Stresses in Orthotropic Deck of Rio-Niteroi Bridge Under Traffic

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The six-lane President Costa e Silva Bridge between Rio de Janeiro and Niteroi over Guanabara Bay in Brazil spans the main shipping channel with two parallel three-span continuous steel box girders 848 m (2782 ft) long. The box girders are joined by an orthotropic steel deck with asphalt surfacing. A thorough field study was conducted of the steel box girders under erection and service load conditions. As part of this investigation, the service life behavior of the orthotropic steel deck was examined. Field studies were carried out on a portion of the orthotropic deck to determine the stress history under a random traffic sample so that the fatigue susceptibility of the welded details could be assessed. The investigation revealed that the orthotropic deck will provide satisfactory service throughout its life. Indications are that certain splices of the trapezoidal deck stiffeners may have the potential for fatigue crack growth. Periodic inspection of these splices will provide ample safeguards and ensure that their capacity is not impaired.

In light of the recent failures during construction of four bridges similar to the Rio-Niteroi bridge (1), a thorough field study of the steel box girders and the orthotopic deck of the bridge was made under erection and service loading conditions (2). Because experiments in England on orthotropic steel bridge deck panels (3) demonstrated that fatigue cracks can be generated in the stiffener-to-floor beam connection welds, the field study included stress measurements on part of the orthotropic deck under a random traffic sample. This study was carried out from May 30 to June 5, 1974.

The orthotropic deck was instrumented over the end support of an end span where the plate thickness is reduced (2). Figure 1 shows the general location of the deck gauges. Local stresses were determined under vehicular traffic, particularly in the stiffening elements and near the welded connection between the deck and the stiffening elements. Traffic flow on the bridge was also recorded, some of it photographically.

Investigations in the United States since 1960 demonstrate the applicability of stress measurements obtained in the field when the serviceability of highway bridge structures is assessed (4, 5, 6, 7). These investigations were also very helpful in the development of the current specifications for failure design that are based on existing data and detailed classifications (8, 9, 10, 11).

INSTRUMENTATION AND TRAFFIC DATA

Thirty-three 0.64-cm-long (¼-in) electrical resistance foil strain gauges were mounted in five groups as shown in Figure 2. A quarter-bridge, three-wire hookup was used, which automatically provided lead-in wire and temperature compensation to all gauges. All gauges were located to provide

1. Representative strain data on the orthotropic deck cross section,
2. Strains near the splice plates on the sides and bottom of the trapezoidal stiffeners,
3. Strains at the junction of the deck plate and web A of the north box (Figure 1), and
4. Strains at the junction of the deck plate and floor beam 17.

Figure 3 shows the analog traces in stress units determined from the strain at gauges 5, 21, and 33. These are typical of the traces from all gauges. The elastic modulus was taken as 207 000 MPa (30×10^6 lbf/in^2). Only truck traffic and other large vehicles generated strains sufficiently large to be detected.

A total of 642 truck records were obtained. These were distributed over 19 daylight hours of the 7-day field study. Each truck record was correlated with each strain record. Of the 642 trucks, 120 of them were photographed as they passed over floor beam 17. A typical photograph is shown in Figure 4. The deck markings in the figure identify the transverse location of each truck in relation to the gauges near floor beam 17. The transverse graduated strip is directly over floor beam 17. The narrow strip parallel to the traffic is directly over web A. The wide strip is a lane marker. The five small crosses are located approximately above the midpoint of the five groups of gauges.

The stress range distribution at gauges 5, 21, and 33 is shown in Figure 5. Stress range is the difference be-

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tween a maximum stress as determined from the analog strain trace and the next minimum stress (Figure 3). Gauges experiencing high maximum stresses also experienced high stress ranges.

A continuous count of all bridge traffic on an hourly basis, 24 h/day and 7 days/week, was made available by the bridge authority from toll booth information acquired at the Niteroi approach from March 4, 1974, to May 31, 1975.

