BRIDGE STRESS-RANGE HISTORY

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A computer system for monitoring stress history data and reducing them to digital form was used to estimate the effect of traffic on the fatigue life of a 3-span, continuous-stringer highway bridge. Tests were performed by using control vehicles to measure the load-carrying characteristics of the bridge. Further, extensive stress-range data on the stringers were obtained under general traffic conditions during a 97-hour period. Results indicate good correlation between the frequency distribution of stress ranges in the most severely stressed stringer at midspan and the frequency distribution of vehicles according to weight. The stress ranges produced at both midspan and the ends of the cover plates near the internal piers were found to be very small relative to the endurance limit of the structural components.

•THE USEFUL life of a highway bridge depends on many factors, among them the fatigue strength of the structural components. Because the fatigue life of a bridge depends on its loading history, the frequency distribution of stress ranges and their relation with the character of the traffic are required. The effect of stress history on the fatigue life of a highway bridge has been presented previously (1, 2). Because the effort of data reduction greatly limited the amount of information that could be considered, a computer system has been developed to monitor strain history data and reduce it to digital form (3). Although that data acquisition system has been used in a pilot study, the investigation described here is believed to represent the most extensive use of the system and the resulting accumulation of data to date (1971).

This report describes a study of collecting and evaluating data related to the response of a 3-span, compositely designed, continuous-stringer bridge subjected to both controlled and general traffic conditions. The bridge is located in the southbound lane of Interstate 35W in Bloomington, Minnesota, which is in the metropolitan area of Minneapolis and S. Paul.

The main objectives of the investigation were the measurement of the bending stresses in the stringers under controlled traffic conditions and the recording of strain-range data under general traffic conditions. The data were used to analyze the specific structural behavior of the bridge and to obtain information regarding the fatigue life of the bridge under repeated loads.

INSTRUMENTATION AND DATA ACQUISITION

The spatial distribution of stress in the bridge under controlled loading and the frequency distribution of stress ranges imposed by the general traffic were determined from strain gauge readings. The strain gauges were mounted on the stringers and diaphragms of the bridge shown in Figure 1.

Although a total of 24 strain gauges were installed, the results of only those 10 gauges on the bottom flanges of the stringers are reported here; a detailed discussion of the behavior of all gauges is contained elsewhere (4). A gauge was placed on each of the 5 stringers at midspan (section A-A) and at a location in the center span, 4 in. from the ends of the tapered cover plates (section B-B).

Two basically different types of instrumentation were employed during this investigation. The specific type of instrumentation depended on whether strain ranges under actual traffic conditions or strains under controlled loading were being recorded. The system for obtaining a record of the structural behavior under controlled loading comprised oscillographs.

The system for measuring the frequencies and magnitudes of strain-range occurrences is owned and operated by the Federal Highway Administration and is described in detail elsewhere (1). Basically, the system consists of Wheatstone bridge completion modules, amplifiers, an analog-to-digital converter, a digital computer, and a teletype machine. The complete system is housed in an environmentally controlled trailer. The computer is programmed to record the frequency and magnitude of a "maximum" strain range, that is, the maximum difference in strain caused by the passage of a single vehicle. As many as 10 strain gauges are read through separate channels, each of which is programmed for 10 individual strain levels. As a vehicle encounters the bridge, the system records the strain at a sampling rate as great as 200 per second and retains the maximum value until a minimum negative (i.e., below a 0 level) value is obtained. The algebraic difference of those 2 readings constitutes a maximum strain range. The system then begins a search for a new maximum strain and an associated strain range. The threshold level is placed at a certain positive strain level so that small strain ranges caused by the passage of light vehicles will not be recorded. In this study the threshold level was set at 15×10^{-6} in./in. The 0 level may be adjusted to separate the temperature effects from those caused by the live load. The 10 gauges on the stringers were recorded with this instrumentation and monitored strain-range data produced by actual traffic for the evaluation of possible fatigue damage.

The acquisition of strain-range data was begun at the beginning of the second week of October 1969 and was maintained continually for 97 hours. The data were accumulated and automatically printed on an hourly basis; the printout time was 4 minutes, during which no data were recorded. During the 97-hour period, truck traffic was counted and classified so that the traffic type with the magnitudes and frequencies of strain ranges could be correlated. During selected time intervals, samples from the truck traffic were weighed at a weigh station located approximately 8 miles south of the bridge. Unfortunately, much of the traffic left the highway before encountering the weigh station. From the weigh-station information, however, an estimate could be made with regard to the general distribution of vehicular loads.

The controlled tests were conducted by using 2 trucks (H20 and HS20) owned and operated by the Minnesota Department of Highways. The trucks encountered the bridge either separately or, in some cases, simultaneously, and the response of the gauges was recorded. The purpose of these tests was to measure the structural behavior of the bridge. In addition to the controlled tests, oscillograph readings were also taken (strain-range measurement continued simultaneously) to obtain some information about the actual stresses incurred under normal traffic conditions. Individual vehicles that happened to be isolated from the other traffic were classified, and their effects on the bridge were recorded. Of those vehicles, those that passed the weigh station were weighed.

