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COLLINGSWOOD AND WESTMONT VIADUCTS STUDY

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ABSTRACT

The Collingswood and Westmont Viaducts are situated at the stations having the same names along the Delaware River Port Authority's (DRPA) Lindenwold-Philadelphia Rapid Transit Line. The Collingswood Viaduct is 2,522 feet long. The Westmont Viaduct is 1,970 feet long. Each viaduct consists of a reinforced concrete deck supported mostly by four simply supported prestressed concrete I-girders. Spans average a little over 70 ft. Both viaducts carry four continuous rails anchored to the deck with no provision for expansion and contraction over or beyond the viaducts. In recent years the two viaducts are observed to have experienced substantial and undesirable structural distress. It is believed that the viaducts may not be functioning as designed with respect to thermal response. Fritz Engineering Laboratory, subcontractors to Gannett, Fleming, Corddry and Carpenter, Inc., of Harrisburg, PA, measured strains in the rails, girders and decks of both viaducts over a six month period; determined approximate stresses from the measured strains using a simplified model; and performed laboratory tests on the compression rail anchor system. A finite element study of the viaducts was also performed outside the scope of the work reported herein. This work is reported in Ref. 2.

1. INTRODUCTION

1.1 Background

The Collingswood and Westmont viaducts are situated at the stations having the same names along the Delaware River Port Authority's (DRPA) Lindenwold-Philadelphia Rapid Transit Line. The viaducts were originally designed in steel in 1966. Alternate bids were allowed and the viaducts were subsequently re-designed and constructed in prestressed concrete.

Each viaduct carries two tracks, with a widened portion to accommodate a station. The Collingswood Viaduct consists of 34 spans with a total length of 2,522 feet. The Westmont Viaduct, one mile east of the Collingswood Viaduct, is a 27 span structure having a total length of 1,970 feet. Both viaducts consist of a reinforced concrete deck supported by four simply supported prestressed concrete I-girders, except in the vicinity of stations, where the number of girders is increased. Both viaducts carry four continuous steel rails anchored to the deck with compression rail anchors, with no provision for expansion or contraction of the rails over the full length of each viaduct.
The Collingswood Viaduct is straight. The Westmont Viaduct is on a horizontal curve. The westbound track is curved to a radius of 9,820 feet throughout. The eastbound track is irregular. Two guard rails are also fastened to each viaduct.

In recent years the Collingswood and Westmont Viaducts are observed to have experienced substantial and undesirable structural distress. This distress has resulted in slight to moderate structural damage.

It is believed that the viaducts may not be functioning as designed with respect to thermal response.

1.2 Scope of Study

Gannett Fleming Corddry and Carpenter, Inc. (GFC&C), Harrisburg, PA, contracted with DRPA to perform engineering services in connection with the above viaducts. A description of these services is contained in their proposal.(1) Lehigh University subcontracted with GFC&C to carry out part of these engineering services. The work was performed by Fritz Engineering Laboratory, Lehigh University.

The scope of work includes:

1. The determination of the change of strains in the rails, concrete deck, and the prestressed concrete I-girders due to temperature changes in the Collingswood and Westmont Viaducts over a six month duration.
2. Conversion of strain data to stress changes in rails, deck and girders by means of a simplified analytic model so that correlation can be made by GFC&C with relevant viaduct component displacement data obtained by GFC&C.
3. Determination of the amount of restraint provided by the compression rail anchors as a function of bolt torque.
4. Preparation of a report.

Instrumentation of the viaducts was carried out in the spring and summer of 1980. Data was obtained during August, September and November 1980 and in February 1981.

Tests were conducted in Fritz Engineering Laboratory to determine the effect of varying compression rail anchor bolt torque on the force required to produce slip between a rail and tie plate.

Lavanchy conducted a parallel analytical study during 1980 of the effect of temperature changes on the Collingswood and Westmont Viaducts as part of a comprehensive study, for the MS degree, of concrete railroad viaducts with maintenance free track support system.[2] Although Lavanchy's study is not part of the scope of work described herein, his results should be studied together with the work reported herein since they do provide valuable insight into the theoretical behavior of the two viaducts.

Possible approaches to comprehensive finite element analyses of the Collingswood and Westmont viaducts following concepts similar to those used by Lavanchy are discussed in Chapter 3. The reduction of measured strains and temperatures by Fritz Engineering Laboratory plus measured displacements by GFC&C, to changes in stresses in the rails, deck and girders, using a finite element analysis is not included in the scope of work and is not performed.

Approximate changes in stresses in the rails, deck and girders are computed using alternative simplified models. These models directly consider the change in measured strains and temperatures but only indirectly consider the other variables which affect the stresses.

2. INSTRUMENTATION OF THE VIADUCTS

2.1 Locations of Instrumented Areas

Figures 1 and 2 show the locations of the instrumented areas of the Collingswood and Westmont Viaducts. These locations were chosen by Lehigh University and Gannett, Fleming, Corddry and Carpenter, Inc., personnel with two general objectives in mind: (1) to instrument 2 spans remote from the station area and on a straight or reasonably straight segment of viaduct, and (2) to instrument over fixed piers at the location of observed distress to the viaducts.

Instrumentation of the Collingswood Viaduct consisted of electrical resistance strain and temperature gages on the rails, deck and girders primarily in spans 12 and 13. Additional strain and temperature gages were placed on the rails over piers 13 and 23.

Instrumentation of the Westmont Viaduct consisted of electrical resistance strain and temperature gages on the rails, deck and girders primarily in spans 10 and 11. Additional strain and temperature gages were placed on the rails over piers 11 and 15.

2.2 Instrumentation of Rails, Deck and Girders

Figure 3 shows details of the instrumentation on the rails of the Collingswood and Westmont Viaducts. The gages are numbered from 1 through 22 and are all mounted at the location of the neutral axis on the inside of the inner rails of the two tracks as shown in the typical cross section. All are strain gages except numbers 12, 15 and 22 which are temperature gages. The temperature gages are mounted adjacent to the strain gages. Gage 10 was not used and is not shown in the figure. All strain gages are located midway between tie plates which are at 30 in. spacing.[2,3]

The strain gages used are 1/4 in., 120 ohm electrical resistance foil gages. The temperature gages are 1/8 in., 50 ohm electrical resistance foil gages. All gages are mounted using epoxy cement. All gages are waterproofed and protected with a coating of butyl rubber covered with an aluminum shield taped to the rail.

The shield is used to protect the gage from thermal radiation coming from direct or reflected sunlight. The strain gages are oriented to measure longitudinal (axial or flexural) strain in the rail. The orientation of the temperature gages is immaterial.

Figures 4 and 5 show details of the instrumentation on the deck and girders of the Collingswood and Westmont Viaducts. All gages are located at midspan of the respective spans indicated in the figures. The gages numbered 1 through 12 are all strain gages except numbers 9, 10, 11 and 12 which are temperature gages. The temperature gages are mounted adjacent to the strain gages. The gages numbered 13 through 24 are all strain gages except numbers 21, 22, 23 and 24 which are temperature gages.

The strain gages used are 2 in., 120 ohm electrical resistance paper backed gages, mounted using Duco cement. The temperature gages are 1/8 in., 50 ohm electrical resistance foil gages mounted using epoxy cement.

All gages are waterproofed and protected as described above for the rail gages. The strain gages are oriented to measure longitudinal (axial or flexural) strains in the deck and girders. The orientation of the temperature gages is immaterial.

A 1/8 in., 50 ohm electrical resistance foil gage was located out of direct sunlight and near the indicator used to measure air temperature. This gage was placed between clear plastic strips and suspended in air clear of surrounding heat sources.

Figure 6 shows the completed installation of a strain gage to a rail. When a strain gage and temperature gage occurred together they were mounted adjacent to each other and protected with one covering. The aluminum protective shield is visible in the figures.

Figure 7 shows a strain gage installation near the bottom of a prestressed girder.

2.3 Data Acquisition System

The data acquisition system was designed and assembled specifically for this project. A major design requirement was that consistent readings be obtained from all strain and temperature gages placed on the viaducts over the duration of the data acquisition phase of the project which was expected to extend over at least six months. Another major design requirement was that the data acquisition system itself not influence the readings.

Rail gages 1 through 22 of the Collingswood Viaduct are connected to a terminal box permanently mounted on a safety rail post above pier 12, using a 3-wire circuit to each gage which automatically compensates for variable wire lengths between the terminal box and each gage. The safety rails are shown in Figs. 4 and 5. The installation of wiring to a terminal box is being performed in Fig. 8. When wiring is completed the terminal box has a weather proof enclosure plate in which are mounted two weather-proofed sockets. During each data acquisition period the portable strain and temperature indicators are plugged into the sockets. A similar system is used at the Westmont Viaduct, where the terminal box is mounted on a safety rail post above pier 10.

The terminal box contains a set of three 120 ohm temperature stable resistors for each active rail strain gage which results in a full bridge circuit for every rail strain gage. Each terminal box also contains one set of four, full bridge, 120 ohm highly stable resistors which establishes an accurate zero reference for every active rail strain gage. The reference for the temperature gages is located near the indicators used to measure temperature and employs a 75°F reference temperature.

For each viaduct, the deck and girder gages numbered 1 through 24 are also connected to a terminal box using a 3-wire hookup to each gage. The terminal boxes are located on top of piers 12 and 10 under the decks of the Collingswood and Westmont Viaducts respectively.

As for the rail strain gages both terminal boxes contain a set of three 120 ohm temperature stable resistors for each active deck and girder strain gage which results in a full bridge circuit for every strain gage. Both of these terminal boxes also contain a set of four, full bridge, 120 ohm resistors to establish an accurate zero reference for every active deck and girder strain gage. Similarly the reference for the temperature gages is also located near the indicators used to measure temperature.

