10. W. Black, D.S. Moss, and M. Emerson. Bridge Temperatures Derived from Measurements of Movement. TRRL Report 748. Transport and Road Research Laboratory, Department of the Environment, Crowthorne, Berkshire, England, 1976.
11. M.J.N. Priestley. Effects of Transverse Temperature Gradients on Bridges. Report 394. Ministry of Works, Wellington, New Zealand, Sept. 1972.
12. M.J.N. Priestley. Temperature Gradients in Bridges--Some Design Considerations. New Zealand Engineering, Vol. 27, Part 7, July 1972, pp. 228-233.
13. D.R.H. Maher. The Effects of Differential Temperature on Continuous Concrete Bridges. Civil Engineering Transactions, Institute of Engineers of Australia, Vol. CE12, Part 1, April 1970, pp. 29-32.
14. W.I.J. Price. Discussion of Papers Entitled Medway Bridge: Design, by O.A. Kerensky and G. Little; and Medway Bridge: Construction, by M.F. Hansen and J.A. Dunster. Proc., Institution of Civil Engineers, Vol. 31, June 1965, pp. 162-166.
15. M.J.N. Priestley. Linear Heat-Flow and Thermal Stress Analysis of Concrete Bridge Decks. Res. Report 76/3. Department of Engineering, University of Canterbury, Christchurch, New Zealand, Feb. 1976.
16. M.J.N. Priestley. Design Thermal Gradients for Concrete Bridges. New Zealand Engineering, Vol. 31, Part 9, Sept. 1976, pp. 213-219.
17. M. Emerson. Temperature Differences in Bridges: Basis of Design Requirements. TRRL Report 765. Transport and Road Research Laboratory, Department of the Environment, Crowthorne, Berkshire, England, 1977.
18. Steel, Concrete, and Composite Bridges--Part I: General Statement. British Standard BS 5400. British Standards Institution, Crowthorne, Berkshire, England, 1978.
19. M. Radolli and R. Green. Thermal Stresses in Concrete Bridge Superstructures Under Summer Conditions. In Transportation Research Record 547, TRB, National Research Council, Washington, D.C., 1975, pp. 23-36.
20. M.J.N. Priestley and I.G. Buckle. Ambient Thermal Response of Concrete Bridges--Bridge Seminar, 1978. Summary Volume 2. Structure Committee, Road Research Unit, National Roads Board, Wellington, New Zealand, 1979.
21. F. Leonhardt, G. Kolbe, and J. Peter. Temperature Differences Endanger Prestressed Concrete Bridges. Brefon-und Stahlbetonau, No. 7, July 1965, pp. 157-163.

Publication of this paper sponsored by Committee on Concrete Bridges.

Fatigue Behavior of Welded Wrought-Iron Bridge Hangers

PETER B. KEATING, JOHN W. FISHER, BEN T. YEN, and WILLIAM J. FRANK

ABSTRACT

The behavior of fatigue crack growth and fatigue strength of welded lap splice wrought-iron hangers in a railroad bridge was studied. The original wrought-iron hangers were lap spliced with steel plates for the purpose of tightening the members. Field inspection revealed cracks in the welded lap splices. Examination of simulated test joints and cracked hanger splices in the laboratory indicated that fatigue cracks would develop from weld deposits at the cut of the wrought-iron bar, propagate into the steel splice plate, and cause failure. Fatigue cracks could also propagate into the wrought-iron bars but would be arrested by the slag (iron silicate) stringers. Breaking of the wrought-iron bar only occurred when the applied stress was quite high in comparison with the yield point. Measured live-load stresses in the actual bridge member were relatively low. Evaluation of traffic and load records indicates that the effective live-load stresses would be well below the fatigue strengths of these spliced joints. No imminent problem of fatigue failure is expected in the hangers.

Before the refinement of the steel-making process, wrought iron was widely used as the principle structural material in bridge construction. It was used in many railroad structures during the period from the late 1800s to the early 1900s. Although the material properties of wrought iron have been well known since the beginning of its use, the welded fatigue behavior has never been adequately quantified. Thus a study on the fatigue behavior of welded wrought-iron splice plate repairs on Norfolk and Western Railway Bridge No. 651 in Hannibal, Missouri, is presented. Welded repairs are known to result in low fatigue strength details with steel components. Because the structural members were wrought iron with steel reinforcement, it was desired to evaluate the seriousness of the resulting welded details and to assess the degree of cumulative damage that may have occurred.

BRIDGE DESCRIPTION

The Norfolk and Western Railway Bridge was originally built for the Wabash Railroad in 1888 by the Detroit Iron and Bridge Works. The bridge spans the Mississippi River with seven truss spans and one continuous swing span for a total length of 1,580 ft (Figure 1). It carries a single track and has a truss spacing of 18 ft. The bridge members are con-

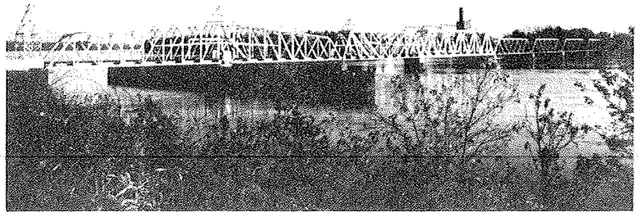

FIGURE 1 View of bridge.

