# Crack Evaluation and Repair of Cantilever Bracket Tie Plates of Edison Bridge

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The Edison Bridge on Route 9 over the Raritan River in New Jersey is a riveted two-girder floor beam structure that was built in 1944. Cracks were discovered in several tie plates connecting the cantilever brackets to the main girder and floor beam in the continuous span, the 25.8-m (84-ft, 6-in.) spans, and the 40.4-m (132-ft, 6-in.) spans. Several cracked tie plates were removed so that the cause of cracking could be evaluated. The investigation included metallographic and fractographic examinations of crack surface areas that were not damaged extensively by corrosion. Striation spacing measurements showed that the cyclic stress driving the crack was independent of crack length, characteristic of displacement-induced fatigue. Field testing to determine the live load stress levels confirmed that in-plane bending of tie plates was the principal cause of the fatigue cracking and that the cantilever bending stresses were negligible. The measurements also suggested that a loss of frictional composite action occurred as the slab deteriorated. This led to high cyclic stresses in tie plates, with the maximum stress range varying between 69 and 138 MPa (10 and 20 ksi). Fatigue cracking initiated at flame-cut plate edges subjected to the maximum in-plane bending stress from distortion. Cracking was also aggravated by corrosion loss of section. Future inspection procedures were developed to enhance crack detection, which is highly variable because of the uncertainty of composite action. Retrofit recommendations for failed tie plate replacement were developed as well as means to eliminate the problem when the structure is rehabilitated.

The Edison Bridge on Route 9 over the Raritan River in New Jersey is a riveted two-girder floor beam structure having a total length of 1321 m (4,332 ft). It consists of two three-span continuous structures of 183 m (600 ft), one three-span continuous structure of 198 m (650 ft), eight simple-span structures of 40.4 m (132 ft 6 in.), six simple spans of 46.5 m (152 ft 6 in.), and six spans of 25.8 m (84 ft 6 in.). The noncomposite concrete slab is directly supported by haunches on the main girders, transverse floor beams, and the cantilever brackets. The main girders are spaced 12 m (39 ft) apart, and the cantilever brackets extend 2.9 m (9 ft 6 in.) beyond the girders. The structure was built in 1944.

Figure 1 shows a portion of one of the continuous-span structures and a typical cross section applicable to all spans. Figure 2 is a view showing the longitudinal girder, a floor beam bracket, and stringer.

Cracks were discovered in several tie plates connecting the cantilever brackets to the main girders and floor beams in simple and continuous spans.

This paper provides an evaluation of the cracking that developed in the tie plates, suggestions for retrofitting the structure for the short term (10 to 15 years), and recommendations for long-term repair and rehabilitation.

## Examination of Cracked Tie Plates

Three cracked tie plates were removed from the structure for examination. A tie plate removed from the eighth 40.4-m (132-ft, 6-in.) span southbound had a crack that extended nearly across the width of the plate. Figure 3 shows one of the plate sections and the diagonal crack that extended about two-thirds across the tie plate. The balance was flame cut during removal. A section of tie plate containing a crack removed from the 183-m (600-ft) continuous span 15.2 m (50 ft) south of Pier 2 was also delivered for examination. A view of the cracked plate is given in Figure 4, with the balance of the plate cut away.

The cracks in all tie plates initiated along the tapered edge of the plate several inches from the start of the taper on the bracket side, as is apparent in Figures 3 and 4. This edge was probably a flame-cut edge given the overall shape of the tie plate, although corrosion has since eroded the original plate edge surface and all evidence of the cutting method. The plates also showed substantial general corrosion damage over the surface as well. The extent of corrosion on all three tie plates indicates that they were debonded from the overlying haunched slab for a substantial period of time. The crack in Tie Plate 3 initiated from the plate edge and propagated to a rivet hole (Figure 4). The crack reinitiated at the rivet hole and continued to propagate across the tie plate.

The length of the crack in Figure 3 was 432 mm (17 in.). The crack shown in Figure 4 for Tie Plate 3 was 279 mm (11 in.) long. No other cracks were detected elsewhere in the tie plate.

## Fractographic Examination

The crack surfaces from each of the tie plates were examined visually and with the scanning electron microscope (SEM) to verify the crack extension mechanism. Measurements of fatigue striations were also obtained where possible along the length of the crack since the striation spacing can be used to indirectly estimate the stress range in the tie plate. This technique has been applied with some success to fatigue cracking in bolts and plate elements in a variety of structures.

The crack surfaces from all tie plates showed varying amounts of damage by corrosion. The damage was most severe at the crack origin and diminished toward the crack tip, where the small crack opening tended to protect the surface from the environment. Before being examined with the SEM, the crack surfaces were cleaned ultrasonically to remove as much of the corrosion product as possible and permit observation of the underlying crack features.

A view of the crack surface from Tie Plate 3 is shown in Figure 5. The smooth and flat surface is typical of fatigue cracking. The crack surface from Tie Plate 3 was also found to be free of corrosion product near the crack tip region. The crack origin region was covered with a heavy layer of corrosion product that could not be lifted from the surface. Beachmarks are clearly seen on the crack surface near the crack tip. A low-magnification (7×) SEM micrograph of this area also shows finer beachmark detail than can be seen with the unaided eye. The surface in this area was also examined at higher magnification for fatigue striations. Figure 6 shows an SEM micrograph at 2,040× magnification that shows the fatigue striations on the crack surface, which is located 241 mm (9.5 in.) from the origin.