Strain and temperature readings of all gages are taken using two portable P-350A Vishay Strain Indicators, one for strains and one for temperatures. Both indicators are connected to a switch box which is capable of switching to any strain or temperature gage on a particular viaduct or to the air temperature gage located near the temperature indicator. The $75^{\circ}F$ reference temperature instrumentation is located near the temperature indicator. Prior to each temperature reading the indicator was "zeroed" to this reference in order to eliminate temperature shifts in the connecting wires.

Figure 9 shows a view of the switch box (left) and the two indicators (right). The instruments are transported in the vehicle shown in the figure, which can be driven into position below the terminal boxes on each viaduct. The switch box is connected to the two terminal boxes at each viaduct by the two cables shown in Fig. 9. One cable consists of wires leading to two plugs which are plugged into the two sockets in the terminal box located on the safety rail post. The other cable consists of wires leading to two plugs which are plugged into the two sockets in the terminal box located on the pier. A four-wire full bridge circuit was used for each gage connection between the terminal boxes and the switch boxes.

Each full bridge circuit containing one active gage is not broken either during the switching operation or when cables are plugged into or unplugged from a terminal box. Thus the gage readings are not influenced by the data acquisition system itself. The full bridge circuits which employ stable resistors to maintain a constant reference for both strain and temperature gages ensures consistent readings over the duration of the data acquisition period.

3. RECORDED DATA AND STRESS COMPUTATION PROCEDURES

3.1 Data Recorded from the Collingswood Viaduct

Table 1 shows the strain and temperature data obtained from the two Vishay Strain Indicators (Art. 2.3) at the Collingswood Viaduct on August 10 and 17, 1980, November 16, 1980 and on February 7, 1981. The start and stop times shown in the table indicate the time of the beginning and the end of taking a set of data. Two sets of data with a minimum time interval between, were taken each day. The air temperature reading is the temperature recorded by the 1/8 in., 50 ohm electrical resistance foil gage located in the shade near the temperature indicator (Art. 2.3). The gage numbers correspond to those shown on Figs. 3, 4, and 5. The sets of strain readings taken on August 10, 1980 are the reference or "zero" set for all other strain data in Table 1 and have no meaning themselves. (The same is true for Tables 2 and 3 discussed below.) The two girder temperature readings shown in parentheses are estimated values. Temperatures are given in degrees Fahrenheit. Strains are given in micro inches per inch, that is, the values shown are to be multiplied by 10^{-6} in./in.

Table 2 shows the strain and temperature data obtained from the two Vishay Strain Indicators at the Collingswood Viaduct at half-hour intervals during the day on September 15, 1980. A complete set of data was taken hourly. Temperature data only was taken on the half-hour, except for the last set of readings for the day where girder strains were also recorded.

3.2 Data Recorded from the Westmont Viaduct

Table 3 shows strain and temperature data obtained from the two Vishay Strain Indicators at the Westmont Viaduct on August 17, and November 16, 1980, and on February 7, 1981. Four sets of data were taken on August 17, 1980. Two sets of data were taken on November 16, 1980 and February 7, 1981.

3.3 Stress Analyses Based on Finite Element Models

3.3.1 Review of Conditions

Before discussing analytical procedures for generating accurate stresses and displacements for the two viaducts, their physical conditions must be reviewed. Each span of each viaduct is a multi-girder span exhibiting the following conditions:

1. The prestressed concrete girders and the concrete slab are most likely acting as one unit with no slip occurring between the girders and the deck.
2. The steel rails are acting in a "composite" manner with the concrete portion (deck and girders). The compression rail anchors and tie plates act as "shear connectors". If there is no slip between the rails and the concrete deck, the composite action is complete. If there is slip, the composite action is incomplete or partial. Examinations of the ends of certain spans show that there are slip marks between the rails and the compression rail anchors.

3. The bearing pads at the ends of the girders permit some amount of horizontal movement. If there are large relative displacements between the bottoms of the prestressed concrete girders and the tops of the piers the bearings are not simply "hinges" or "rollers" as often assumed in design.
4. The "simple span" composite concrete deck and girders are not statically determinate because the rails are continuous over the spans. Even if shear and flexural stresses in the rails are neglected, axial stress will have to be released at each end of the span in each of the six rails fastened to the deck in order to reduce the span to a "simply supported" condition. The degree of indeterminacy of a single span, is therefore, at least 12, and likely somewhat more.
5. Although the girders and decks are straight in each span, horizontal curvature exists in each viaduct, especially in the Westmont Viaduct (see Figs. 1 and 2).
6. The daily change of stresses and displacements are primarily due to change of temperature. The effects of creep, shrinkage, settlement, etc., are likely negligible. This may not be true for seasonal changes in stresses where effects other then temperature may be contributing.
7. The changes of temperature at different parts of a viaduct are different from the variation of air temperature, as well as different from each other.

Because of these highly indeterminate conditions for analyzing the viaducts, a finite element procedure must be employed. A finite element procedure can be developed as was done in Ref. 2 to generate stresses and displacements from assumed or measured temperature changes. Alternatively, a finite element procedure can be developed which will generate stresses from measured temperature changes and measured displacements.

3.3.2 Finite Element Analysis by Using Temperature Changes

This procedure was employed in Ref. 2. In that study the total viaduct is first analyzed, then the individual spans are analyzed. The procedure can be briefly summarized in the following steps:

1. Establish a model with continuous line elements along a viaduct to represent the continuous rails and separate line elements to represent the concrete spans. In constructing the model the geometrical properties of all the rails fastened to the deck, such as area and moment of inertia, are combined to provide similar properties for one continuous line of elements representing the rails. The same is done for the girder and deck spans. This results in a condensation of the entire three dimensional viaduct into a two dimensional structure consisting of a continuous rail plus girder and deck spans (see Ref. 2).
2. Assume a linear shear-versus-slip relationship for the compression rail anchors and ties as was done in Ref. 2. This allows slip between the rail and the concrete portions to be considered in the finite element model. The concrete deck and the prestressed concrete girders are assumed to be composite and to have complete interaction.

3. Assume appropriate boundary elements for the rails at the ends of the viaduct. They are assumed to provide fixity against displacement in the direction of the rails, or partial fixity depending on the degree of rail anchorage beyond the viaducts.
4. Assume suitable restraints at the girder bearings, such as pin, roller, or elastic.
5. Determine realistic temperature changes in the rails and in the concrete portion, and input daily and seasonal temperature changes into the finite element model.
6. The output from the finite element analysis can include displacements and forces in the component parts of the entire viaduct.
7. Establish a finite element model of a single span with line elements representing the rails and two dimensional elements representing the concrete portion of one span as was done in Ref. 2.
8. Utilize the output from the viaduct analysis as boundary conditions to the finite element model of the single span and input the realistic temperature changes for this span.
9. The output from the finite element analysis of the single span can include the displacements and forces in the component parts of the single span.

Examples of this procedure are given in Ref. 2. The inherent inaccuracy due to modelling the concrete portions of the entire viaduct by line elements in Step 1 can be overcome by the use of two dimensional elements such as was done in Step 7 for the single span. However, this will require a large capacity computer. The results, nevertheless should be very much improved from those for the entire viaduct using only line elements. Strains generated from the finite element analysis can be compared with the measured strains presented herein.

3.3.3 Finite Element Analysis by Using Measured Displacements

For each of the viaduct spans, if field measurements of displacements are made in conjunction with temperature variations in the structural components, the procedure can be summarized as follows:

1. Establish a finite element model with line elements representing the rails and two dimensional elements representing the concrete portion of a single span.
2. The connecting elements between the rails and the concrete span should represent the shear-versus-slip characteristics of the compression rail anchors and the ties, as in Step 2 of Art. 3.3.2.
3. The measured movements of the concrete girders at the bearings, of the top of the deck at the end of spans, and of the rails at the gaps between decks, are input as boundary conditions. Absolute movements should be used, that is, movements referenced to a fixed point. The corresponding temperature changes of the rails and concrete span are also input as conditions of analysis.
4. The output of the anlaysis can include the forces in the rails and strains in the concrete portions. These forces and strains can be compared with the measured values.

The difficulty inherent in this procedure is the required accuracy of the field measurements, which should be comparable to the accuracy of the analysis. On the other hand, this procedure provides the simplest and most direct approach to evaluate the forces in the rail.

The creation of suitable finite element models for calculating stresses in the viaducts from measured changes in temperatures, strains and displacements is a time consuming and expensive task. The work reported in Ref. 2 required approximately one man-year and over five thousand dollars of computer charges on the CDC 6400 located at Lehigh University. However, the results of such an analysis will be quite accurate providing the finite element model is properly constructed and the input data has been measured to suitable accuracy.

3.4 Alternative Procedures for Calculating Stresses

As an alternative to the procedures discussed in Art. 3.3, the measured changes in temperature and strains in the rails and girder/deck can be used together with simplified analytic models to provide stresses in the rails, girder and deck but with much less accuracy.

Articles 3.4.1 and 3.4.2 describe the use of such analytic models to compute approximate stresses in the rails, girders and deck. Due to the nature of the simplifications used to generate the analytic models, the resulting rail stresses, although approximate, are believed to be considerably more accurate than the stresses obtained for the girders and deck. The relative approximations of the stresses produced using the procedures described in Arts. 3.4.1 and 3.4.2 is not known, however, without comparing them to the results of a finite element analysis.