CONTROLLED TESTS

The structural response under controlled conditions was determined by driving 2 vehicles, located at various lateral positions, across the bridge at various speeds. The H20 vehicle (double-tandem dump truck) had front and rear axle weights of 8.20 and 32.20 kips respectively; front-to-rear axle spacings were 13 ft 10 in. and 4 ft 2 in. The HS20 vehicle (5-axle semitrailer) had front-to-rear axle loads of 6.14, 32.00, and 32.16 kips; front-to-rear axle spacing was 10 ft, 4 ft, 10 ft 3 in., and 4 ft.

Test runs were made at approximately 3 speeds: 5, 25, and 40 mph. Three primary lateral vehicle positions were considered. In the first case each truck's right wheels ran a few inches from the right lane curb, and in the second case each truck was centered in the right lane. In both cases only one truck at a time was on the bridge. In the third case, both vehicles crossed the bridge simultaneously; the HS20 vehicle was centered in the right lane, and the H20 vehicle in the left lane.

Figure 1. Plan, elevation, cross section, and location of strain gauges.

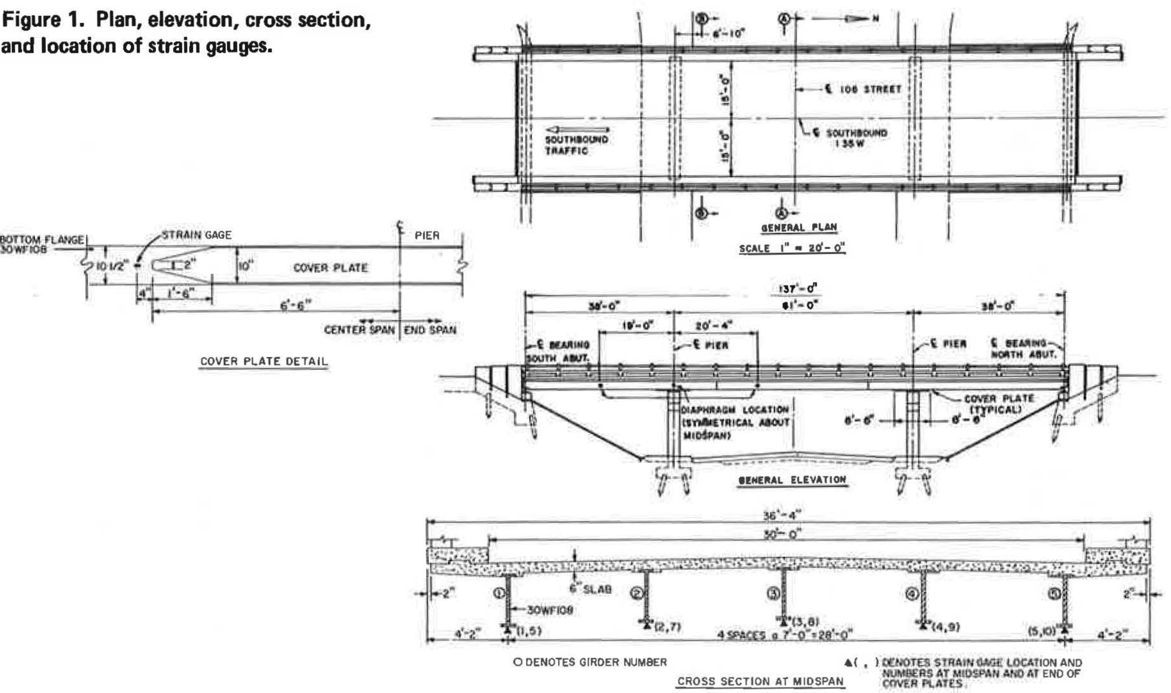


Figure 2. Positive bending stresses at midspan under controlled loading.

