High Cycle Fatigue of Welded Bridge Details

STRESS HISTORY OF A CURVED BOX BRIDGE

by

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1. INTRODUCTION

A stress history investigation determines two significant variables used in evaluating the fatigue life of a structural detail, the stress range and its frequency of occurrence. These variables have been incorporated in the strength and fatigue crack growth analyses of structural details in a number of bridges (1,2,3,4). The recorded stress range variations under vehicular loads on the bridge and the corresponding frequencies of occurrence were converted to root-mean-square (RMS) values (5), or were coupled with Miner's Hypothesis (6), to give equivalent constant amplitude stress ranges. These values were then compared with laboratory results on the fatigue strength of the details (7).

Stress history data are scarce for structural details of curved bridges. In an effort to obtain some field data, measurement of strain variations were taken at a number of details on a curved, composite box girder bridge under vehicular loads. The bridge is part of an approach ramp to the Fort Duquesne Bridge (I279) in Pittsburgh, Pennsylvania.

The field study also provided an opportunity to inspect the bridge details. Two details were of primary interest: the termination of transverse stiffeners at intermediate diaphragms and the discontinuous backup bars at flange-to-web joints. The results of analyzing these details are summarized in the report, together with the results of stress history studies.
2. DESCRIPTION OF BRIDGE AND TESTING PROCEDURE

2.1 The Bridge

The bridge under investigation is an approach ramp north of the Fort Duquesne Bridge in Pittsburgh, Pennsylvania. It carries southbound traffic onto the Fort Duquesne Bridge over the Allegheny River (Fig. 1). The bridge is anticipated to be well traveled by all types of vehicles including heavily loaded trucks from the northern portions of Pittsburgh.

The approach ramp consists of a number of three-span, continuous, curved bridges. The portion under investigation has two concentric rectangular steel box girders with a composite reinforced concrete deck (Fig. 2). Each span has a centerline arc length of 30.48 m (100 ft.). The radius of curvature for each span is constant but varies between spans, being 260 m (853 ft.), 263 m (863 ft.), and 260 m (853 ft.), respectively. The bridge is supported on radial steel bents. The test span was the middle span, between Bents SB11 and SB12.

The cross-sectional dimensions of the box girders vary along the length of each span. A cross section at an intermediate diaphragm is shown in Fig. 3. Intermediate diaphragms are spaced at 3.048 m (10 ft.) intervals. The box girders contain typical transverse and longitudinal stiffeners.
2.2 Instrumentation

Forty-eight electrical resistance strain gages of foil type were mounted at various details and at a few nominal cross sections of the box girders (see Figs. 4a and 4b). The details under investigation included the transverse and longitudinal stiffeners and the backup bars at longitudinal butt welds.

Out-of-plane forces exerted from diaphragm members onto the transverse stiffeners may cause large plate-bending stresses in the web between the end of the stiffeners and the flange. Strain gages were placed around this area. The bottom flange longitudinal stiffeners in the test span extend throughout the bridge with some terminating in the positive moment region, 9.1 m (30 ft.) from the interior support bents. Strain gages were applied on the bottom flange at the end of a number of these stiffeners.

The backup bars are small rectangular steel bars tack-welded at the bottom flange-to-web connections along the length of the box girder. The bars were placed inside the box girder to aid fabrication and were left in place in the bridge, thus becoming an integral part of the bridge box girder. The small gap between two adjoining steel bars constitutes a discontinuity for stress flow hence is a structural detail potentially weak in fatigue strength. Strains in the flange at some of these gaps were monitored.

Cross sections where strain gages were installed for stress evaluation were at one end, at a quarter point, and at the middle of the test span (Figs. 4a and 4b). All strain gages were connected to analog
trace recorders which depict strain variations on oscillographs. Some tracings of oscillographs are shown in Fig. 5 as examples. From these graphs, stress ranges and frequencies could be determined.

2.3 Field Testing

The field testing of the bridge included a control load test and the stress history acquisition. During the control load test, the Federal Highway Administration's test truck traveled across the test span in each of the three lanes (Fig. 2) at two different speeds (crawl runs and speed runs). The test truck simulated the standard HS20 loading (Fig. 6). During the test truck runs, the bridge was cleared of all traffic. Table 1 summarizes the test truck run conditions.