FIGURE 4 Toe crack in transverse weld.

structed of riveted built-up wrought-iron sections and eyebars. Span F, which contains the most severe welded repair detail, is a nine-panel, 176-ft, 4-in.-long simply supported span (Figure 2). All diagonals, bottom chord members, and the U1-M1 and U8-M8 hangers are comprised of eyebar pairs.

Throughout its history, the bridge has undergone an extensive number of repairs and revisions (note that these data are from the records and summarizations of repairs to Bridge 651 of the Norfolk and Western Railroad). This includes span shortening and relocation, stringer replacement, and eyebar tightening. Beginning in 1937, a period during the early development of the welding process, many of the repairs incorporated welding, some to an extreme. Common to old, pin-ended truss bridges, many of the eyebar members loosened with time and required tightening. The corrective repair procedure used on this bridge was to cut the eyebar body and then to weld steel lap shear splices over the cut. A small length was cut out of the eyebar body and the two cut ends were drawn together so as to retension the member. Splice plates were welded on either side, thus overlapping the cut (Figure 3). The steel lap plates were of A7 material.

A continuous fillet weld was placed around the entire perimeter of each lap plate, which resulted

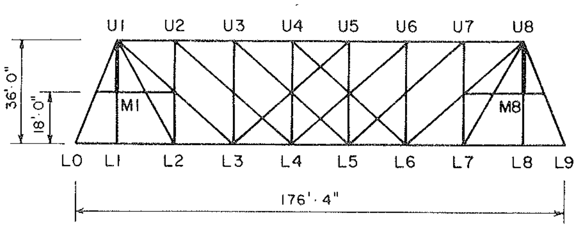

FIGURE 2 Profile of span F.

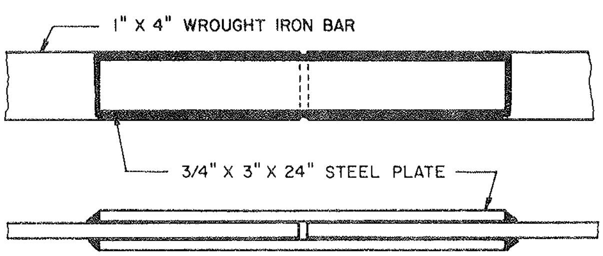

FIGURE 3 Plan of lap splice repair.

in both transverse and longitudinal welds, the latter bridging the exposed gap between the two wrought-iron plate ends. This was done on many of the diagonals, bottom chord members, and on six of the eight eyebar hangers in span F, as well as eyebar members in the other seven spans. For the hangers in span F, the original cross-sectional area of the wrought-iron bar is 4.0 in.^{2}. The addition of the splice plates increases the total area by 12 percent to 4.5 in.^{2}. This type of tightening procedure was a common method of repair before the fatigue strength of weldments became known. The American Railway Engineering Association (AREA) specifications now provide procedures for eyebar tightening by heating and upsetting of the eyebar body.

FIELD INSPECTION

A detailed inspection of the bridge revealed fatigue cracks in the welded lap splices. The most severe cracking was found in the transverse welds in the hangers of span F, where the cracks had coalesced across the toe of the transverse weld. Figure 4 shows the presence of the crack along the entire length of the transverse fillet weld. Also, fatigue cracks were found in the welded gap at the center of the lap splice (Figure 5). Although each hanger is comprised of a pair of eyebars, with a combined cross-sectional area of 8.0 in.^{2}, the inspection revealed that only one eyebar was carrying the load. The other eyebar was found to be slack, even during live loading of the span. This may have occurred from pin wear, pin rotation, or improper tensioning during repair. This condition effectively doubled the stress range level in the active member, which resulted in a more serious situation. A study was undertaken to determine the severity of the cracking and to determine what corrective measures were necessary to prevent failure of the member.

FIELD MEASUREMENTS

Concurrent with the field inspection was the strain gauging and monitoring of one hanger in span F. An electrical resistance strain gauge was mounted on the load-carrying member of hanger U1-M1. Strain readings were taken during passage of several freight trains and of a test train of known weight at various speeds. This allowed for the correlation of the field-measured stresses with the stresses ob-

