Rio-Niteroi Bridge: Thermal Field Studies

A. Ostapenko, Lehigh University

One side span of the three-span (200, 300, 200 m) continuous steel box girder bridge in Rio de Janeiro was instrumented for measuring strains and temperature during construction and in service. Electrical, mechanical, and scratch gauges were used. The field measurements, conducted in winter and summer, show that the daily temperature increase is the largest from 6:30 a.m. to 3:00 p.m. The high temperature change in the top flange drops within 0.30 m by as much as 13 to 15°C and then remains essentially constant in the rest of the cross section. A computer program was developed for generating a complete thermal field in the bridge from the temperatures specified at two cross sections and for performing thermoelastic analysis. The highest thermal stress was found to be 14.2 MPa. Recommendations are made for a temperature change pattern to be used in design of steel bridges under the Rio de Janeiro conditions and for the intensification of research in the United States on thermal effects in bridge structures.

Most commonly the effect of temperature on bridge girders has been considered by providing adequate travel at the expansion joints and bearings. The temperature gradient from the top of the roadway to the bottom fiber of the girder has generally been considered to be linear. Most specifications permit greater allowable stresses when the effect of temperature is included (1). Probably the most important consequence of temperature effects is the deformations, which must be accurately considered in construction. It is only recently that engineers have begun to give attention to the thermal phenomena and that a worldwide trend has developed toward better understanding of the behavior and strength of bridges.

Field measurements of temperature distribution have been conducted mainly on prestressed concrete bridges but also on some steel and composite bridges (2). A theoretical approach has been proposed for computing the temperature distribution in a bridge girder from the ambient conditions (3). These studies have shown that the temperature in a bridge cross section may deviate considerably from the linear pattern usually assumed in design. Reactions of a continuous concrete girder were measured in France as recently as 1975. When a sim-

plified version of the method developed by Emerson (3) was used to calculate thermal stresses from these reactions, the stresses were found to be of potential intensity to induce cracking of concrete (4). Analysis of a small concrete model tested under a thermal load gave good correlation with theory (5). Some sample concrete bridges were also analyzed for thermal conditions in North America (6).

Although the differential equations and principles of thermoelastic analysis are well established, their application to bridge girders is hampered by the complexity of the cross sections, uncertainties of thermal material properties, uncertainty about the surface interaction of the structure, air, and sun rays, and uncertainty about the actual temperature and wind conditions. The problem is unmanageable unless many approximations and simplifying assumptions are made, and therefore the validity of such analyses cannot be accepted unless the results are confirmed by field measurements of the actual temperature distribution in bridge girders. However, only a few and very limited measurements have been conducted so far (2).

The instrumentation program of the Rio-Niteroi bridge (Ponte Presidente Costa e Silva) in Brazil presented an opportunity for obtaining extensive temperature and strain data on a steel box girder bridge of unprecedented dimensions (7, 14).

RIO-NITEROI BRIDGE AND SCOPE OF FIELD STUDIES

The Rio-Niteroi bridge joins the cities of Rio de Janeiro and Niteroi over the Guanabara Bay and has an overall length of 13 km with 8.5 km over water. Most of the bridge is composed of 80-m prestressed concrete spans of segmented construction. Portions continuous over five spans are provided with expansion joints. Except in the overland approaches, the bridge has a uniform twin-box cross section. The main shipping channel is bridged by a three-span continuous steel box girder of a matching twin-box cross section but of a variable depth. This steel girder has two 200-m side spans and a 300-m center span, which is the longest in the world for unstayed box or plate girder bridges.