The test truck crawl runs (8 km/h) (5 mph) simulated a static load condition, load without impact. Data obtained from this portion of the test were used for comparing with computed results. Data from the speed runs (69-80 km/h) (43-50 mph) were used in the comparison of static and dynamic responses of the bridge.

The stress histories of the strain-gaged details were compiled over five continuous days of measurement, giving a representative sample of stress variations due to different vehicles. When trucks, semi-trailers, and buses capable of producing stress fluctuations were approaching, the trace recorders were started simultaneously. The resulting oscillographs were labeled with the vehicle type according to the standard FHWA truck classification (Fig. 7). Each vehicle produced a set of traces which were to be measured for stress ranges. A stress range was the difference between the maximum and minimum stress for each trace.
3. TEST RESULTS

3.1 Controlled Load Test

The two groups of controlled load tests conducted on the bridge were the crawl run (static) and the speed run (dynamic).

Oscillograph traces for the crawl runs were measured for strains as a function of the truck's position on the bridge. The strains can be converted to stresses resulting in a plot analogous to the influence line for a given gage location. An example is given in Fig. 8 comparing the oscillograph trace of a longitudinal flange gage to the influence line of bottom flange stress at midspan of a three-span continuous beam.

Stresses measured at a given truck location can be used to determine the accuracy of the stresses computed through the finite element analysis of the bridge. A finite element analysis of the structure was made using the program SAP IV(8). In simplifying the problem, the bridge was assumed prismatic in cross-sectional dimensions with nominal wall thicknesses described by the design drawings. Symmetry about midspan was used to reduce the model size. The concentric boxes were assumed to act independent of each other when the load was applied over one box. The inner box was chosen for discretization. The model was discretized into plate bending elements for the flanges and webs, beam elements for the stiffeners and truss elements for the diaphragms. Stress at various details could be determined by substructuring.
A comparison of web stresses in the longitudinal direction was made between the measurements from oscillograph traces and the values predicted from the finite element model, Fig. 9. For the model, two concentrated loads placed over the webs at midspan simulated the test truck. The total load was equal to the test truck gross weight of 298 kN (67 kips) distributed to the webs for a lane 1 loading. The results in Fig. 9 indicate the model is adequate for the evaluation of stress distribution pattern in single box girders. The measured stress values were, however, lower than the computed values due to the assumption of concentrated total truck loads versus actual truck load distribution in the longitudinal and radial directions, as well as in the participation of the other box.

The speed runs were used in the determination of dynamic effects on the stresses in the structure. The oscillograph trace from a static (crawl) run and that from a dynamic (speed) run are compared in Fig. 10 for a point on the bridge. The time scale is different for the two runs. It can be seen that the curves are identical in overall shape. The speed run contained dynamic strains which were superimposed on the static strain curve. The dynamic strains had a higher frequency than the static strain but were smaller in magnitude. The difference between the dynamic and static curves reveals the dynamic effects of the truck on the stress and strain at the point of the gage and is an indication of the impact factor for the bridge.

The lateral position of a truck influences the stress at a point of a multi-lane bridge. Three oscillograph traces for a point
on the bottom flange of the inner box girder are compared in Fig. 11, each for a different lateral position of the test truck over the box girder (Lane 1). The stress magnitudes decreased when the vehicle was further away from the gage (Lanes 2 and 3). When the test truck was in Lane 3, the stress magnitude remained one-fourth of the maximum Lane 1 stress. This is an indication of the load distribution characteristic of the box girder bridge, and implies that trucks in all lanes of the bridge must be considered in evaluating the stress history of bridge details.

3.2 Stress History

Oscillograph recordings were made for all large vehicles crossing the test span during a five day period. Small vehicles such as cars and pickup trucks produced negligible stress ranges in the bridge and were not recorded. Details which sustained only very low stresses during the control load test were excluded from further measurements. Each of the significant oscillograph traces were measured for stress range under the assumption of one stress range per vehicle passage (2). The stress ranges were grouped according to magnitude and were plotted as stress range histograms. The histogram for the bottom flange at a longitudinal stiffener cutoff is shown in Fig. 12. Histograms were prepared for each gage and are presented in the Appendix.

