Fatigue Damage in the Lehigh Canal Bridge From Displacement-Induced Secondary Stresses

John W. Fisher, Ben T. Yen, and J. Hartley Daniels, Fritz Engineering Laboratory, Lehigh University

Strains were measured at several structural steel details on one of the Lehigh Canal bridges under normal traffic. Inasmuch as these bridges have several fatigue cracks in the tie plates connecting the floor beams to the outrigger cantilever brackets, the primary focus was on the tie plates and the cause of fatigue cracking. Strain gauges were mounted on five tie plates, on a stringer, and on the longitudinal girders. An automatic computer-controlled data acquisition system was used to record the strain range occurrences. In addition, an analog trace recorder was used to determine the live load strain variations with time. Stress ranges in the stringer and girders were comparable to those observed by others in girder bridges. However, the horizontal in-plane bending stresses in the tie plates were found to be two to three times as high. The higher stress range was attributed to differential displacements between the deck-stringer system and the girders, which were transmitted through the tie plates. The strain measurements on the tie plates and the volume of truck traffic during the structure's life were used to estimate the cumulative damage in several tie plates. Good correlation was obtained with the root mean square stress range and constant cycle laboratory fatigue test results. Miner's rule was also found to provide a good correlation.

Fatigue cracks have recently been detected at steel bridge details such as the ends of cover plates, web or flange attachments, and tie plates connecting transverse floor beams and brackets across the main girders. Among bridges that have sustained some of these cracks are the Yellow Mill Pond Bridge on the Connecticut Turnpike (1), the Lehigh River and Lehigh Canal bridges in Pennsylvania, and the Allegheny River bridge on the Pennsylvania Turnpike. All of these bridges carried large volumes of truck traffic.

Recent laboratory studies on the fatigue strength of beams with cover plates or welded attachments indicate that the stress range under live loads controls the fatigue behavior of structural details (2, 3). To further examine the fatigue behavior of several steel bridge details under traffic loading and correlate the stress range history of these details with laboratory fatigue data, pilot field tests of one of the Lehigh canal bridges were undertaken. This paper summarizes the results of these pilot studies.

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DESCRIPTION OF BRIDGES

The Lehigh Canal bridges consist of twin bridges that carry the eastbound and westbound lanes of US-22 near Bethlehem, Pennsylvania. Each bridge is continuous for three spans and has small haunches at the interior piers (Figure 1). Each bridge has two riveted longitudinal girders with a floor beam-stringer system and a noncomposite concrete deck. The 44-m (144-ft) end span of the eastbound bridge was chosen for testing because of its accessibility. A typical cross section of the two-lane bridge is shown in Figure 2a. The details of tie plates, which connect the outrigger brackets to the floor beams and girders, are shown in Figure 2b. The bridges were constructed in 1951 to 1953 and opened to traffic in November 1953.

Inspections by Pennsylvania Department of Transportation personnel revealed several cracks in the tie plates in spring of 1972. The approximate location and length of these tie plate cracks in the test span are shown in Figure 3. Throughout the length of the Lehigh Canal bridges and the Lehigh River bridges, most of the tie plate cracks were at or near the outside edge of the longitudinal girders. Several of the plates had cracked across their entire width; some had only fine hairline cracks. All observed cracks appeared to be through the thickness of the tie plates as shown in Figure 4. All cracks started at the edge of the tie plates from a tack weld that was used to connect the tie plates to the outrigger bracket during fabrication.

STRAIN DATA ACQUISITION

Strain gauges were mounted on the tie plates, the longitudinal girders, and a stringer. Most of the gauges were placed on the tie plates parallel to the edges of these plates and above the cutoff point of the top flange of the cantilever brackets (shown in insert to Figure 5). All gauges were 6.4-mm (¼-in) electrical resistance foil gauges.

Strain variations due to traffic were recorded by using the FHWA automatic data acquisition system and an analog trace recorder. The FHWA system consists of an amplifier, an analog-to-digital signal converter, and a computer (4). The computer was connected to a teletype machine for input-output. In this study the strain range frequencies at the gauge points were sought. As each vehicle traveled over the bridge, the magnitudes of strain ranges were recorded. After a period of time, the number of strain range excursions (occurrences) from predetermined values (levels) was printed. Very low strain ranges due to vibration and automobile traffic were excluded.