Figure 1. Location of instrumented sections in Rio de
Janeiro side span.
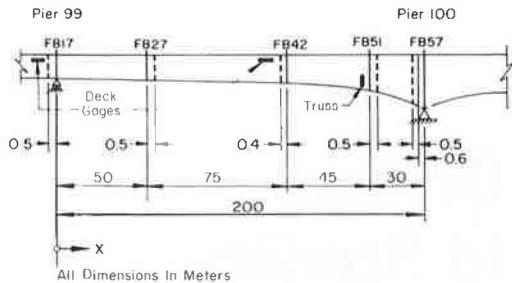
Figure 2. Wiring system for static readings on
completed bridge.
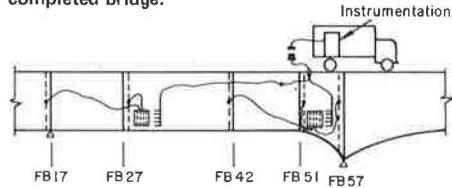
A special bridge authority, S. A. Empresa de
Engenharia e Construção de Obras Especiais (ECEX),
originally named S. A. Empresa de Construção e
Exploração da Ponte Presidente Costa e Silva, was
charged by the Brazilian department of highways (DNER)
with the planning and construction of the whole bridge (7).
As described by Graham in a paper in this Record,
structural design and construction supervision of the
steel part of the bridge were carried out by an American
engineering firm (8). The steel girders were fabricated
in large panels in Great Britain and then shipped to Rio
de Janeiro. There, the panels were assembled and
erected by the joint venture of three companies, two of
Great Britain and one of Brazil (9, 10, 11).

Many novel aspects of both the concrete and the steel
portions of the bridge made it advisable to conduct a field
investigation during and after construction. Laboratorio
Nacional de Engenharia Civil (Portugal) was contracted
for the concrete portion and Lehigh University for the
steel portion (7, 12).

The principal parts of the field study on the steel
bridge were as follows:

1. Monitoring of some selected stress points during
construction for deviations from computed values,
2. Measurement of stresses during construction over
full cross sections,
3. Measurement of stresses under test loads on the
completed bridge,
4. Study of stresses in the orthotropic deck under
regular traffic conditions, and
5. Study of the temperature distribution and its ef-
fects on the bridge.

The thermal study (item 5) is reported here.

INSTRUMENTATION OF STEEL STRUCTURE

Location of Instrumented Sections

The instrumentation on the steel portion of the bridge
consisted of electric resistance gauges (strain and tem-
perature), a mechanical gauge, and scratch gauges.
The instrumentation was installed in the Rio de Janeiro

side span primarily because of the logistics of construc-
tion at the time when the field work on instrumentation
was started in September 1973.

Figure 1 shows the location of the instrumented sec-
tions. The view is from the south with the 300-m center
span adjoining on the right. The five instrumented cross
sections are indicated by the vertical dashed lines posi-
tioned to the right or left of the cross sections labeled
FB17, FB27, FB42, FB51, and FB57 (the floor beams in
the orthotropic deck). The sections at FB27 and FB57
have strain and temperature gauges; the sections at FB17,
FB42, and FB51 have only strain gauges. Strain gauges
were also placed in the north box on the components of
the orthotropic deck at FB17 and FB42 (labeled deck
gauges in Figure 1) and on the truss reinforcing the bot-
tom flange at FB51.

Electric Resistance Gauge System

The electric resistance gauges consisted of (a) 52 longi-
tudinal strain gauges parallel to the girder axis, (b) 82
transverse strain gauges prependicular to the girder
axis, mainly for measuring bending strains in the trans-
verse stiffeners and floor beams, (c) 57 three-gauge
strain rosettes 45 deg to each other for measuring shear-
ing and principal stresses (57 × 3 = 171 single-gauge
equivalents), and (d) 52 temperature gauges. Thus, total
single-gauge equivalents were 422. This number included
8 longitudinal gauges, 36 transverse gauges, and 7 ro-
settes (21 single-gauge equivalents) used for the dynamic
strain readings in the orthotropic deck (13). The re-
maining 357 single-gauge equivalents were read statically
in four groups by using an automatic data acquisition unit
with 100 channels.