Fatigue crack growth was also detected in Tie Plate 1, seen in Figure 3. The crack origin was covered completely with corrosion product, which obscured any detail of crack initiation. Efforts to remove the corrosion product at the crack origin and over most of the crack surface were not successful since the corrosion process had completely consumed the underlying fracture features. Closer to the crack tip, some areas were free of corrosion and some evidence of fatigue beachmarks could be seen (Figure 7). The crack surface in this area was examined further with the SEM. Figure 8 shows a high-magnification micrograph (2,040×) showing well-defined fatigue striations.

The crack surface from Tie Plate 2 was consumed entirely by corrosion except near the crack origin. Corrosion damage was severe since this tie plate was located near an expansion joint, which provided frequent wetting from the weather. Evidence of grout was seen on the crack surface, suggesting that the crack existed at the time the bridge was last resurfaced, in 1985. Despite efforts to remove the corrosion product, no fracture features could be discerned on the crack surface.

## Material Properties

Although none of the tie plate crack surfaces examined showed evidence of crack instability, limited material property tests were conducted on Tie Plate 1 to characterize the strength and notch toughness properties of the plate. Considering the date of erection, the material was most likely supplied under the ASTM A7 specification. Tensile test results confirmed this, providing a yield strength of 300 MPa (43.4 ksi), ultimate strength of 456 MPa (66.2 ksi), and elongation of 27 percent.

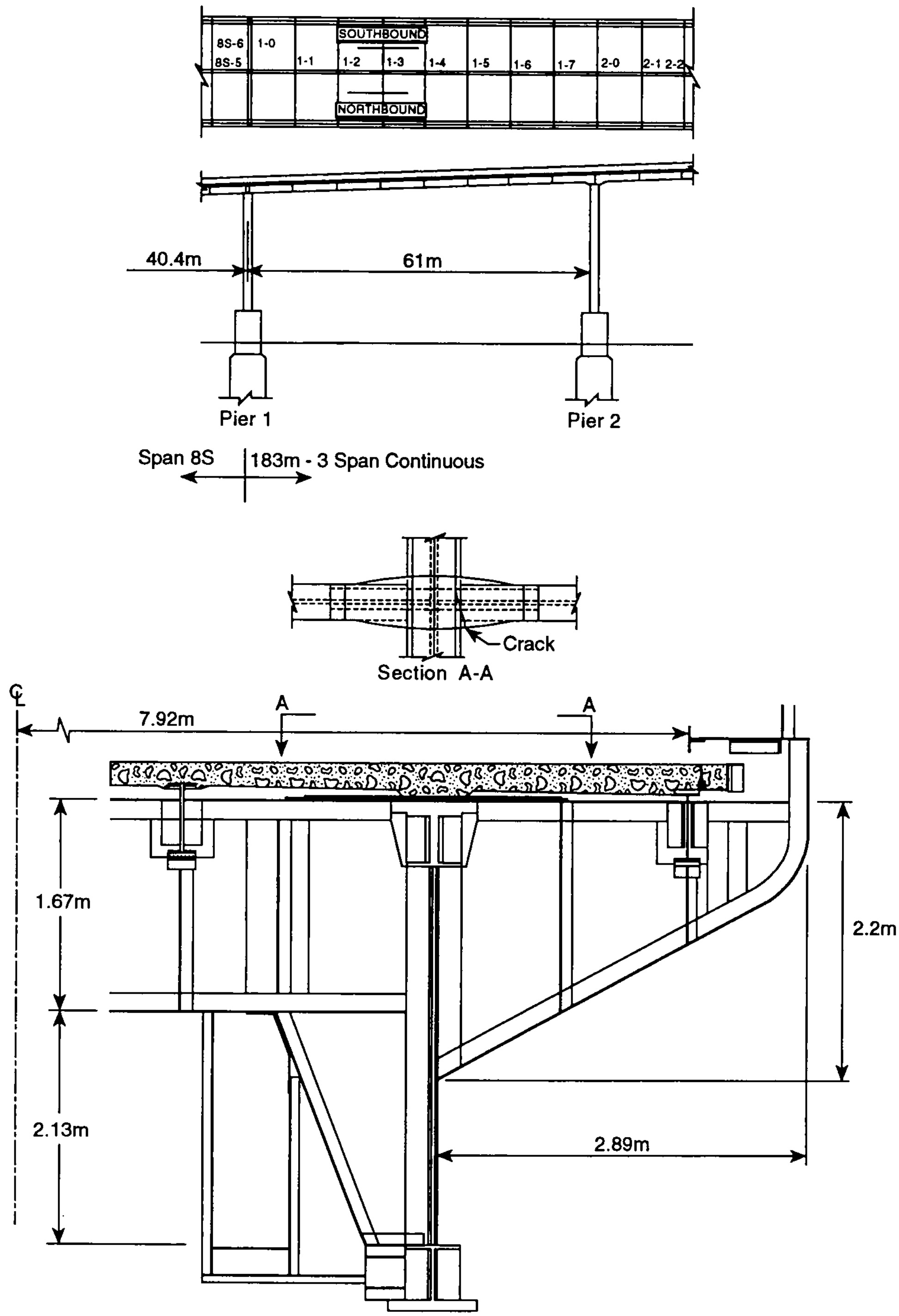


FIGURE 1 Typical span and elevation of Edison Bridge.

The relatively high yield strength (for ASTM A7) is not uncommon in the thin plate of this material.

Subsize Charpy V-notch (CVN) specimens were also fabricated from the tie plate. These tests provided about 135 J (100 ft-lb) at 4°C (40°F). Approximate standard CVN test specimen results, corrected for specimen thickness, averaged 182 J (135 ft-lb). The results show that crack instability would not be expected in the tie plate at minimum service temperatures as low as −34°C (−30°F) under the low strain rates that these elements are subjected to and because of the high toughness of the material.