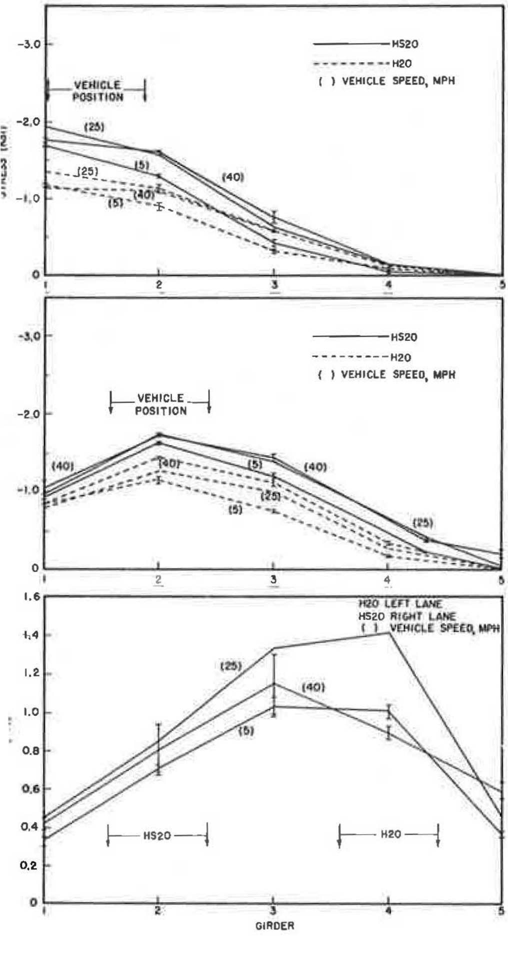
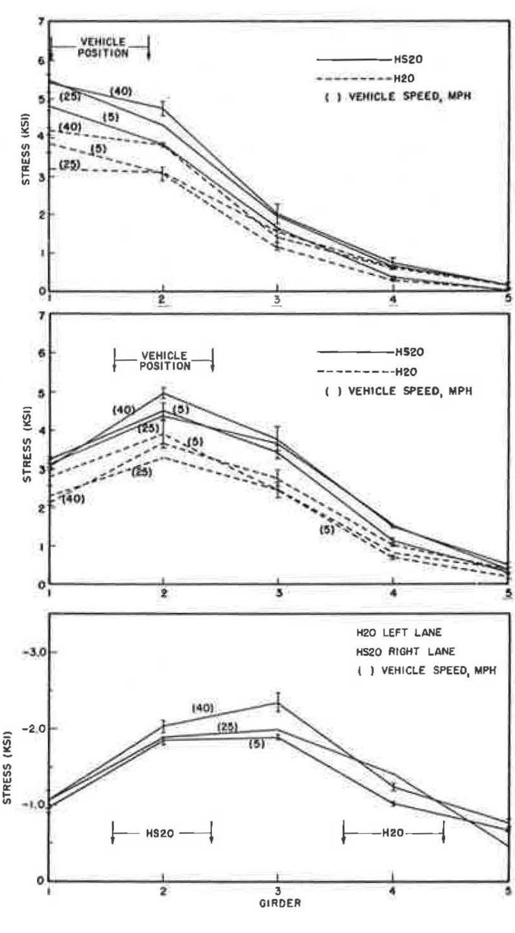


Figure 3. Negative bending stresses at ends of cover plates under controlled loading.



The tests involving each of the 3 primary positions were performed twice at both the crawl speed (5 mph) and the fastest speed (40 mph). Because of increasingly heavy traffic, only one set of tests was made at the intermediate speed (25 mph).

BENDING STRESSES

Midspan

Figure 2 shows the magnitudes of the bending stresses in each girder at midspan for each of the 5 vehicle positions and 3 speeds. When either vehicle ran next to the right lane curb, the greatest stress was incurred in girders 1 and 2. Girder 1 experienced, on an average, approximately 15 percent more stress than did girder 2. Furthermore, the magnitudes of the stresses in girder 3 were about 30 percent of those in girder 1. Girders 4 and 5 carried very little load in this case. The values of stresses in girder 1 ranged from 4.8 to 5.5 ksi under HS20 loading at 5 and 25 mph. Because the tests run at 25 mph were made only once, no indication of repeatability could be obtained in these cases. For both the H20 and HS20 runs, the impact factors associated with 40 mph in girders 1 through 3 were approximately 1.1, 1.2, and 1.2 respectively. These factors generally compare favorably with the results of other tests (1, 5) and with the value of 1.27 predicted by the ASSHO formula.

Figure 2 also shows the stresses incurred under a load centered in the right traffic lane. The impact factors for girders 2 and 3 are somewhat less than those indicated in the previous case. Although girder 2 assumes the greatest stress (4.4 to 5.0 ksi for HS20 loading and 3.3 to 3.9 ksi for H20 loading), girder 3 assumes more stress (3.4 to 3.8 ksi and 2.4 to 2.8 ksi) in this loading condition than in that of the curb loading. Girder 1, however, is less heavily stressed (3.0 to 3.2 ksi and 2.1 to 2.8 ksi) than girder 3 in this case or than girder 1 itself in the previous case. The maximum stress (5.5 ksi) in girder 1 under curb loading is greater than the maximum stress (5.0 ksi) in girder 2 under the right-lane loading, which compares favorably with results obtained for a type of simple-span bridge (6). Additional tests with the load located in the left lane showed the structure to behave in a symmetrical manner.

The effects of loading both lanes simultaneously reveal several interesting features. For the 3 most heavily stressed girders (1, 2, and 3), the stresses incurred at the crawl speed are essentially the same or even slightly greater than the corresponding stresses experienced under vehicular speeds of 25 mph. Of more significance, however, is that at the crawl speed the stresses measured in every girder are almost precisely those than would be obtained by superposition of the individual loadings described tn the preceding case (loads centered in right lane). The structure behaves linearly (i.e., superposition of loads applies), therefore, under static loading. However, if a similar study is made of the stresses incurred at vehicular speeds of 40 mph, it is found that the actual combination of loads yields considerably less stress (about 15 percent less) than that predicted by superposition. It may be concluded, therefore, that the impact factor is less for the combined tests than for those where the vehicles cross the bridge separately.