3.4.1 Rail Axial Stress

Figure 10(a) shows an elevation view of a short length of bridge span chosen to include a rail strain gage and consisting of the girder/deck system supporting the steel rails. The rails are carried on tie plates spaced 30 in. apart as shown. Two compression rail anchors clamp the rails to each tie plate (see Chap. 4 and Ref. 2). The compression rail anchors are designed to allow slip between the rail and tie plate at a sufficiently large shear force on the contact surfaces. Thus the behavior of the structure cannot be expected to be elastic under all temperature change conditions. The following approximate method of calculating changes in axial stress in the rail from measured changes in strain at the neutral axis level of the rail and measured temperature changes of the rail, will automatically account for the elastic and inelastic behavior of the structure from all effects.

Consider a short segment of one rail of length L centrally located between tie plates as shown in Fig. 10(a). The segment contains a strain gage at mid-length of the segment and on the neutral axis of the rail as shown in Fig. 3. Temperature changes of the rail segment are measured by temperature gages placed on the rail on or near the segment. Assume that the rail segment is cut out and supported as shown in Fig. 10(b). Since only axial strains are measured the segment can be assumed pin supported at the neutral axis at each end by a boundary structure which simulates the behavior of the entire viaduct structure including rails (over and beyond the viaducts), deck, girders, bearings, piers and foundations, outside the boundaries of the segment.

Axial distortions of the rail segment are shown in Fig. 10(c). Axial distortion e_m is computed from the measured change of strain and is given by

$$e_m = (\varepsilon_2-\varepsilon_1)L \tag{1}$$

where ε_2 and ε_1 are the strains measured on the segment. The distortion e_m thus acccunts for all effects occurring in the boundary structure including the effects of temperature change.

The axial distortion e_t is the computed unrestrained distortion of the segment (the segment is assumed standing alone free of the boundary structure) due to the measured temperature change in the segment and is given by

$$e_t = \alpha_s(T_2-T_1)L \tag{2}$$

where T_2 and T_1 are the temperatures measured on or near the segment and α_s is the coefficient of thermal expansion of the steel rail.

Referring to Fig. 10(c), the elongation e is given by

$$e = e_m - e_t \tag{3}$$

Substituting Eqs. 1 and 2

$$e = (\varepsilon_2-\varepsilon_1)L - \alpha_s(T_2-T_1)L \tag{4}$$

Dividing Eq. 4 by L the complimentary strain ε is given by

$$\varepsilon = (\varepsilon_2 - \varepsilon_1) - \alpha_s(T_2 - T_1) \tag{5}$$

Unrestrained distortion of the segment e_t due to temperature change occurs without axial stress. Thus axial stress σ is computed only from the complimentary strain ε and is given by $\sigma = \varepsilon E_s$, or

$$\sigma = [(\varepsilon_2 - \varepsilon_1) - \alpha_s(T_2 - T_1)]E_s \tag{6}$$

where E_s is the modulus of elasticity of steel.

Table 4 shows the computed change in axial stress in psi at the location of the rail strain gages on the Collingswood Viaduct from August 1980 to February 1981. The data used is contained in the upper part of Table 1.

Table 5 shows the computed change in axial stress in the rails of the Collingswood Viaduct during September 15, 1980. The data used is contained in Table 2. In Table 5 the readings taken at 6:56 am are used as the reference readings for computing all changes in stress.

Table 6 shows the computed change in axial stress in the rails of the Westmont Viaduct. The data used is contained in Table 3.

Referring to Table 4, each column provides the change in axial stress in the rail at a particular strain gage due to the change in measured temperature of the rail in the vicinity of the strain gage over six periods of time. For example, the third column headed 0810-0207 shows the changes in axial stress over the time interval August 10, 1980 to February 7, 1981.

Referring to Table 1 the two sets of readings on any given day were condensed to one set by averaging the two readings.

In the computer analyses used to generate Table 4, 5 and 6, the following pairing of measured temperature and strain readings was assumed (see Appendix A and B and also Fig. 3).

(a) Temperature readings from gage 12 were used with strain readings from gages 9 and 11,

(b) Temperature readings from gage 15 were used with strain readings from gages 5, 6, 7, 14, 16, 17,

(c) Temperature readings from gage 22 were used with strain readings from gages 1, 2, 3, 19, 20, 21,

(d) The average of the temperature readings from gages 15 and 22 were used with strain readings from gages 4 and 18.

Referring to Table 2, only the complete sets of hourly readings were used.

Referring to Table 3, the readings in columns 1 and 2 were averaged. Similarly the readings in columns 3 and 4 were averaged. Thus two sets of data were used for August 17, 1980. The two sets are labelled A817 and B817 in Table 6 and refer to the earlier and later data sets.

The computer programs used to generate Tables 4 and 5 are provided in Appendix A and B, respectively. The program used to generate Table 6 is the same as the one shown in Appendix A.

3.4.2 Girder and Deck Stresses

Two sets of results were produced. The first set is shown in Tables 7, 8 and 9. The second set is shown in Tables, 10, 11 and 12.

The first set of results were generated in a manner identical to that used for the rails, using the following equation for stress, σ, which has the same form as Eq. 6:

$$\sigma = [(\varepsilon_2-\varepsilon_1) - \alpha_c(T_2-T_1)]E_c \quad (7)$$

where ε_2 and ε_1 are the measured strains at a particular gage location (Figs. 4 and 5); T_2 and T_1 are the measured temperatures assumed to exist at the gage locations; α_c is the coefficient of thermal expansion of concrete and E_c is the modulus of elasticity of concrete (assumed to be the same for the girders and the deck).

Table 7 shows the computed change in stress in psi at a particular strain gage location on the Collingswood Viaduct from August 1980 to February 1981. The data used is contained in the lower part of Table 1. The temperatures in parentheses for gage 9 are estimated values. The corresponding results in Table 7 are shown in parentheses. In Table 10 all Span 12 results involving 0810 are affected by the estimated temperatures in Table 1.

Table 8 shows the computed change in stress at a strain gage location on the Collingswood Viaduct during September 15, 1980. The data used is contained in Table 2.

Table 9 shows the computed change in stress at a strain gage location on the Westmont Viaduct. The data used is contained in Table 3.

Referring to Table 1, the two sets of readings on any given day were condensed to one set by averaging the two readings as discussed in Art. 3.4.1. However for gage 20 the single reading is used.

In the computer analysis used to generate Tables 7, 8 and 9, the following pairing of measured temperature and strain gage readings is assumed (see Appendix C and D and also Figs. 4 and 5).

(a) Referring to Fig. 4 temperature readings from gage 9 were used with strain gages 1 and 5.

(b) Similarly, temperature gage 10 was used with strain gages 2 and 6; 11 with 3 and 7 and 12 with 4 and 8.

(c) Referring to Fig. 5, the pairings were as follows: 21 with 13 and 17; 22 with 14 and 18; 23 with 15 and 19 and 24 with 16 and 20.

The girder and deck readings in Tables 2 and 3 were used in the same manner as previously discussed for the rail readings in Art. 3.4.1.

The computer programs used to generate Tables 7 and 8 are provided in Appendix C and D respectively. The program used to generate Table 9 is the same as the one shown in Appendix C.

The second set of results shown in Tables 10, 11 and 12 were also generated using Eq. 7. However a linear representation of the temperature and strain data was assumed through the depth of the girders and deck.

Figure 11(a) shows an elevation view of a short length of girder similar to Fig. 10(a) together with its tributary width of deck. The segment contains four strain gages positioned as shown in Figs. 4 and 5.

Consider that the girder/deck segment is cut out and supported as shown in Fig. 11(b). As discussed in Art. 3.4.1 the support simulates the boundary structure but is connected to the girder/deck segment so that axial as well as flexural strains are accounted for. The boundary structure shown in the figure therefore simulates the behavior of the entire viaduct structure outside the boundaries of the segment.

The distribution of the change in measured strain, between two intervals of time, is not expected to be linear over the depth of the girder/deck segment. Similarly with the distribution of the change in temperature. However, a preliminary analysis of the data indicated that it would be possible to estimate stress changes based on an assumed linear distribution. The least squares method was used to obtain a linear representation of the data.

Linear distortions of the girder/deck segment are shown in Fig. 11(c). The distortion e_m at an arbitrary depth is computed using the assumed least squares linear distribution of measured strains as discussed above. Similarly e_t is computed from the assumed linear distribution of temperature strains. The distortion e is given by Eq. 3. The resulting change in stress, σ, is given by Eq. 7.

Table 10 shows the computed change in midspan stress in psi at the top of deck, bottom of girder and at the gage locations for the two instrumented girders (Tables 4 and 5) for spans 12 and 13 of the Collingswood Viaduct for the same six time periods discussed before. The computer program used to compute the stresses and perform the least squares fit of the data is provided in Appendix E. The data used is contained in Table 1.

Table 11 shows the computed change in midspan stress at top of deck bottom of girder and at the gage locations in spans 12 and 13 of the Collingswood Viaduct during September 15, 1980. The program used is shown in Appendix F. The data used is contained in Table 2.

Table 12 shows the computed change in midspan stress at top of deck, bottom of girder and at gage locations for the two instrumented girders (Tables 4 and 5) for spans 10 and 11 of the Westmont Viaduct. The computer program used is similar to the one shown in Appendix E. The data used is contained in Table 3.

The data in Tables 1, 2 and 3 were used in the same manner as previously discussed for the rails and discussed earlier in this article.

3.5 Discussion of Results

The strain and temperature data in Tables 1, 2 and 3 are shown just as they were recorded (Art. 2.3). Experience has shown that, even with care, a small percentage of data, recorded by hand, will be in error. Experience also shows that it is to be expected that a small number of gages installed in the field will not function properly, even though they all appear to be transmitting data as well as any other gage. It is next to impossible to detect these problems during the data gathering phase. It is during the data analysis phase that "out-of-line" values may be a result of such causes. This may account for the values for gage 16, Table 9, for example, which appear incorrect.