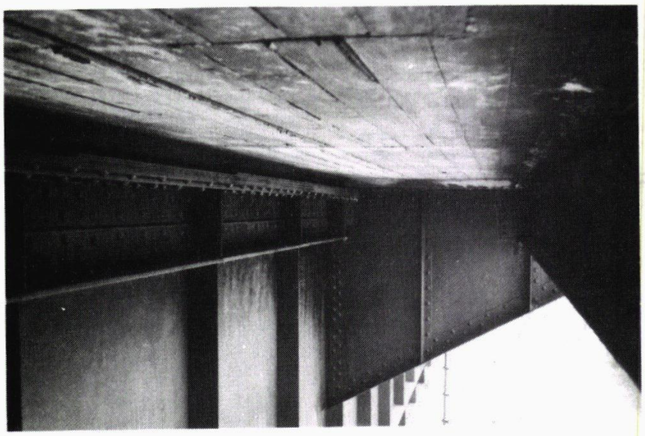

**FIGURE 2** View of girder-cantilever bracket.

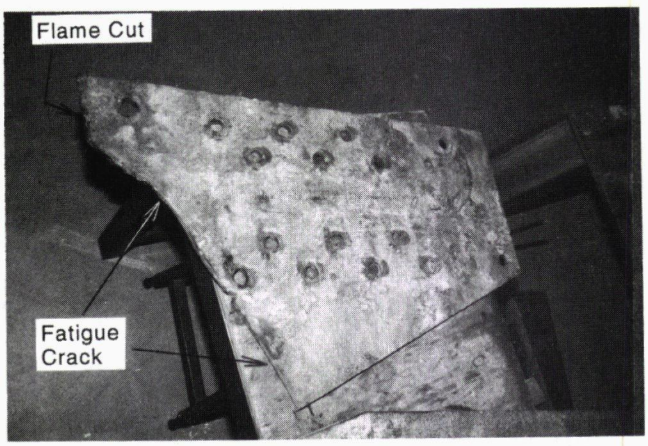


**FIGURE 3** View of cracked tie plate 1 from 30.4-m span.

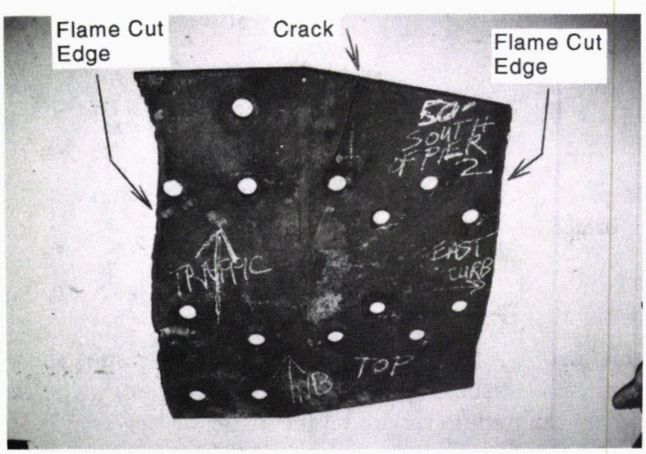


**FIGURE 4** Crack in Tie Plate 3 from 183-m continuous span.

## Results of Striation Measurements

Each of the micrographs showing striations was measured to determine the average striation spacing. Since the spacing varies under variable loading, measurements were taken by counting striations on the micrograph and computing an average spacing. This count was performed over several areas. The location on the crack surface relative to the crack origin of each of the micrographs shown was also recorded. The corresponding stress intensity range, $\Delta K$, was calculated using the crack growth relationship