The distribution of westbound trucks by number of axles is shown in Figure 6. A comparison of the data shows similar axle distributions during both sample periods. The eastbound distribution (Rio to Niteroi) during the 14-month period is similar. Hence, it is reasonable to assume that the composition of truck traffic in the field sample is typical for the entire structure at least up to May 31, 1975.

No loadometer survey is available in Brazil. Therefore, the frequency of occurrence of axle weight or gross weight could not be determined directly. These were indirectly computed by comparing the strain response at selected gauges, due to passage of a sample of trucks of known lateral (lane) position, with the strain response from controlled load tests using a test truck of known axle weights and known lateral position. The test truck had two axles.

The two-axle truck made 13 crawl run passes and 7 speed run passes over the deck gauge locations near floor beam 17. The speed runs were made at approximately 55 to 60 km/h (35 to 38 mph).

Figure 7 shows the variation in maximum stress ranges at gauges 21 and 33 during the crawl and speed runs. Each curve is essentially an experimental influence line for stress range. Similar curves were obtained for the other gauges. The double hump in each curve is a result of the influence of wheels (front axle) or wheel groups (duals on rear axle). For most gauges, particularly the transverse gauges on the deck plate and stiffeners, the strains produced by the front and rear axles do not interact. A relatively small interaction did occur at gauge 33 as expected.

The axle and gross vehicle weights of selected trucks were computed by using the influence line for gauge 33. The results are shown in Figure 8. Inasmuch as closely spaced (tandem) axles cause a single strain response, the distribution of axle group weights instead of individual axle weights is presented. Figure 8 shows a skewed distribution of axle group weights, which indicates that large numbers of relatively small axle group weights can be expected. Because the orthotropic deck is mainly responsive to axle group weights and wheel or wheel group loads, each truck may generate two or more cycles. However, given the lateral position of traffic and the frequency and loading of trucks, probably only one stress cycle is significant in the fatigue analysis.

Figure 1. Location of deck gauges adjacent to floor beam 17.

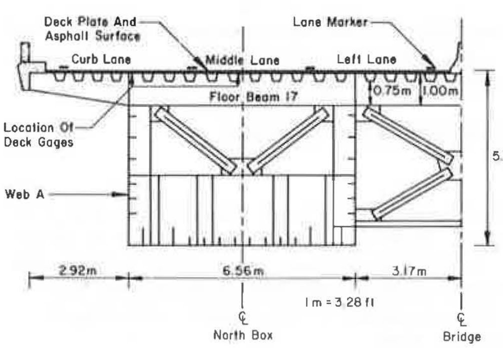


Figure 2. Location and type of strain gauges on underside of orthotropic deck as viewed from below.

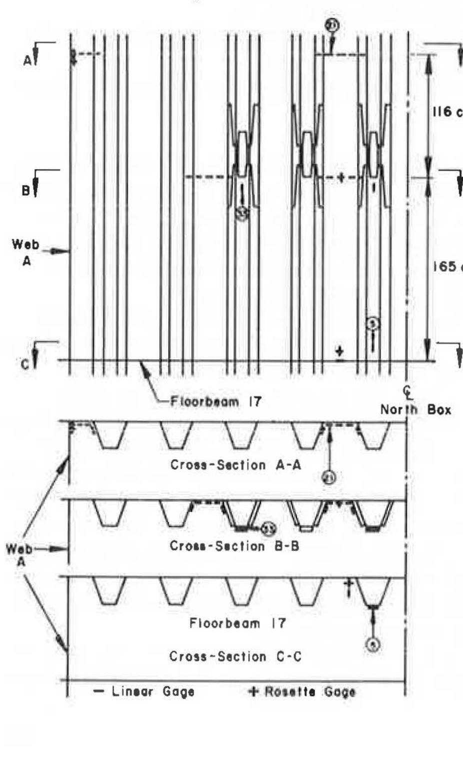


Figure 3. Typical analog traces for gauges 5, 21, and 33 produced by a 12-channel ultraviolet oscillograph trace recorder.