Analog traces were recorded periodically. The analog recorder used the amplifiers of the FHWA system so that strain variations of several gauges could be monitored by the two systems simultaneously.

Traffic on the bridge was correlated with recorded strains in the tie plates and girders by visually observing traffic flow and by recording the number and types of trucks for short periods of time. The standard FHWA truck classification was used to identify truck traffic. Altogether 170 h of strain data were acquired for statistical evaluation and stress analysis.

STRESSES AND DISPLACEMENTS

Stress History Measurements

From analog traces of strain variation and observations of the traffic on the bridge, it was evident that each truck crossing the bridge caused one primary stress range at all gauge points. This is true of the tie plates and the girders (Figure 6).

The recorded live load stress distributions in the tie plates with gauges are shown in Figure 3 for a given instance. The stress distribution indicates that the tie plates were subjected to bending in the horizontal plane. The gauges at the centerline of the tie plates had low stress magnitudes, implying that truck loading caused little axial elongation or vertical bending of the plates.

It is also apparent from Figure 3 that the maximum live load horizontal bending stresses in a tie plate depended on the location of the tie plate in the bridge span. Maximum stresses were higher in the tie plates near the abutment and toward the pier. This distribution agrees with the crack pattern observed in the tie plates.

Live load stresses in the longitudinal girders and in a stringer of the bridge were low compared to those observed in the tie plates. The magnitudes of maximum live load stresses in the girders were about 55 MPa (8000 lbf/in^2). These values are stress ranges resulting from stress reversal in continuous girders and are slightly higher than the maximum stresses recorded by other investigators in main longitudinal members (4, 5, 6, 7, 8).

In-Plane Bending of Tie Plates

The time variation of stresses shown in Figure 6 was the sum of the static response to a truck and the vibrational stresses. The static stress response at a point on a girder flange is analogous to the stress influence line for that point. All stress-time response records shown in Figure 6 for a gauge on a girder are typical stress influence lines for a point in the side span of a three-span continuous beam.

For tie plates, the stress-time response records (Figure 6) indicate that the pattern of stress variation was the same for all types of trucks; only the magnitude of the stress range changed. Furthermore, the relationship between the stress variation in the tie plate and those in the girder was the same for all trucks. This suggested that the stress variation in the tie plates was directly related to the behavior of the longitudinal girders. Preliminary analysis indicates that longitudinal displacement at the top flange of the girder causes horizontal bending in the tie plates (Figure 7). Because the longitudinal displacement of a point on the top flange of the girder is the product of the slope of the deflection curve and the distance from the neutral axis to that point, the strain influence line for the tie plates is analogous to the influence lines for slope in the girder at the tie plate. Figure 8 shows a comparison of the strain variation at gauges 5 and 10 on tie plates C4S and C6N and the influence lines for the slopes of the girder at the corresponding points. The similarity is apparent.

Effect of Tie Plate Geometry and Connection of Plate to Girder

Horizontal in-plane bending stresses in the tie plates and the longitudinal displacements of the top flanges of the girders were further examined by removing cracked tie plates C0S and C1S (Figure 3). New tie plates of different widths and thicknesses were placed at these locations in different combinations. The plates were bolted to the floor beam and cantilever bracket in all cases but sometimes were not connected to the top flange of the girder. Strain gauges were mounted at middepth on the edge of these tie plates to measure the horizontal in-plane bending strains due to normal truck traffic and a test truck. Simultaneously, the deflections of floor beam-outrigger bracket C0S and girder ES (Figure 5) were measured against the back wall of abutment C with dial gauges. The relative longitudinal displacements between the girder top flange and the outside stringer near floor beams C0 and C1 were also measured. The test truck was a two-axle truck with front and rear axle loads of 47 and 103 kN (10 600 and 23 200 lbf) respectively.