$$\frac{da}{dN} = C\Delta K^3 \qquad (1)$$

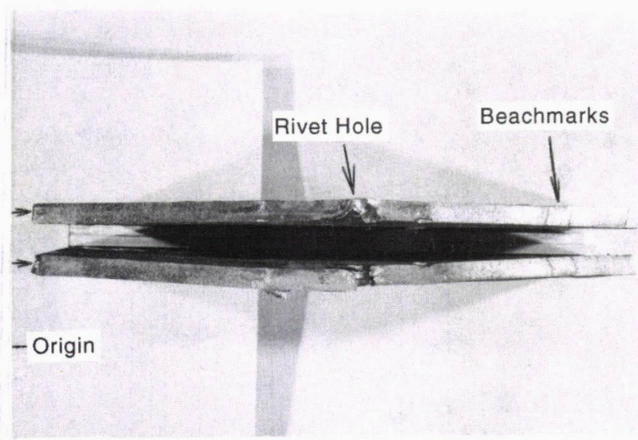


**FIGURE 5** Crack surface of Tie Plate 3 showing beachmarks, rivet hole, and crack origin.

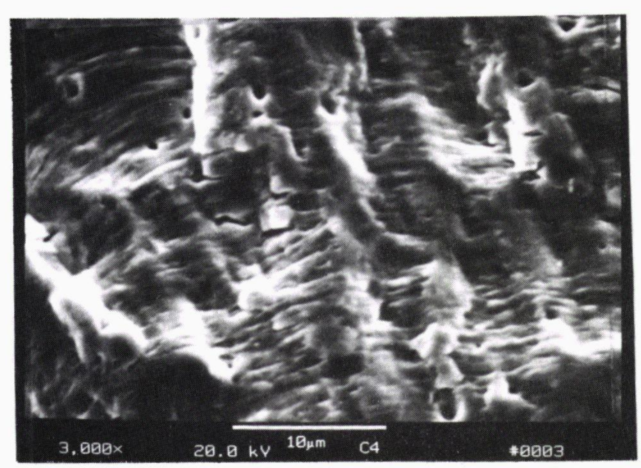


**FIGURE 6** SEM micrograph showing fatigue striations in Tie Plate 3 beachmark area (2,040×).

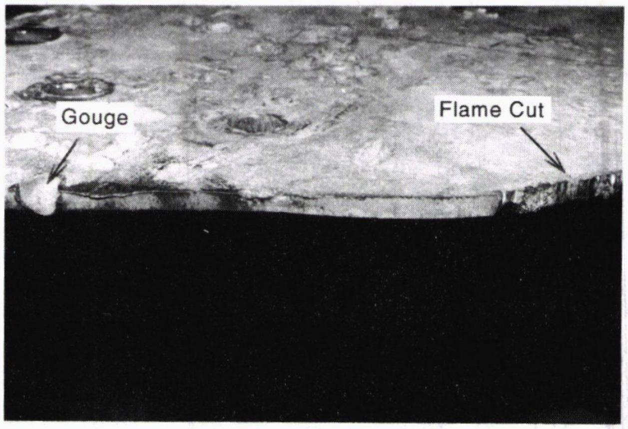


**FIGURE 7** **Crack surface in Tie Plate 1 near flame-cut end.**

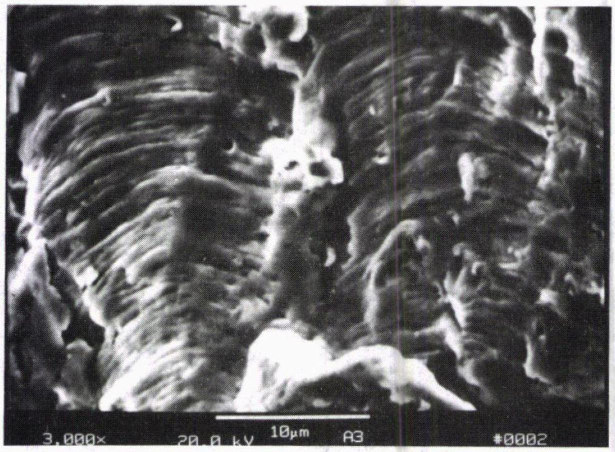


**FIGURE 8** **SEM micrograph showing fatigue striations in Tie Plate 1 near crack tip (2,040×).**

$C = 2.18 \times 10^{-13}$ where $a$ and $\Delta K$ are in units of millimeters and MPa $\sqrt{\text{m}}$, respectively. Because of corrosion over most of the crack surfaces, measurements could be obtained over only a limited range of crack lengths in each plate that varied from 203 to 318 mm (8 to 12.5 in.). The calculated stress intensity range of 37 MPa $\sqrt{\text{m}}$ (34 ksi $\sqrt{\text{in.}}$) is nearly the same in both tie plates and was independent of crack size. The estimated stress ranges in the tie plates calculated using an edge crack model in pure bending are as follows:

$$\Delta K = \frac{\left[0.923 + 0.199\left(1 - \sin\frac{\pi a}{2b}\right)^4\right]}{\cos\frac{\pi a}{2b}} \Delta\sigma\sqrt{\pi a} \qquad (2)$$

where $b$ is the width of the tie plate along the plane of the crack (Figure 9). The effect of shear is to cause the crack to follow the principal stress that is apparent in the crack paths seen in Figures 3 and 4. This measured 610 mm (24 in.) for Tie Plate 1 and 559 mm (22 in.) for Tie Plate 3. Stress ranges of 29 to 51 MPa (4.17 to 7.4 ksi) were calculated from the striation measurements. These most likely represent the most frequently occurring stress cycles. Finer striations not discernible using the SEM are undoubtedly present on the crack surfaces as well as infrequent larger stress cycles not detected over the small area of fracture surface observed.

## Field Measurements

To assess the causes of crack initiation and growth, field measurements of the live load strain response were obtained on several tie plates of the spans shown in Figure 1. Gauges were also placed on the web copes of the floor beam cantilever brackets and on one cross section of the east girder. Figure 10 shows the strain gauges on the bracket web cope and the edge of the tie plate near the location at which the fatigue cracks were observed to develop before repair. The gauges were installed on each edge of the tie plate as well as on the floor beam side.

The strain gauges were connected to a 21-channel recorder through cables and signal conditioners. The recording equipment was placed on Pier 1 or Pier 2, under the bridge deck, so as to clear the roadway for full traffic. Altogether, about 3 hr 20 min of strain response was obtained on each gauge on August 24 and 25, 1993.

## Results of Strain Measurement

The recorded strain variation at each strain gauge was examined through an oscilloscope. Figure 11 shows strain-time records for several gauges in the area of Pier 2 during passage of a northbound truck. The upward excursion of a trace indicates live load compression; downward is tension. The stress range was defined by