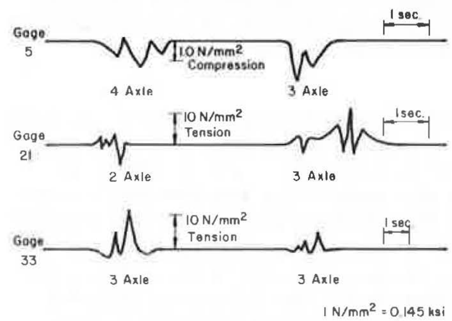


Figure 4. Typical trucks (traveling west from Niteroi to Rio de Janeiro) passing over floor beam 17 of north box.

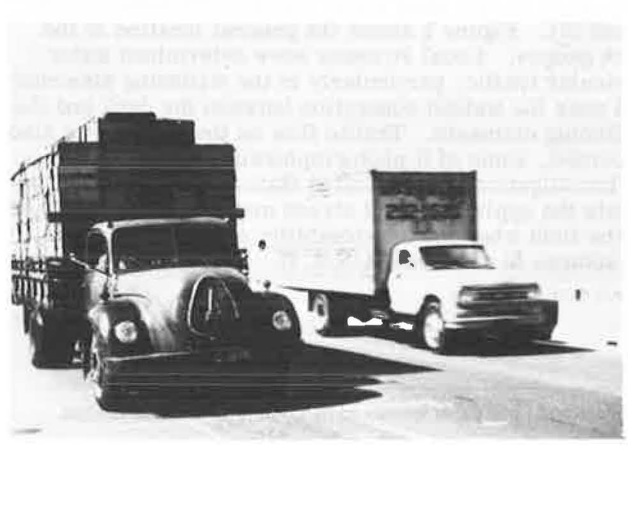


Figure 5. Stress range distribution (a) at gauge 5, measured parallel to stiffener and adjacent to web of floor beam 17, (b) at gauge 21, measured transverse to stiffener and on deck plate, and (c) at gauge 33, measured parallel to stiffener and 25 mm from edge of splice plate.

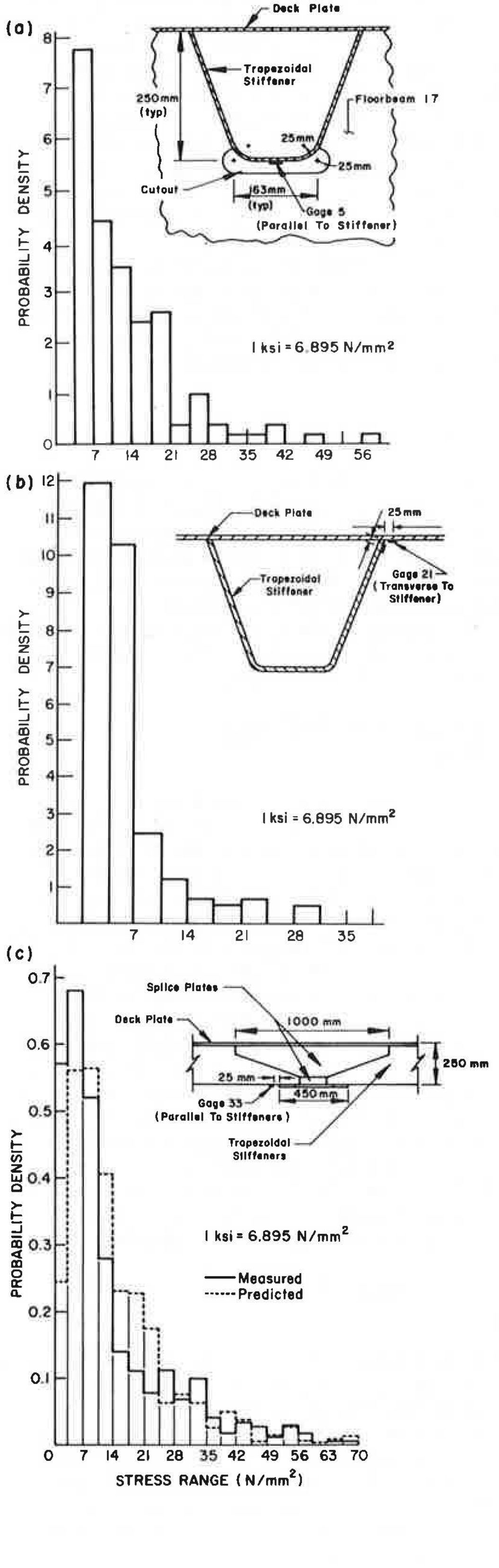


Figure 6. Distribution of axles for (a) 642 trucks, May 30 to June 5, 1974, and (b) 570 525 trucks, March 4, 1974, to May 31, 1975.