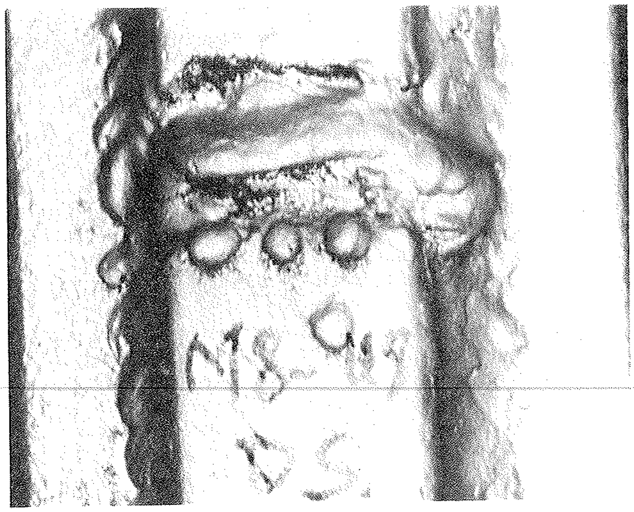

FIGURE 5 Crack in welded gap.

tained from a computer model of the structure. Also, it permitted the effective stress range for the recording period and, subsequently, for the entire life of the bridge to be estimated.

Figure 6 shows the frequency of maximum stress for 10 trains during the recording period, excluding stress ranges less than 7 ksi. The maximum recorded stress range was 16 ksi, which accounted for only 3.5 percent of the total number of cycles. The effective stress range (S_r Miner) was calculated to be 10.6 ksi. Examination of the stress histories indicated that there occurred one load cycle per car or locomotive for the hanger members. Each wheel group (the two trucks from adjoining cars) gave one complete cycle.

PHYSICAL PROPERTIES

Wrought iron is a two-component metal that consists of high purity iron and iron silicate, a particular type of glasslike slag. Originally, the slag content of wrought iron (2.5 percent by weight) was thought to be an undesirable impurity. But it was

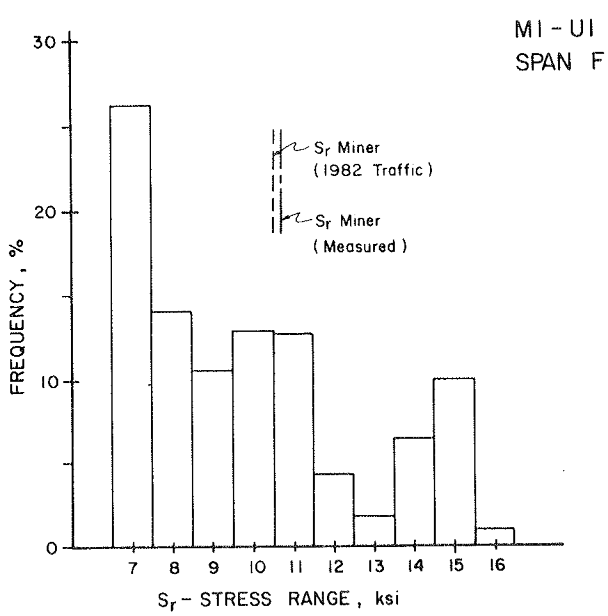

FIGURE 6 Histogram for hanger, span F.

eventually realized that the slag was responsible for the desirable properties of wrought iron--its resistance to corrosion and to fatigue.

Nineteenth century furnace temperatures were not high enough to keep the refined iron from solidifying and trapping some of the molten slag during the final stages of the refining operation. Bessemer's development of steel making was originally intended to produce wrought iron of higher quality and lower cost than what was currently available. But the process resulted in steel that did not contain the slag. It had superior mechanical properties and could be manufactured in greater quantities and at a lower cost.

In wrought iron, slag is distributed throughout the iron matrix, generally in the form of threads of fibers that extend in the direction of rolling. Approximately 250,000 of the siliceous fibers are present in each cross-sectional square inch. Corrosion resistance is attributable to the purity of the iron base, freedom from segregated impurities, and the presence of the slag fibers distributed throughout the metal base. The slag fibers are present in sufficient numbers to form an effective barrier to the process of corrosion, forcing it to spread over the surface of the metal rather than to pit or penetrate (1).

Tensile tests of wrought iron bars taken from the structure gave a minimum yield stress of 26 ksi at 0.002 offset. The tensile strength was measured as 45 ksi. These results agree with ASTM specifications for A42-13 wrought-iron plates.

FATIGUE TESTS

The laboratory fatigue tests of the hanger lap splice were conducted in two parts to accurately assess the severity of the transverse weld cracks. Initially, specimens were fabricated and tested by using pieces of wrought-iron bars salvaged from span E. This span was replaced when the original span was knocked into the river during a barge collision. Later, as a result of the replacement of the cracked hangers, the actual welded joints from span F became available. These were also fatigue tested, examined, and then compared with the fabricated specimens. Figure 7 shows a lap splice repair of span F in the alternating stress testing machine.

Testing of Fabricated Specimens

A total of seven tests were conducted by using three different constant amplitude stress ranges and two variations of the weld orientation. The first three test specimens were run at a stress range of 12 ksi. Although there was noticeable cracking along the toe of the transverse weld, failure of these three specimens did not occur at this location. The first two specimens incorporated the center junction, which was subjected to a stress range of 10 ksi. This resulted in failure of the specimen at this location at 2.1 million cycles for specimen 1 and 3.6 million cycles for specimen 2. The cracks initiated in the longitudinal weld that bridged the gap between the wrought-iron plates and propagated into the steel plates. These failures correspond to the fatigue life for ordinary steel and plot near the category D fatigue strength curve, as can be seen in Figure 8. However, the test sample is so small that assessing the resistance using category E is a reasonable lower bound. Clearly, it was this location that controlled the fatigue life of the lap splice connection.