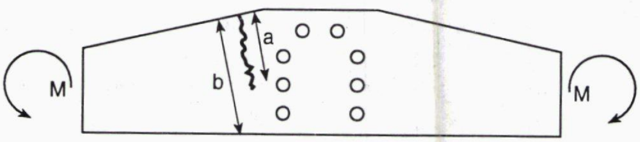


**FIGURE 9** **Schematic of crack in tie plate model.**

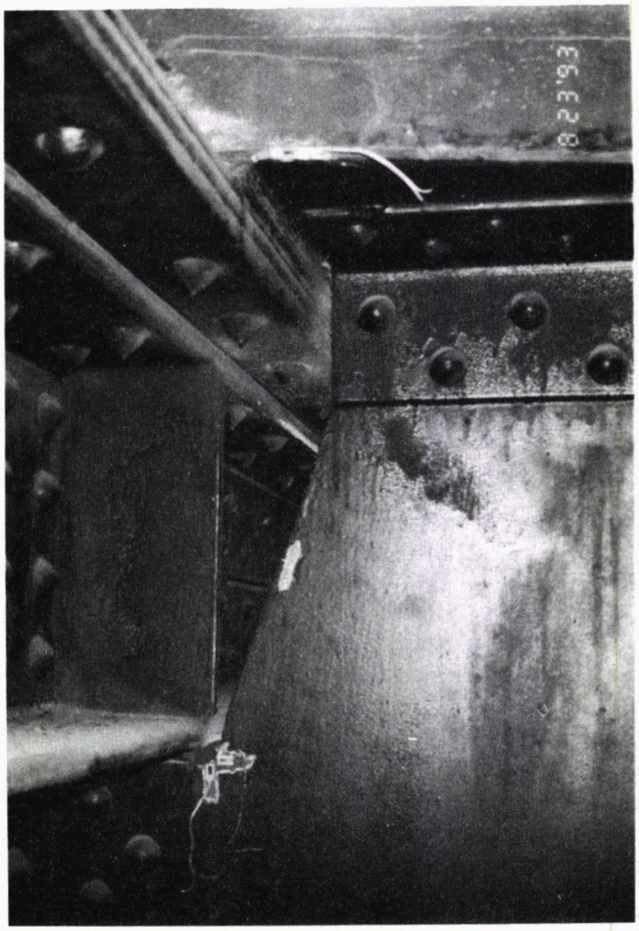


**FIGURE 10** **View of gauges on tie plate edge and at bracket cope.**

the peak-to-peak excursion while the vehicle crossed the structure.

From examination of these traces, and from results of other bridges with similar conditions of tie plate stresses, a number of observations can be made:

- The variations of live load strains in the tie plates are analogous to those in the girder flanges. This indicates that the stresses in the tie plates are generated by the relative longitudinal displacement between the girder top flange and the bridge deck (*1,2*).
- The tie plates are bending horizontally when trucks travel on the bridge. This is depicted by the opposite excursion of strain traces for Gauges 1 and 2, 3 and 4, and 7 and 8, in Figure 12, as well as for other strain gauge pairs in the north-south direction on all tie plates. From the response of gauges at Pier 2 and at floor beam 1-6E, 15.2 m (50 ft) south of Pier 2, it is apparent that the vehicles are traveling about 15.2 m/sec (50 ft/sec). The peak excursion at Pier 2 in one direction occurs as the vehicle moves from the end span at Pier 1 to Pier 2. The stress cycle reverses as the vehicle continues for about 45.7 m (150 ft) in 3 sec and then returns gradually to its starting point after about 4 more seconds (~61 m or 200 ft). Figure 12 (*top*) shows the simultaneous stresses in Gauges 1 through 4 at the moments of peak stresses due to a northbound truck, corresponding to the strain traces at times $t_3$ and $t_4$, which are identified by the vertical lines in Figure 11. These stress gradients show the full reversal of the stress cycle at Pier 2-0E. A similar plot of the instantaneous gradients at Tie Plate 1-6E is shown in Figure 12 (*bottom*). The maximum stress range due to this truck was 93 MPa (13.5 ksi) at Gauge 4. The same truck induced the maximum live load stresses in Tie Plate 1-6E with the maximum stress range equal to 116 MPa (16.8 ksi) at Gauge 8.
- Northbound trucks generated higher stress ranges in tie plates over the east (northbound) girder. Southbound trucks induced higher stress ranges in the west girder tie plates. The measurements suggest that the tie plates near Pier 2 act as though a significant amount of composite behavior has been lost with time. The loss of composite action increases the in-plane bending that the tie plate experiences. The low levels of stress range measured at Pier 1 indicate that there is little loss of composite action in either span at Pier 1. It is also apparent that the axial stress in the tie plates produced by the cantilever load on the bracket is small compared with the in-plane bending.

The HS-20 design axial stress range in the tie plate is estimated to be about 30 MPa (4.3 ksi). The measured axial stress in the tie plate was less than 50 percent of the design estimate. The in-plane bending of the tie plate that results from the relative longitudinal movement is dominant but highly variable because of differences in composite behavior.

- The magnitudes of stress range due to the same truck differ from tie plate to tie plate. Experience at other bridges has shown that tie plates over end bearings and piers would be subject to more horizontal bending, thus higher stress ranges (*1,2*). This condition did not seem to hold true for this bridge system. The tie plates at Pier 1 (end bearings) had very low stress ranges, whereas the tie plates at FB1-6 near the contraflexure point had the highest.

The stress range spectra for Gauges 4 and 8 on the east side near Pier 2 are shown in Figures 13 and 14. These plots show the skewed distribution characteristics of the truck gross vehicle weight distribution. Between 526 and 689 stress cycles greater than 10 MPa (1.5 ksi) were observed at these gauges during the 3 hr 20 min of record at Pier 2. None of the stress cycles at Pier 1 was high enough to warrant developing a stress range spectra, as the maximum stress range was 23 MPa (3.4 ksi).