Application of the results reported in this chapter by GFC&C personnel should take into consideration the following discussion relevant to the apparent stress changes in the rails and in the girders and deck.

3.5.1 Rail Stresses

The following points are relevant to the interpretation of the apparent stress changes in the rails:

(a) Because the rails are relatively straight and expected to be subjected to tensile changes in stress, and because the webs are relatively thin, cross bending effects at gage locations are considered to be small. Thus gages are placed only on one side of the rail web. Two sided gaging could have been used and the readings averaged to eliminate cross bending effects but a doubling of the number of rail gages would result, significantly increasing data acquisition costs at an expected minor benefit. However it is quite possible that some results may include a local cross bending effect.

(b) Employing a temperature gage adjacent to each strain gage would nearly double the number of gages. Since the rail temperature is not expected to vary significantly within the length of a gaged section (gage 1 through 7, for example) only a few were used. The error is not expected to be significant.

(c) The data spanning the six month interval from August 1980 to February 1981 includes the seasonal effects of changes in all variables including temperature. Other variables may include displacements of the rails beyond the viaducts, foundations displacements and the effects of train operations. The data obtained on September 15, 1980 at the Collingswood Viaduct includes primarily the effect of daily temperature change. Since the girder temperature remains fairly constant the change in rail stress is essentially due to temperature change of the rail and deck only.

(d) Examination of the results shown in Tables 4 and 6 indicate that those involving the November 16, 1980 readings appear inconsistent. It is possible that short term changes in rail temperatures that day were ahead of the bridge response. Steady state conditions are needed for consistent results.

3.5.2 Girder and Deck Stresses

Many of the points made in Art. 3.5.1 also apply to the change in stress in the girders and deck. However the following additional points are relevant to the apparent stress changes in the girder and deck.

(a) Experience has shown that it is difficult to obtain reliable strains on concrete using electrical resistance strain gages. A 2 in. gage length was used to overcome some of the difficulties usually encountered with very short gage lengths.

(b) It is possible that some of the strain gages cover hairline surface cracks. Under the forces introduced by the continuous rails the strain readings will therefore include a crack opening or closing displacement, rendering the reading questionable.

(c) The effects of moisture content changes, creep and shrinkage of the concrete may also distort the readings but are not expected to be significant.

(d) The stress changes obtained from the September 15, 1980 readings on the Collingswood Viaduct are consistent with the results for the rails and appear reliable.

(e) More reliable stress changes in the girders from August 1980 to February 1981 can be determined using the procedures discussed in Ref. 2 and Art. 3.3.

4. BOLT FORCE vs TURN-OF-THE-NUT TESTS

Three tests were performed in Fritz Engineering Laboratory on December 2, 1980 to determine the relationship between the bolt force and turn-of-the-nut for one compression rail anchor. The results of these tests were used to design the Compression Rail Anchor Restraint Tests discussed in Chap. 5. A description of the tests and the test results are as follows:

4.1 Test Setup and Instrumentation

Figure 12 shows a view of the test setup. A six foot length of rail (132 RE rail) was obtained from the Delaware River Port Authority (DRPA) and placed on two tie plates as shown in the figure. The tie plates obtained from DRPA were spaced 30 in. on centers. Four compression rail anchors with bolts and nuts, provided by DRPA, were used to clamp the rail to the tie plates in the same manner as used on the Collingswood and Westmont Viaducts. Two compression rail anchors and their bolts and nuts are visible in the figure. Three of the compression rail anchors (the two not visible in the figure plus the one visible on the left) were tightened to specification. The compression rail anchor location visible on the right was used for the test.

The specification for the installation and tightening of the compression rail anchors was provided by The Rails Co., Maplewood, NJ 07040. The anchors are properly installed when the nut is tightened so that the top of the anchor (or spring) is almost flat across the top surface and conforms to the shape of a plastic template supplied by

The Rails Co. This is referred to herein as the "best fit". The template used is designated "Compression Rail Anchor Template, Rails Co., 43-A Springs".

The bolt used in the tests was instrumented using two 1/4 in., 120 ohm electrical resistance foil gages attached on opposite sides of the bolt shank midway between the bolt head and the threads. This portion of the bolt rests in a slotted hole in the tie plate well clear of edges of the hole. Prior to the tests the bolt was calibrated in a tension testing machine to obtain a bolt force vs measured strain relationship. The bolt and nut were then cleaned, greased and installed to a snug fit (by hand). A different compression rail anchor and nut were used for each of the three tests. Marks were placed on the anchors and nuts to enable the turn of the nut to be monitored. The Vishay Strain Indicator used to measure strain in the bolt during each test is visible above the rail in Fig. 12.

Figure 13 shows a view of the test location. The compression rail anchor, end of the bolt and the nut are visible in the figure. The wires lead from the gages on the bolt shank to the Vishay Strain Indicator.

4.2 Test Procedure and Results

Test 1: Figure 14 shows the results of Test 1. In this test the nut was turned down against the compression rail anchor from the snug position until the top surface of the anchor matched the template shape as closely as could be judged by eye with a light shining from behind the template. At 2½ turns a "best fit" was attained. This is shown as point 1 in the figure. The bolt force at this point is determined by converting the bolt strain to bolt force using the calibration relationship previously determined (Art. 4.1).

The nut was then turned an additional 1/2 turn to point 2. Since it was believed that the anchor would remain elastic during the test no attempt was made to perform a "slow" test to account for possible yielding of the anchor. At point 2 the gap between the center of the template and the anchor measured 3/128 in. The nut was then backed off 1/2 turn which resulted in point 3 in Fig. 14. A "best fit" again was obtained. The nut was backed off an additional 1/2 turn to point 4. The gap under the end of the template nearest the rail measured 1/32 in. at point 4 in the figure. The nut was then backed off to a snug position.

At point 5 it was obvious that inelastic action had occurred since only one turn at the nut was required to unload the bolt. A plot of the results up to point 5 was then made which confirmed this behavior.

The nut was tightened and released four more times to determine if the behavior of the anchor would be essentially elastic under additional cycles. The first of these additional cycles is defined by points 5 through 11. The second is defined by points 11 through 17. The third and fourth additional cycles were both close to a loop passing through points 17-4-3-14-15-16-17 showing nearly elastic behavior was obtained in the last cycles.

For all cycles comprising Test 1, a "best fit" was always obtained at 2½ turns of the nut from the original snug position, point 0 in the figure.

<u>Test 2</u>: Figure 15 shows the results of Test 2. This test essentially repeats Test 1 but was performed slowly to incorporate the influence of inelastic behavior of the anchor.

In Test 2 the nut was turned in 1/6 increments (corresponding to the points on the hexagonal nut) from snug. At the first attainment of "best fit", point 1 in the figure, which occurred at 2½ turns of the nut, the bolt force was allowed to decrease with time as the anchor yielded. At point 2 after 15 minutes elapsed time, no further reduction in bolt force was detected. The nut was tightened again resulting in points 3 and 4. Upon reaching point 4 the bolt force was again allowed to stabilize which resulted in point 5 after a 10 min. wait. The nut was loosened to slightly above snug and retightened producing points 5-6-5. After reaching point 5 the second time the bolt was allowed to stabilize to point 7, requiring 10 min. The nut was then unloaded to point 8.

"Best fit" was also obtained with 2½ turns of the nut.

<u>Test 3</u>: Figure 16 shows the results of Test 3. This test is similar to Test 2 except that the bolt force was allowed to stabilize after each 1/6 increment in turn of the nut thus producing a "slow" test. Also, the bolt was unloaded at first attainment of the "best fit", then loaded beyond "best fit" twice.

"Best fit" was again obtained with $2\frac{1}{2}$ turns of the nut.

4.3 Discussion of Test Results

It is evident from the test results plotted in Figs. 14, 15 and 16 that the compression rail anchors provided by DRPA behave inelastically under first loading up to the attainment of the template shape as specified by The Rails Co. Subsequent loading and unloading of the anchors is essentially elastic. Thus the bolt force is not a unique function of the turn-of-the-nut or of the required template shape. The force in the bolt is "path dependent", that is, it is a function of the previous loading history.

For example, if the compression rail anchor is tightened, at any time, sufficiently beyond the "best fit" shape, it is possible that the "best fit" can then be obtained under zero bolt force providing the anchor does not fracture. The zero bolt force situation would occur if the nut were tightened to about $3\frac{1}{2}$ to 4 turns.

Because of the inelastic behavior of the compression rail anchors it may therefore not be possible to ensure that a given bolt force is obtained in practice by checking periodically with the template.

5. COMPRESSION RAIL ANCHOR RESTRAINT TESTS

Five tests were performed in Fritz Engineering Laboratory on February 18, 1981, to determine the force required to produce significant slip between the rail and tie plate. The expected design loads were calculated from the test results reported in Chap. 4. The following individuals were present:

John McDonnell	DRPA
Jim Costello	DRPA
Donald Wolfe	PATCO
Tom Benner	GFC&C
Russ Ricker	GFC&C
Herb Greenberg	The Rails Co.
Min Yzou	The Rails Co.

A further set of four tests were performed in Fritz Engineering Laboratory on April 14, 1981. The above individuals were not present. These tests were also performed to determine the force required to produce significant slip between the rail and tie plate but under a smaller value of bolt force.

A description of the tests and the test results are as follows:

5.1 Test Setup and Instrumentation

5.1.1 February 18 Tests

Figure 17 shows a schematic of the test setup and instrumentation. Four tie plates were mounted as shown on a reinforced concrete block, 20 in. by 20 in. in plan, and 65 in. high. Two 72 in. long rails (132 RE) were clamped to the tie plates. The rails, tie plates, anchor bolts, elastomeric insulation rail pad under the tie plates, compression rail anchors and all other hardware were supplied by DRPA and mounted to the concrete block in accordance with the construction drawings for 132 RE rail Type "V" installation.[3]

Load was applied to the spreader beam shown in the figure by the 5,000,000 lb. Baldwin testing machine located in Fritz Engineering Laboratory. Four Ames dials, reading to 0.001 in., were mounted between the rails and tie plates as shown to measure slip.