Ends of Cover Plates

Figure 3 shows the magnitudes of the negative bending stresses at the end of the cover plate in each girder for each vehicle position and speed. For loading along the right curb and in the center of the right lane, the most severe negative stresses are produced by the HS20 loading. The most heavily stressed girder for the case of the load centered in the right lane is girder 2; for the case of the curb loading, the negative stresses in girder 2 are slightly less than those in girder 1. The negative stresses at the ends of the cover plates were approximately 50 percent greater than the positive stresses at this section. Those negative stresses were, however, between 50 and 70 percent less than the positive stresses measured at the midspan.

STRESSES INCURRED UNDER SAMPLE TRAFFIC

For one 6-hour interval, during which the bridge was subjected to general traffic conditions, stress measurements were obtained from the passage of selected commercial vehicles. A vehicle was chosen for this part of the study if it was sufficiently isolated from the rest of the traffic so that it alone crossed the bridge during the time that strains were measured.

Most of the commercial vehicles that crossed the bridge during the 6-hour period left the highway before encountering the weigh station; therefore, stresses could not be directly related to vehicular gross weight. Generally, higher stresses were produced by the larger classes of vehicles than by the smaller trucks. Of significance is that, in all but 1 of the 38 cases observed, girder 2 experienced the largest stress at midspan. The maximum positive bending stress incurred by girder 2 was 3.94 ksi. Only in the case where a truck crossed the bridge in the left traffic lane was the greatest stress (4.03 ksi) produced in girder 5; this was the only case where bending stress in a girder exceeded 4.00 ksi. As may be expected, the negative bending stresses at midspan were significantly less than the positive stresses.

The bending stresses at the ends of the cover plates were generally less than those at midspan. The maximum negative stress (-1.59 ksi) at the end of a cover plate occurred in girder 2. The maximum positive stress was 1.32 ksi; however, this stress is considerably larger than the average positive stress at the ends of the cover plates.

The largest stress ranges (algebraic difference between positive and negative stresses) usually occurred in girder 2. In that member the maximum stress ranges were 4.46 and 2.78 ksi at the midspan and the end of the cover plate respectively.

At midspan the maximum positive stress (4.62 ksi) produced by the HS20 vehicle occurred in girder 1; when this load was located in the center of the right lane, the maximum stress (4.94 ksi) occurred in girder 2. When both control vehicles crossed tha bridge simultaneously, the maximum stress, 6.22 ksi, was produced in girder 3. The maximum positive and negative stresses produced at the ends of the cover plates under HS20 loading were 1.41 and -1.94 ksi respectively; combined loading produced maximum stresses of 1.41 and -2.47 ksi. In all cases the stresses incurred under general selected traffic were considerably less than those produced in the controlled tests.

EFFECT OF TRAFFIC ON STRESS RANGES

A summary of the stress ranges computed from strain-range data and recorded for 97 one-hour periods is given in Table 1. Gauges 1 through 5 correspond to those placed at midspan (Fig. 1), and gauges 6 through 10 are those at the ends of the cover plates and are in the same order as the first 5 gauges. The stress ranges for each gauge are divided into 9 increments designated in units of ksi.

With the exception of gauge 5, all the gauges at midspan experienced comparable total numbers of stress-range occurrences. This may be interpreted to mean that with the passage of a single vehicle nearly the same number of stress-range values (i.e., the number of recorded peaks and valleys) occurs in each girder. Occurrences in the 2 right-lane girders (1 and 2) were approximately 16 and 6 percent more respectively than the total average occurrences in the 4 girders; occurrences in the other girders (3 and 4) were about 14 and 10 percent less than the average. The comparison of the total number of occurrences among the 5 girders gives some indication of the distribution of loading across the bridge at midspan. That the fifth girder experienced so many fewer stress-range events seems to indicate that the outer girders are effective only when the loading is in the lane corresponding to that member.

Although the total numbers of stress-range events among the various structural members may be of similar magnitudes, the distribution of these events relative to particular stress ranges may be distinctly different. Figures 4 and 5 show the percentage frequency distribution of strain-range occurrences for each gauge. The values corresponding to stress-range events between 0 and 2.85 ksi (i.e., the first 4 increments of stress ranges) were, for gauges 1, 3, 4, and 5 respectively 98.8, 95.0, 98.5, and 97.8; for the second girder (gauge 2) the corresponding value was 90.8. This comparison indicates that the second girder experiences 9.2 percent of its stress-range events over the 2.85-ksi range, whereas the other gauges experience considerably fewer events above

Table 1. Stress-range occurrences for 10 gauges.

