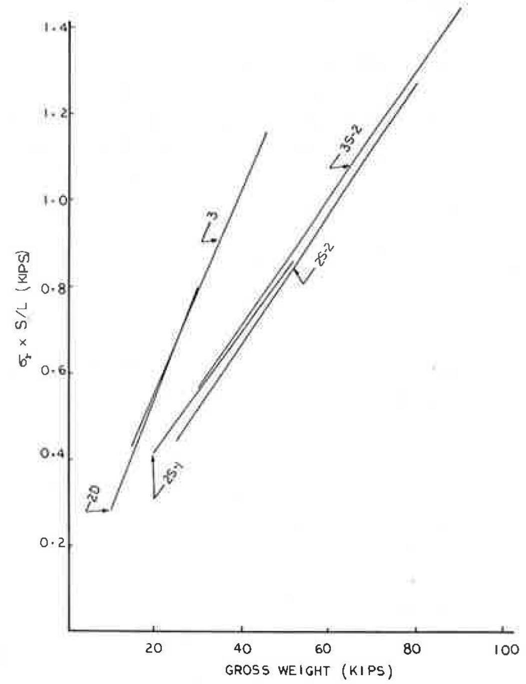

Figure 4. Stress range at midspan, Eq. 1.

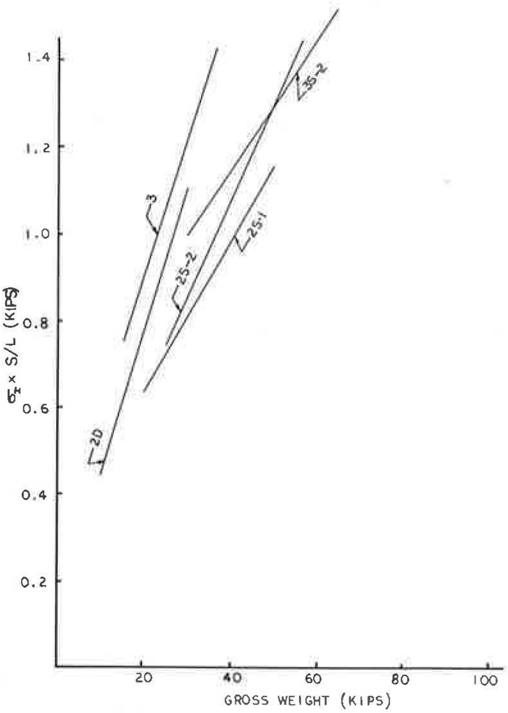

Figure 5. Stress range off cover plate, Eqs. 2 and 3.

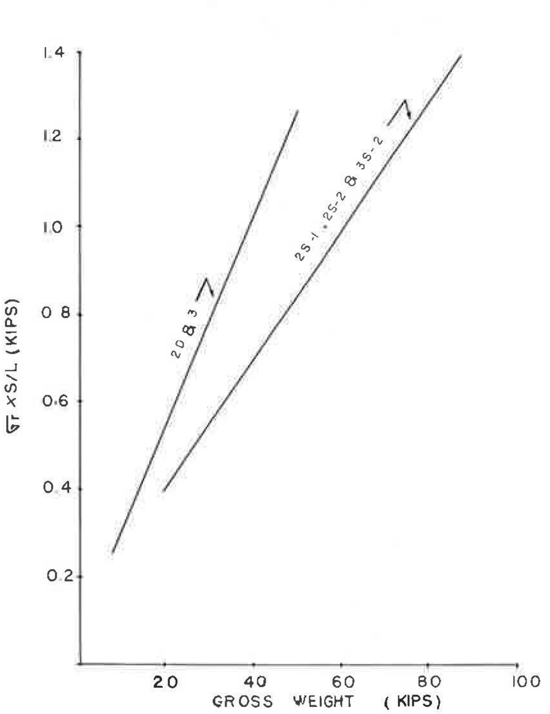

Table 5. Estimated fatigue life.

Bridge	Annual Damage	Existing Service Life (year)	Estimated Fatigue Life (year)
1	0.000006175	21	162,000
2	0.007900	20	127
3	0.00006025	7	16,650
4	0.001064	12	940

that will predict the frequency of types and gross weights of vehicles for an average day at a given sector in Maryland. This technique utilizes the loadometer data collected throughout the state by the state transportation department. Applying the computer program (5) yielded the traffic data for the respective bridges.

Based on the assumption that the frequency of loads for a typical day was the same throughout the year, the estimated damage for a year was determined and is given in Table 5. If the damage or vehicle loadings do not vary from year to year, the fatigue life of the bridge can be estimated by the following equation:

$$N \text{ life} = 1 / \sum (n/N_f) \tag{5}$$

Certainly, the great variation in predicted life indicates a need for better truck volume data and continued fatigue studies.

SUMMARY AND CONCLUSIONS

Examination of field data, induced girder strains, and corresponding vehicle type and gross weight from 4 simply supported girder slab bridges has yielded a series of empirical equations relating induced stresses and vehicle gross weights. These equations were then employed in conjunction with a linear damage theory and estimated vehicle weight and volume data to predict the fatigue life of the 4 bridge structures. A wide variation in fatigue life of those bridges resulted from this analysis.

The methodology for using the collected load history data to develop equations that can be used in fatigue analysis and eventually design is promising. Additional data are certainly required in order to provide some degree of confidence in the empirical equations. Possibly an integrated analysis of all loading history data now being collected nationwide (9) should be considered.

ACKNOWLEDGMENTS

The collection and reduction of data and the fatigue analysis are part of a project at the University of Maryland sponsored by the Maryland Department of Transportation and the Federal Highway Administration. Appreciation is expressed for their support and guidance during this research.

REFERENCES

1. Heins, C. P., and Sartwell, A. D. Tabulation of 24-Hour Dynamic Strain Data on Four Simple Span Girder-Slab Bridge Structures. Univ. of Maryland, College Park, Civil Eng. Rept. 29, June 1969.
2. Sartwell, A. D., and Heins, C. P. Tabulation of Dynamic Strain Data on a Girder-Slab Bridge Structure During Seven Continuous Days. Univ. of Maryland, College Park, Civil Eng. Rept. 31, Sept. 1969.
3. Murad, F. A., and Heins, C. P. Fatigue of Beams With Welded Cover Plates. Univ. of Maryland, College Park, Civil Eng. Rept. 38, Sept. 1970.
4. Khosa, R. L., and Heins, C. P. Study of Truck Weights and the Corresponding Induced Bridge Girder Stresses. Univ. of Maryland, College Park, Civil Eng. Rept. 40, Feb. 1971.
5. Fisher, J. W., et al. Effects of Weldments on the Fatigue Strength of Steel Beams. Lehigh Univ., Bethlehem, Penn., Rept. 334.2, Sept. 1969.
6. Galambos, C. F., and Heins, C. P. Loading History of Highway Bridges, Comparison of Stress Range Histograms. Highway Research Record 354, 1971, pp. 1-12.
7. Miner, M. A. Cumulative Damage in Fatigue. Jour. of Applied Mechanics, June 1945.
8. Desrosiers, R. D. The Development of a Technique for Determining the Magnitude and Frequency of Truck Loadings on Bridges. Univ. of Maryland, College Park, Civil Eng. Rept. 24, April 1969.
9. Galambos, C. F., and Armstrong, W. L. Acquisition of Loading History Data on Highway Bridges. Public Roads, Vol. 35, No. 8, June 1969.