High Cycle Fatigue of Welded Bridge Details

FATIGUE DAMAGE IN THE LEHIGH CANAL BRIDGE

by

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Strains were measured at several structural steel details of one of the Lehigh Canal Bridges under normal traffic during a pilot study. Since 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 gages 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 measurements observed by others in girder bridges. However, the ranges of the horizontal in-plane bending stresses in the tie plates were found to be two to three times as high. The cause of the higher stress range was attributed to differential longitudinal displacements between the deck-stringer system and the girders transmitted through the tie plates. The strain measurements on the tie plates and estimated truck traffic during the structure's life were used to estimate the cumulative damage in several tie plates. Good correlation was obtained between the root-mean-square stress range and constant cycle laboratory fatigue test results. Miners rule was also found to provide a good correlation.
Project 72-3: High Cycle Fatigue of Welded Bridge Details

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The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Pennsylvania Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

LEHIGH UNIVERSITY
Office of Research
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LIST OF FIGURES

Figure                                                                 Page
1  Lehigh Canal Bridge - Instrumentation Installed in Left Side Span    20
2  Plan and Elevation of the Test Span                                  21
3a Cross-section of Bridge                                             22
3b Tie Plate Detail at Floor Beam Bracket Connection                   22
4  Schematic of Cracked Tie Plates in the Test Span                    23
5a Crack in Tie Plate Originating at Tack Weld                         24
5b Schematic of Instrumentation on Tie Plates and Girder               25
6  Typical Strain History Response in Tie Plates, Stringer and Girder  26
7  Comparative Response of Tie Plate C4S and South Girder for Various Loads 27
8  Instantaneous Stress Distribution in Tie Plate C6N                   28
9  Instantaneous Stress Distribution in Tie Plate C6S                   29
10 Measured Stress Distributions in the Gaged Tie Plates during Passage of Truck 30
11 Strain History with Truck in the Passing Lane                        31
12 Comparison of Measured Strain History in Tie Plates with Influence Lines for Girder Slope 32
13 Stress Histogram for Tie Plate C4S                                    33
14 Stress Histogram for Tie Plate C6N                                    34
15 Stress Histogram for North Girder                                    35
16 1972 PennDOT Loadometer Survey Gross Vehicle Weight Distribution    36
17 1970 FHWA Nationwide Gross Vehicle Weight Distribution              37
18 Estimated ADTT at the Lehigh Canal Bridge Site                       38
19 Comparison of Estimated Equivalent Stress Range with Laboratory Test Data 39
1. INTRODUCTION

1.1 Background and Objectives

Fatigue cracks have recently been detected on several steel highway bridges in the United States. These cracks have occurred at bridge details such as the ends of coverplates, at web or flange attachments, and in tie plates connecting transverse floor beams and brackets. Among bridges which have sustained some of these cracks are the Yellow Mill Pond Bridge on the Connecticut Turnpike\(^1\), the Lehigh River and Lehigh Canal Bridges on U.S. Route 22 in Pennsylvania (reported herein), and the Allegheny River Bridge on the Pennsylvania Turnpike\(^2\). All of these bridges carried large volumes of truck traffic. The occurrence of cracks in the tie plates of the Allegheny River Bridge and the Lehigh River and Lehigh Canal Bridges provided a unique opportunity to observe the stress history of details which were actually experiencing fatigue crack growth.

Recent laboratory studies on the fatigue strength of beams with coverplates or welded attachments indicate that stress range (live load and impact stress) controls the fatigue behavior of structural details\(^3,4\). No "run-out" or fatigue endurance limit was observed when testing these beams up to 6,000,000 cycles of load application at 6 ksi stress range.

For the purpose of further exploring the fatigue behavior of steel bridge details under relatively low stress ranges and high numbers of cyclic loading, and to correlate the stress range history of bridge
components with laboratory fatigue data, pilot field tests of one of the Lehigh Canal Bridges were undertaken.

This report summarizes briefly the method of testing, the recorded stresses, traffic on the bridge, the fatigue cracks, the correlation between laboratory and field test results, and a brief discussion of the causes of the high cyclic stresses in the tie plates.

1.2 Description of Bridges

The Lehigh Canal Bridges consist of twin bridges which carry the eastbound and the westbound lanes of U.S. Route 22 near Bethlehem, Pennsylvania. Each bridge is continuous for three spans with small haunches at the interior piers as illustrated in Figs. 1 and 2. Each bridge has two riveted steel longitudinal girders with a floor beam-stringer system and a non-composite concrete deck. An end span of the eastbound bridge, Fig. 1, was chosen for testing because of its accessibility. The plan and elevation of the 144 foot end span are shown in Fig. 2.

