LOADING HISTORY OF SPAN 10 ON YELLOW MILL POND VIADUCT

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Two simple-span structures on heavily traveled Interstate 95 in Bridgeport, Connecticut, were tested electronically to determine the magnitude and frequency of stress ranges induced by normal live loading. The bridges were designed in accordance with composite-action techniques and consisted of $7\frac{1}{2}$-in. concrete decking on rolled cover-plated beams. Strain gauges were mounted at the critical ends of cover plates and at midspan on selected beams and on one diaphragm. A computer-controlled data acquisition system made it possible to record strains continually. As a supplement to strain data, lane counts were made and truck classifications and weights and measurements obtained. Gross truck weights were fairly evenly distributed between 10,000 and 70,000 lb with maximum weight recorded at between 90,000 and 100,000 lb. Distribution of truck traffic was approximately 55 percent in the outer lane, 45 percent in the middle lane, and less than 1 percent in the inner lane. On the basis of current popular methods of fatigue analysis, which tend to neglect stress ranges below 3 ksi, fatigue failure of the beams tested would be considered a remote possibility for the near future. The numerous low stress ranges induced by live loading, although their effect on the integrity of cover-plated beams is unknown, could perhaps drastically shorten the service lives of these members. /AUTHOR/

•IN LATE 1970, catastrophic failure of an internal, cover-plated steel beam occurred in span 11 of the 14-span Yellow Mill Pond Viaduct on Interstate 95 in Bridgeport. The failure, shown in Figure 1, originated as a crack at the toe of the fillet weld along the end of the primary cover plate and propagated through the flange and 16 in. up into the web before detection and subsequent repair. Also noted in connection with this failure was a high incidence of missing bolts in the diaphragm-to-beam connections. Follow-up inspection of other cover-plated beams in other bridges on I-95 revealed the presence of extremely small cracks (less than 1 in. long) along the weld toes at the cover plate ends.

Certain investigators suggested that the cracking might have resulted from fatigue action generated by normal live loading. In this connection, a study was undertaken to determine (a) the frequency of ranges of dynamic stresses induced in selected bridges by normal live loading and (b) the general composition and weight of the traffic causing these stresses and to estimate (c) the fatigue lives of the structures from the data acquired in (a) and (b). This report is devoted to discussion of the equipment and methods employed to acquire appropriate strain data and to the findings derived therefrom.

TEST BRIDGES

Two simple-span, cover-plated, steel-beam bridges were selected for acquisition of data that would permit assessment of structural behavior under normal loading. These test bridges were the eastbound and westbound structures in span 10 of the Yellow Mill Pond Viaduct. Selection of these bridges was based on the following:

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1. Span length and steel types similar to those in span 11,
2. Live loading the same as on span 11, and
3. Easy access to the underside of the bridge.

Figure 2 shows a plan view of the layout of beams and diaphragms in span 10; Table 1 gives data on beam detail. As can be seen, the internal beams are the same in both eastbound and westbound structures. The fascia beams, however, vary not only in size but also in cover plate detail. The ends of the partial cover plates are not tapered but are rounded to a radius of 3 in. at the corners. Fillet welds $1\frac{1}{2}$ in. in size extend across and around the ends and along the edges of the plates for a distance of 2 ft, at which point $\frac{5}{16}$-in. fillet welds begin and continue along the remainder of the edge.

Both bridges carry three lanes of traffic on $7\frac{1}{4}$-in. concrete decks. The roadways in span 10 are on tangent and have a positive gradient of approximately 1 percent to the west. In 1969, a "thick" two-course bituminous concrete overlay was placed on the concrete deck in both roadways. The bridges were built between 1956 and 1957 and were opened to traffic in January 1958.

INSTRUMENTATION

Strain Gauges

Electrical-resistance strain gauges were mounted at various points on the tension flanges and tension flange cover plates, as well as at the midpoint on one of the diaphragms. Strain gauge placement is shown in Figure 2. As can be seen, gauges were placed primarily at two locations on the beams: on the tension flange cover plate at midspan and on the tension flange 4 in. off the leading edge (with respect to traffic flow) of the primary cover plate. Auxiliary gauges were mounted at the secondary plate terminus on beam 3 on the westbound structure and on the full-length primary plate 4 in. off the secondary plate on the external fascia beam in the westbound roadway.