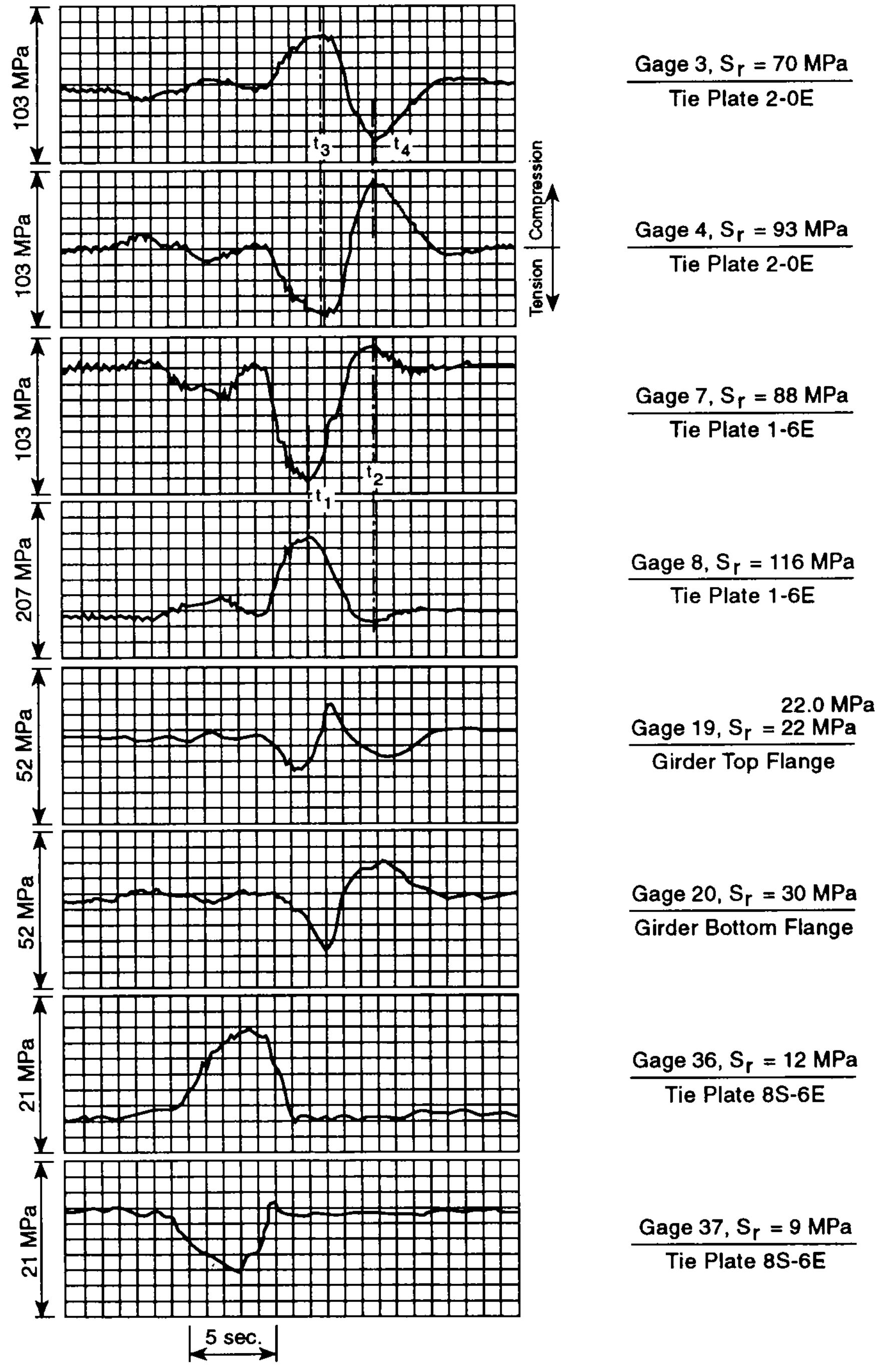


**FIGURE 11** Typical strain-time response of tie plates and main girder in 61-m span (see Figure 1).

- The stress ranges in the flanges of the east girder are comparable to those recorded at similar girders. The maximum stress range was 30 MPa (4.4 ksi) at the Bottom Flange Gauge 20, and 22 MPa (3.2 ksi) at the Top Flange Gauge 19 (see Figure 11).
- The stress ranges at the strain gauges at the cope of cantilever were not high. The maximum was 16 MPa (2.3 ksi) at a horizontal gauge.

## Discussion of Results

From the experimental results and field examination of the bridge, it was apparent that the concrete deck of the bridge was in contact with the top flange of the girders and the top flange of the floor beams. However, the width of the concrete "pedestal" along the girder top flange is different in different spans. For the three-span