In most of the test span, the longitudinal girders are 8'-1/2" deep. Nine floor beams (W27 x 94) and outrigger brackets, two exterior stringers (W14 x 34 and W16 x 40), and four interior stringers (two W16 x 40 and two 16 x 45) constitute the load carrying steel system. A typical cross section of the two-lane bridge is shown in Fig. 3a. The details of tie plates which connect the brackets to the floor beams are shown in Fig. 3b. The bridges were constructed in 1951-53 and opened to traffic in November 1953.
Inspections by Pennsylvania Department of Transportation personnel had revealed several cracks in the tie plates in the spring of 1972, prior to in-service testing. The approximate location and length of these cracked tie plates in the test span are summarized in Fig. 4. The cracks detected throughout the length of the Lehigh Canal Bridges and the Lehigh River Bridges are summarized in Appendix A. Most of the cracked tie plates were at or near the piers and abutments. All cracks were along 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 illustrated in Fig. 5a. The cracks did not pass through the rivet holes. All cracks started at the edge of the tie plates from a tack weld toe. The tack welds were used to connect the plates to the bracket prior to riveting.
2. STRAIN DATA ACQUISITION

2.1 Strain Gages and Recording Systems

In order to monitor stresses, electrical resistance strain gages were mounted on the tie plates, on the main longitudinal girders, and on a stringer beam. Most of the gages were mounted on the tie plates. The tie plate gages were orientated parallel to the edges of the tie plates and were generally above the cut-off point of the top flange of the cantilever brackets, Fig. 5b.

The approximate location of the strain gages is indicated schematically in Fig. 5b. Gages were placed on tie plates, C1N, C4S, C5S, C6S, and C6N. The gages on the girders were located on the top and bottom flanges between floor beams 4, 5, 6 and 7. A stringer was also instrumented between floor beams 5 and 6.

All gages were 1/4 in. long electrical resistance foil gages. Weatherproof coatings were applied to protect the gages from moisture and the environment. The gages were connected to temperature compensation gages and plates, so as to minimize the effect of temperature changes.

Strain variations due to traffic were recorded using two separate systems: the FHWA automatic data acquisition system and an analog trace recorder. The FHWA system, which is installed in a van, consists of an amplifier, an analog-to-digital signal converter, and a computer. The computer is connected to a teletype machine for input-output. In this study the strain histories at the gage 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 occurrences when the strain range was between predetermined values (levels) were printed. To exclude recording large numbers of very low strains due to vibration and automobile traffic, a test level was defined for each gage. The strain range had to exceed this level (between 15 to 50 micro-inches per inch) to be recorded. For accurate recording of strain ranges, the zero-level of strains was checked periodically to prevent drifting.

Variation of strains due to traffic were also recorded by an analog trace recorder. A typical recorded analog trace is shown in Fig. 6 in which the live-load strain magnitudes of six gages were recorded as a function of time. The analog recorder utilized the amplifiers of the FHWA system, thus strain variations of several gages could be monitored by the two systems simultaneously.

2.2 Traffic Identification and Recording Periods

In order to correlate traffic on the bridge with recorded strains in the tie plates and girders, truck traffic counts were made. This was done visually by observing the traffic flow and recording the number and types of trucks for short periods of time. These observations were also made while the analog traces were being taken so that strain magnitudes could be correlated with various types of trucks.

Strain data were compiled during two intervals of time. Altogether 170 hours of data were acquired for statistical evaluations and stress analysis.
3. STRESSES IN VARIOUS COMPONENTS

From observing the analog traces of strain variations and the traffic on the bridge, it was evident that each truck crossing the bridge caused one stress range excursion at all gage points on the bridge. This was true of the tie plates and the girders as illustrated in Fig. 7. The strain magnitudes in most of the tie plates were much higher than those observed in the stringer and girders.

The recorded live load stress distributions in typical tie plates are shown in Figs. 8 and 9 for a given time. The stress distributions indicate that all tie plates were subjected to bending in the horizontal plane. The gages at the centerline of the tie plates had low stress magnitudes, implying that there was little axial elongation or vertical bending of the plates due to truck loading.