The gauges were paper-backed and were cemented to the beam after recommended preparation of the steel. After waterproofing, the gauges were connected via transducer cable to electronic strain-monitoring equipment housed in an FHWA trailer.

Strain-Monitoring Equipment

The data acquisition system employed in the tests was developed for the FHWA by Scientific Data Systems. The system is largely automated and is computer controlled. It has been employed successfully in a number of loading history tests conducted by the FHWA on various bridges throughout the country.

Briefly, the equipment takes variations in analog voltages produced in a maximum of 10 resistance strain gauges, digitizes the magnitude of this variation for each gauge, stores the values and tabulates them as strains within certain preselected ranges, and prints out the total number of strains that fall within these ranges for each gauge over a specified time interval. The levels of strain that define the individual ranges can be manipulated to produce a meaningful picture of the stress events that take place under a given set of conditions.

TEST PROCEDURES

Acquisition of Strain Data

Strain-range data were acquired and printed out during 64-min cycles. The computer was programmed to classify and count strain ranges for 60 min and print out the stored data during the last 4 min of the cycle. Stress ranges were manipulated so that minimum stress range, i.e., the stress below which the computer will disregard an event, was set at 0.6 ksi; thus, negligible strains produced from passage of light vehicles and damped vibrations induced by heavy trucks were eliminated from the count. In the eastbound span, stress levels were increased in increments of 0.45 ksi from 0.6 to 4.65 ksi, whereas in the westbound span levels increased in increments of 0.6 ksi from 0.6 to 6.0 ksi. One exception to this rule occurred at midspan of the external fascia beam in the westbound roadway where higher stress levels were encountered.

Figure 1. Failure in internal cover-plated steel beam.

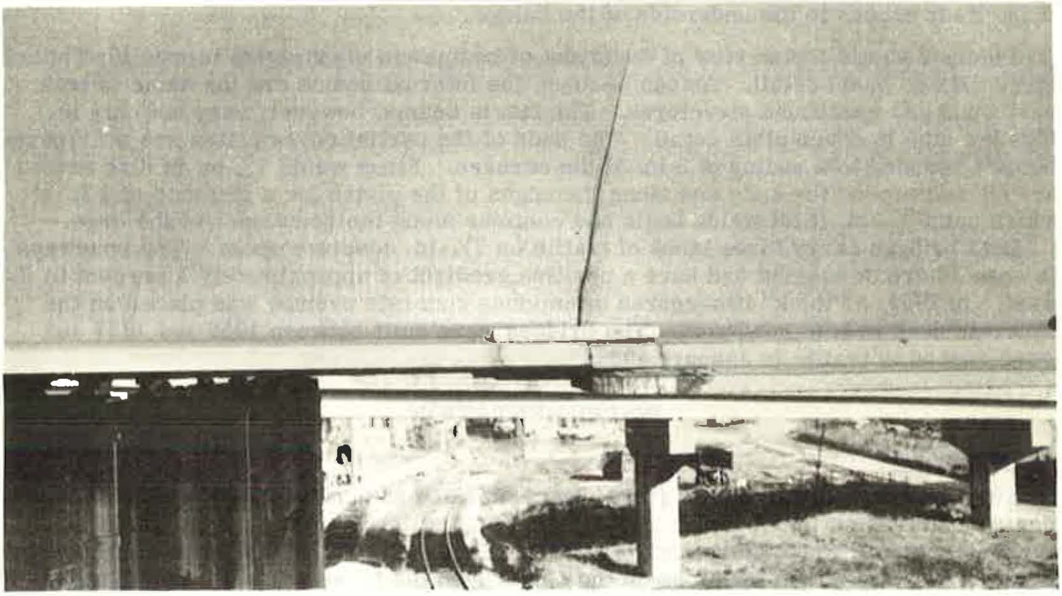
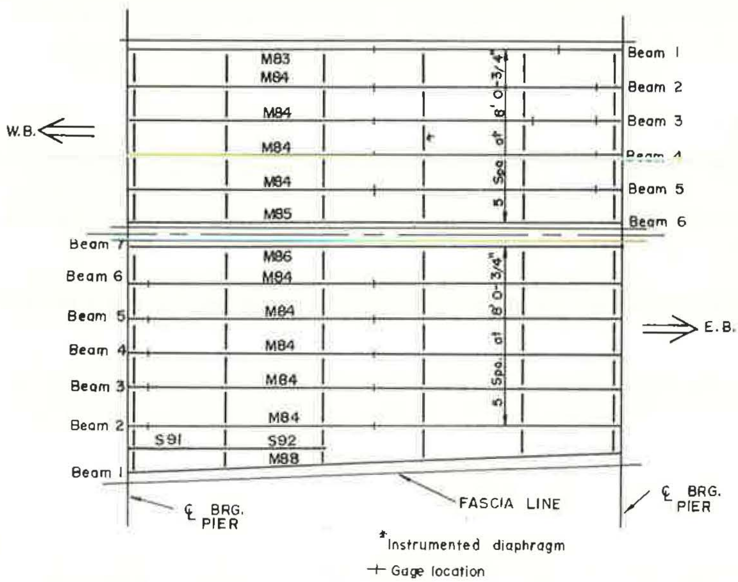