Table 1 gives the average maximum stress range in the tie plates under loading of a test truck and random traffic. The horizontal deflections at floor beam C0S under random truck traffic are given below (location refers to area between abutment wall and component listed):

Location	Maximum Deflection (mm)	East Departure (mm)	West Departure (mm)
Stringer	0.711	0.483	0.229
Outrigger bracket	0.508	0.381	0.127
Girder top flange	1.422	1.143	0.279
Stringer	0.432	0.406	0.025
Girder bottom flange	1.219	0.229	0.991

The results showed that

1. Unbolting the tie plate from the main girder generally decreased the stress range in the tie plates,
2. The stress range was further decreased when narrow plates were used, and
3. Changes in plate thickness did not alter the magnitude of the stress range in the tie plates.

These tests further confirmed the fact that displacements were primarily responsible for the horizontal, in-plane bending stresses in the tie plates.

The average maximum horizontal deflection of floor beam-bracket C0S and girder ES against the back wall of abutment C is given above. The horizontal displacement at the top of the girder was more than twice those occurring at the stringers and the outrigger bracket, indicating a relative displacement. Unbolting the tie plate from the girder flange and reducing the tie plate width increased the relative displacement between the top flange of the girder and the outside stringer slightly because a more flexible connection was generated, which, actually, is compatible with the design

Figure 1. Lehigh Canal bridge (instrumentation installed in left side span).

Figure 2. (a) Cross section of bridge and (b) tie plate detail at floor beam-bracket connection to girder.

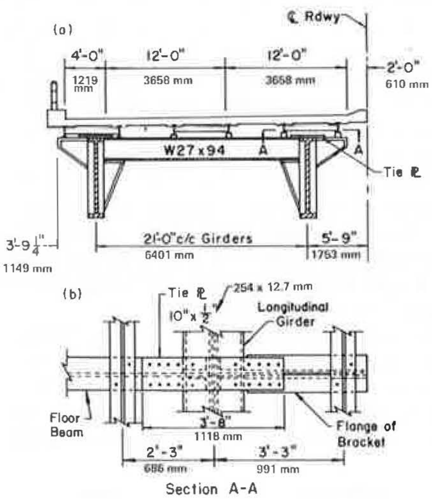


Figure 3. Cracks and stresses in tie plates of test span.

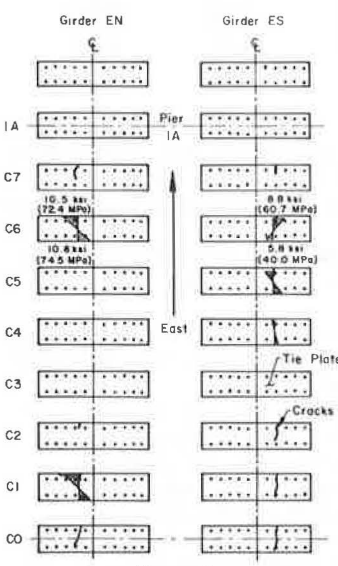


Figure 4. Crack in tie plate originating at tack weld.

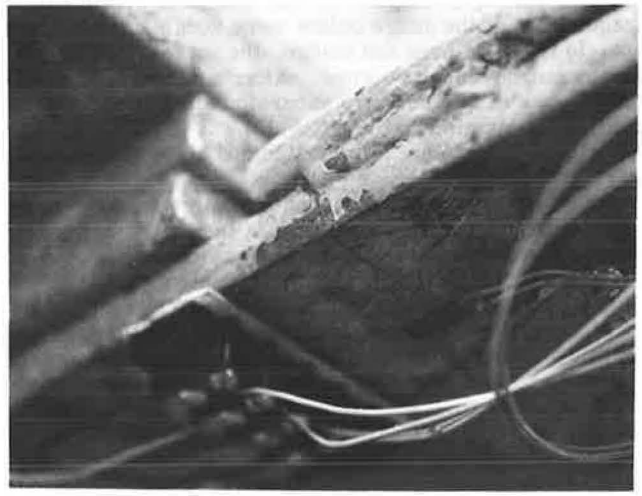


Figure 5. Instrumentation on tie plates and girder.