From reviewing the stress range histograms, it can be found that the frequency of higher stress ranges was relatively low for most
of the bridge details. One of the factors contributing to this condition was the distribution of traffic volume. During the five days of stress history recording, a total of 1424 trucks traveled on this section of the bridge. Lane 1 is at the end of an entrance lane with a small divider separating it from the other two lanes, thus only carrying oncoming trucks. About one-third (34.3 percent) of the total truck traffic came on the bridge from here. Lanes 2 and 3 contained the major flow, with 52.5 percent in the curb lane and 13.2 percent in the passing lane, Lane 3. Since Lane 2 is between the two box girders, loads in this lane were distributed to both box girders resulting in lower stress magnitudes. Hence only a small percent of the traffic generated large strain ranges in the box girders. Further discussion on the use of stress history will be made later in Chapter 6.
4. TERMINATION OF TRANSVERSE STIFFENER DETAIL

One of the structural details which incurred relatively high stress ranges was the termination of transverse stiffeners. The largest measured stresses occurred in the vertical direction where the transverse stiffeners were part of an intermediate diaphragm in the box girders. The vertical plate bending stress near the gap between the flange and the end of the stiffener had a maximum value of 32 MN/m² (4.6 ksi).

Both AISC(9) and AASHTO(10) design specifications permit the termination of transverse stiffeners short of the tension flange at all intermediate locations. When a transverse stiffener is part of an intermediate diaphragm (Fig. 3), forces may be introduced to the stiffener, moving it out of the plane of the web. Stiffeners not connected to the flange will displace relative to the flange. This relative displacement concentrates in the area between the stiffener and flange, producing large vertical bending stresses on the web surfaces.

All intermediate transverse stiffeners in the Fort Duquesne approach spans are terminated short of the tension flange. Some of these stiffeners serve as a part of the intermediate diaphragms. Strain gages at one of these stiffener ends recorded the relatively high stresses as reported above.
Stresses at the transverse stiffener gap were computed from the substructuring of the overall finite element model (Fig. 13). The finite element model was a replica of the detail in the test span. This substructure contained node points which corresponded to nodes on the overall bridge analysis model. Nodal displacements and rotations from the overall analysis with a simulated HS20 loading were used as input on the substructure. Results from the substructure are given in Table 2 with a comparison to the field test results. A good correlation exists between the computed and measured stresses at the gage points. From this outcome, it can be assumed that the substructure model accurately describes the out-of-plane bending behavior of the actual structural detail. The maximum computed vertical bending stresses occur at the ends of the stiffener gap and are much higher than that at the strain gage.

To explore further the magnitudes of the out-of-plane vertical bending stresses at the transverse stiffener gap, a parameter study was made using a finite element model similar to that used for the box girder detail substructuring. The variables that were studied were the thickness of the flange, the thickness of the web, and the stiffener to flange gap length. Model boundary conditions were assumed hinged, providing least restraint to the model. A concentrated horizontal load was placed on the stiffener to simulate the effect produced by a diaphragm. The stresses then obtained would be an upper bound value due to the under-estimation of actual restraint at model boundaries and the disregard of stress relaxation of the diaphragm.
The results of a 9.5 mm (3/8 in.) web subjected to a horizontal load of 4.45 kN (1 kip) are shown in Fig. 14. The out-of-plane vertical bending stresses increase with the length of the gap until it reaches a maximum. For a given gap length, the maximum stress occurs at the top or the bottom of the gap, depending on the thickness of the flange.

For an identical model subjected to a given lateral displacement (instead of a given load) at the transverse stiffener, the stresses at the gap decrease with increasing gap length, as is shown in Fig. 15. In other words, a larger gap would be preferable to a smaller one with regard to lateral displacement.