The stress distributions in the tie plates are superimposed on the plan-view sketch of these plates in Fig. 10. From the stress data for these plates, it was observed that the maximum live load horizontal bending stresses were higher in the tie plates near the abutment and towards the pier. This distribution agrees with the crack pattern observed in the tie plates. Some cracks occurred at locations where the live load stresses were primarily compressive in nature. This reflects the dead load and residual tensile stresses which made the stress range fully effective in crack growth\(^3,4\).
In order to examine the relative stress magnitudes at different
tie plates when a truck was crossing the bridge, one gage on each of the
tie plates was connected to the recording instruments simultaneously.
Examples of strain variation with time as recorded on the analog trace are
shown in Fig. 11 which shows the response when a five-axle tractor semi­
trailer was traveling in the passing lane. As the vehicle crossed the
bridge, tie plates in the slow lane (gages 5, 7 and 11) registered lower
strains than tie plates in the passing lane (gages 1 and 10). Gage 20
was on the top flange of the slow lane girder. The small fluctuations of
strain for each gage were due to vibration of the bridge under traffic.

The strain responses shown in Figs. 6 and 11 confirm that
strains in the tie plates were dependent on the location of the tie
plates in the bridge span, and that maximum stresses were higher in tie
plates near the abutment and towards the pier.

Live load stresses in the longitudinal girders and a stringer
of the bridge were low compared to those observed in the tie plates. The
magnitude of maximum live load stresses in the girders were in the order
of 8 and 9 ksi. These values are comparable to strains recorded by other
investigators in main longitudinal members\textsuperscript{6,7,8,9}.

Examples of the time variation of stresses due to a truck were
shown in Figs. 6, 7 and 11. The total stresses were the superposition of
the static response and the vibrational stresses. The static stress
variation for a point on a girder flange is analogous to the stress influ­
ence line for that point. The stress-time tracing for gage 19 in Fig. 6

\textsuperscript{6,7,8,9}
is obviously similar to the stress influence line for a point in the left span of a three-span continuous beam.

The strain variation traces shown in Fig. 7 indicate that the pattern of stress excursion at a tie plate on the bridge was the same for all types of trucks, only the magnitude of the strain range changed. This suggested that the strain variation in the tie plates was directly related to the strain occurring in the longitudinal girders. Preliminary analysis indicates that longitudinal displacement at the top flange of the girder would cause horizontal bending in the tie plates. Since 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 would be analogous to the influence lines for slope in the longitudinal girder at the tie plate. Figure 12 compares the strain variation at gages 5 and 10 on tie plates C4S and C6N with the influence lines for the slopes of the girder at the corresponding points. It is readily apparent that these curves are compatible with each other. Detailed evaluations of this behavior are under way.
4. STRESS RANGE UNDER RANDOM TRAFFIC

The data acquisition system was programmed to count the number of strain range occurrences. More trucks traveled in the slow lane than in the passing lane. Hence, the strain range occurrences for tie plates under the slow lane were higher than those for the tie plates under the passing lane.

The detailed strain range occurrence data are summarized in Tables B1 and B2 of Appendix B. Table B1 shows the stress range levels and frequency of occurrence of the stress excursions for the tie plates. Similar data is given in Table B2 for the girders and stringers. The total recording time and number of occurrences is also indicated.

The stress range occurrence data were plotted as histograms, depicting the percentage of frequency of occurrence between the stress range levels. Examples are shown in Figs. 13 to 15 for gages on tie plates C4S and C6N and on the north girder. Although the recording periods were not very long, some general observations could still be made. The gage on tie plate C4S was subjected to stress reversal and the maximum stress magnitudes were smaller than observations on other tie plates (see Figs. 11 and 12). A large percentage of occurrences were observed at lower stress range levels. Tie plate C6N was subjected to maximum stress magnitudes that were higher and more frequent at higher stress range levels. There was also a tendency for two peaks on the histograms. The gages on the main longitudinal girders indicated much lower stress
range levels. From the analog traces and traffic monitoring, it was observed that only larger trucks induced stresses above one ksi in the girders. The single-peaked histograms reflect this response.
5. TRAFFIC RECORDS

During the in-service testing of the Lehigh Canal Bridge, traffic counts were taken on the bridge on three consecutive weekdays. All counts were carried out in the afternoon. The highest volume of trucks consisted of five-axle tractor semi-trailers. Because the sample time was only limited in length the results were compared with existing PennDOT traffic surveys.