Figure 18 shows a test in progress. The red spreader beam, brown rails and the white concrete block are clearly visible. The Ames dials are the small white circles. Figure 19 shows a close-up view of Bolt 2 location (Art. 5.1.2 and Fig. 17).

5.1.2 April 14 Tests

The test setup and instrumentation was identical to that described in Art. 5.1.1 except for the following.

For these tests the two compression rail anchor bolts marked Bolt 1 and Bolt 2 were instrumented and calibrated as described in Chap. 4 so that the forces in the bolts could be measured during the tests.

5.2 Test Procedure and Results

5.2.1 February 18 Tests

Only one rail at a time was loaded to slip. This was accomplished by placing a mechanical jack under the other rail to prevent slip. In Fig. 18 the left or north rail is being tested. The mechanical jack is visible below the right or south rail in the figure. The load transferred from the test rail to one tie plate was calculated as one quarter of the total load applied to the spreader beam.

Figure 20 shows the results of the five tests. A description of the tests is as follows:

<u>Test 1: South Rail</u>

(a) Nuts on all compression rail anchors torqued to "best fit" using The Rails Co. template (Chap. 4).

(b) South rail loaded to significant slip.

(c) South rail unloaded.

<u>Test 2: North Rail</u>

(a) Nuts on all compression rail anchors torqued to "best fit".

(b) North rail loaded to significant slip.

(c) North rail unloaded.

<u>Test 3: South Rail</u>

(a) Nuts on all compression rail anchors torqued an additional 1/3 turn past "best fit".

(b) South rail loaded to significant slip.

(c) South rail unloaded.

<u>Test 4: North Rail</u>

(a) Nuts on all compression rail anchors torqued an additional 1/3 turn past "best fit".

(b) North rail loaded. Just prior to reaching significant slip a compression rail anchor at the lower tie plate fractured.

Failure apparently was caused by excessive rotation of the anchor as it followed the motion of the rail.

(c) North rail unloaded.

Test 5: South Rail

(a) Nuts on all compression rail anchors loosened to snug then torqued again to 7/6 turn beyond snug. The 7/6 turn was determined from the last unloading curve of Fig. 16 and the estimated amount needed to again achieve best fit.

(b) South rail loaded to slip.

(c) South rail unloaded.

5.2.2 April 14 Tests

Following the February 18 tests, the rails and tie plates were removed from the concrete block. The elastomeric pads were realigned and the tie plates and rails reassembled using all new compression rail anchors which were supplied by The Rails Co. and replacing any hardware items that were damaged during the previous tests.

The April 14 test procedure was identical to that described in Art. 5.2.1. However, The Rails Co. template (Chapter 4) was not used to control torque in the compression rail anchor bolts. Discussions with The Rails Co. following the February 18 tests revealed that they obtain the "best fit" template shape after advancing the nut on the compression rail anchor 0.12 in. from snug. We agreed on the definition of snug. Since 0.12 in. corresponds to approximately 7/6 turn of the nut for a pitch of 10 threads per inch it was agreed to use the 7/6 turn from snug rather than the "best fit" criterion to control bolt torque in the April 14 tests. In addition the forces in Bolt 1 and Bolt 2 shown in Fig. 17 were measured during the tests.

Figure 21 shows the results of the four tests. A description of the tests is as follows:

<u>Test 1: North Rail</u>

(a) Nuts on all compression rail anchors torqued 7/6 turn from snug for the reason stated above (a "Best fit" was not obtained). Force in Bolt 1 is 3.2 kips.

(b) North rail loaded to significant slip. Force in Bolt 1 is 3.1 kips.

(c) North rail unloaded.

<u>Test 2: South Rail</u>

(a) Nuts on all compression rail anchors torqued 7/6 turn from snug (a "best fit" was not obtained). Force in Bolt 2 is 5.4 kips.

(b) South rail loaded to significant slip. Force in Bolt 2 is 4.8 kips.

(c) South rail unloaded.

<u>Test 3: North Rail</u>

(a) Nuts on all compression rail anchors torqued an additional 1/2 turn to 5/3 turn total from snug (a "best fit" was not obtained). Force in Bolt 1 is 4.3 kips.

(b) North rail loaded to significant slip. Force in Bolt 1 is 4.1 kips.

(c) North rail unloaded.

Test 4: South Rail

(a) Nuts on all compression rail anchors torqued an additional 1/2 turn to 5/3 turn total from snug (a "best fit" was not obtained). Force in Bolt 2 is 7.1 kips.

(b) South rail loaded to significant slip. Force in Bolt 2 is 6.3 kips.

(c) South rail unloaded.

6. SUMMARY AND CONCLUSIONS

Fritz Engineering Laboratory, Lehigh University, under subcontract to Gannett, Fleming, Corddry and Carpenter, Inc. of Harrisburg, PA, performed the following study reported herein:

1. The determination of the change of strains in the rails, concrete deck, and the prestressed concrete I-girders of the Collingswood and Westmont Viaducts on the Delaware River Port Authority's Lindenwold-Philadelphia Rapid Transit Line. The viaducts were instrumented in the spring and summer of 1980. Strain data was obtained on August 10 and 17, September 15 and November 16, 1980, and again on February 7, 1981, using a portable data acquisition system specifically designed and built for this study.
2. Conversion of strain data to stress changes in the rails, deck and girders by means of simplified analytic models. Procedures for more accurate determination of stresses and displacements using finite element analyses are described. Although not part of the scope of this project, a finite element study of the two viaducts was carried out during 1980 and is reported in Ref. 2.
3. The experimental determination, in Fritz Laboratory, of the amount of restraint provided by the compression rail anchors as a function of bolt torque.

The following general conclusions are based on the results of the work reported herein:

1. Accurate determination of changes in stress in all viaduct components will require finite element analyses conducted along the lines described in Chapter 3 and presented in Ref. 2.
2. Changes in stress, computed using the simplified analytic models, are believed to be reasonably accurate when applied to the steel rails but less so when applied to the concrete deck and prestressed concrete I-girders.
3. Changes in stress, computed for August 1980 and February 1981, appear reasonable and fairly reliable. The results for November 1980 appear erratic, but no explanation for this behavior was found.
4. The compression rail anchor tests revealed that the compression rail anchors behave inelastically well before attainment of "best fit" as defined by the template provided by The Rails Co. Thus, except for small bolt forces, the bolt force is not a unique function of the required template shape.

7. TABLES AND FIGURES

TABLE 1 COLLINGSWOOD VIADUCT DATA

Gauge Location	Gauge No.	1	2	3	4	5	6	7	8
Date		Aug. 10, 1980		Aug. 17, 1980		Nov. 16, 1980		Feb. 7, 1981	
Start Time				16:55	17:11	11:08	11:23	10:17	10:34
Air Temp. (^{o}F)				85.2	84.4	54.0	52.5	44.7	45.2
Rail	12	109.2	103.3	101.6	100.0	58.1	60.5	47.3	48.8
Temp.	15	117.5	108.0	101.5	99.5	63.9	65.8	50.3	53.0
(^{o}F)	22	116.6	108.1	101.4	99.1	64.9	66.3	50.9	53.7
	1	+0512	+0578	+0673	+0687	+0452	+0443	+0883	+0868
	2	+1190	+1256	+1372	+1385	+0813	+0810	+1563	+1547
	3	+1440	+1505	+1637	+1648	+1244	+1243	+2226	+2210
	4	+0980	+1041	+1156	+1166	+0770	+0758	+1402	+1382
	5	+0636	+0700	+0785	+0797	+0569	+0558	+1082	+1064
	6	+0497	+0545	+0621	+0631	+0417	+0402	+0808	+0792
	7	+0348	+0400	+0496	+0525	+0297	+0286	+0700	+0684
Rail	8	+0110	+0168	+0260	+0270	+0273	+0257	+0428	+0494
Strain	9	-0400	-0316	-0253	-0507	-0065	-0079	+0112	+0074
μ in/in	11	+0384	+0443	+0488	+0503	+0441	+0424	+0766	+0747
	13	+0082	+0132	+0185	+0193	+0277	+0264	+0524	+0509
	14	-0276	-0208	-0152	-0143	-0050	-0054	+0150	+0131
	16	+0426	+0485	+0556	+0567	+0412	+0417	+0828	+0813
	17	-0068	-0000	+0046	+0060	+0192	+0180	+0383	+0362
	18	+0166	+0232	+0278	+0280	+0305	+0297	+0591	+0570
	19	-0889	-0840	-0726	-0708	-0396	-0438	-0515	-0555
	20	+0300	+0383	+0437	+0418	+0281	+0280	+0592	+0576
	21	+0352	+0414	+0509	+0526	+0384	+0373	+0800	+0780
	9	(110.0)	(112.0)	89.3	89.6	43.6	44.0	27.9	28.4
	10	10.14	102.7	79.6	80.1	44.6	44.8	31.0	31.2
Girder	11	90.7	91.6	77.9	78.0	44.3	44.4	31.1	31.5
Temp.	12	88.6	89.3	79.6	79.5	44.5	44.7	31.4	32.0
(^{o}F)	21	89.7	89.4	87.5	87.8	43.0	43.3	27.7	27.8
	22	99.8	101.4	81.6	81.9	43.5	43.4	28.3	28.6
	23	93.9	95.2	78.3	78.4	42.8	42.6	28.6	28.8
	24	90.6	91.1	79.0	78.8	43.8	44.3	31.0	31.6
	1	-2578	-2579	-2622	-2620	-6429	-6436	-2053	-2052
	2	-4166	-4162	-4309	-4312	-7300	-7305	-3986	-3984
	3	-3281	-3280	-3385	-3388	-6958	-6972	-2535	-2539
	4	-2410	-2395	-2464	-2465	-6353	-6364	-2102	-2104
	5	-3200	-3206	-3313	-3314	-5760	-5754	-2790	-2788
	6	-1170	-1174	-1168	-1170	-4626	-4629	-1084	-1081
Girder	7	+0273	+0282	+0319	+0317	-4040	-4043	+0388	+0384
Strain	8	-2640	-2634	-2730	-2729	-5442	-5447	-2478	-2482
μ in/in	13	-0780	-0783	-0679	-0686	-5516	-5509	-0107	-0110
	14	-1826	-1848	-1840	-1846	-6115	-6133	-1766	-1768
	15	-2856	-2860	-2919	-2921	-6776	-6864	-2677	-2680
	16	-4747	-4744	-4930	-4926	-7664	-7660	-4507	-4508
	17	+0654	+0643	+0932	+0935	-1215	-1208	+5522	+5525
	18	-0354	-0367	-0307	-0308	-4060	-4057	-0163	-0164
	19	-1757	-1764	-1806	-1806	-4833	-4847	-1483	-1484
	20	-	-1192	-1207	-1206	-4445	-4474	-0720	-0730
Stop Time				17:10	17:30	11:20	11:36	10:33	10:49