After failure of the steel plates, both specimens 1 and 2 were regripped to continue testing of the

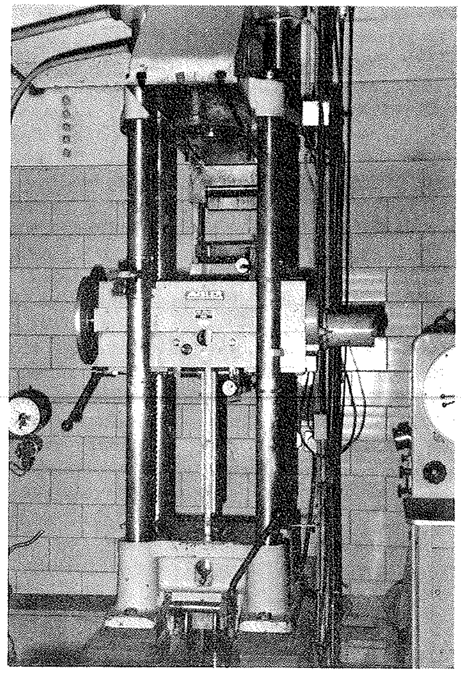

FIGURE 7 Alternating stress test machine.

Because the properties of wrought iron are not isotropic but are directionally dependent, a seventh test specimen was fabricated with the steel plates welded to the edges of the wrought-iron bar. The specimen was tested at a constant stress range of 15 ksi, and the fatigue crack propagated perpendicular to the edges of the elongated stringers. This did not result in crack arrest, and the failure occurred at 0.46 million cycles. This failure life plots within the scatter band of a category E detail and is shown by the cross in Figure 9.

In order to evaluate the fracture toughness of the toe crack, the temperature was lowered on two specimens during testing. Specimens 1 and 4, run at stress ranges of 12 and 18 ksi, respectively, did not fail by fracture with a reduction in temperature to -40°F. The test results indicate that the welded wrought iron had a relatively high resistance to crack instability at reduced temperatures. It was concluded that the toe-cracked eyebars would not fail because of brittle fracture.

Testing of Span F Hangers

Six of the eight hangers in span F that contained the lap splice repair were removed from the structure. The six lap splice joints were cut from the members and shipped to the laboratory for detailed examination. Several were fatigue tested, and the remainder were cut open to reveal the extent of the cracking.

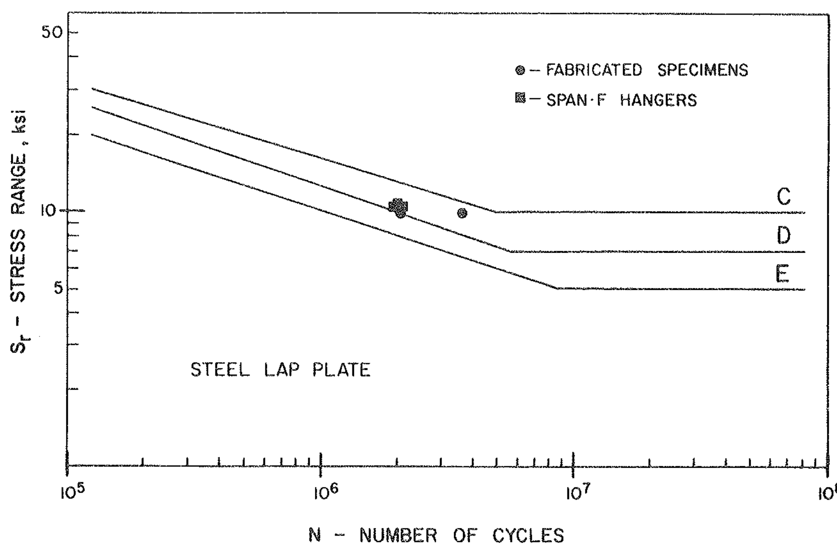

FIGURE 8 Plot of steel lap plate data on S-N curve.

transverse welds that provided a cover plate detail. Both specimens were tested to 20 million cycles without failure at this location, although noticeable cracking at the weld toe and in the weld occurred. Specimen 3, which incorporated only the end detail, was tested to 6 million cycles without failure and then was destructively examined to evaluate the cracking.

Because failure at the toe weld of the wrought-iron plate did not result in failure at 12 ksi, the stress range was increased to 18 ksi for the next three specimens. At the higher stress level failure occurred at the toe of the transverse weld beginning at approximately 0.74 million cycles. This was still well above the fatigue strength expected at a category E detail. All fatigue test results applicable to the wrought-iron members are plotted in Figure 9. Those tests that were stopped before failure are indicated with an arrow.