Gauge	0.45 to 1.05 ksi	1.05 to 1.65 ksi	1.65 to 2.25 ksi	2.25 to 2.85 ksi	2.85 to 3.45 ksi	3.45 to 4.05 ksi	4.05 to 4.65 ksi	4.65 to 5.25 ksi	5.25 to 5.85 ksi	Total
1	6,901	10,679	3,375	1,201	241	18	5	0	0	22,420
2	9,816	5,125	2,698	1,090	718	728	349	100	10	20,634
3	7,434	5,352	1,897	1,130	651	154	23	5	1	16,647
4	12,756	3,277	866	337	99	60	54	37	11	17,497
5	2,840	2,947	1,282	410	122	42	3	1	0	7,647
6	290	101	19	5	3	0	0	0	0	418
7	2,252	1,808	1,016	734	53	3	0	0	0	5,866
8	910	933	400	59	19	0	0	0	0	2,322
9	597	685	244	69	5	0	0	0	0	1,600
10	155	28	7	3	1	0	0	0	0	194

Figure 4. Stress-range occurrences at midspan.

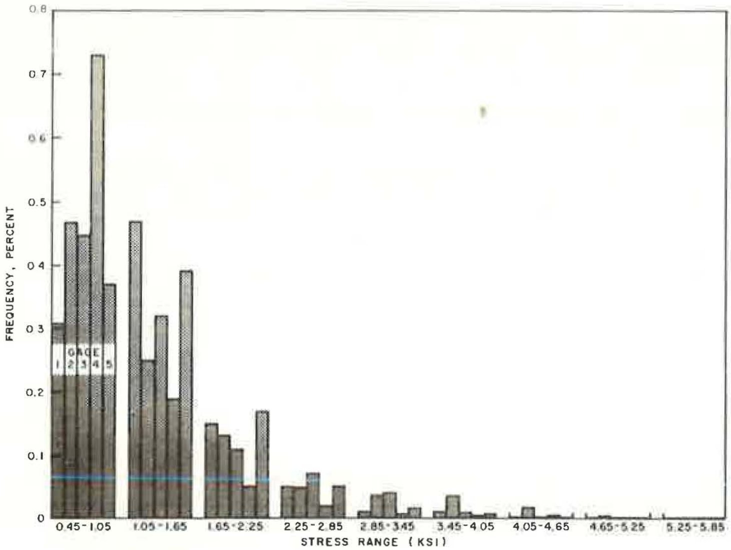


Figure 5. Stress-range occurrences at ends of cover plates.

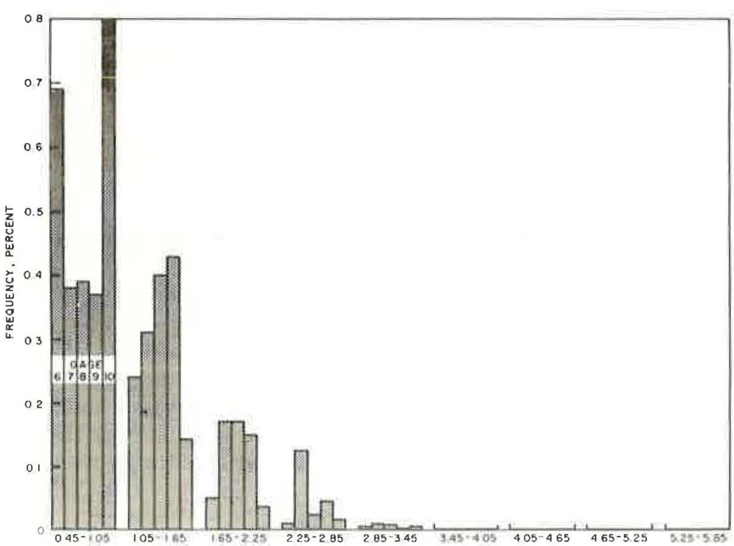


Table 2. Average 24-hour traffic volumes.

Vehicle Type[a]	24-Hour Weekday Average	Average Annual Daily Volumes 1969	1975
0	14,967	15,992	23,416
1	791	549	676
2	689	592	912
3	65	56	58
4	149	100	133
5	857	661	1,089
6	170	149	217
Total	17,688	18,099	26,501

[a]0 = passenger cars and 4-tire trucks; 1 = single unit, 2 axle, 6-tire trucks; 2 = single unit, 3-axle trucks; 3 = truck tractor-semitrailers, 3 axle; 4 = truck tractor-semitrailers, 4 axle; 5 = truck tractor-semitrailers, 5 axle; and 6 = buses and trucks with trailers.

this range. Furthermore, in the range above 3.45 ksi the second girder experiences 5.8 percent of the total occurrences; the next highest percentage above this range is 1.1 percent in girder 3. Although in girder 2 the percentage occurrence above 3.45 ksi may be large compared with that in the other girders both at midspan and at the ends of the cover plates, it is still too small to affect the fatigue life of the member.

The total number of stress-range occurrences at the end of the cover plate in each girder was considerably less than that at midspan. Again, the gauge on the second girder recorded the most stress events (5,866) and the highest percentage (28.4 percent) relative to the total number of events in that girder at midspan. Gauges 9 and 10 recorded about a third fewer events than gauges 6 and 7 because there were statistically fewer vehicles crossing in the left lane than in the right lane.