In box girder design and analysis, whether force or displacement is computed is a matter of analytical procedure. By using force as the controlling unit, various combinations of the parameters were investigated in this study. The results show a linear relationship between the logarithm of vertical bending stresses and the logarithm of web thickness for any ratio of gap length to web thickness. This relationship is depicted in Figs. 16 to 23. Most of the lines are parallel.

In algebraic form, the relationship is

\[ \frac{\sigma_g}{P} = C t_w^{-k} \]  

(1)

where

- \( \sigma_g \) = vertical bending stress at top or bottom of gap
- \( P \) = diaphragm force
C = constant for a given flange-to-web
   thickness ratio
k = exponential constant for top or bottom
   of gap

This equation is valid for conventional transverse stiffeners which
are cut short from the flange and are a part of an intermediate dia-
phragm in a rectangular box girder.

Figures 16 to 23 indicate large stresses will be produced in
thin webs if lateral force is introduced at a transverse stiffener gap.
Large stresses due to live loads may cause fatigue cracks to occur at
the gap.

For given box girder component dimensions, the vertical bending
may be reduced by decreasing the gap length. A positive solution to the
problem would be the elimination of the relative displacement between
the stiffener and the flange. This could be achieved by welding the
transverse stiffener to the flange.

In the Fort Duquesne Bridge, the horizontal members of the
intermediate diaphragms were attached to the longitudinal stiffeners
of the bottom flange as shown in Fig. 3. This provided sufficient
restraint so that the force transmitted into the transverse stiffener
was minimized.
5. DISCONTINUOUS BACKUP BAR

The backup bar is a fabrication aid used to facilitate the welding of two plates. It is usually tack welded to the back of the joint to contain molten weld metal. The backup bars in the test span were used at the bottom flange to web plate joints (Fig. 24). Each of the bars is rectangular in cross section 1/4" x 3/4" (6 x 19 mm). They were tack welded to the inside of the joint and the groove weld was made from the outside. The backup bars run the full length of the three span box girders.

The AWS Structural Welding Code\(^{(11)}\) states that all backing strips must be made continuous. In practice, sometimes backup bars are placed butted against each other without welding. This procedure results in a continuous strip to contain the molten weld metal but discontinuities remain in the backup bar itself. When the discontinuities are oriented perpendicular to the stress field, consideration must be given to fatigue crack propagation.

The backup bar discontinuities in the test span are oriented perpendicular to the longitudinal bending stress field. By compatibility, the backup bar experiences the same high magnitude of strain and stress as the web-to-flange joint.

To examine the fatigue strength of the discontinuous backup bar detail, subcritical crack growth is considered. The crack growth
equation (12) is rearranged as

\[ dN = \frac{da}{C \Delta k^n} \]  \hspace{1cm} (2)

where

- \( a \) = crack size
- \( N \) = life, in cycles
- \( C \) and \( n \) = material constants
- \( \Delta k \) = change in stress intensity factor

The change in stress intensity factor may be computed using the expression (13)

\[ \Delta k = F_e F_w F_s g \Delta \sigma \sqrt{\pi a} \]  \hspace{1cm} (3)

where

- \( F_e = \frac{2}{\pi} \), for penny-shaped cracks
- \( F_w = \sqrt{\sec(\pi a/2t)} \), for finite width plate \( (14) \)
- \( F_s = 1.12 \), for surface crack \( (14) \)
- \( F_g \) = correction factor for stress concentration at the detail, a function of the crack size
- \( \Delta \sigma \) = applied normal stress range at the detail

A finite element analysis of the discontinuous backup bar detail has been initiated to determine the stress concentration \( (15) \). From the preliminary results, a stress concentration decay curve was determined in terms of the crack size "a", as shown in Fig. 25. A polynomial equation for the stress concentration correction factor was derived from this decay curve \( (13) \).
By using the values of the correction factors $F_e$, $F_w$, $F_s$, and $F_g$, Eq. 2 is then in terms of $N$, $\Delta\sigma$, and $a$. Integration from the initial flaw size to the tolerable flaw size results in an estimate of the fatigue life of the detail. The initial flaw size was assumed 0.762 mm (0.03 in.). The tolerable crack size was taken as the effective thickness of the material (Fig. 24). The result of the integration is a relation between stress range and life as given in Fig. 26. The relationship is very close to the AASHTO fatigue category E\(^{(15)}\) detail. This implies that care must be taken to control the stress at the backup bar discontinuity or alternately, such discontinuities must be avoided.