The actual bridge lap splices were found to be of lower quality with respect to the welding and workmanship when compared to the laboratory-fabricated specimens. These repairs were conducted in 1937, when the welding process had not been adequately developed, and they were made under field conditions. The majority of the transverse welds were found to be undersized for the given plate thickness. The weld sizes ranged from 0.1875 to 0.25 in. for a 0.875-in. plate. Current design specifications require a minimum weld size of 0.3125 in. In addition, the welds exhibited porosity and undercutting of the wrought-iron plate. Many of the weld profiles had a contact angle that greatly exceeded 45 degrees.

One full-sized joint was placed in the alternating stress machine and fatigue tested at a stress

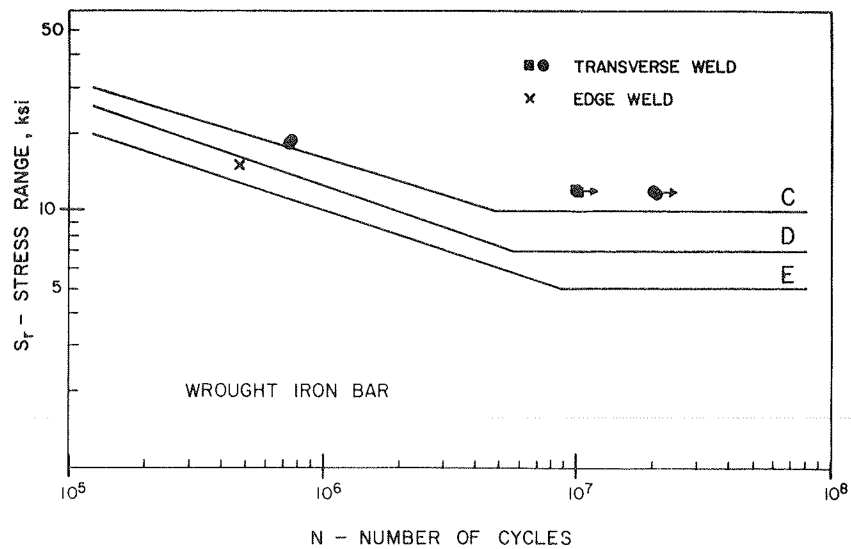

FIGURE 9 Plot of wrought-iron data on S-N curve.

range of 12 ksi in the eyebar. As expected, failure occurred, with a fatigue crack orginating at the center gap, where the stress range was 10.7 ksi, and propagated into the steel plates. Final failure occurred at 1.9 million cycles; the results are plotted in Figure 8. This obviously is a lower-bound estimate of the life, as it did not consider the stress cycles experienced by the detail in the bridge. Each of the two resulting halves of the original joint were then tested separately to evaluate the fatigue resistance of the visible transverse weld toe cracks. Although each piece experienced toe cracks in the wrought iron and root cracking of the welds, the tests were carried to 10.0 million cycles without any evidence of failure or inability to continue to resist the cyclic loads. These tests are also plotted in Figure 9; the arrows indicate that they had not failed. Two additional joints were tested to determine the fatigue strength of the steel splices. These tests are plotted in Figure 8 as solid squares. All three joints provided slightly less fatigue resistance than the fabricated specimens as a result of their prior cyclic loading in the structure.

The remaining lap splice joints were cut open to expose cracks that might have occurred in the welds during service. For most joints, cracks were found on only one side of the joint. This was generally observed at the weld with the greater reinforcement angle. Also, toe cracks were found toward the center portion of the transverse weld, and root cracks were found at the outer ends of the weld. No cracking was found in the longitudinal welds, except at the center gap location. Here the crack had propagated through the entire weld, but in no case was the crack observed to enter the steel lap plates. Figure 10 shows an exposed surface at the member gap. The two lap plates were partly cut with a saw before pulling the member apart at a reduced temperature. No significant crack extension can be seen in the splice plates. Cracking was visible in the welds at each edge only.

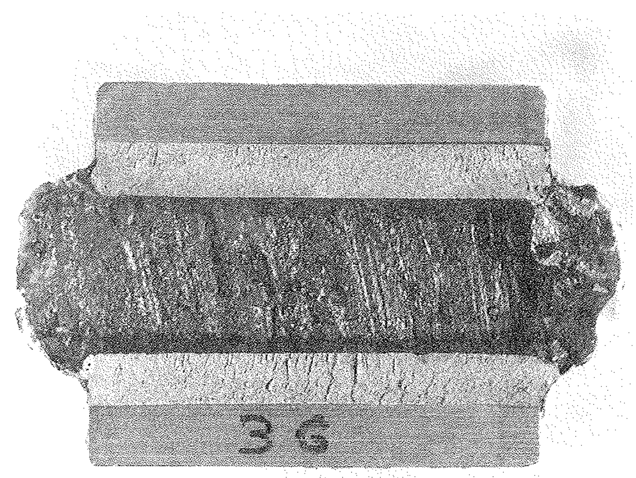

FIGURE 10 Exposed welded gap.