A schematic of the wiring system used for making the
static readings of the 357 strain and temperature gauges
on the completed bridge is shown in Figure 2. The cables
leading from the gauges at FB17 and FB27 were as-
sembled on a switching panel at FB27 and from FB42,
FB51, and FB57 on a switching panel at FB51. For tak-
ing readings, one-quarter of the gauges at each switching
station were connected to the cables leading to a set of
plugs housed in a special junction box installed in the
concrete curb near FB51. The data acquisition unit was
housed in a truck and plugged into the junction box.

To minimize inaccuracies, the switching from one
group of gauges to another was accomplished by soldering.
When it was desirable to take readings in quick succes-
sion, as many as 76 strain and 14 temperature gauges
were left connected continuously.

Sections Instrumented for Temperature Readings

The cross sections chosen for mounting temperature
gauges were at FB27 and FB57. As shown in Figures 1
and 2, the section at FB27 has an average depth of 6.34 m,
is within the span, and thus is subjected to the cooling
effect of wind. The other section at FB57 is the deepest
one in the bridge (13.04 m), has a large mass, and is
partially shielded from the direct effect of wind by the
turbulence created by the pier shafts below it. Thus,
the thermal conditions at these two sections were ex-
pected to be most contrasting.

The temperature gauges were of the electric resis-
tance type and were read by using the same equipment as
used for strain gauges. In addition to temperature gauges,
a few thermocouples were also installed in the girder,
but essentially all of them were damaged during construc-
tion and no systematic readings could be taken.

Mechanical Gauge System

Mechanical and scratch gauges provided alternate backup systems to the electric resistance gauges and supplied some additional information on the behavior of the bridge.

In the mechanical gauge system, the distance between two small target holes predrilled at a series of locations in the bridge girder was measured by means of a special mechanical gauge. The difference between two readings taken at different times at a particular location divided by the distance between the two target holes gave the strain change produced by a change in loading and temperature between the readings. A total of 83 locations were prepared for these readings.

By using a standard temperature-stable reference bar, the errors that may have been introduced by the variation in the adjustment of the mechanical gauge were eliminated, and two sets of readings taken at a considerable time interval—days, weeks, or months apart—could be directly compared. Thus, unlike the electric gauges, the mechanical gauge provided a means for measuring strain changes over long periods of time.

The three sets of mechanical gauge readings on the completed bridge, taken in March 1974, June 1974, and January 1975, confirmed the gross elongation or shortening of the girder produced by the seasonal temperature changes and indicated that no unexpected strain changes had taken place.

Scratch Gauges

Eight scratch gauges with 1-m gauge length (initially, 0.305 m) were installed in the Rio-Niteroi Bridge. They were an important backup system during construction. On the completed bridge, their principal function has been to record the stress fluctuations caused by the diurnal temperature changes and unexpected occurrences. Although not so accurate as the electric or mechanical gauges, the scratch gauges can provide a permanent and continuous record of stress changes for several months or even years.

TEMPERATURE READINGS

The temperature gauges were read concurrently with strain gauges. However, because construction and test load readings for the most part were taken at night when the temperature in the bridge was essentially uniform, these readings are not discussed here. After the bridge was completed, the following three sets of readings were taken specifically to study the thermal behavior of the bridge: (a) in February 1974 for finding the temperature distribution on a summer day, (b) in June 1974 for determining the temperature variation at some selected locations with respect to time and (c) in January 1975 for finding the total diurnal temperature change.

Additional information on the thermal behavior of the bridge was obtained from scratch gauge targets.

Temperature Distribution in Cross Section

The readings on February 27, 1974, were taken to establish the temperature distribution in bridge cross sections on a typical hot summer day. The temperature was measured around noon (Figure 3). The heavy solid circles and solid ordinates indicate the places where particular gauges were located. A small circle with a dashed ordinate gives the value transferred from an adjoining plate where the gauge was actually located. The readings show some scatter, but this can be readily ascribed to the later recognized fact that the temperature-time gradient from 11 a.m. to 1 p.m. during which the readings were taken is quite steep and can readily account for discrepancies of 3 to 4°C.