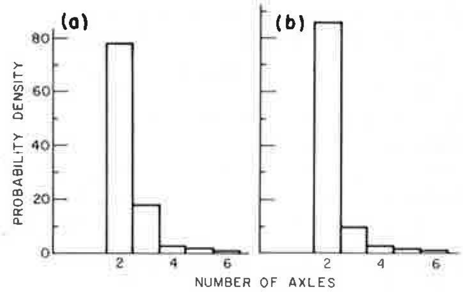


Figure 7. Maximum crawl and speed run stress ranges as a function of the lateral position of the front right tire of the test truck.

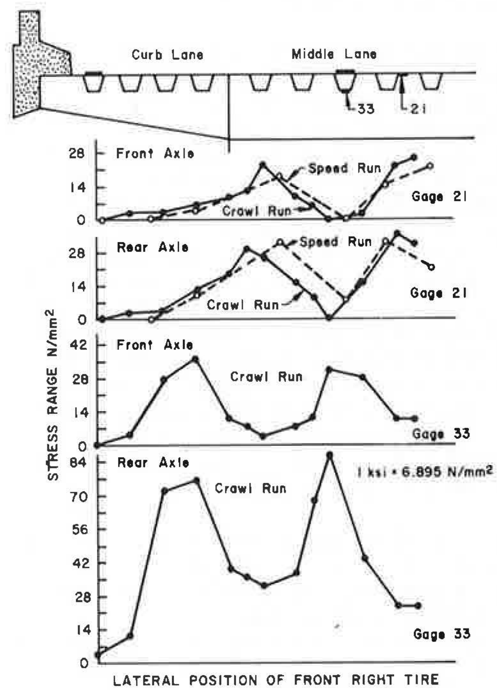
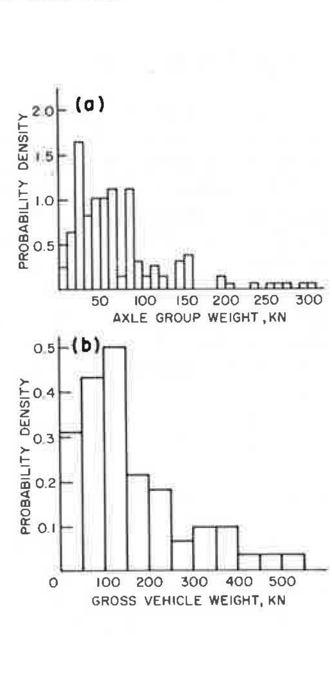


Figure 8. Distribution of axle and gross vehicle weights of (a) 128 axle groups and (b) 64 vehicles.



In Figure 8b, the gross vehicle weight distribution essentially exhibits two peaks (bimodal), indicating that a number of multiple-axle trucks at 300 to 400 kN (67 500 to 90 000 lbf) and a larger number of buses and two-axle trucks at 50 to 150 kN (11 200 to 33 700 lbf) may be expected. In Brazil, at present truck traffic at the higher load levels is not so frequent as it is in the United States. However, the observed frequency distribution can be used together with U.S. experience to estimate the probable future distribution of truck traffic over the bridge. Figure 9 shows the estimated average daily truck traffic (ADTT), both observed and projected volume. A rate of increase of 1.5 percent is assumed based on Fisher's experience (8), described in a paper in this Record. If all the main arteries leading to the bridge are assumed to be complete by 1984, a higher rate of increase (3 percent) should then be assumed.