TABLE 2 COLLINGSWOOD VIADUCT DATA

Date		September 15, 1980						
Start Time		6:56	7:30	8:00	8:30	9:00	9:30	10:00
Air Temp. (°F)		69.6	69.0	69.4	69.0	69.1	69.6	69.4
Gauge Location	Gauge No.	1	2	3	4	5	6	7
Rail Temp. (°F)	12	69.6	-	73.3	75.3	77.6	79.9	78.2
	15	67.2	-	71.0	73.4	76.0	76.7	77.7
	22	67.7	-	71.6	74.2	76.6	77.6	78.6
Rail Strain μ in/in	1	+0963		+0950		+0914		+0890
	2	+1672		+1645		+1620		+1593
	3	+2739		+2728		+2686		+2673
	4	+1475		+1454		+1410		+1399
	5	+1116		+1100		+1053		+1032
	6	+0880		+0864		+0834		+0815
	7	+0745		+0734		+0710		+0689
	8	+0562		+0553		+0512		+0488
	9	-0163		-0152		-0350		-0203
	11	+0770		+0740		+0693		+0687
	13	+0498		+0486		+0461		+0430
	14	+0118		+0108		+0083		+0055
	16	+0837		+0828		+0784		+0775
	17	+0340		+0343		+0273		+0258
	18	+0567		+0541		+0500		+0479
	19	-0533		-0735		-0766		-0711
	20	+0643		+0596		+0550		+0529
	21	+0798		+0780		+0773		+0753
Girder Temp. (°F)	9	74.6	73.9	73.5	73.2	73.1	73.0	73.0
	10	74.5	74.0	73.8	73.5	73.1	72.8	72.8
	11	74.0	73.6	73.2	73.0	72.8	72.4	72.4
	12	72.6	72.0	71.7	71.8	71.7	71.7	71.6
	21	73.7	73.3	73.0	72.7	72.4	72.3	72.5
	22	75.8	75.2	74.6	74.3	73.8	73.5	73.6
	23	74.5	74.0	73.5	73.4	73.1	72.8	72.7
	24	72.4	72.0	71.7	71.9	71.7	71.7	71.7
Girder Strain μ in/in	1	-2413		-2415		-2407		-2402
	2	-4293		-4295		-4289		-4278
	3	-3345		-3345		-3342		-3339
	4	-2313		-2315		-2317		-2322
	5	-3205		-3202		-3191		-3185
	6	-1158		-1150		-1146		-1140
	7	+0380		+0386		+0387		+0389
	8	-2623		-2620		-2623		-2627
	13	-0498		-0486		-0480		-0465
	14	-1862		-1851		-1834		-1820
	15	-2863		-2861		-2854		-2853
	16	-4825		-4814		-4819		-4816
	17	+3275		+3283		+3286		+3300
	18	-0297		-0291		-0286		-0276
	19	-1749		-1748		-1748		-1746
	20	-1099		-1100		-1100		-1104
Stop Time		7:08	7:35	8:07	8:32	9:07	9:32	9:05

TABLE 2 Continued

Date		September 15, 1980						
Start Time		10:30	11:00	11:30	12:00	12:29	13:02	13:30
Air Temp. (^{O}F)		69.6	71.1	71.2	70.0	72.0	73.0	72.4
Gauge Location	Gauge No.	8	9	10	11	12	13	14
Rail Temp. (^{O}F)	12	80.6	82.7	82.7	81.9	86.8	89.5	89.4
	15	81.0	84.3	85.4	84.5	90.5	94.6	93.2
	22	81.8	85.0	85.9	85.5	91.1	95.3	94.2
Rail Strain μ in/in	1		+0852		+0856		+0796	
	2		+1576		+1564		+1488	
	3		+2638		+2638		+2578	
	4		+1350		+1348		+1273	
	5		+0997		+0991		+0917	
	6		+0786		+0777		+0733	
	7		+0658		+0647		+0590	
	8		+0443		+0442		+0380	
	9		-0231		-0212		-0385	
	11		+0641		+0649		+0570	
	13		+0377		+0387		+0288	
	14		-0000		-0000		-0091	
	16		+0709		+0720		+0636	
	17		+0201		+0210		+0122	
	18		+0430		+0420		+0350	
	19		-0855		-0732		-0731	
	20		+0499		+0496		+0414	
	21		+0677		+0690		+0599	
Girder Temp. (^{O}F)	9	73.5	73.9	74.4	75.0	75.9	76.6	77.0
	10	72.9	73.0	72.9	72.5	73.5	73.6	73.8
	11	72.6	72.8	72.7	72.6	73.1	73.4	73.5
	12	72.2	72.8	72.4	72.5	73.7	74.1	74.0
	21	72.7	73.2	73.3	73.6	74.7	75.3	75.8
	22	73.5	73.5	73.7	73.5	74.3	74.4	74.6
	23	72.9	73.1	73.0	72.8	73.5	73.5	73.8
	24	72.0	73.5	72.5	72.3	73.1	73.6	73.6
Girder Strain μ in/in	1		-2395		-2402		-2396	
	2		-4266		-4268		-4263	
	3		-3338		-3336		-3330	
	4		-2327		-2331		-2337	
	5		-3172		-3170		-3164	
	6		-1128		-1124		-1115	
	7		+0430		+0390		+0388	
	8		-2633		-2638		-2645	
	13		-0460		-0479		-0457	
	14		-1824		-1811		-1793	
	15		-2854		-2857		-2846	
	16		-4831		-4838		-4834	
	17		+3310		+3306		+3316	
	18		-0267		-0266		-0255	
	19		-1743		-1741		-1740	
	20		-1111		-1111		-1119	
Stop Time		10:32	11:08	11:32	12:08	12:32	13:09	13:33

TABLE 2 Continued

Date		September 15, 1980					
Start Time		14:01	14:30	14:58	15:30	15:59	16:27
Air Temp. (°F)		72.8	72.2	72.7	72.2	72.3	71.3
Gauge Location	Gauge No.	15	16	17	18	19	20
Rail Temp. (°F)	12	88.8	87.2	86.3	84.9	82.3	-
	15	90.7	88.2	86.8	83.8	81.8	-
	22	91.4	88.6	87.3	85.5	82.5	-
Rail Strain μ in/in	1	+0815		+0839		+0890	
	2	+1530		+1546		+1583	
	3	+2595		+2619		+2644	
	4	+1302		+1331		+1399	
	5	+0950		+0968		+1012	
	6	+0750		+0770		+0802	
	7	+0620		+0636		+0683	
	8	+0412		+0430		+0464	
	9	-0431		-0287		-0264	
	11	+0593		+0621		+0672	
	13	+0323		+0352		+0423	
	14	-0057		-0040		+0072	
	16	+0672		+0697		+0783	
	17	+0142		+0197		+0255	
	18	+0386		+0408		+0469	
	19	-0696		-0707		-0773	
	20	+0445		+0479		+0511	
	21	+0647		+0669		+0718	
Girder Temp. (°F)	9	78.0	78.7	79.3	79.6	79.7	80.0
	10	74.1	74.0	74.5	74.9	74.7	74.7
	11	73.7	73.9	74.1	73.9	74.0	74.0
	12	74.2	74.0	74.0	74.1	73.5	73.4
	21	76.6	77.0	77.5	77.7	77.8	78.1
	22	75.0	75.2	75.5	75.9	75.7	75.8
	23	74.0	74.1	74.2	74.3	74.3	74.2
	24	73.5	73.6	73.5	73.6	73.4	72.9
Girder Strain μ in/in	1	-2386		-2392		-2393	-2391
	2	-4245		-4243		-4255	-4258
	3	-3328		-3327		-3333	-3334
	4	-2335		-2329		-2328	-2332
	5	-3167		-3172		-3185	-3188
	6	-1111		-1112		-1117	-1123
	7	+0393		+0396		+0391	+0388
	8	-2642		-2640		-2640	-2643
	13	-0473		-0451		-0448	-0453
	14	-1784		-1783		-1790	-1804
	15	-2838		-2838		-2840	-2851
	16	-4827		-4825		-4820	-4836
	17	+3322		+3314		+3312	+3322
	18	-0254		-0254		-0263	-0273
	19	-1740		-1740		-1740	-1751
	20	-1118		-1116		-1116	-1116
Stop Time		14:08	14:32	15:05	15:32	16:04	16:31