Figure 2. Framing plan of span 10 with locations of strain gauges.



In this case, the initial stress range was 0.6 to 2.4 ksi with successive ranges increased in increments of 0.6 ksi to 7.2 ksi. Thus, the computer was programmed to scan for a peak stress whenever the analog voltage exceeded that corresponding to the minimum test level of 0.6 ksi. Once the peak strain was sensed, the computer would scan for a valley only when the signal voltage dropped below that corresponding to a no-load situation. The resulting stress range would therefore be the absolute value measured between the peak and valley and would be counted in the appropriate stress category. Secondary ranges were counted only when the minimum and zero stress levels were exceeded.

Original plans called for 24-hour data acquisition for a period of 5 to 6 days on each structure. Problems with the computer, however, reduced actual strain-sensing time to 33 and 65 hours on the eastbound and westbound structures respectively. Despite this reduction, the volume of data acquired was quite adequate to develop a clear picture of the loading to which the individual members are subjected.

Truck Classification and Lane Counts

All trucks and buses with more than four tires were classified and counted according to the lane in which they crossed the test spans. Normal highway lighting provided sufficient illumination for dark-hour classification and counting. Thus, a continual record of truck crossings was obtained over a 6-day period for both eastbound and westbound spans. Data on multiple truck crossings, i.e., instances where any part of more than one truck was on the span at one time, were obtained for shorter time intervals on each bridge. Truck-count periods corresponded to computer recording intervals, e.g., counting for 1 hour followed by a 4-min lapse during which the computer printed out strain data.

Truck Weights

Unfortunately, off-highway space for truck-weighing operations in the Bridgeport area was limited to the Stratford Toll Station. The truck-weighing site selected was a rest area in Westport, approximately 9.25 miles, and 10 exits, west of the test spans. At Westport, weighing in both roadways was carried out over three nonconsecutive 8-hour shifts. Trucks were flagged down at random and driven across portable scales that weighed individual axles. Axle spacings were measured simultaneously with the weighing. A traffic count was also conducted at the Westport site during the weighing operations. This count took into consideration all types of vehicles, including passenger cars and motorcycles, but did not classify vehicles according to lane traveled.

TEST RESULTS

Eastbound Structure

Table 2 gives the output of the 11 gauges monitored in the eastbound span and also the number of trucks that traversed the structure during the total recording period. As previously indicated, malfunctions in the computer system resulted in intermittent recording of stress ranges, which totaled 33 hours for the majority of gauges.

As can be seen from the table, 94 to 99 percent of the stress ranges occurring at the end of the cover plates fell within the 0.60- to 1.95-ksi levels for all beams. Very few events were recorded in excess of 2.40 ksi, and only a single even above 3.3 ksi (beam 3). Total events divided by the total trucks indicates to some extent the distribution of loading across the span at the ends of the cover plate. It should be noted that beams 3 and 4, which are directly under the outside and middle lanes, sustained the highest value of stress events per truck at this location.