Examination of Results

The mechanism of crack formation and propagation is as follows. Fatigue cracks were found to begin at the toe of the transverse weld in the welded center portion of the wrought-iron plate. Visible cracks were first noticed at approximately 0.5 million cycles at a stress range of 12 ksi. Initially, the toe cracks propagated perpendicular to the thickness of the wrought-iron plate until they encountered a significant slag stringer. The crack may then reinitiate out of the stringer at a location that is further into the joint. The crack continues to encounter stringers, and this process continues to a depth of approximately 0.1875 to 0.125 in. at an approximate angle of 45 degrees. At this point the crack is arrested as the crack turns and extends vertically into the joint, parallel to the stress. The angle of growth corresponds to the plane perpendicular to the stress field through the fillet weld. The crack growth results in a staircase effect, as can be seen in Figures 11 and 12.

The importance of the stringers in the wrought iron appears to be their ability to arrest the transverse crack and deflect the growth of the crack into the joint. A similar joint geometry in steel would result in the toe crack propagating through the thickness of the main plate. In the welded wrought iron, the crack is arrested and turned parallel to the plate and cyclic stress. There is an obvious redistribution of the stress in the vi-

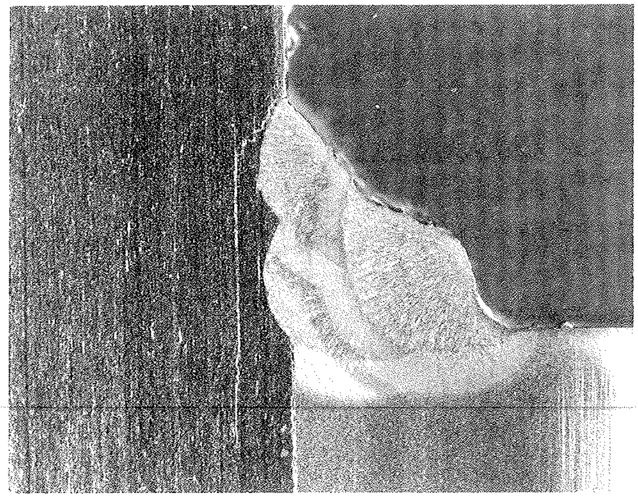

FIGURE 11 Crack growth in test specimen.

FIGURE 12 Crack growth in span F hanger.

cinity of the crack front. As the crack moves into the joint following the laminated stringers, the stress is redistributed to the longitudinal welds. The redistribution of the stress to the outer portions of the joint results in root cracking in both the longitudinal welds and in the outer portions of the transverse welds because the weld sizes were small. Eventually all the load was transferred through the longitudinal welds, which resulted in continued crack extension along the weld throat.

Fatigue testing at a high stress range of 18 ksi resulted in a maximum stress level of 21 ksi. This maximum stress corresponds to approximately 80 percent of the measured yield stress of the wrought iron. As the fatigue crack propagated into the joint, the cross section of the wrought-iron plate was reduced. This resulted in yielding of the net section of the wrought-iron plate and also resulted in continued crack extension, with the eventual failure of the section. Such high stress ranges would not be expected in a wrought-iron structure.

The test results from the welded wrought-iron specimens and the actual hangers are summarized in Figures 8 and 9. These tests demonstrate that the fatigue resistance is much greater than that provided by category E--the expected fatigue category for such welded steel details. None of the details tested at a stress range of 12 ksi failed as a result of the crack that formed at the weld terminations. These cracks were arrested and did not impair the load-carrying capability of the joint. Failure, should it occur, was shown to develop at the gap region.

TRAFFIC STUDY AND FATIGUE ASSESSMENT

To accurately assess the severity of fatigue cracking in welded lap splices, the stress history spectrum for the hanger member must be determined. By reviewing the traffic that the structure has experienced and its relation to the stress levels in the hanger, an estimate can be made of the cumulative fatigue damage and the remaining life of the structure.

Because of the long history of the bridge and a change in ownership, limited traffic data were obtainable from the Norfolk and Western Railway. Available data included the following: train timetables from 1936 to 1963, annual gross tonnage figures from 1971 to 1981, and a listing of all train movements during 1982. Because these data were insufficient for an adequate traffic study, it became necessary to supplement it with other sources of data. Correlation was made with traffic data from Canadian National (CN) Rail at bridges that were on lines similar to that of the line that Bridge 651 traverses (2,3). CN has done extensive traffic studies on many of their lines, which allows them to develop useful averages and to indicate particular trends from an extensive data base.

Given the limited amount of traffic data, the traffic frequency and distribution must be estimated. There are several important parameters that can be used to generate the necessary information. These are the number trains per day, locomotive weights, engine units per train, annual gross tonnage, and carload distribution. Although not all parameters are known for each year under study, not all are needed for any one particular year. The method used in a particular time period was dictated by the available information.