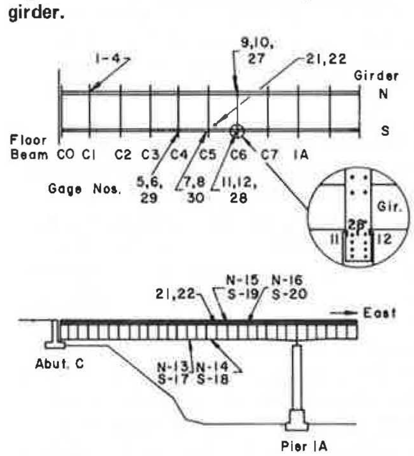
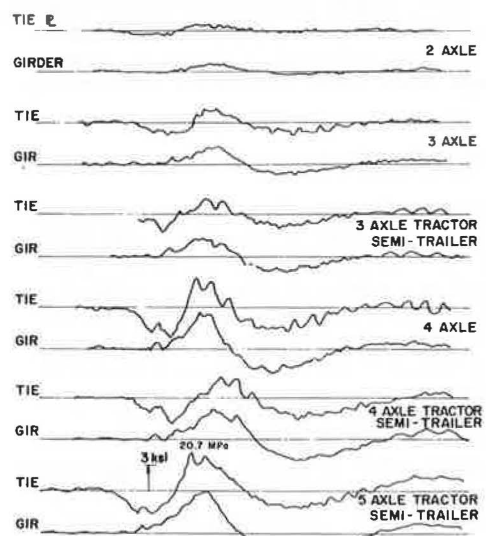


Figure 6. Comparative response of tie plate C4S and south girder under various loads.



assumptions. Unbolting and reducing tie plate widths, on the other hand, greatly reduce the bending stresses in the tie plates.

STRESS RANGE UNDER RANDOM TRAFFIC

The stress range occurrence data from the FHWA system were plotted as histograms depicting the frequency of occurrence between the stress range levels. Examples are shown in Figure 9 for gauges on tie plates C4S and C6N and on girder EN. The gauge on tie plate C4S was subjected to stress reversal, and the maximum stress magnitudes were smaller than those recorded on other tie plates (Figure 3). A large percentage of the stress range occurrences was at low stress levels. Tie plate C6N was subjected to high maximum stresses, and the stress range was relatively more frequent at higher levels. There was also a tendency for two peaks on the histograms. The gauges on the main girders indicated much lower stress range levels. It was observed that only larger trucks induced stresses of more than 6.9 MPa

Figure 7. Longitudinal displacement at top flange of girder and horizontal bending of tie plate.

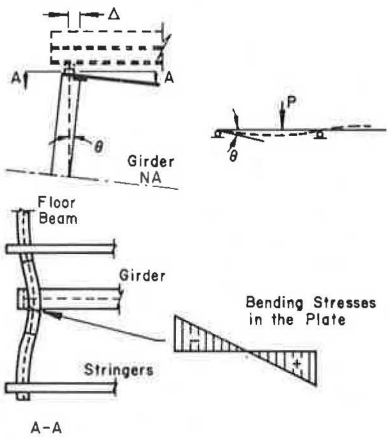


Figure 8. Comparison of measured strain history in tie plates and influence lines for girder slope.

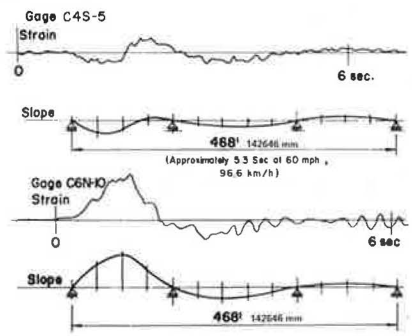


Table 1. Stress range in new tie plates.