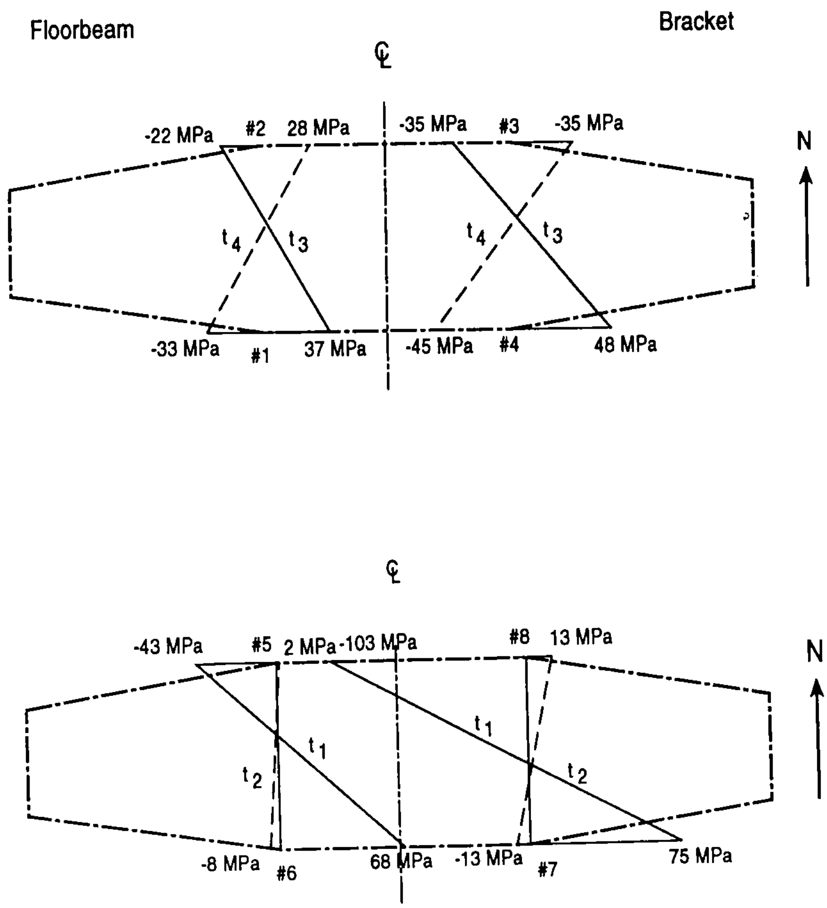


**FIGURE 12 Stress gradients in tie plates of 61-m Span 1 during passage of northbound truck: *top*, Tie Plate 2-0E; *bottom*, Tie Plate 1-6E.**

183-m (600-ft) continuous structure between Piers 1 and 4, the bottom width of this "pedestal" is narrower than the width of the girder top flange. There were signs of separation and relative vertical and horizontal movement between the deck and the girder near Pier 2. At some floor beams, a visible gap existed between the two components. On the other hand, the pedestals of the deck above the girders of the simple 40.4-m (132-ft, 6-in.) spans were the same width as the girder top flange. There was no visible separation between the deck and the top flange at Pier 1 for either span at that location. This separation contributes to the loss of composite behavior and enhances the longitudinal movement between the girder and the deck of the 183-m structure and thus the horizontal bending of the tie plates near Pier 2.

The highest magnitude of stress range in a tie plate was measured at Tie Plate 1-6E, which was a replacement plate installed in August 1993. The new tie plate is not in contact with the concrete deck. At Tie Plate 8S-6W, the deck appears to be in direct contact with the girder flange, and the measured stress range was less than 21 MPa (3 ksi). This reinforces the assumption that tight contact between the deck and the girder top flange generates interaction between these components and decreases the horizontal bending of the tie plates because of the composite action. The experimental results also demonstrate that the axial stress in the tie plates from wheel loading on the cantilever bracket is small compared with the in-plane bending. In the continuous span near Pier 2 where loss of composite behavior was most apparent, the in-plane bending stresses were typically four to six times as great as the axial stresses in the tie plates.

The relative horizontal movement between the deck and the girder at Tie Plate 1-0E was 0.94 mm (0.037 in.). This is smaller than the magnitudes observed on other bridges where the deck is not in contact with the girder flange. The existence of relative movement indicates that the tie plate is being bent horizontally, which was confirmed by the strain measurements.

The stress range data summarized in Figures 13 and

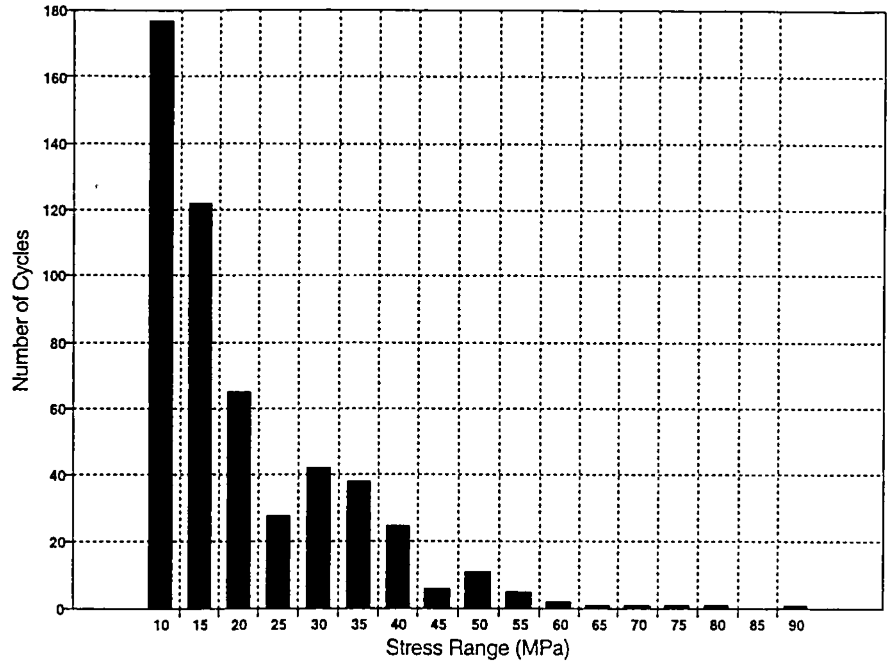


FIGURE 13 Stress range histogram for Gauge 4 on Tie Plate FB2-0E.