For girders 1 and 5, the cumulative percentage stress-range occurrences less than 1.05 ksi were 30.8 and 37.1 respectively at midspan and 69.4 and 79.9 respectively at the cover plates. Further, in the range less than 1.65 ksi the cumulative percentages of occurrences were 78.4 and 75.7 at midspan and 93.5 and 94.5 at the cover plates. These comparisons indicate a symmetrical structural action about the center girder. It is again emphasized, however, that, because of the loading pattern and varying structural behavior along the length of the bridge, the bases for these percentages are distinctly different. A similar comparison may be made of the behavior of girders 2 and 4.

The cumulative percentage stress-range occurrence above 2.85 ksi in girder 2 (gauge 7) was 1.0 percent. Although this percentage is much smaller than that (9.2 percent) in the same girder at midspan, the lower fatigue life of the cover plate connections makes such a location the more critical one. As shown subsequently, although the fatigue life of the connection is less than that of the midspan section, it is not critical with regard to the useful life of the bridge. Such behavior was previously demonstrated in a similar study (1).

VEHICLE COUNT AND CLASSIFICATION

A nearly continuous traffic count and classification were made at the bridge site and tabulated hourly. Table 2 gives traffic counts for an average 24-hour interval; then data were supplied by the Minnesota Department of Highways. The second column represents an average based on the 97-hour interval during which a traffic count was conducted. Corrected average annual daily volumes to account for seasonal variation are also given for 1969 and for 1975.

FREQUENCY DISTRIBUTIONS OF VEHICLE TYPES AND GROSS WEIGHTS

Through a regression analysis a relation may be obtained between the number of occurrences in each stress range of a gauge and each type of vehicle crossing the bridge. Such an analysis fits the best (in the least squares sense) hyperplane to the acquired data of hourly strain-range events and hourly classified vehicle counts. However, the period of time over which data were obtained was too short to yield reliable results, and an alternative approximate method was used.

During a 32-hour period a total of 427 vehicles that encountered the weigh station were weighed. The number of vehicles that were weighed did not include all the vehicles that crossed the bridge because normally many trucks leave the highway before reaching the weigh station. Further, of those vehicles that did encounter the weigh station, the larger vehicles were weighed but many smaller ones were permitted to pass by. Therefore, the frequency distribution of vehicle gross weights established at the weigh station is intentionally biased toward the larger trucks. The percentage distribution of all vehicles weighed by type in each weight range is given in Table 3. The fraction of each vehicle type in each weight category is given in Table 4. These fractional values are further used to estimate the frequency distribution of vehicle

Table 3. Percentage distribution of all vehicles by type and weight category.

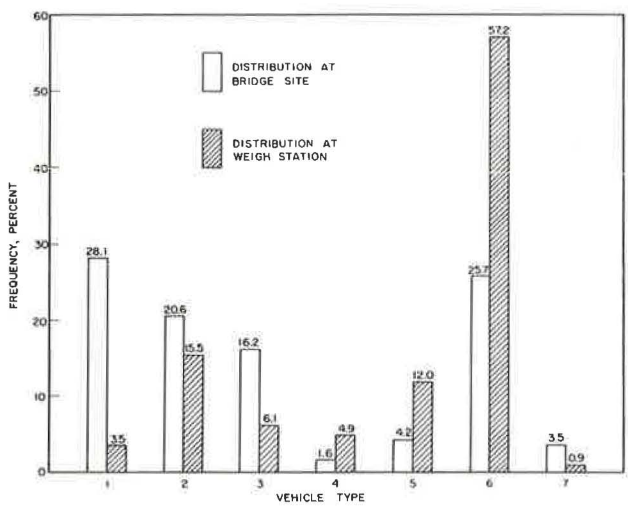
Vehicle Weight (kip)	Total Vehicles		Percentage Distribution by Type						
	Number	Percent	1	2	3	4	5	6	7
0-10	30	7.0	3.3	3.5	–	0.2	–	–	–
10-20	54	12.6	0.2	8.0	2.8	0.2	0.7	0.2	0.5
20-30	107	25.1	–	4.0	0.9	1.6	3.8	14.3	0.5
30-40	63	14.8	–	–	1.2	1.4	2.8	9.4	–
40-50	43	10.2	–	–	1.2	1.2	2.6	5.2	–
50-60	32	7.5	–	–	–	0.2	1.4	5.9	–
60-70	34	8.0	–	–	–	–	0.5	7.5	–
70-80	64	15.0	–	–	–	–	0.2	14.8	–
Total									
Percent		100.0	3.5	15.5	6.1	4.8	12.0	57.3	1.0
Number	427		15	66	26	21	51	244	4
Avg weight, kip			6.16	14.88	27.04	31.72	37.36	49.48	19.70

Note: Discrepancy in percentages total due to rounding.

Table 4. Fraction distribution of each vehicle type by weight category.