The following observations can be made from the figure:

1. The temperature on each of the four webs is essentially constant over the full depth except for a very sharp increase at the top to the temperature of the top flange;

2. The temperatures in the two outside webs are equal to each other and are approximately 4°C higher than in the two inner webs;

3. The air temperature in the boxes at the lower levels, as measured by the gauges on the bracing diagonals, is equal to the temperature of the outside webs, and, at the top, it is higher and equal to the air temperature outside between the boxes;

4. The temperature in the top flange plate is the highest in the middle of a box (up to more than 42°C) and then drops a few degrees at the webs because of their heat-sink function;

5. The flange tips have markedly lower temperatures than the rest of the flange (more than 10°C lower) because of the concrete curb and railing over them; and

6. Comparison of Figure 3 with a similar plot for FB27 showed no recognizable difference between the temperature distributions that could be ascribed to the different locations along the girder.

Variation of Temperature Distribution With Time

Temperature variation in the bridge cross section with respect to time was determined on June 10 and 11, 1974. No soldering switchings were made, and the same group of 12 temperature and the accompanying 76 strain gauges were read over and over at short time intervals. The selection of gauges was based on the fact that the sun at this time of the year (Brazilian winter) is low in the north (to the left of the cross sections in the figures).

The temperature readings of June 10, 1974, at FB27 were plotted against time (Figure 4). The temperature in the section is essentially uniform and equal to the air temperature at about 9 a.m.; it reaches the highest value at about 2 to 3 p.m. and then decreases toward 5 p.m. The gauge on the inner web gave somewhat lower overall readings than other gauges, apparently because of a malfunction.

Although the air temperature changed through the day by only 5°C, the bridge temperature increased by as much as 14°C. The observation, relevant to the readings made in February 1974, is that the temperature rose quite rapidly between 11 a.m. and 1 p.m. and developed the greatest difference between individual points in the cross section only later, at 2 to 3 p.m.

Figure 5 shows the temperature variation at FB57 on the same day. The pattern is very similar, but it exhibits some additional characteristics. The gauge at the bottom of the north web (hollow circles) shows a very rapid temperature increase under the early exposure to sun rays. At 9 a.m., when the temperature of the top flange was equal to the air temperature, the temperature of the bottom of the web had already risen 5°C and was increasing faster than in other components. Gradually, the temperature in the web leveled off and started decreasing at 2 p.m., that is, before the other points reached their maxima. The gauge at the left tip of the top flange (hollow triangles) hardly warmed up, and its temperature differed from that of the flange itself by about 10°C—the same as was observed in February 1974.

Although the temperature difference in Figures 4 and 5 between the points on the flange and those immediately under it on the web is only about one-third or one-half of

that observed in February, the patterns are in good agreement.

Maximum Diurnal Temperature Change

A diurnal temperature change needed for thermoelastic stress analysis of the bridge could not be properly established either in February or in June 1974, and the January 1975 readings were designed to accomplish this. The readings were taken from 11 p.m. on January 16, 1975, until 5 p.m. the next day to determine not only the temperature variation at certain locations but also the full temperature distribution at times several hours apart.

Figure 6 shows the maximum diurnal temperature change at FB57 between 6:30 a.m. and 3 p.m. on January 17, 1975. The pattern is very similar to the temperature distribution pattern in Figure 3 for February 1974, except that the distribution in the top flange is less accurately defined, inasmuch as many gauges had been damaged in the interim.

Given the symmetry about the vertical axis and the equivalence of temperatures at FB57 and FB27 (not shown here), all the readings of the January 1975 temperature change are shown in Figure 7 on the outline of the north box (the flange tip is on the left side). A pattern of the maximum temperature change was then approximated from the composite plot as shown in the figure by the dashed line. This pattern is also superimposed on the cross section of Figure 6 for a direct comparison with the readings on it.

Inasmuch as meteorological records indicate that air temperature on January 17, 1975, was the highest since 1973, the pattern of the diurnal temperature change in Figure 7 was accepted as a typical maximum pattern.