TABLE 3 WESTMONT VIADUCT DATA

Gauge Location	Gauge No.	1	2	3	4	5	6	7	8
Date			Aug. 17, 1980			Nov. 16, 1980		Feb. 7, 1981	
Start Time		14:30	14:50	15:40	16:05	9:20	9:38	13:08	13:22
Air Temp. (^{o}F)		81.6	81.4	82.6	84.1	42.4	42.1	51.4	49.3
Rail Temp. (^{o}F)	12	112.3	113.5	109.7	106.3	44.1	46.3	62.8	63.3
	15	117.7	119.0	114.3	110.4	38.9	48.9	70.2	71.8
	22	122.0	118.2	113.7	109.8	33.5	48.9	70.0	71.4
Rail Strain μ in/in	1	-	-	+0960	+1005	+0633	+0660	+0840	+0846
	2	+1200	+1325	+1384	+1398	+0984	+0930	+1300	+1292
	3	+1627	+1660	+1673	+1717	+1044	+1055	+1686	+1682
	4	+1563	+1521	+1628	+1616	+1080	+1065	+1635	+1632
	5	+1499	+1466	+1510	+1525	+0967	+0967	+1641	+1640
	6	+1181	+1162	+1190	+1207	+0739	+0732	+1306	+1300
	7	+1502	+1575	+1630	+1642	+1056	+1059	+1616	+1613
	8	+0105	+0232	+0296	+0323	+0390	+0411	+0426	+0424
	9	+0094	-0256	-0182	+0000	+0311	+0320	+0348	+0348
	11	+0500	+0483	+0520	+0535	+0460	+0453	+0671	+0667
	13	+0662	+0769	+0778	+0800	+0626	+0603	+0873	+0865
	14	+0577	+0560	+0606	+0616	+0455	+0455	+0743	+0740
	16	+0653	+0683	+0715	+0717	+0534	+0550	+0803	+0804
	17	+1031	+1005	+1060	+1070	+0754	+0758	+1221	+1217
	18	-0204	-0243	-0190	-0181	+0140	+0130	+0084	+0079
	19	-0400	-0300	-0370	-0235	+0623	+0645	+0000	-0050
	20	+0006	+0023	+0074	+0075	+0252	+0243	+0259	+0254
	21	+0678	+0756	+0792	+0810	+0595	+0603	+0882	+0882
Girder Temp. (^{o}F)	9	83.4	84.6	86.6	87.5	43.7	43.9	37.3	38.0
	10	77.4	77.9	79.0	79.5	44.9	44.7	35.0	35.3
	11	79.0	79.5	80.1	80.0	43.0	43.1	34.6	34.7
	12	80.2	80.7	81.0	81.3	42.1	42.2	37.0	37.5
	21	83.2	84.0	86.3	87.2	42.9	43.0	36.4	37.3
	22	76.6	77.0	78.0	78.6	41.7	41.8	31.2	31.6
	23	76.6	76.9	77.5	79.9	41.2	41.3	31.7	32.2
	24	78.4	78.7	79.1	79.4	41.1	41.1	34.4	34.9
Girder Strain μ in/in	1	-1533	-1458	-1460	-1456	+2292	+2545	+1475	+1470
	2	-1932	-1875	-1870	-1871	-0700	-0690	-1702	-1692
	3	-1486	-1390	-1370	-1380	-0638	-0640	-1143	-1146
	4	-0413	-0278	-0306	-0309	-0040	-0037	-0032	-0034
	5	-1612	-1545	-1564	-1572	-0470	-0468	-0818	-0822
	6	-3955	-3895	-3904	-3910	-1930	-1914	-3505	-3508
	7	-2766	-2717	-2708	-2712	-1313	-1300	-2328	-2332
	8	-1740	-1688	-1686	-1687	-0770	-0774	-1576	-1580
	13	-2248	-2140	-2159	-2176	-0208	-0170	+0942	+0940
	14	-3826	-3731	-3737	-3735	-2000	-1938	-3476	-3474
	15	-2058	-1967	-1962	-1958	-1010	-1000	-1885	-1887
	16	+2133	+2218	-2228	+2234	+0980	+0972	+2102	+2102
	17	+1863	+1904	+1920	+1912	-	-	-	-
	18	+2064	+2085	+2088	+2087	+1056	+1072	+1916	+1916
	19	-3260	-3212	-3204	-3220	-1562	+1568	-2933	-2934
	20	-3180	-3148	-3144	-3154	-	-	-	-
Stop Time		14:49	15:04	16:00	16:17	9:37	9:48	13:20	13:32

TABLE 4 COLLINGSWOOD VIADUCT - CHANGE IN RAIL STRESS (psi)
August 10, 1980 to February 7, 1981

	0810-0817	0810-1116	0810-0207	0817-1116	0817-0207	1116-0207
1	6302.7	6088.1	21264.3	-214.6	14961.7	15176.3
2	6907.4	-3174.9	21308.6	-10082.4	14401.2	24483.5
3	7335.2	2208.8	33506.8	-5126.4	26171.7	31298.0
4	6779.1	1807.6	22874.3	-4971.5	16095.2	21066.7
5	5977.4	6102.1	23663.4	124.6	17686.0	17561.4
6	5446.4	5895.6	19946.4	449.1	14500.0	14050.9
7	6375.7	6751.1	21096.9	375.4	14721.2	14345.9
8	6065.9	12901.8	21214.9	6835.9	15149.0	8313.1
9	396.0	17439.7	24464.4	17043.6	24068.3	7024.7
11	3464.0	9563.2	21278.4	6099.1	17814.3	11715.2
13	4767.9	14008.1	23796.2	9240.1	19028.2	9788.1
14	5136.7	14789.8	22999.7	9653.1	17863.0	8209.8
16	5475.9	7975.3	22483.4	2499.4	17007.5	14508.1
17	4915.4	15674.8	23707.7	10759.4	18792.2	8032.8
18	4699.3	12088.4	22874.3	7389.0	18175.0	10785.9
19	6671.4	22165.6	21234.8	15494.1	14563.4	-930.7
20	4857.2	7164.8	18668.3	2307.6	13811.2	11503.5
21	6287.9	8831.6	23521.1	2543.6	17233.2	14689.5

(Sign Convention: Tension stress positive)

TABLE 5 COLLINGSWOOD VIADUCT - CHANGE IN RAIL STRESS (psi)
September 15, 1980

	0656-0800	0656-0900	0656-1000	0656-1100	0656-1200
1	-1131.3	-3152.1	-4243.6	-6591.8	-6569.7
2	-1544.3	-3240.6	-4420.6	-6149.3	-6599.2
3	-1072.3	-3270.1	-4037.1	-6296.8	-6392.7
4	-1348.2	-3604.9	-4293.7	-6985.6	-7102.1
5	-1200.7	-3454.9	-4491.4	-6789.4	-7004.8
6	-1200.7	-3044.4	-3930.9	-6051.9	-6355.8
7	-1053.2	-2719.9	-3665.4	-5845.4	-6208.3
8	-994.2	-3162.4	-4196.4	-6789.4	-6857.3
9	-385.0	-7050.5	-2829.1	-4517.9	-3804.0
11	-1594.5	-3805.5	-4097.5	-6317.4	-5928.0
13	-1082.7	-2778.9	-4019.4	-6848.4	-6591.8
14	-1023.7	-2719.9	-3871.9	-6759.9	-6798.3
16	-994.2	-3250.9	-3842.4	-7054.9	-6768.8
17	-640.2	-3663.9	-4432.4	-7379.4	-7152.3
18	-1495.7	-3663.9	-4647.7	-7339.6	-7692.1
19	-6706.8	-8580.1	-7341.1	-12816.3	-9283.7
20	-2134.3	-4450.1	-5453.1	-7565.3	-7749.7
21	-1278.8	-2444.1	-3417.6	-6886.8	-6599.2

(Sign Convention: Tension stress positive)

TABLE 5 Continued

	0656-1302	0656-1401	0656-1458	0656-1559
1	-10218.8	-8910.5	-7416.3	-4991.4
2	-10702.3	-8733.5	-7475.3	-5463.4
3	-10041.8	-8792.5	-7298.3	-5640.4
4	-11232.1	-9628.8	-8006.3	-5060.7
5	-11124.5	-9403.1	-8124.3	-5867.6
6	-9590.4	-8341.1	-7003.3	-5100.6
7	-9826.4	-8193.6	-6973.8	-4628.6
8	-10623.0	-8931.1	-7652.3	-5690.6
9	-10364.8	-11587.6	-6860.2	-5414.7
11	-9715.8	-8903.1	-7597.7	-5326.2
13	-11449.0	-9668.6	-8065.3	-5012.1
14	-11419.5	-9668.6	-8419.3	-4156.6
16	-11183.5	-9373.6	-7888.3	-4392.6
17	-11685.0	-10347.1	-7976.8	-5307.1
18	-11674.6	-9864.8	-8448.8	-5709.7
19	-11133.3	-9353.0	-8891.3	-9917.9
20	-12047.8	-10385.5	-8596.3	-6731.9
21	-11162.8	-8999.0	-7563.8	-5197.9

(Sign Convention: Tension stress positive)

TABLE 6 WESTMONT VIADUCT - CHANGE IN RAIL STRESS (psi)
August 17, 1980 to February 7, 1981