As for the midspan gauges, approximately 97 to 99 percent of all stress ranges recorded fell within the limits 0.60 to 2.85 ksi. As is apparent, the value of events per truck increases with increasing beam number from outside to inside of the bridge; the number of events greater than 2.85 ksi is also much higher than expected in the higher numbered beams (5 and 6). The distribution of high stress ranges would indicate that beam 4, which would normally be expected to carry 25 to 30 percent of the total internal live load moment generated in the five beams in question, is now sustaining considerably less than its share of high-stress events. Conversely, beam 6, which is under the inside lane and would normally carry approximately 12 to 20 percent of the live load moment induced in these five members, is sustaining about 33 percent of the events greater than 2.85 ksi. Assignation of one specific cause of this unusual distribution is difficult inasmuch as a combination of effects, e.g., bridge and traffic dynamics, interaction of longitudinal and torsional motions, and construction differences, could make themselves felt in unpredictable ways.

During computer operation, approximate lane distribution of all trucks in the eastbound roadway of span 10 (Table 2) was as follows: 55 percent in the outer lane, 45 percent in the middle, and less than 1 percent in the inner. Weekday truck volume was high and exhibited a certain periodicity with peak volume occurring at about 900 to 1,000 hours. Weekend truck traffic was low and favored the outside lane. Multiple crossings were generally less than the number of total commercial vehicles by an order of magnitude. Considerable jumps in multiple crossings occurred during peak-hour traffic flows on the weekdays. Most of the multiple crossings involved single trucks in the outer and middle lanes.

In Westport, a total of 845 trucks were weighed and measured. The majority of gross truck weights were fairly uniformly distributed between 20 and 70 kip, with no loading exceeding 100,000 pounds. As for overweight vehicles, 21 of the 845 trucks weighed exceeded the allowable maximum weights for their respective classes as established in the Connecticut code. It is interesting to note, however, that the greatest percentage of illegal overweights was not in gross loadings but in the single drive axles on the 2S-1 and 2S-2 vehicles. Of the 311 2S-2 trucks weighed for example, 56, or 18 percent, carried excessive weight on the single drive axle.

Westbound Structure

Table 3 gives the output of the 13 gauges monitored in the westbound span and also the number of trucks that traversed the structure during the total recording period. Computer recording time for the gauges varied from 65 hours 10 min to 15 hours 10 min due to channel sharing and, in certain cases, gauge failure. In the table, the a gauges are those placed on the flange next to the primary cover plate, and the e gauges are those mounted on the primary plate just off the secondary one. As in the eastbound structure, recording of stress ranges was not continuous as planned but intermittent due to minor system malfunctions. According to the table, from 88 to 97 percent of the total stress events occurring at the end of the primary cover plates fell within the 0.6- to 1.8-ksi range. Only 35 events greater than 3.0 ksi were recorded at these locations, and, of these, 28 occurred on beam 2. In the case of the two e gauges, the one mounted on the fascia beam (beam 1) produced considerably more high stress ranges than the one on beam 3, despite the significantly greater section modulus and exterior position of beam 1.

As for midspan gauges, approximately 95 to 99.9 percent of all stress ranges recorded fell within the limits 0.60 to 3.6 ksi. It follows from Table 3 that the number of stress events greater than 3.0 ksi decreases with increasing beam number (outside to inside). It will be noted that the frequency of high-stress events occurred in just the opposite manner: increasing with increasing beam number.

The output of the gauge on the diaphragm between beams 3 and 4 produced some of the most surprising results. Each truck that traversed the test span during the recording period gave rise to an average of 1.9 stress-range events in this member. More surprising yet was the magnitude of the stress ranges. In 63 hours, the bottom flange at midspan of the diaphragm was stressed to 3.0 ksi more than 2,400 times. Included among these were 27 stress events greater than 5.4 ksi. This might explain the high incidence of bolt failures in the diaphragm-to-beam connections that have occurred in this viaduct.