Twenty-four hour traffic counts taken during 1972-73 at a site near the Lehigh Canal Bridge were obtained from PennDOT. A comparison showed that the twenty-four hour traffic surveys resulted in more two-axle trucks and five-axle tractor semi-trailers than were observed at the bridge site during the short sample periods. Overall, the percentages of various types of trucks from the twenty-four hour records were consistent with the observations at the bridge site. The PennDOT records were also consistent with results from loadometer surveys throughout the state and the nation.

The results of a 1972 PennDOT loadometer survey for twenty stations on main arteries in Pennsylvania are summarized in Fig. 16. The frequencies of occurrences of trucks by weight are plotted in the figure. There are two peaks in the histogram, indicating large numbers of loaded trucks at 60 to 75 kips, and large numbers of two-axle or unloaded tractor semi-trailers at 24 to 36 kips.
Figure 17 summarizes the results of a 1970 FHWA Nationwide Loadometer survey. The relative distribution of truck frequencies from this survey are comparable to the distribution shown in Fig. 16. There are two peaks at approximately 25 kips and 70 kips, similar to the twenty station survey from Pennsylvania.

Since the results of traffic counts at the bridge were consistent with the results of the twenty station main artery survey, which in turn were comparable to results from other surveys, the results of traffic counts at the bridge were considered representative of the normal traffic crossing the Lehigh Canal Bridge.

The total number of trucks that have traveled over the bridge during the nineteen year period (1953-1972) was estimated from the PennDOT traffic count data. The estimated average daily truck traffic counts (ADTT) from the PennDOT data are plotted in Fig. 18 on a yearly basis. This ranged from 2038 trucks per day in 1953 to 3709 in 1972. The average rate of increase was about 1.5% prior to 1960 and 4.5% 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.
6. CORRELATION WITH FATIGUE TEST RESULTS

6.1 Fatigue Test Data

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. If all other conditions are the same, an increased attachment length results in a shorter fatigue life. Results of the laboratory studies on beams with short attachments are plotted as a function of stress range and cycle life in Fig. 19. The dotted lines represent the 95% confidence limits for the test specimens. The specimens used for determining the stress range-cycle life relationship were all subjected to constant stress range cycles. These test data correspond to Category D in the 1974 AASHTO Specifications and were used to evaluate the tie plate performance because several pilot bending tests of tie plates provided comparable test results.

The riveted tie plates of the Lehigh Canal Bridge were subjected to horizontal in-plane bending under random applied loads. Most of the tie plates have tack welds one to two inches long connecting the edges of the tie plates to the bracket top flange as was shown in Fig. 5a. Load transfer between the flange of the outrigger bracket and the tie plate was provided by both the rivets and the tack welds. Since the tack welds were at the end of the joint which is the most highly stressed region, 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. The initial discontinuity at the tack weld toe was probably greater than discontinuities that commonly exist at weld terminations and may be the reason for this low fatigue strength. When comparing laboratory fatigue test results with the 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.

6.2 Root-Mean-Square Estimates

One procedure to account for the stress range spectrum is by the root-mean-square (RMS) method\textsuperscript{3,12,13}. In this method the root-mean-square 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 stress ranges. The root-mean-square stress range is defined as

\[ S_{\text{RMS}} = \left( \sum \alpha_i S_{r_i}^2 \right)^{1/2} \]  

(1)

where \( \alpha_i \) is the frequency of occurrence of stress range \( S_{r_i} \).

The stress range occurrences of the gages on the tie plates were used to determine the RMS values of stress ranges. These values were then adjusted by the distance from the gages to the plate edges where the crack growth originated.
The $S_{RMS}$ at the plate edge and the corresponding fatigue cycles corresponding to the truck traffic are plotted in Fig. 19 and are compared with the constant stress range laboratory data. The data from the Lehigh Canal Bridge is seen to lie above or near the lower confidence limit for Category D. Cracks were observed in tie plates C1N, C5S and C6S. The cracked plates are seen to plot on or above the lower confidence limit in Fig. 19. Tie plates C4S and C6N had no visible cracks at the time of measurement. The data points for these two plates are below the lower confidence limit.

The comparison given in Fig. 19 indicates that the RMS stress range provides a reasonable correlation between the field data and the laboratory test results. 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 had $S_{RMS}$ and cycle lives above this limit could be expected to exhibit visible fatigue cracking. Tie plates C4S and C6N can be anticipated to develop fatigue cracks under additional truck traffic.

6.3 Correlation by Miner's Rule

The stresses at gages on tie plates C1N, C4S, C5S, C6N and C6S were adjusted to give the stress range at the edge of the tie plates due to truck traffic. The resulting distribution of stress range at the tie plate edges was used with Miner's cumulative damage hypothesis$^{14}$ to determine the total damage caused by the truck traffic.
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 the tie plates C4S and C6N, which had damage ratios less than unity.