Vehicle Weight (kip)	Type 1	Type 2	Type 3	Type 4	Type 5	Type 6	Type 7
0-10	0.933	0.227	-	0.048	-	-	-
10-20	0.067	0.515	0.462	0.048	0.059	0.004	0.500
20-30	-	0.258	0.154	0.333	0.314	0.250	0.500
30-40	-	-	0.192	0.286	0.235	0.164	-
40-50	-	-	0.192	0.238	0.216	0.090	-
50-60	-	-	-	0.048	0.118	0.102	-
60-70	-	-	-	-	0.039	0.131	-
70-80	-	-	-	-	0.020	0.258	-
Total	1.000	1.000	1.000	1.000	1.000	1.000	1.000

Figure 6. Distribution of vehicles by type at bridge site and weigh station.



gross weights at the bridge site. Vehicle types in these tables and subsequent figures are as follows:

Vehicle	Code Number
Light truck, 2-axle, single-tire	1
Single-unit, 2-axle truck	2
Single-unit, 3- to 4-axle truck	3
Truck tractor-semitrailer, 3-axle	4
Truck tractor-semitrailer, 4-axle	5
Truck tractor-semitrailer, 5- to 6-axle	6
Truck with trailer	7

Figure 6 shows a comparison between frequency distribution at the weigh station and at the bridge site according to vehicle type. The frequency distribution by vehicle type at the bridge site differs considerably from that at the weigh station. Because of the bias sampling, there was at the weigh station a smaller percentage of lighter (single unit) trucks and a greater percentage of heavier (truck-semitrailer) vehicles.

An estimate was made of the frequency distribution at the bridge site according to vehicle gross weight by converting the distribution according to vehicle type 5 with the use of the fractional values given in Table 4. The distribution, according to vehicle gross weight and the stress-range distribution at the midspan of girder 2 (from Fig. 3), is shown in Figure 7. Except for the lowest and highest ranges, good correlation exists between the frequency distributions of vehicle gross weights and stress ranges. Therefore, it appears that the behavior of the most heavily stressed member (girder 2) at midspan is very closely related to the frequency and magnitude of the loading. This comparison is essentially a simplified regression analysis that is useful when sufficient data for a more rigorous analysis are unavailable. A comparison of stress-range distribution in a member other than girder 2 with the vehicle gross weight distribution shows poorer correlation. Girder 2 experiences the broadest distribution and the greatest number of stress-range occurrences; therefore, this member should serve as the best indicator in such a comparison.

FATIGUE LIFE ANALYSIS

The fatigue life of a structure is affected by a combination of many factors. The most important considerations are loading history and loading expectation, fatigue strength and structural details, stress range and mean stress, and temperature variation and corrosion.

Although neither loading history nor future loading is known with certainty, they may be estimated from data related to the current pattern of dynamic loading. These data specify the frequencies and magnitudes of stress ranges corresponding to traffic volume. As stated previously, girder 2 experienced the greatest frequencies and magnitudes of stress ranges. Table 5 gives the number of occurrences of the live load stress ranges in girder 2 for the 97-hour test period. These data are used to obtain current (1969) annual and future (1975) frequency estimates. Because the method of collecting strain-range data grouped the data in discrete intervals, the stress ranges shown in the following represent the mean value in each interval. For example, the stress range 0.45 to 1.05 ksi is replaced by 0.75 ksi.

The fatigue strength of any structural component depends on the mean stress level as well as the stress range. If temperature effects are ignored, the minimum stress at any point in the structure may be taken as that produced by the dead load. If no composite action is assumed to exist in positive moment zones, the dead-load stresses in an inner girder at midspan and at the end of the cover plate are 5.39 ksi and 2.56 ksi respectively. Because of the uncertainty of composite action, these stress values were used to develop fatigue curves.

The fatigue curves may be developed with data corresponding to test specimens subjected to zero-to-tension loading; they indicate the number of cycles at a given stress

Figure 7. Distribution of vehicle gross weights and stress ranges in girder 2.

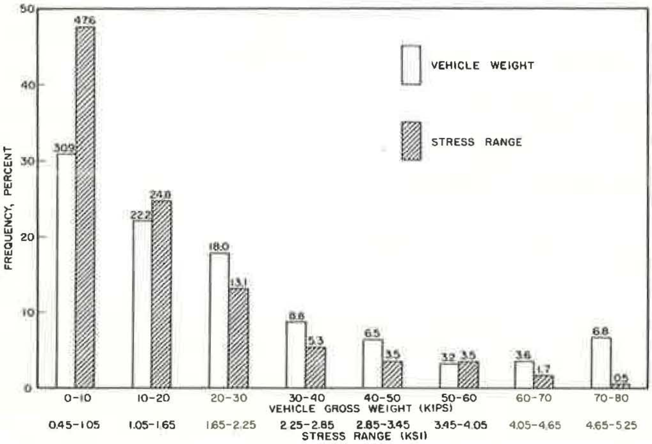


Figure 8. Fatigue curve for rolled beam with partial-length-tapered end cover plate.