ANALYSIS OF WELDED DETAILS FOR FATIGUE SUSCEPTIBILITY

The results of the field stress measurements were used to evaluate the fatigue behavior of several welded details in the orthotropic deck. To examine the adequacy of the measured stress range histograms (for example, Figure 5c), the lateral (lane) positions of trucks were used together with the axle group weight probability density (Figure 8a) and the influence line for stress range at gauge 33 (Figure 7) to construct the probability density function for stress range at gauge 33. The computed results shown in Figure 5c are in reasonable conformity with the measured stress range spectrum for gauge 33 shown by solid lines in the figure. This comparison indicates that the measured stress range spectra at all gauge locations are a reasonable estimate of the expected stress ranges generated by the truck traffic on the bridge deck.

An examination of the orthotropic deck showed three major details requiring investigation:

1. The end weld of the splice plate on the bottom of the trapezoidal stiffeners (near gauge 33),
2. The weld connecting the trapezoidal stiffeners to the floor beams (near gauge 5), and
3. The partial-penetration rib-to-deck weld between the deck plate and the trapezoidal stiffeners (near gauge 21).

End Weld of Splice Plate

The splice plate constitutes an attachment plate more than 30 cm (12 in) long, which places it in category E of the 1974 AASHTO fatigue provisions (8, 11).

Two methods were used to evaluate cumulative damage due to random application of stress range and its frequency of occurrence. One is the root mean square method (10, 12) in which the root mean square (RMS) of the stress ranges in a spectrum becomes the equivalent constant cycle stress range and is correlated directly with the number of stress cycles in the spectrum. The RMS stress range S_{rRMS} is defined as

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

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

An RMS stress range of 22 MPa (3200 lbf/in^2) was computed for the spectrum shown in Figure 5c. This value is shown in Figure 10 and compared with the constant cycle stress range data (S-N curve) from laboratory studies (10). Figure 10 shows that the (99 percent survival) fatigue strength will be reached in about 40 000 000 cycles. If, at this detail, a truck is assumed to produce one stress cycle, this corresponds to about 70 years if the current ADTT is maintained.

Two conditions suggest that the fatigue strength may be reached earlier. First, the ADTT probably will increase with time (Figure 9). Second, the frequency of occurrence of heavier trucks will probably increase. The latter will cause S_{rRMS} to increase over the life of the structure. If the ADTT increases at a constant annual rate of 1.5 percent, the fatigue strength will be approached in about 50 years; if the rate of increase is 3.0 percent after 1984, it will occur in about 40 years. Higher rates of growth will obviously decrease the time required to reach the fatigue strength.

The stress range histogram in Figure 5c can also be evaluated by using Miner's hypothesis for cumulative damage (13). By combining the relationships provided by constant cycle data and Miner's rule, an equivalent stress range S_{rMINER} can be estimated as

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

This results in an equivalent stress range of 26 MPa (3800 lbf/in^2), which corresponds to about 20 000 000 cycles of constant cycle stress range as shown in Figure 10. This number of cycles will be reached in about 30 years based on the present frequency of truck traffic and an annual increase rate of 1.5 percent.

Based on the assumed rate of increase in truck traffic, both methods suggest that fatigue cracks might develop in about 30 to 40 years at this detail.

Connection of Trapezoidal Stiffeners to the Floor Beam

The trapezoidal stiffeners pass through the floor beams with cutouts as shown in Figure 5a. The floor beam web is fillet welded to the sloping sides of the stiffener. This detail is directly analogous to a transverse stiffener attached to the web of a girder. It constitutes a non-load-carrying connection and is classified as a category C detail by the current AASHTO fatigue specification (11).

As shown in Figure 5a, the highest recorded stress range in the bottom flange of the trapezoidal stiffener is about 59 MPa (8600 lbf/in^2). The RMS stress range of the stress spectrum is 17 MPa (2500 lbf/in^2); the Miner's equivalent stress range S_{rMiner} is 20.7 MPa (3000 lbf/in^2). The critical point for fatigue is the weld toe termination at the top of the cutout 25 mm (1 in) above the bottom surface. The stress range at this point is even lower. Since both values above are considerably less than the fatigue limit of 82.7 MPa (12 000 lbf/in^2) for category C details, no fatigue damage is expected at this connection, even with substantial increases in truck weights.