Loading	Gauged Section on Tie Plate	Plate Thickness (mm)	Stress (MPa) 254-mm Plate Bolted	254-mm Plate Unbolted	203-mm Plate Bolted	203-mm Plate Unbolted	152-mm Plate, Unbolted
Test truck	Near bracket	25.4	72.4	65.5	–	31.0	24.8
		12.7	72.4	58.6	–	34.5	24.8
	Centerline of girder	25.4	34.5	48.3	48.3	31.0	17.2
		12.7	34.5	34.5	58.6	31.0	25.5
	Near floor beam	25.4	48.3	48.3	–	34.5	13.8
		12.7	64.1	37.9	–	34.5	21.4
Random traffic	Near bracket	25.4	141.0	124.0	–	62.1	58.6
		12.7	148.0	82.7	–	62.1	44.8
	Centerline of girder	25.4	69.0	107.0	55.2	44.8	34.5
		12.7	75.9	62.1	75.9	53.1	37.9
	Near floor beam	25.4	75.9	82.7	–	–	27.6
		12.7	82.7	58.6	–	55.2	31.0

Note: 1 mm = 0.039 in; 1 MPa = 145 lbf/in².

Figure 9. Stress histograms.

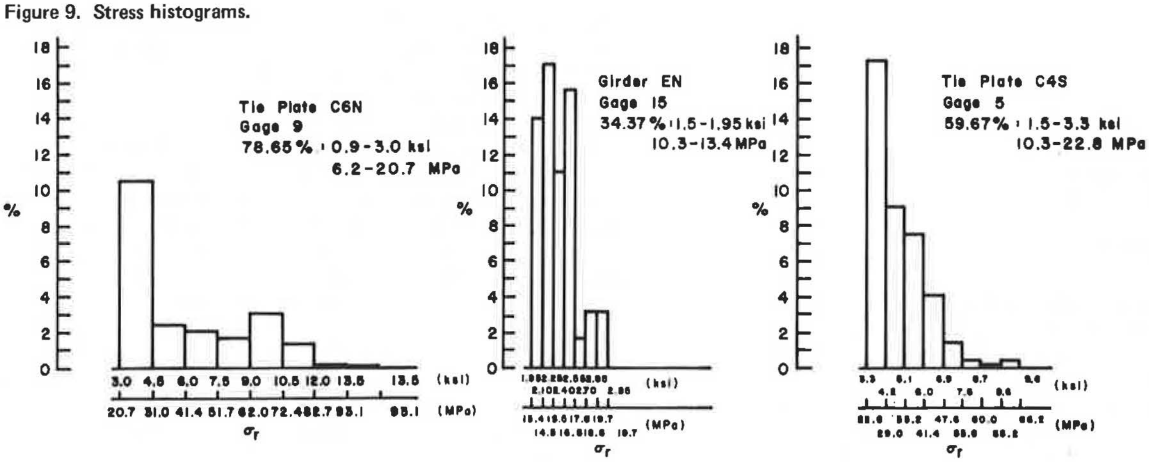


Figure 10. Gross vehicle weight distribution.

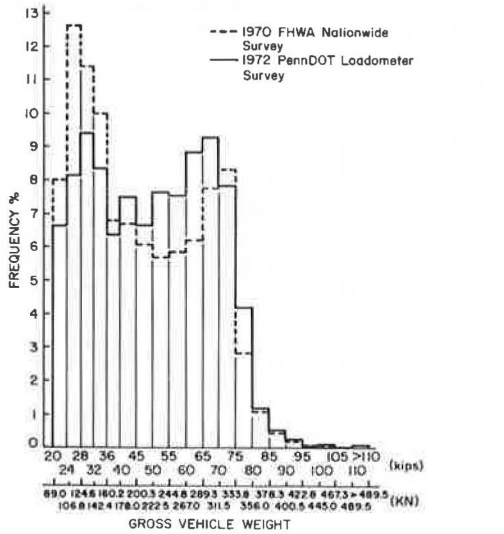


Figure 11. Estimated ADTT at the Lehigh Canal bridge.