By combining the relationship provided by constant cycle data and Miner’s rule, an equivalent stress range $S_{r_{\text{ Miner}}}^{1/3}$ was estimated as:

$$S_{r_{\text{ Miner}}} = (\sum \alpha_i S_{s_i}^3)^{1/3}$$

(2)

where $\alpha_i$ is the frequency of occurrence of stress range $S_{s_i}$.

The $S_{r_{\text{ Miner}}}$ values of the tie plates are also plotted in Fig. 19. The results also demonstrate that Miner’s Rule provides a reasonable correlation between the observed tie plate behavior and the laboratory results.

The results in Fig. 19 indicate that the Miner’s Hypothesis can be used to correlate field measurements and laboratory test results in terms of stress range and cycle life. The lower confidence limit for Category D of AASHTO Interim Specification 1974 provides a reasonable estimate of the fatigue strength of the tie plates of the Lehigh Canal Bridge.
7. SUMMARY AND CONCLUSIONS

From the pilot field studies and the analyses 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. Seldom did the live load stresses exceed 8 ksi.

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 indicate 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. The crack pattern in the tie plates agreed with the measured strain ranges. In addition it appeared to be compatible with the predicted stresses due to displacement.

5. The measured stress history for short time periods was compatible with the frequency distribution of trucks reported by PennDOT on comparable roads.
6. The root-mean-square stress range, $S_{RMS}'$, and an estimated traffic volume for the period of time from 1953 to 1972 provided good correlation between crack occurrence on several tie plates and the fatigue test results from laboratory studies.

7. Miner's Hypothesis also provided a reasonable estimate of the damage that had occurred in the tie plates.

8. No stress measurements were obtained on completely cracked tie plates. However, the crack pattern was compatible with the expected stress distribution due to the in-plane displacements.

A more detailed analysis of the stress development mechanism in the tie plates of outrigger bracket connections is being developed. This should permit a better evaluation of the cause of the high stresses and their possible occurrence in other structures. 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 results 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 in order 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.
8. ACKNOWLEDGMENTS

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Fig. 1 Lehigh Canal Bridge
(Instrumentation Installed in Left Side Span)
Fig. 2 Plan and Elevation of the Test Span
Fig. 3a Cross-Section of Bridge

Fig. 3b Tie Plate Detail at Floor Beam Bracket
Connection to Girder
Fig. 4 Schematic of Cracked Tie Plates in the Test Span
Fig. 5a  Crack in Tie Plate Originating at Tack Weld
Fig. 5b Schematic of Instrumentation on Tie Plates and Girder
Fig. 6 Typical Strain History Response in Tie Plates, Stringer and Girder
Fig. 7 Comparative Response of Tie Plate C4S and South Girder for Various Loads
Fig. 8 Instantaneous Stress Distribution in Tie Plate C6N
Fig. 9 Instantaneous Stress Distribution in Tie Plate C6S
Fig. 10 Measured Stress Distributions in the Gaged Tie Plates During Passage of Truck
Fig. 11 Strain History with Truck in the Passing Lane
Fig. 12 Comparison of Measured Strain History in Tie Plates With Influence Lines for Girder Slope
Gage C4S-5
59.67% 1.5-3.3 ksi
40.33% 3.3->9.6 ksi

Fig.13 Stress Histogram for Tie Plate C4S
Gage C6N - 9
78.65 % 0.9-3.0 ksi
21.35 % 3.0->13.5 ksi

Fig. 14 Stress Histogram for Tie Plate C6N
Fig. 15 Stress Histogram for North Girder
Fig. 16 1972 PennDOT Loadometer Survey Gross Vehicle Weight Distribution
Fig. 17 1970 FHWA Nationwide Gross Vehicle Weight Distribution
Fig. 18 Estimated ADTT at the Lehigh Canal Bridge Site
Fig. 19 Comparison of Estimated Equivalent Stress Range With Laboratory Test Data
Fig. A.1 Cracks in Tie Plates, Lehigh River Bridges (PennDOT)
Fig. A.1 Cracks in Tie Plates, Lehigh River Bridges (PennDOT) - continued -41-
Fig. A.2 Cracks in Tie Plates, Lehigh Canal Bridges (PennDOT)
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* Number of occurrences when stress range equaled or exceeded the stress range for level 1
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