Connection of Trapezoidal Stiffeners to Deck Plate

Work in England on experimental orthotropic bridge deck panels showed that high compressive cyclic stresses transverse to the trapezoidal stiffener occur in the deck near the connection welds (3). This work led to an experimental program reported by Maddox (14). Studies were also made in the United States because of differing opinions on the requirements for the deck-to-stiffener connection. The resistance of full and partial penetration welds was examined (15).

The results of these studies are shown in Figure 11 where the bending stress range at the weld root is plotted

Figure 9. Estimated average daily truck traffic.

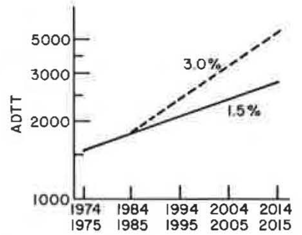


Figure 10. Fatigue strength estimated at end weld of splice plate near gauge 33.

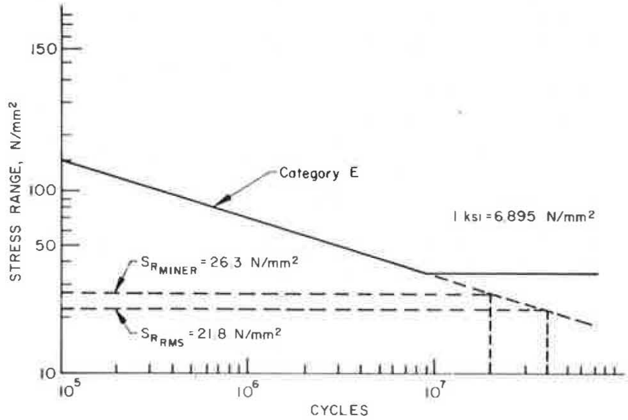


Figure 11. Fatigue strength estimate at connection of trapezoidal stiffener to deck plate at gauge 21.

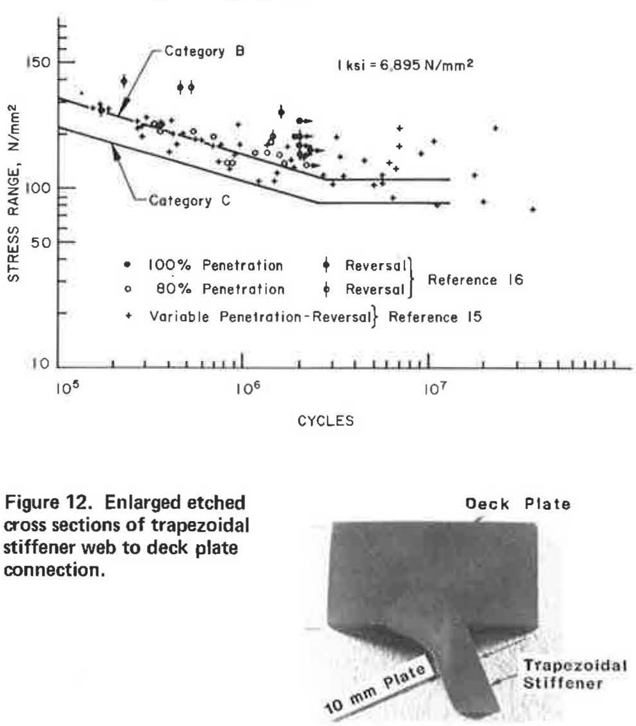
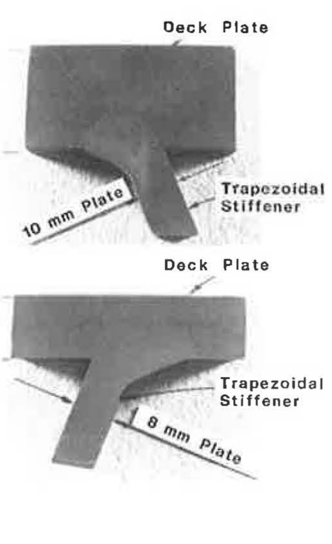