Like the eastbound roadway, percentage of trucks on the outer, middle, and inner lanes in the westbound roadway averaged 55, 45, and 1 percent respectively, with variations of 4 to 5 percent occurring in the outer and middle lanes depending on hours of

Table 1. Details of beams shown in Figure 2.

Member	Cover Plate Dimensions (ft) Top	Cover Plate Dimensions (ft) Bottom	Size	Center-to-Center Bearing (ft, in.)
M88	14 × $1\frac{1}{4}$ × 64	15 × $1\frac{1}{4}$ × F. L. 14 × 1 × 77	36WF300	113 $7\frac{13}{16}$
M84	14 × 1 × 65	15 × $1\frac{1}{4}$ × 95 14 × $1\frac{1}{8}$ × 77	36WF230	113 6
M86		14 × $\frac{3}{4}$ × 75	36WF280	113 6
M85	14 × $\frac{7}{8}$ × $58\frac{1}{2}$	15 × 1 × 87 14 × $\frac{3}{4}$ × 74	36WF280	113 6
M83	14 × $1\frac{1}{4}$ × 64	15 × $1\frac{1}{4}$ × F. L. 14 × 1 × 76	36WF300	113 6
S91			18WF60	
S92			18WF60	

Table 2. Total number of events at each stress range.

Item	End of Cover Plate					Midspan				
	2a	3a	4a	5a	6a	2d	3d	4d	5d	6d
Stress level and stress, ksi										
0 at >4.65	0	0	0	0	0	0	0	0	0	0
1 at 4.65										
2 at 4.20	0	0	0	0	0	0	0	0	1	2
3 at 3.75	0	0	0	0	0	2	5	2	5	8
4 at 3.30	0	1	0	0	0	9	23	11	17	41
5 at 2.85	0	6	0	0	1	67	151	36	103	171
6 at 2.40	0	91	2	3	0	278	464	245	302	396
7 at 1.95	9	176	19	34	1	644	635	858	485	724
8 at 1.50	110	897	251	384	55	1,022	1,094	1,266	940	1,783
9 at 1.05	937	1,307	1,827	1,236	444	1,931	1,798	1,963	2,032	3,428
Minimum 0.60	2,154	1,947	2,542	2,221	1,013	1,906	1,726	1,835	1,357	2,023
Total	3,210	4,425	4,641	3,878	1,514	5,859	5,896	6,216	5,242	8,576
Total trucks	7,783	7,783	7,783	7,783	3,792	7,783	7,783	7,783	6,543	7,783
Events/truck	0.41	0.57	0.60	0.50	0.40	0.75	0.76	0.80	0.80	1.10
Total recording time, hours	33	33	33	33	17	33	33	33	28	33

Table 3. Total number of events at each stress level.

Item	End of Cover Plate						Dia-phragm	Midspan				Stress for 1d (ksi)
	5a	4a	3a	2a	1e	3e		4d	3d	2d	1d	
Stress level and stress, ksi												
0 at >6.0	0	0	0	0	0	0	2	0	0	1	3	>7.2
1 at 6.0												7.2
2 at 5.4	0	0	0	0	7	0	25	0	0	11	5	6.6
3 at 4.8	0	0	0	0	9	0	79	0	0	21	13	6.0
4 at 4.2	0	0	0	0	30	0	248	0	10	54	32	5.4
5 at 3.6	0	1	1	3	72	0	673	7	26	241	57	4.8
6 at 3.0	0	3	2	25	146	17	1,375	42	124	556	99	4.2
7 at 2.4	1	34	8	106	402	96	2,797	392	452	801	225	3.6
8 at 1.8	36	331	127	838	1,027	516	5,077	1,412	992	1,559	415	3.0
9 at 1.2	682	1,946	1,244	2,654	3,064	1,184	9,122	4,091	2,223	4,358	689	2.4
Minimum 0.6	3,551	5,127	3,756	4,807	3,839	1,527	8,304	6,323	3,263	5,001	7,908	0.6
Total	4,270	7,442	5,138	8,431	8,596	3,340	27,702	12,267	7,090	12,603	9,452	
Total trucks	14,877	14,582	7,654	14,582	10,538	3,386	14,582	14,582	8,393	14,385	7,744	
Events/truck	0.29	0.51	0.67	0.58	0.82	0.99	1.90	0.84	0.84	0.88	1.22	
Total recording time, hour:min	65:10	63:10	34:10	63:10	45:00	15:10	63:10	63:10	36:10	62:10	30:00	

computer operation. Periodic fluctuation in commercial traffic and in multiple crossings also followed closely the patterns that developed in the eastbound flow. Commercial vehicles accounted for approximately 13.5 percent of the total traffic flow.