A refined analysis is being conducted to assess more accurately the fatigue strength of the discontinuous backup bar detail of Fig. 4\(^{(15)}\). There are no known available experimental data on the fatigue strength of this detail. Results from testing butt weld on plates with backup bar strips\(^{(16)}\) indicated a comparable fatigue strength.
6. CUMULATIVE DAMAGE

Magnitudes of stress ranges were low at most of the gage locations on the Fort Duquesne Bridge approach test span. The maximum recorded stress range of most gages, as shown in the stress range histograms (Appendix A) are far below the corresponding permissible stress range of AASHTO(10). Therefore, it is not likely that any fatigue failure will occur in this test span.

The highest stress range value from measurements was from the vertical bending gage near the transverse stiffener cutoff. Its value was 32 MN/m^2 (4.6 ksi). The stress in the gap is likely higher. The results of the finite element analysis indicate that the stress in the gap would be 3.5 times greater than the stress at the vertical gage location. Caution must be exercised when using the measured results.

Because of the uncertainty in the long term, high cycle fatigue behavior of structure details, the cumulative effects of stress ranges at a number of details were examined, ignoring the "thresholds" of the fatigue strength categories. The stress range histograms of these gages were converted to root-mean-square (RMS) stress ranges (5) and are listed in Table 3. These equivalent constant amplitude stress ranges intercept the extended fatigue strength lines at very high number of cycles (> 10^8), indicating long life. If Miner's hypothesis (6) is used with the fatigue strength lines, similar results are obtained. Figure 27 summarizes some of these comparisons.
The number of trucks monitored during the test period of 84 hours was 1424, giving an average daily truck traffic (ADTT) of 407. By assuming a high annual increase rate of 3 percent, the total truck volume in 40 years is 11.54 million. This is far lower than the number of cycles which might cause fatigue damage as predicted from Fig. 27.

If the RMS gross vehicle weight of trucks increases 30 percent in the future due to reasons such as the development of industrial plants or increase of traffic directly over the box girders, the RMS stress ranges or the equivalent stress ranges incorporating the Miner's hypothesis can be conservatively increased by 30 percent. The resulting life of possible fatigue damage is still very high, and is higher than the projected total traffic volume (Fig. 27). Cumulative damage due to fatigue thus does not appear to be a matter of concern for the test span and spans under similar conditions in the Fort Duquesne Bridge approaches.
7. CONCLUSIONS AND RECOMMENDATIONS

The field testing and subsequent data reduction and analysis of the Fort Duquesne approach spans have resulted in the following conclusions.

1. The strain variation at a point of the curved box girder under a vehicular load was analogous to the stress influence line of that point (Fig. 8).

2. Limited measurements of primary stresses in cross sections of the box girder due to live load and impact were in agreement with, but lower than the corresponding values which were computed using simplified analytical model and loading (Fig. 9).

3. The lateral (lane) position of vehicles was observed to influence the stress magnitude at bridge details. For these two box spans, trucks directly over one box generated proportionally significant stresses in the other box (Fig. 11).

4. The maximum stress range at all details of this study were relatively low and infrequent (Figs. 12 and A1 to All). The highest measured stress range was 32 MN/m$^2$ (4.6 ksi) in the vertical direction, near the end of a transverse stiffener which was a part of an intermediate diaphragm.

5. Computed bending stresses were high in the vertical direction at the gap between the end of an intermediate diaphragm
connection plate (vertical stiffener), and the bottom flange, confirming the measured values (Table 2).

6. Results from an analytical study using a finite element model indicated that, for a given geometry and lateral force from the diaphragm, the vertical bending stresses increase with the gap opening (Fig. 14). For a given geometry and lateral displacement at the gap, the stresses decrease with the gap size (Fig. 15).