Scratch Gauge Records of Thermal Stresses

After the completion of the bridge, a typical scratch gauge record was in the form of a continuous zigzag trace, indicating periodic stress changes. Because a day-by-day correlation could be established between the zigzags of the scratch gauge traces and of the meteorological temperature record, these stress changes were ascribed to daily temperature fluctuations. The effect of traffic had to be excluded because the stresses computed for the current traffic level were found to be much lower than those recorded, and also they fluctuated many times rather than once during the day.

Figure 8 shows two example segments of scratch gauge traces in which the highest stress changes were recorded during 1974. One is from the gauge on the bottom of the web at FB27, and the other is from the gauge on the bottom of the web at FB42. The daily stress change of approximately 52 MPa shown by these traces is much higher than the thermal stress computed for the daily temperature changes measured in the bridge. This discrepancy can be plausibly and approximately quantitatively explained by the temperature difference between the gauge and the girder metal produced by the insulating effect of the paint and the air clearance between them. Thus, the temperature in the gauge was much closer to the temperature of the air inside the box than that of the plate to which the gauge was attached. Consequently, the much slower temperature changes of the inside air through the day had a significant magnifying effect on the scratch gauge trace (14). Only a 5 to 9°C difference was sufficient to explain the discrepancy.

THERMOELASTIC ANALYSIS

General thermoelastic analysis of bridges requires that two problems be resolved: (a) the determination of the temperature change pattern and (b) the actual stress analysis. The temperature change pattern can be computed theoretically, but at present only a few simple cases could be successfully analyzed (3, 4, 5, 6). Or the pattern can be measured on the actual structure as was done in this study.

If the commonly made assumptions about the linearity of stress-strain and temperature-strain relationships and about the planar nature of deformations of a cross section are valid, two types of normal stress develop in a girder cross section subjected to a nonplanar temperature change. One represents the self-balancing stress field resulting from the nonplanar nature of the temperature change, and it is affine to the temperature pattern. The other type of stress is produced by the redundant forces of the statically indeterminate structure, is planar, and is zero for statically determinate girders (5, 6). In equation 1, these two types of stresses are given by the first and second bracketed terms.

$$\sigma(y,z) = \left[-\alpha ET + (1/A) \int \int E\, \alpha\, T\, dydz + (y/I_z) \int \int E\, \alpha\, Ty\, dydz \right.$$
$$+ (z/I_y) \int \int E\, \alpha\, Tz\, dydz \left. \right]$$
$$+ [(1/A)(\|n_i\|^T\, \|X_i\|) + (y/I_z)(\|m_{iz}\|^T\, \|X_i\|)$$
$$+ (z/I_y)(\|m_{iy}\|^T\, \|X_i\|)] \qquad (1)$$

where

a, E, T = coefficient of thermal expansion, modulus of elasticity, and temperature change (each may be a function of the coordinates in the cross section),

A, I_z, I_y = area and moments of inertia of the cross section,

n_i, m_{iz}, m_{iy} = axial force and banding moments produced by the i th redundant $X_i = 1$,

X_i = final value of the i th redundant, and

T = transpose of the $\|n_i\|$, $\|m_{iz}\|$, and $\|m_{iy}\|$ column matrices.

Method Developed for the Rio-Niteroi Bridge

The method used for thermoelastic analysis of the Rio-Niteroi bridge was based on the above principles. The bridge is a three-span continuous girder with 5 deg of indeterminacy, three in the vertical plane (two vertical reactions and one horizontal, at the center pier) and two in the horizontal plane (the four piers are treated as elastic supports for twisting and horizontal bending of the girder produced by an unsymmetrical variation of temperature across the width). Longitudinally, the girder is subdivided into thirty-five 20-m segments.

Each midsegment cross section is broken down into 171 elements as shown in Figure 9 for a section near FB57. The flange and web plates consist of relatively wide elements except at the top of the webs, where, in anticipation of a steep temperature gradient, the elements are only 0.10 m wide. Each longitudinal stiffener is treated as an individual element. The numbering system of shallower sections is kept the same, but the dimensions are modified by reducing the width of or completely eliminating the lowest web elements.