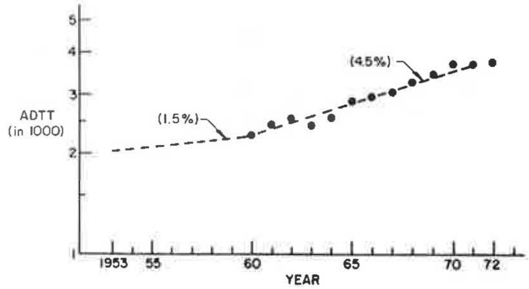
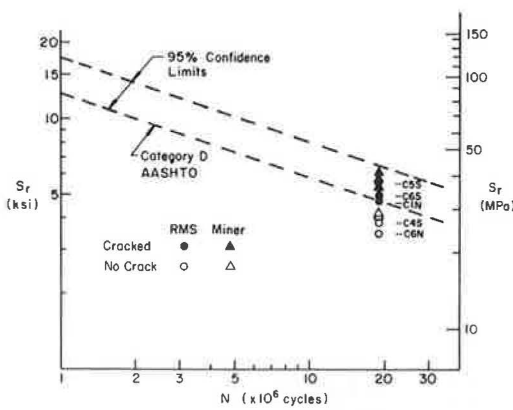


Figure 12. Comparison of estimated equivalent stress range with laboratory test data.



(1000 lbf/in^2) in the girders. The single-peaked histograms reflect this response.

TRAFFIC RECORDS

Traffic counts taken during the in-service testing indicated that the highest volume of trucks consisted of five-axle tractor semitrailers (3S-2). The Pennsylvania DOT provided 24-h traffic counts taken during 1972-73 at a site near the Lehigh Canal bridge. A comparison showed that the 24-h traffic surveys resulted in more two-axle trucks and five-axle tractor semitrailers than were observed at the bridge site during the short sample periods. Overall, the percentages of various types of trucks from the 24-h records were consistent with the observations at the bridge site. Pennsylvania DOT records were also consistent with results from loadometer surveys throughout the state and the nation.

The results of a 1970 FHWA nationwide loadometer survey and the results of a 1972 Pennsylvania DOT loadometer survey for 20 stations on main arteries in Pennsylvania are shown in Figure 10. The relative distributions of truck weight frequencies from the two surveys are comparable. The two peaks in the histograms indicate a large number of loaded trucks of 267 to 334 kN (60 000 to 75 000 lbf) and large numbers of two-axle or unloaded tractor semitrailers of 107 to 160 kN (24 000 to 36 000 lbf).

Inasmuch as the results of traffic counts at the bridge were consistent with traffic counts of the Pennsylvania DOT, which in turn were comparable to results from state and nationwide loadometer surveys, the results of traffic counts at the bridge were considered representative of the normal traffic crossing the Lehigh Canal bridge. Furthermore, the measured stress range histograms were considered to be representative of the stress range spectra due to the truck traffic crossing the bridge.

The total number of trucks that have traveled over the bridge during the 19-year period from 1953 to 1972 was estimated from the Pennsylvania DOT traffic count data. These estimated average daily truck traffic (ADTT) counts are shown in Figure 11 and range from 2038 trucks per day in 1953 to 3709 in 1972. The average rate of increase was about 1.5 percent before and 4.5 percent after 1960 when main arteries leading to the bridge were open to traffic. The total volume from 1953 to 1972 was estimated to be 18.9 million trucks.

CORRELATION WITH FATIGUE TEST RESULTS

Results from beam tests and girder tests in the laboratory have indicated that stress range and the length of welded attachments are controlling factors for fatigue strength (2, 3). Design specifications are derived from available test results (9). Bending tests of tie plates with tack welds (10) showed that category D of the 1974 AASHTO specification is applicable to tack-welded tie plates.

The riveted tie plates of the Lehigh Canal bridge were subjected to horizontal in-plane bending under random truck traffic. Most of the tie plates have 25 to 50-mm-long (1 to 2-in) tack welds connecting the edges of the tie plates to the top flange of the bracket as shown in Figure 4. Load transfer between the flange of the outrigger bracket and the tie plate was provided by both the rivets and the tack welds. Because the tack welds were at the end of the joint, which is the most highly stressed region (11), the stress concentration at the end of the tack weld appears to be as severe as the condition at short attachments to beam flanges as was confirmed by the pilot studies (10). In comparisons of laboratory fatigue test results and measured stresses at the bridge, adjustments must be made to the measured stresses because cracks occurred at the edges of the tie plates where stresses were higher. Furthermore, the measured stresses were not of constant magnitude.