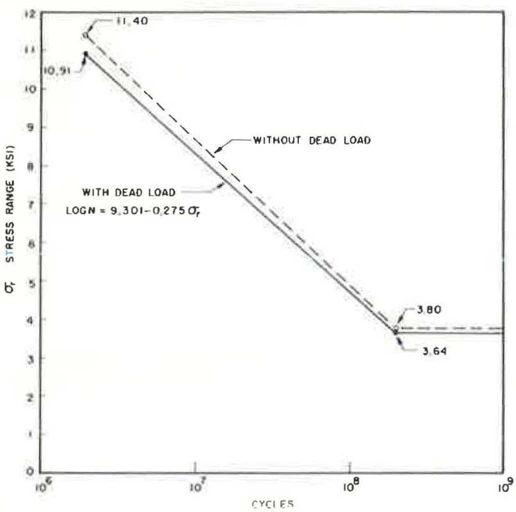


Table 5. Stress-range occurrences in girder 2.

Stress Range (ksi)	Occurrences at Midspan			Occurrences at End of Cover Plate		
		Annual Estimate ($\times 10^6$)			Annual Estimate ($\times 10^6$)	
	97-Hour	1969	1975	97-Hour	1969	1975
0.75	9,816	0.866	1.307	2,252	0.203	0.300
1.35	5,125	0.463	0.683	1,808	0.162	0.241
1.95	2,698	0.244	0.359	1,016	0.092	0.135
2.55	1,090	0.098	0.145	734	0.066	0.098
3.15	718	0.065	0.096	53	0.005	0.007
3.75	728	0.066	0.097	3	0.0003	0.0004
4.35	349	0.032	0.046	0	0	0
4.95	100	0.009	0.013	0	0	0
5.55	10	0.001	0.001	0	0	0

range required to bring the specimens to failure, Fatigue curves were constructed for the points at midspan and at the end of the cover plate. The fatigue curve for a structural element may be developed by first obtaining the fatigue strength at 2×10^6 cycles and then by assuming that the fatigue strength at 200×10^6 cycles is equal to a third of this value (7). Within these 2 limits a linear relationship is assumed between the stress range and the logarithm of the number of cycles.

The fatigue curves shown in Figure 8, corresponding to the end of the cover plate, were constructed by using data related to fatigue strength of flexural members with partial-length-tapered end cover plates welded all the way around (8, 9). The fatigue strength at 2×10^6 cycles, neglecting the effect of dead-load stress, is given as 11.4 ksi. Using the modified Goodman law and assuming an ultimate strength of 60 ksi give 10.9 ksi as the fatigue strength accounting for dead-load stress. The endurance limit, assuming a dead-load stress of 2.56 ksi, is 3.64 ksi. Because the only stress range in girder 2 higher than this limit was 3.75 ksi and occurred only 3 times during the 97-hour test period, the fatigue life at the end of the cover plate is about 7×10^6 years.

In the same manner the fatigue curves corresponding to the midspan of the girder were obtained from data for ASTM A-7 steel (8). If the effect of the mean stress is neglected, the fatigue strength at 2×10^6 cycles is 31.2 ksi. The fatigue strength is 24.8 ksi if dead load is taken into account. The endurance limit for a plain rolled beam of A-7 steel with a minimum stress of 5.39 ksi is 9.47 ksi; and, because the maximum recorded stress range is 5.55 ksi, it may be concluded that at midspan (the point of maximum stress) there exists almost an unlimited fatigue life.

CONCLUSIONS

The primary objectives of this study were to collect strain-range data on a specific highway bridge of a common type, to determine the implications of those data for the fatigue life of the bridge, and to measure stresses produced in the structure by actual traffic. Some conclusions drawn from the results of this investigation are as follows:

1. The greatest number of stress-range events produced by general traffic during a 97-hour period occurred at midspan in the external stringer under the right traffic lane (girder 1). The most severely stressed member at midspan was girder 2, the stringer intermediate between the right external stringer and the centerline stringer. Girder 2 experienced nearly as many stress-range events as did girder 1 and had more events occurring in the higher stress ranges (up to 5.55 ksi).
2. The number of stress-range events occurring in the most heavily stressed member (girder 2) at the end of the cover plate was approximately one-fourth that produced at midspan. Further, a negligible percentage of stress-range events occurred above 3.45 ksi, and no events greater than 4.05 ksi were produced at this section.
3. In only one case among those vehicles from the general traffic that were recorded individually was the peak midspan live load bending stress as great as 4.0 ksi. At the end of the cover plate on girder 2, the most heavily stressed girder, the maximum negative live-load stress under general traffic was -1.59 ksi. The stresses produced by the general traffic were considerably less than those produced by either the H20 or HS20 control vehicles. The HS20 control vehicle produced a midspan bending stress of 5.62 ksi and a stress of -2.47 ksi at the end of the cover plate on girder 2.
4. The stress ranges produced at both the midspan and the ends of the cover plates are very small relative to the endurance limit of the structural components. It may be concluded that the effect of traffic volume similar to that currently encountered is insignificant with regard to the fatigue life of the longitudinal stringers.
5. Good correlation exists between the behavior of the most heavily stressed member (girder 2) in terms of frequency distribution of stress-range occurrences and the estimated frequency distribution of vehicle gross weights at the bridge site.

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