The temperature distribution is taken in the computer program to be symmetrical about the midlength of the bridge although it does not have to be symmetrical in the cross section. The temperature values for all the seg-

ments are generated from the temperatures at FB27 and FB57, which are used as input data.

Although the computer program was developed primarily for the Rio-Niteroi bridge, it can be modified for other three-span, four-web box or plate girder bridges.

Thermal Stresses

Thermoelastic analysis of the bridge girder was performed for a number of temperature change patterns. The thermal stresses computed at FB42 for the January 1975 pattern (Figure 7) are shown in Figure 10. The

maximum compression in the top flange and at top of the webs is 11.6 to 14.2 MPa. The tension in the web below the top flange is about 11.6 MPa and in the bottom flange 7.5 MPa.

The stresses at the section of FB42 are representative of the higher stresses obtained for other sections along the girder. Yet, for this bridge, the stress levels are quite low; they can be readily covered by the 25 to 33 percent increase in allowable stresses generally permitted when thermal effects are added to the effects of dead and live loads.

Figure 3. Temperature distribution at FB57 on February 27, 1974.

Temperature Ordinates are Reduced by 20° C
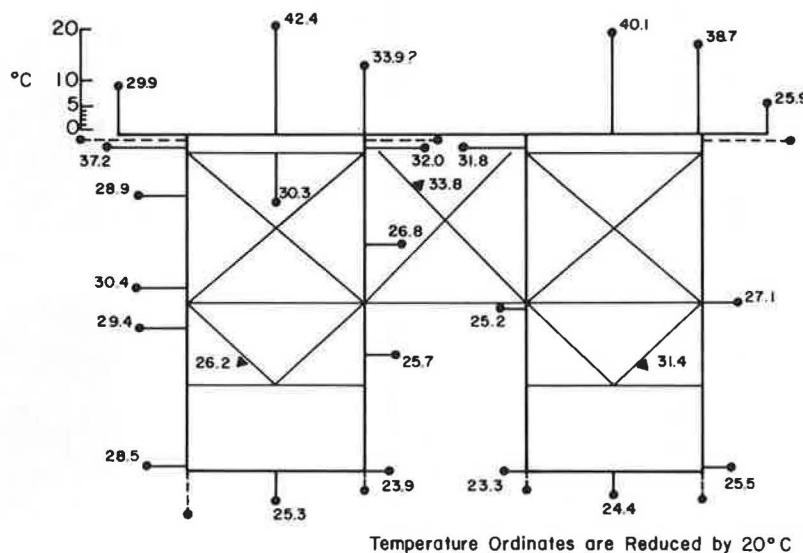
Figure 4. Temperature versus time at FB27 on June 10, 1974.
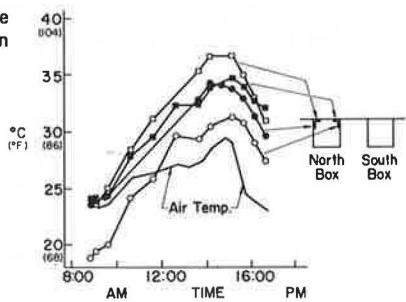
Figure 5. Temperature versus time at FB57 on June 10, 1974.
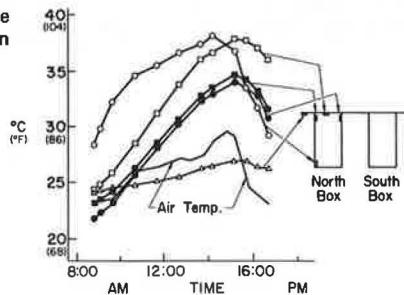
Figure 6. Diurnal temperature change at FB57 on January 16 and 17, 1975.
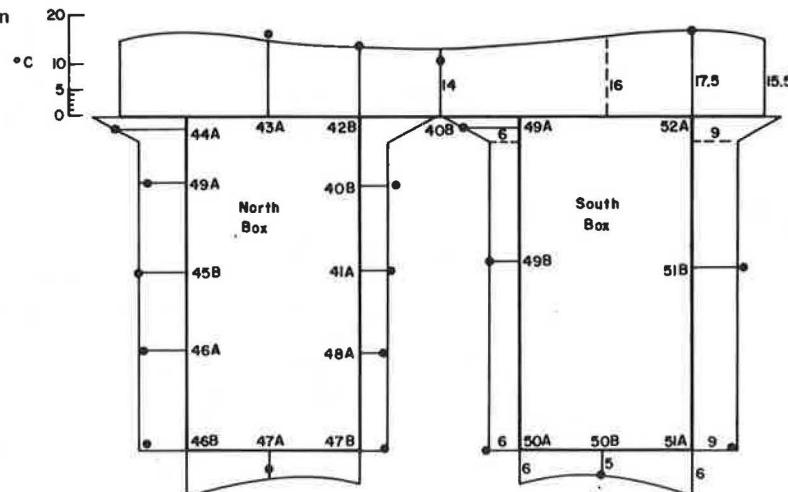
Design Recommendations for Diurnal Temperature Change in Steel Bridges