Figure 12. Enlarged etched cross sections of trapezoidal stiffener web to deck plate connection.



as a function of fatigue life. A comparison of the data points with current AASHTO design curves indicates that category C provides a reasonable lower bound to the test data. Maddox noted that results providing higher fatigue strength generally result from compressive stress at the weld root or the absence of tensile residual stress. This is particularly noticeable with the reversal specimens reported by Seim and Ferwerda (15). It is apparent that the full stress range is not effective in these specimens. The residual tensile stresses in the orthotropic deck of the Rio-Niteroi bridge should be high enough that the category C fatigue strength should realistically model the expected fatigue behavior.

Figure 12 shows etched cross sections of the stiffener to deck plate connection near floor beam 17 showing partial penetration welds. The average ratio of weld throat thickness to the sloping web plate thickness is 0.83. Hence, for bending moment about an axis parallel to the weld, flexural stress in the weld throat is about $(1/0.83)^2 = 1.45$ times greater than the web plate stress. Because of flexural stress gradient in the stiffener web between the weld and the bottom of the stiffener, the stress recorded at gauge 21 (Figure 6a) must be adjusted upward to provide the stress range at the weld. Because only a single gauge was attached to the surface of the stiffener, the stress gradient cannot be adjusted to account for stress gradient through the web plate thickness. However, based on the gauge location and the plate geometry, the stress at the weld was estimated to be about 15 percent higher than that at gauge 21. The weld throat stress therefore is about $1.15 \times 1.45 = 1.67$ times the stress in the stiffener at gauge 21.

The stress range spectrum for gauge 21 shown in Figure 5b is typical for stiffener webs in the vicinity of floor beam 17. In Figure 5b, the maximum stress range is about 32 MPa (4600 lbf/in^2) and S_{rRMS} is about 18 MPa (2600 lbf/in^2). At all locations of measurement, the maximum stress range seldom exceeded 33.8 MPa (4900 lbf/in^2); a single measured stress was 42 MPa (6100 lbf/in^2). Thus, on the average the adjusted maximum stress range on the weld throat is about 69 MPa (10 000 lbf/in^2), which is less than the fatigue limit shown in Figure 11. No failure should occur at any of the stiffener-to-deck plate connections because of transverse stress range. Even if heavier vehicles in the future generate occasional stress ranges exceeding the fatigue limit of the connection, the low value of RMS stress range (less than 40 MPa or 5800 lbf/in^2) suggests that no fatigue damage should occur throughout the life of the structure.

CONCLUSIONS AND RECOMMENDATIONS

A study of the stresses at selected locations of the orthotropic steel deck of the Rio-Niteroi bridge under a test truck of known weight and under random truck traffic indicates that the deck should provide satisfactory service throughout its life. However, the splice detail on the trapezoidal stiffener may have the potential for fatigue crack growth.

Specific conclusions developed from this study follow.

1. The observed stress range spectra at the welded details exhibited the same basic characteristics of those at welded details of many other highway bridges. The stress range spectra are highly skewed, which is comparable to the axle group load spectrum.
2. The strains developed in the orthotropic steel deck are essentially the same under static and moving traffic.
3. The bimodal distribution of truck and bus traffic exhibits the same characteristics as truck traffic in the

United States although the frequency of heavier trucks is currently not so great in Brazil.

4. Given the lateral position of traffic, the frequency and loading of trucks, probably only one stress cycle is significant in the fatigue analysis.

5. End welds of the trapezoidal stiffener splice plates are subjected to stress cycles that exceed the fatigue limit. At the current rate of loading visible fatigue damage may occur in about 30 to 40 years. If the frequency of heavier loads increases, such cumulative damage may occur at an earlier date.

6. There is little probability of fatigue damage to the partial penetration stiffener-to-deck plate welded connections. When fatigue cracks are detected then retrofitting by welding (16) could be effected. Very few of the random stress cycles exceed the fatigue limit.

7. There is little probability of fatigue damage to the stiffener-to-floor beam connections.

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