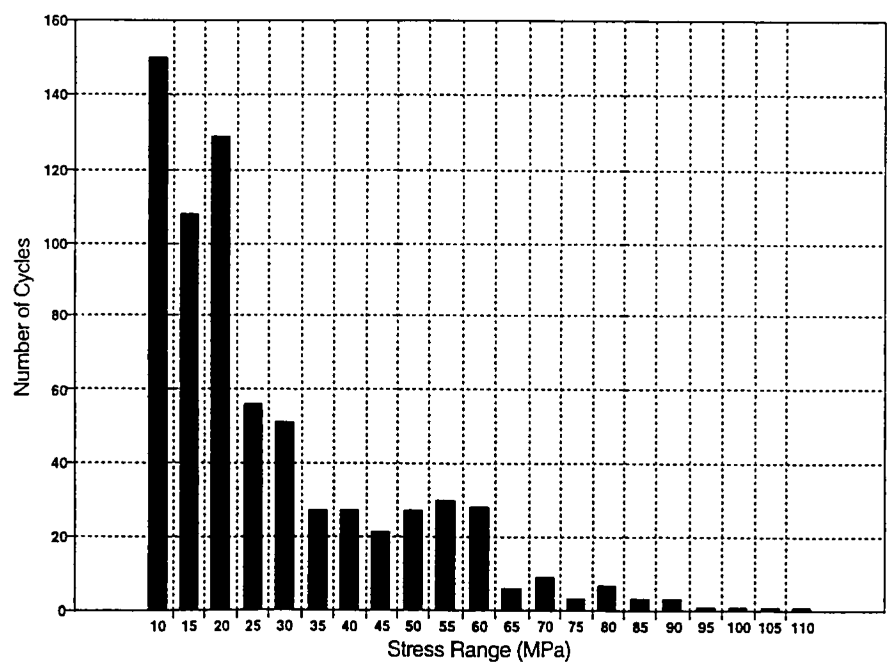


FIGURE 14 Stress range histogram for Gauge 8 on Tie Plate FB1-6E.

14 show the stress cycles greater than 10 MPa (1.5 ksi). The effective stress range can be calculated as

$$S_{re} = \left[\sum \alpha_i \, S_{ri}^{3}\right]^{1/3} \tag{3}$$

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

For Gauge 4 with a maximum stress range of 93 MPa (13.5 ksi), the effective stress range for the spectrum shown in Figure 13 was 25 MPa (3.6 ksi). For Gauge 8 shown in Figure 14, the maximum stress range was 116 MPa (16.8 ksi) and the effective stress range was 36 MPa (5.2 ksi).

Fatigue cracks are initiated only from the flame-cut edge or corrosion-notched region of the tie plate by stress cycles higher than 69 to 110 MPa (10 to 16 ksi). These are the fatigue limit resistance levels applicable to Categories C and B, which apply to these conditions. Cracking of the tie plate at FB1-6E is consistent with the stress range levels shown in Figure 14. The other tie plates in the vicinity of Pier 2 have smaller and fewer stress cycles, and this has prevented additional cracks up to now. However, the infrequent stress cycles of high magnitude will continue to accumulate so that other tie plates can be expected to develop cracks in the future where there is clear evidence of loss of composite behavior.

The composite behavior in the Edison Bridge appears to be highly variable, based on the measurements at Piers 1 and 2. The response at Pier 1 demonstrated that the continuous 61-m (200-ft) span as well as the 40.4-m (132-ft, 6-in.) simple span behaved more like a composite system, with few stress cycles exceeding 14 MPa (2 ksi). No cracks will develop at those stress range levels. Furthermore, neither of these locations exhibited the degree of movement that was observed at Pier 2.

Because of the uncertainty of the degree of composite behavior as a function of time, it is difficult to analytically characterize the structural behavior and predict damage accumulation in the tie plates for the short term (10 to 15 years). It is necessary to make a more qualitative assessment that relies on visual observations of the movement between the slab and the girder that may be apparent from the separation or generation of powder. This can be related to the experimental measurements made at Piers 1 and 2. Where there is little evidence of movement, cracking will not develop and inspection of the tie plates need not be carried out. Where movement is apparent, cracks will be more likely to develop and the tie plates should be inspected at reasonable intervals.

At FB1-6E with reduced composite behavior, the stress range spectrum plotted in Figure 14 suggests that stress cycles large enough to initiate a fatigue crack are occurring more than 20 times each hour. This corresponds to about 175,000 damaging stress cycles a year. At flame-cut tie plate edges and corrosion-notched locations, fatigue cracks are likely to develop after 2 to $4 \times 10^6$ cycles. This would correspond to 11 to 15 years of service after such stress cycles start to occur. It is not possible to establish when this loss of composite action occurred in the past.

It should also be noted that the measured effective stress range is consistent with the magnitude of stress range associated with fatigue crack extension in the tie plates.

## Summary

1. Examination of the crack surfaces in two of the tie plates confirmed that all the cracks were propagated in fatigue. No evidence of rapid crack extension was detected. Severe corrosion of the third crack surface destroyed all evidence of crack extension.
2. The fatigue crack growth characteristics of the tie plates were consistent with the stress range measurements.
3. The fatigue cracks in all three plates initiated from the flame-cut tapered edge. Corrosion had eroded the original edge and destroyed the heat-affected zone.
4. All locations with evidence of movement (i.e., powder visible and separation apparent) that have original tie plates are likely to have stress cycles that exceed their fatigue limit. Hence, cracks will eventually develop at these locations. Nine tie plates had cracked as of 1993, and additional cracks will form in the future. Because it is uncertain when composite action was lost, locations with movement should be inspected annually for cracks.
5. The studies of the cracked tie plates suggest that crack growth after a crack initiates will not cause the tie plate to crack in two during a year of service. The remaining net section can contribute to the resistance of the cantilever bracket web connection should such cracks form.
6. Tie plate locations of the structures with no significant evidence of movement will not experience fatigue cracking for at least 10 years. These tie plate locations need not be given annual inspections.
7. New tie plates with the same width and flame-cut edges installed to replace cracked plates will provide 10 to 15 years of service before cracks initiate.
8. New tie plates that are installed in the future as a short-term fix (i.e., 10 to 15 years) will provide longer lives if the flame-cut edges are ground to provide a smooth edge without the flame-cut serration. They should not crack in less than 15 years.

9. When the structure is rehabilitated and the deck replaced for a long-term fix, a positive shear connection should be provided between the longitudinal girders, transverse floor beams and bracket, and the concrete slab. This will reduce the out-of-plane bending well below the fatigue limit and prevent these cracks from forming.

## ACKNOWLEDGMENTS

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