This thermal study leads to the conclusion that the di-

urnal temperature change pattern observed is typical of steel girder bridges under the climatic conditions of Rio de Janeiro. A simplified and conservative version of the recommended pattern is shown in Figure 11.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions can be made from this work.

1. The temperature distribution in the cross section

Figure 7. Composite temperature changes at FB57 for January 16 and 17, 1975.

Δ FB 57 North
• FB57 South
○ FB 27 North
□ FB 27 South

Figure 8. Scratch gauge traces with diurnal stress changes 235X; 0.305-m gauge length.

FB 27 BOTTOM

FB 42 BOTTOM
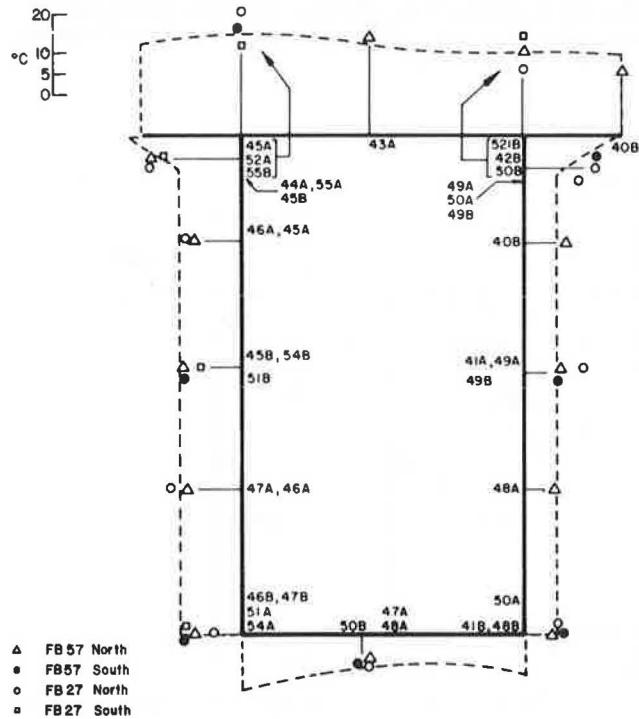
Figure 9. Breakdown at FB57 for thermoelastic analysis.
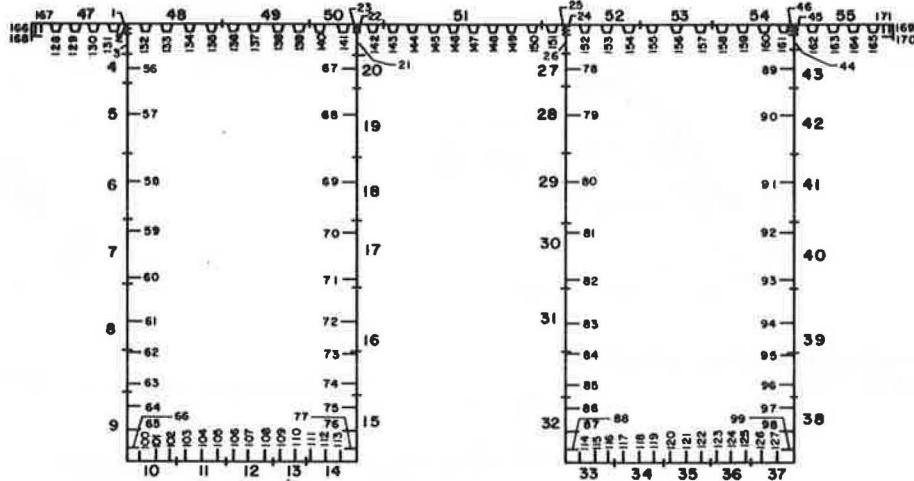
**Figure 10.** Thermal stresses at FB42 during January 1975.
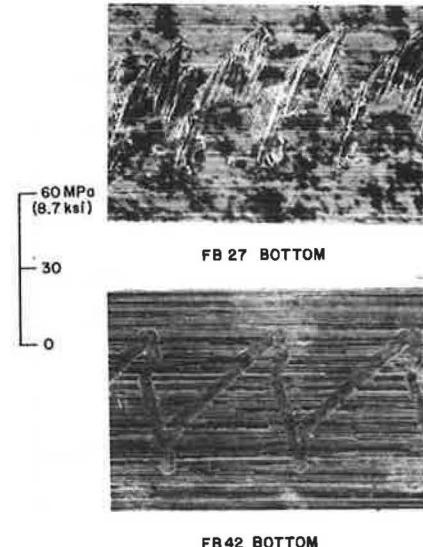
Figure 11. Temperature change recommended for climatic conditions of Rio de Janeiro.
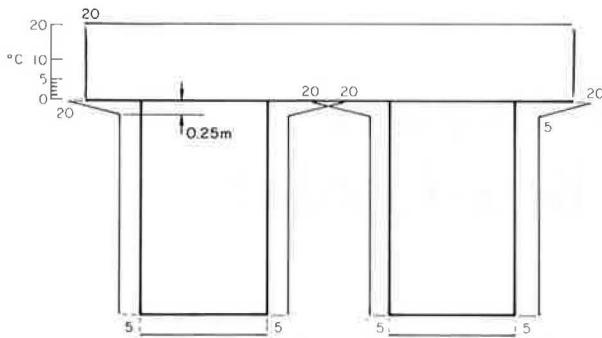
showed a sharp gradient at the very top of the webs and an essentially constant temperature for the rest of the web.

2. The greatest change in temperature took place between 6:30 a.m. and 3 p.m.

3. There was no detectable difference in temperature distribution between a deep section at the pier and a shallow section at midspan.

4. The greatest normal stress due to a diurnal change of temperature was found to be only 14.2 MPa. However, a girder of different proportions may develop much higher stresses.

5. Scratch gauges were effective in recording stresses, but they require very good insulation from the outside air if they are to have the same temperature as the structure and measure thermal stresses accurately.

6. A prudently conservative temperature change to be considered in design of similar steel bridges under the climatic conditions of Rio de Janeiro is recommended as shown in Figure 11.

The lessons learned in this field study conducted under the conditions in Rio de Janeiro point to the need for similar investigations in the United States so that the AASHTO design recommendations can reflect more realistically the true field conditions. Specifically, the following recommendations are made.

1. Conduct extensive theoretical and field studies on concrete and steel bridges under at least two extreme climatic conditions in the United States.

2. Install a few temperature measuring and recording devices in some bridges in the vicinity of meteorological stations. Information gathered should help in the correlation of theoretical and short-term experimental work recommended above.

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