The period of concern for the hangers is from 1937, when the welded repairs were performed, to the present. Also, an estimation for the future (to the year 2000) was made. For each year, the number of locomotives and cars that crossed the bridge was estimated. Studies by CN concluded that only cars more than 60 gross tons have a significant effect on the fatigue life, and all other lighter traffic could be ignored (4). From 1937 to 1954 it was assumed that only the locomotives contributed to fatigue damage because carloads seldom exceeded 60 tons. Assuming one locomotive per train (typical with steam locomotives), the yearly total was calculated from the timetables. Because extra freight trains were run in a given time slot, the number of scheduled trains, both passenger and freight, was increased by 10 percent. Beginning in 1955, with the complete conversion to diesel, carloads began to increase. The number of cycles per year from the carloads was determined by multiplying the annual gross tonnage by the percentage of cars greater than 60 tons and dividing by the root-mean-square value for carloads greater than 60 tons. The number of locomotive loadings was determined from available timetables, with the realization that more than one diesel unit could be used on one train. Beginning in 1962, when train tables were no longer available, the number of locomotive units was determined by dividing the annual gross tonnage by the estimated average gross tons pulled by one unit. Carload cycles were determined as previously mentioned.

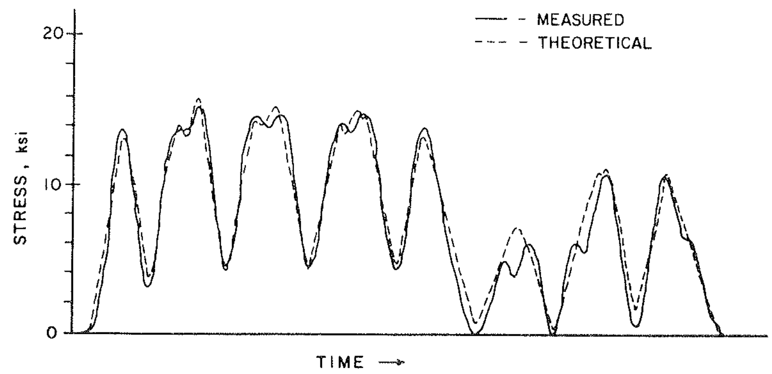

FIGURE 13 Comparison of measured and theoretical response.

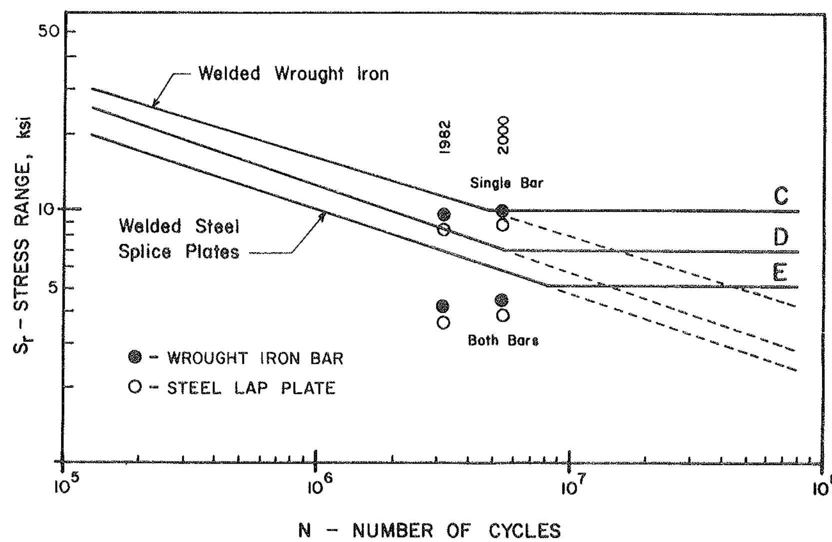

FIGURE 14 Plot of estimated fatigue life for hangers, span F.

Having the number of loading cycles caused by both locomotives and carloads for each year, it was necessary to determine the stress ranges caused by them. A three-dimensional linear finite-element model of the span was developed and calibrated with the strain-gauge data obtained during the field investigation. This examination demonstrated that impact effects were negligible and that a space-frame analysis was applicable to the structure. The influence of dynamic effects is reduced on the structure because of the presence of a sharply curved tunnel immediately adjacent to the bridge site. This imposes speed restrictions on all train traffic. Figure 13 shows the correlation between the theoretical and measured stress history for a given train passage. From the computer analysis an influence diagram was developed for the hanger member. A typical wheel-spacing configuration was assumed for both a steam and diesel locomotive and a typical car. Each was given a unit weight and passed across the influence diagram; then the stress range was determined. Then, by using the estimated static values for locomotive and car weights, the effective stress range could be determined for each year. By using Miner's linear fatigue damage relationship, the extent of the fatigue damage could be estimated by summing the number of cycles at a particular stress range for a given year.