7. A family of log-log straight lines were generated from a parametric study to correlate stresses, gap size, and girder component geometry at the end of transverse stiffeners of intermediate diaphragms. Most of the lines are parallel (Figs. 16 to 23).

8. Backup bars butting against each other without welding might constitute initial discontinuities with respect to stresses and fatigue crack growth. Preliminary analysis indicate that the fatigue strength of such structural details could be less than AASHTO fatigue category E (Fig. 26).

9. It does not appear that fatigue cracks will occur in the test span. The equivalent constant amplitude root-mean-square stress ranges, or those by the Miner's hypothesis, were fairly low, such that the anticipated fatigue life is much higher than the projected stress cycles (Fig. 27).

From the above, it is recommended that:

10. Transverse stiffeners which serve as a part of a diaphragm should be arranged to reduce local stresses at the end. For new bridges, connecting such stiffeners to the flange is advised. For existing
bridges with this structural detail, evaluations may be made with the aid of Figs. 16 to 23.

11. Backup bars should be made continuous as specified by AWS, and treated as structural components of the bridge girder.

12. Further studies need to be undertaken to evaluate the fatigue strength characteristics of these two details. Experimental results to define their fatigue strength categories are essential.
ACKNOWLEDGMENTS

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The help of Mrs. Dorothy Fielding in typing and processing this report is appreciated.
### TABLE 1 SUMMARY OF CONTROL TRUCK RUNS

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### TABLE 2 STRESSES IN TRANSVERSE STIFFENER GAP

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<td>- 43 (-6.2)</td>
<td>- 32 (-4.6)</td>
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<td>Horizontal Gage*</td>
<td>9.3 (1.3)</td>
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<td>Maximum Gap Stress*</td>
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*See Fig. 4 for orientation
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**Legend:**
- **IB** Inner box
- **OB** Outer box
- **END** End of span
- **1/4** Quarter span
- **MID** Midspan
- **BF** Bottom flange
- **BB** Backup bar
- **LS** Longitudinal Stiffener
- **TS** Transverse stiffener
- **H** Horizontal gage
- **V** Vertical gage
Fig. 1 Test Site
Fig. 2 Test Span
Fig. 3 Typical Cross Section Showing
One Box and Details
Fig. 4a Strain Gage Locations

1 BENDING STRESS
2 BACKUP BAR
3 LONGITUDINAL STIFFENER
Fig. 4b. Strain Gage Locations

① TRANSVERSE STIFFENER HORIZONTAL GAGE
② TRANSVERSE STIFFENER VERTICAL GAGE
Fig. 5 Oscillograph Traces
Fig. 6 Comparison of Control Truck to HS20-44
Fig. 7  FHWA Truck Classification
Fig. 8 Comparison of Influence Line and Static Run
Oscillograph Trace
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Fig. 9 Comparison of Finite Element Model Results to Measured Test Results
Fig. 10 Comparison of Static and Dynamic Oscillograph Traces
Fig. 11 Effect of Lateral Position of Load on Bottom Flange Stress

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Fig. 12 Histogram for Longitudinal Stiffener Cut-off
Fig. 13 Transverse Stiffener Detail Substructure
Fig. 14 Results of Parameter Study with Constant Diaphragm Force = 4.4 kN, $t_w = 9.5$ mm
Fig. 15 Results of Parameter Study with Constant Relative Displacement = $2.5 \times 10^{-2}$ mm, $t_w = 9.5$ mm

$\sigma_{BOTTOM}$ $t_f = 9.5$ mm

$\sigma_{TOP}$ $t_f = 38$ mm

$\sigma_{TOP}$ $t_f = 9.5$ mm

$t_w = 9.5$ mm

($3/8$ IN.)