In Westport, 588 trucks were sampled for gross and axle weights in the westbound roadway. In the popular truck classes, the average gross weights were generally equal to, or less than, those measured in the eastbound roadway. Distribution of gross truck weights was fairly uniform between 30 and 70 kip. Approximately 10 percent more westbound trucks weighed between 20 and 30 kip than eastbound trucks. Gross truck weights in excess of 70 kip were about 8 percent of the total trucks weighed; this percentage was about double that in the eastbound roadway. Average westbound gross truck weight was, however, slightly lower than the eastbound as a result of the higher percentage of trucks weighing 20 to 30 kip.

As in the eastbound roadway, the majority of illegal loadings occurred in the single drive axle on the 2S-2 trucks. Approximately 10.3 percent of the 3S-2 vehicles were above the maximum allowable gross weight of 73,000 pounds, which amounted to three times the overweight trucks in this class in the eastbound roadway. Percentages of overweights in other classes were similar in both roadways.

FATIGUE CONSIDERATIONS

In analysis of the fatigue lives of the two spans under consideration, various assumptions were made. First, the percentage of trucks in the total traffic stream was considered constant from 1958 to 1971 (about 13.5 percent). Second, the 1967 ADT was assumed to represent the average ADT for the period 1958 to 1971. Third, the stress conditions given in Tables 2 and 3 were assumed to prevail since 1958. Fourth, the stress level below which there would be a negligible effect on fatigue was assumed to be 2.85 and 3.0 ksi for the eastbound and westbound roadways respectively. Following these assumptions, estimated values of total stress ranges greater than 2.85 (eastbound) and 3.0 (westbound) ksi that have occurred since 1958 were obtained for each gauge.

The results of this exercise indicate that, based on present behavior under live loading, the two test structures are adequately designed to resist fatigue for some time to come. For example, the critical end-of-cover-plate location monitored by gauge 1e in the westbound roadway was subjected to more than 586,000 events greater than 3.0 ksi, the greatest number sustained for this location on both bridges. Even if it is assumed that all 586,000 events were registered at the maximum range recorded (6.0 ksi), this particular detail would be in no immediate danger of fatigue failure because the 95 percent lower confidence limit for cycles to failure has been estimated at about 4×10^6 cycles at $S = 6$ ksi for beams with end-welded cover plates.

Considering the results obtained from the tests and subsequent fatigue analyses on span 10, it is difficult to comprehend the apparent fatigue failure in adjacent span 11. The two spans are almost identical in detail and completely identical with regard to length, traffic, age, environment, and steel. There are, however, conjectural differences, such as construction technique and workmanship, that may very well have entered into the failure in span 11.

If we disregard the popular theory that the effect of low stress levels on beam-fatigue failure is negligible, another possibility exists. The shape of the S-N curve below $S = 3$ ksi is not known for cover-plated steel beams. All known laboratory tests in this area have been carried out above this stress level. Thus, if the S-N curve does not flatten out below $S = 3$ ksi as is currently assumed but maintains the same slope, it is entirely possible that the large number of low-level stress events, say from 3.0 to 0.6 ksi, that have occurred at critical points in the members may exert a considerable influence on the fatigue lives of the structures in span 10. The number of these events, together with the low-amplitude stress cycles that occurred in the damping phase and were not recorded as events in this study, can be estimated at close to 200,000,000 for certain members.

From the test data obtained in this study, it is safe to assume that the fatigue behavior of one bridge cannot necessarily be used to assess the fatigue behavior of a similar bridge subject to the same loading and environmental conditions.