A problem of determining the correct effective stress range that corresponded to all damage cycles arose because of the slackness in some in the eyebars. The actual accumulated fatigue damage estimate for a given loaded member is bounded by the condition of both bars carrying the load and by the condition that only one carries the entire load for the member. Assuming that both eyebars were loaded resulted in an effective stress range (S_r Miner) equal to 4.3 ksi for 3.1 million variable load cycles. Projected to the year 2000, the figure increases to 4.5 ksi for 5.2 million cycles. With only one bar participating, the 1982 value increases to 9.8 ksi for 3.1 million cycles, and the future damage increases to 10.3 ksi for 5.2 million cycles. These points are plotted in Figure 14 and are compared to AASHTO fatigue categories and the fatigue test data developed on the welded wrought-iron joints.

The predicted effective stress range and accumulated cycles of the variable loading plot well below the fatigue resistance provided by welded wrought-iron end details. The tests on laboratory-welded specimens and on cracked details removed from span F demonstrated that a stress range of 12 ksi would not produce failure, even after 20 million cycles of loading. The effective stress range and cumulative cycles in 1982, as well as those projected to 2000, are well below those resistance values. Had both hangers been effective and the stress range decreased, an even greater margin of reserve life would exist.

Because the lap plate splices are more critical at the cross section at the center of the joint where the eyebar was cut, those values are plotted in Figure 14. The increased cross-sectional area decreases the effective stress range to 3.8 ksi in 1982 if both bars are effective and to 8.7 ksi if only a single bar is carrying the load. The small degree of cracking observed in the field and in the eyebars removed from the structure suggests that both hangers were carrying the load throughout much of their life. Had both hangers shared the load, no appreciable damage would have developed for the traffic projected to the year 2000.

SUMMARY AND CONCLUSIONS

As a result of the fatigue tests of welded wrought iron with welded lap splice repairs and the correlation with the load history of the bridge, the following conclusions were reached.

1. Surface cracks were found to develop at the toe of the splice plates that were welded to the surface of the wrought-iron eyebars at cycle lives comparable to welded steel components. These cracks were found to have no adverse effect on the fatigue resistance of the wrought-iron members because they were arrested by the wrought-iron slag stringers.
2. An evaluation of the load history to which the members were subjected indicated that the weld toe cracking would be expected. It primarily resulted because the two eyebar hangers were not sharing the load. One eyebar was found to be loose, and this increased the cyclic stress in the other eyebar by slightly less than 50 percent.
3. Laboratory fatigue tests were carried out on simulated welded joints and on several of the cracked eyebar hanger splices. The simulated test joints demonstrated that the wrought iron would arrest the fatigue cracks that formed at the weld toe at stress levels expected in the structure. The weld toe cracks encountered the flattened longitudinal stringers and were arrested as the crack deflected parallel to the cyclic stress. The crack-arrested details were able to sustain 20 million stress cycles at 12 ksi without any further evidence of distress. The cracked hangers were also fatigue tested, and they yielded comparable results.
4. The laboratory tests also demonstrated that the steel splice plates were the more critical detail. The four tests carried out provided a fatigue resistance comparable to category D, although they are classified as category E details. Because the stress range at the critical steel section was less than the wrought-iron bar, the hangers had not experienced much growth at those sections.
5. The stress history analysis and laboratory evaluation demonstrated that the other weld-repaired members (such as chord and diagonals) were not susceptible to any appreciable crack growth and would not be fatigue critical.
6. A pilot test on the edge-welded wrought-iron splice demonstrated that the stringers were not effective in arresting fatigue crack growth.

ACKNOWLEDGMENT

This study was sponsored by the Norfolk and Western Railway. The authors are indebted to E. Bond, assistant chief engineer of structures, for his cooperation and assistance with this study. The authors are also indebted to the staff of Fritz Engineering Laboratory, Lehigh University, for their assistance throughout this study. H.T. Sutherland assisted with both the field studies and the fatigue tests, R. Sopko with photography, and R. Dales and C. Hittinger with the laboratory testing.

REFERENCES

1. J. Aston and E.B. Story. Wrought Iron, Its Manufacture Characteristics and Application. A.M. Byers Company, 1939.
2. I.A.S. Elkholy. Estimated Fatigue Damage in the 83'-0 Deck Truss Span of the Muskoka River Bridge, Mile 116.8, Newmarket Subdivision, Huntsville, Ontario, Canada. Technical Report. Canadian National Rail, Montreal, Quebec, Canada, April 1981.
3. R.A.P. Sweeney and I.A.S. Elkholy. Estimated Fatigue Damage in the 206'-3 Through Truss Span of the Ottawa River Bridge, Ste Anne de Bellevue, Quebec, Canada. Technical Report. Canadian National Rail, Montreal, Quebec, Canada, June 1980.
4. R.A.P. Sweeney. The Load Spectrum for the Fraser River Bridge at New Westminster, B.C. Proc., American Railway Engineering Association, Washington, D.C., 1976.

Publication of this paper sponsored by Committee on Dynamics and Field Testing of Bridges.