$\Delta = 0.025$ mm

($0.001$ IN.)
Fig. 16 Maximum Stiffener-Flange Gap Stress, Bottom of Gap,
\[ \frac{t_f}{t_w} = 1.0 \]
Fig. 17 Maximum Stiffener-Flange Gap Stress, Top of Gap, 
\( \frac{t_f}{t_w} = 1.0 \)
Fig. 18 Maximum Stiffener-Flange Gap Stress, Bottom Gap,
\( \frac{g}{t_w} = 1.5 \)
Fig. 19 Maximum Stiffener-Flange Gap Stress, Top of Gap,
\[
\frac{g}{t_w} = 1.5
\]

\[\frac{t_f}{t_w} = 1.5\]
Fig. 20  Maximum Stiffener-Flange Gap Stress, Bottom of Gap,
\[
t_f/\frac{t}{t_w} = 2.0
\]
Fig. 21  Maximum Stiffener-Flange Gap Stress, Top of Gap,
\[ \frac{g}{t_w} = 2.0 \]
Fig. 22 Maximum Stiffener-Flange Gap Stress, Bottom of Gap,
\( \frac{g}{t_w} \), where \( t_f/t_w = 2.5 \)
FIG. 23 Maximum Stiffener-Flange Gap Stress, Top of Gap,
\( \frac{g}{t_w} = 2.5 \)

\( t_f/t_w = 2.5 \)
Fig. 24 Discontinuous Backup Bar Detail
Fig. 25 Stress Concentration Decay Curve
Fig. 26 Estimate of Fatigue Life of Discontinuous
Backup Bar Detail

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Fig. 27  S-N Curve Showing Estimated Fatigue Life of Details

Primed Gage Numbers Account for Future Load Increase of 30%
APPENDIX

STRESS RANGE HISTOGRAMS AT VARIOUS GAGE LOCATIONS
Fig. A.1 Histogram for Gage 5
Fig. A.2 Histogram for Gage 8
Fig. A.3 Histogram for Gage 12

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Fig. A.4 Histogram for Gage 13
Fig. A.5 Histogram for Gage 19
Fig. A.6 Histogram for Gage 21
Fig. A.7 Histogram for Gage 26
Fig. A.8 Histogram for Gage 27
Fig. A.9 Histogram for Gage 31

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Fig. A.10 Histogram for Gage 46
Fig. A.11 Histogram for Gage 50

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REFERENCES

1. Bowers, D. G.

2. Fisher, J. W., Yen, B. T. and Daniels, J. H.
FATIGUE DAMAGE IN THE LEHIGH CANAL BRIDGE FROM DISPLACEMENT-INDUCED SECONDARY STRESSES, Transportation Research Record 607, Transportation Research Board, 1976.

STUDIES ON BRIDGES WITH CANTILEVER BRACKETS, Fritz Engineering Laboratory Report No. 386.3, Lehigh University, April 1977.

4. Daniels, J. H., Yen, B. T. and Fisher, J. W.
STRESSES IN ORTHOTROPIC DECK OF RIO-NITEROI BRIDGE UNDER TRAFFIC, Transportation Research Record 607, Transportation Research Board, 1976.

5. Swanson, S. R.

6. Miner, M. A.

7. Fisher, J. W.
BRIDGE FATIGUE GUIDE, DESIGN AND DETAIL, American Institute of Steel Construction, New York, N. Y., 1977

8. Bathe, K., Wilson, E. L., Peterson, F. E.

9. American Institute of Steel Construction
MANUAL OF STEEL CONSTRUCTION, AISC, New York, N. Y., 1973
REFERENCES (continued)

10. American Association of State High and Transportation Officials,  
STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, AASHTO,  

11. American Welding Society  

FATIGUE - AN INTERDISCIPLINARY APPROACH,  
Proceedings 10th Sagamore Conference, page 107, Syracuse  

13. Zettlemoyer, N.  
STRESS CONCENTRATION AND FATIGUE OF WELDED DETAILS,  
Ph.D. Dissertation, Department of Civil Engineering,  
Lehigh University, October 1976.

14. Hertzberg, R. W.  
DEFORMATION AND FRACTURE MECHANICS OF ENGINEERING MATERIALS,  

15. Batcheler, R. P.  
FATIGUE STRENGTH OF DISCONTINUOUS BACKUP BAR DETAILS,  
M.S. Thesis, Department of Civil Engineering, Lehigh  
University, October 1977.

16. Gurney, T. R.  
FATIGUE TESTS ON AUTOMATIC LONGITUDINAL BUTT WELDS,  