One procedure to account for the stress range spectrum is the root mean square (RMS) method (2, 12, 13). In this method, the RMS of the stress ranges in a spectrum is considered equivalent to a constant cyclic stress range and is correlated with the number of stress cycles corresponding to the spectrum. The RMS stress range is defined as

$$S_{rRMS} = \left(\sum \alpha_i S_{ri}^2\right)^{1/2} \quad (1)$$

where α_i is the frequency of occurrence of stress range S_{ri}.

The stress range occurrences of the gauges of the tie plates were used to determine the RMS values of stress ranges. These values were then adjusted to the plate edges where the crack growth originated.

The S_{rRMS} at the plate edge and the fatigue cycles corresponding to the truck traffic (18.9 million) are shown in Figure 12 and are compared with the constant stress range laboratory data. The data from the Lehigh Canal bridge lie above or near the lower confidence limit for category D. Cracks were observed in tie plates C1N, C5S, and C6S; the data (circles) are on or above the lower confidence limit. Tie plates C4S and C6N had no visible cracks at the time of measurement; the data points for these two plates are below the category D line.

The comparison shown in Figure 12 indicates that, when the RMS stress range is used, the lower confidence limit for category D of AASHTO provides a reasonable lower bound to the fatigue strength of the tack-welded tie plates of the Lehigh Canal bridge. Tie plates that plotted above this limit could be expected to exhibit visible fatigue cracking. The plates C4S and C6N can be expected to develop fatigue cracks under additional truck traffic.

Another procedure to account for cumulative damage is Miner's hypothesis (12). Tie plates C1N, C5S, and C6S had cracks; the cumulative damage ratio based on Miner's hypothesis was close to or higher than 1.0. No visible cracks were detected at tie plates C4S and C6N, which had damage ratios less than unity.

By combining the relationship provided by constant cycle data (2) and Miner's rule (12), an equivalent stress range S_{rMiner} was estimated (13) as

$$S_{rMiner} = \left(\sum \alpha_i S_{ri}^3\right)^{1/3} \quad (2)$$

The S_{rMiner} values of the tie plates are also shown in Figure 12. The results demonstrate that by using Miner's rule the lower confidence limit for category D of AASHTO 1974 specifications also provides a reasonable estimate of the fatigue strength of the tie plates of the Lehigh Canal bridge.

SUMMARY AND CONCLUSIONS

From the pilot field studies and the analysis of results of the Lehigh Canal bridges, the following conclusions can be reached.

1. The live load stresses in the main longitudinal girders were similar to those observed in longitudinal members of other bridges. The maximum live load stresses (stress ranges) were 55 MPa (8000 lbf/in^2).
2. The live load stresses in the tie plates connecting the outrigger brackets and floor beams were much higher than those observed in the longitudinal members. The distribution of these stresses in the plates indicates that horizontal in-plane bending was occurring with little axial extension or twisting of the plates.
3. The development of live load stress in the tie plates was related to displacements caused by longitudinal strain of the main girder under bending. The relative longitudinal movement of the top flange of the main girder with respect to the stringers, brackets, and concrete slab caused a horizontal bending of the tie plates.
4. Unbolting the tie plates from the girder and reducing the tie plate width both decreased the magnitude of the in-plane bending stresses in the tie plates. Bridges using this system in the future should avoid connecting the tie plate to the girder.
5. The crack pattern in the tie plates agreed with the measured strain ranges.
6. The measured stress history for short time periods was compatible with the frequency distribution of trucks reported by the Pennsylvania DOT and FHWA on comparable roads.
7. The RMS stress range and an estimated traffic volume for 1953 to 1972 indicated that category D of 1974 AASHTO interim specifications provides good correlation with crack occurrence on several tie plates.
8. Miner's hypothesis also showed that category D provides a reasonable estimate of the damage that had occurred in the tie plates.

It is apparent that care should be taken to minimize displacement-induced moment and stresses such as those occurring in the tie plates. Very short distances between the first stringer on a bracket and a longitudinal girder result in high displacement-induced moments in the tie plates. These moments are reduced when this distance is increased.

On the Lehigh Canal bridge, more comprehensive studies are being undertaken to develop more complete knowledge of the stress range-life relationship as the bridge provides a unique opportunity to relate laboratory studies to the random loading and corresponding bridge response.

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