	A817-B817	A817-1116	A817-0207	B817-1116	B817-0207	1116-0207
1	-	-	-	3616.0	3755.1	140.1
2	5391.9	6116.8	10460.7	725.0	5068.8	4343.9
3	3120.4	-2393.9	10667.2	-5514.3	7545.8	13061.1
4	3740.6	857.0	11980.0	-2883.6	8239.3	11123.0
5	2183.0	-931.5	13740.4	-3114.5	11557.4	14671.8
6	1947.0	1413.8	12958.6	-533.2	11011.6	11544.8
7	4026.8	86.3	11321.4	-3940.5	7294.6	11235.1
8	5310.0	21119.8	16646.1	15809.8	11336.1	-4473.7
9	644.6	24678.2	22214.2	24033.7	21563.7	-2464.0
11	2001.6	11949.0	14795.0	9947.4	12793.4	2846.0
13	3318.8	11296.3	13607.6	7977.5	10288.9	2311.3
14	2404.3	10927.5	14182.9	8523.3	11778.6	3255.3
16	2566.5	10558.8	13076.6	7992.3	10510.1	2517.8
17	2537.0	6546.8	15008.9	4009.8	12471.9	8462.1
18	2501.6	25283.0	18278.2	22781.4	15776.6	-7004.8
19	3002.4	44157.1	19060.0	41154.7	16057.6	-25097.1
20	3371.1	22002.6	16611.5	18631.5	13240.3	-5391.1
21	4079.1	11648.1	14340.0	7569.0	10260.8	2691.9

(Sign Convention: Tension stress positive)

TABLE 7 COLLINGSWOOD VIADUCT - CHANGE IN GIRDER STRESS (psi)
August 10, 1980 to February 7, 1981

	0810-0817	0810-1116	0810-0207	0817-1116	0817-0207	1116-0207
1	(349.8)	(-13906.7)	(4123.1)	-14256.5	3773.3	18029.8
2	-53.6	-11261.4	2436.9	-11207.8	2490.5	13698.4
3	-108.0	-13716.9	4443.5	-13608.9	4551.5	18160.4
4	-22.6	-14870.3	2591.3	-14847.7	2613.9	17461.6
5	(75.8)	(-8667.7)	(3671.7)	-8743.5	3596.0	12339.5
6	548.9	-12538.9	2076.3	-13087.8	1527.4	14615.2
7	482.4	-16273.9	1884.4	-16756.3	1402.0	18158.4
8	-145.5	-10241.8	2017.0	-10096.4	2162.5	12258.9
13	444.9	-17944.0	4206.5	-18388.9	3761.6	22150.5
14	431.6	-15894.7	2026.7	-16326.3	1595.1	17921.4
15	141.9	-14713.1	2315.6	-14855.0	2173.8	17028.8
16	-446.5	-10621.9	2399.1	-10175.3	2845.6	13020.9
17	1194.5	-6373.8	21140.6	-7568.3	19946.1	27514.4
18	669.4	-13521.1	2538.5	-14190.4	1869.1	16059.6
19	208.4	-11156.7	2708.6	-11365.0	2500.2	13865.2
20	230.5	-12036.4	3321.9	-12266.9	3091.4	15358.3

(Sign Convention: Tension stress positive)

TABLE 8 COLLINGSWOOD VIADUCT - CHANGE IN GIRDER STRESS (psi)
September 15, 1980

	0656-0800	0656-0900	0656-1000	0656-1100	0656-1200
1	18.5	60.5	83.0	89.5	34.7
2	8.9	50.0	101.6	145.1	149.1
3	19.3	41.1	62.9	57.2	70.1
4	13.7	5.6	-12.1	-61.3	-70.1
5	38.7	92.7	119.3	149.9	131.4
6	49.2	82.2	113.6	157.2	185.4
7	43.5	57.2	75.0	230.5	74.2
8	33.9	21.8	8.1	-45.1	-58.0
13	65.3	104.0	162.0	165.2	79.0
14	73.3	161.2	222.5	208.8	261.1
15	32.2	70.1	83.8	70.1	65.3
16	61.3	41.1	53.2	-50.8	-50.0
17	49.2	75.8	129.8	153.1	127.3
18	53.2	92.7	137.8	176.5	180.5
19	28.2	37.9	55.6	58.0	73.3
20	12.9	12.9	-3.2	-75.0	-45.9

(Sign Convention: Tension stress positive)

TABLE 8 Continued

	0656-1302	0656-1401	0656-1458	0656-1559
1	20.1	26.6	-29.0	-42.7
2	142.7	203.1	201.5	148.3
3	75.0	75.8	70.1	48.4
4	-133.0	-127.3	-98.3	-82.2
5	116.9	70.9	19.3	-42.7
6	195.1	199.1	185.4	160.4
7	46.7	59.6	62.1	44.3
8	-124.9	-115.3	-102.4	-90.3
13	126.5	30.6	97.5	102.4
14	311.9	333.7	325.6	292.6
15	92.7	112.8	108.0	97.5
16	-65.3	-34.7	-26.6	-4.0
17	126.5	119.3	65.3	50.0
18	203.1	192.6	180.5	139.4
19	60.5	48.4	43.5	41.1
20	-109.6	-103.2	-95.1	-92.7

(Sign Convention: Tension stress positive)

TABLE 9 WESTMONT VIADUCT - CHANGE IN GIRDER STRESS (psi)
August 17, 1980 to February 7, 1981

	A817-B817	A817-1116	A817-0207	B817-1116	B817-0207	1116-0207
1	77.4	16745.5	13081.8	16668.1	13004.4	-3663.7
2	94.3	5664.6	1859.8	5570.3	1765.5	-3804.7
3	234.5	4095.3	2261.2	3860.7	2026.7	-1834.1
4	136.2	2163.3	2304.0	2027.1	2167.7	140.6
5	-31.4	5443.3	4177.5	5474.8	4208.9	-1265.8
6	33.9	8866.4	2714.2	8832.6	2680.4	-6152.2
7	107.6	6658.4	2736.8	6550.8	2629.2	-3921.6
8	93.9	4722.4	1592.7	4628.5	1498.8	-3129.7
13	30.6	9063.1	13764.5	9032.4	13733.8	4701.4
14	135.0	8139.8	2320.9	8004.8	2185.9	-5818.9
15	164.4	4918.6	1593.1	4754.2	1428.6	-3325.6
16	-8772.1	-3928.4	765.3	4843.7	9537.4	4693.7
17	54.8	-	-	-	-	-
18	16.1	-3224.8	459.0	-3240.9	442.9	3683.8
19	49.6	13911.6	2302.3	13862.0	2252.8	-11609.2
20	43.5	-	-	-	-	-

(Sign Convention: Tension stress positive)

TABLE 10 COLLINGSWOOD VIADUCT - CHANGE IN GIRDER STRESS (psi)
August 10, 1980 to February 7, 1981

SPAN 12 SOUTH GIRDER

	0810-0817	0810-1116	0810-0207	0817-1116	0817-02-7	1116-0207
TOP SLAB	201.7	-12154.5	3624.8	-12356.2	3423.1	15799.3
1	155.5	-12524.7	3559.6	-12680.2	3403.1	16084.4
2	126.6	-12756.2	3518.9	-12882.8	3392.3	16275.0
3	-12.0	-13867.0	3323.3	-13854.9	3335.3	17190.3
4	-104.4	-14607.5	3129.9	-14503.1	3297.4	17800.5
BTM GIRD	-121.8	-14746.4	3168.5	-14624.6	3290.3	17914.9

SPAN 12 NORTH GIRDER

	0810-0817	0810-1116	0810-0207	0817-1116	0817-0207	1116-0207
TOP SLAB	424.0	-10825.3	3137.6	-11249.3	2713.7	13963.0
5	371.0	-11144.0	2928.6	-11515.0	2557.5	14072.5
6	338.0	-11343.1	2797.9	-11681.1	2459.9	14141.0
7	179.2	-12299.0	2170.6	-12478.2	1991.4	14469.6
8	73.3	-12936.3	1752.4	-13009.7	1679.1	14688.7
BTM GIRD	53.5	-13055.8	1674.0	-13109.3	1620.5	14729.8

SPAN 13 SOUTH GIRDER

	0810-0817	0810-1116	0810-0207	0817-1116	0817-2027	1116-0207
TOP SLAB	666.7	-18641.7	3382.9	-19308.4	2716.2	22024.6
13	515.7	-17532.3	3196.7	-18048.0	2681.0	20729.0
14	421.3	-16838.9	3080.3	-17260.2	2659.0	19919.2
15	-31.6	-13510.7	2521.7	-13479.1	2553.3	16032.3
16	-333.6	-11291.9	2149.2	-10958.3	2482.8	13441.1
BTM GIRD	-390.2	-10875.8	2079.4	-10485.6	2469.6	12955.2

SPAN 13 NORTH GIRDER

	0810-0817	0810-1116	0810-0207	0817-1116	0817-0207	1116-0207
TOP SLAB	1110.3	-9077.8	14613.4	-10188.1	13503.1	23691.2
17	957.6	-9561.8	12560.3	-10519.4	11602.7	22122.1
18	862.1	-9864.4	11277.1	-10726.5	10415.0	21141.4
19	394.3	-11346.8	4989.3	-11741.1	4595.0	16336.1
20	88.8	-12314.9	882.9	-12403.7	794.2	13197.9
BTM GIRD	31.5	-12496.5	113.0	-12528.0	81.5	12609.5

(Sign Convention: Tension stress positive)

TABLE 11 COLLINGSWOOD VIADUCT - CHANGE IN GIRDER STRESS (psi)
September 15, 1980

SPAN 12 SOUTH GIRDER

	0656-0800	0656-0900	0656-1000	0656-1100	0656-1200
TOP SLAB	14.5	68.7	118.5	160.3	129.5
1	14.7	60.2	101.3	130.7	105.4
2	14.8	54.9	90.6	112.2	90.3
3	15.3	29.5	38.9	23.4	18.1
4	15.6	12.6	4.5	035.8	-30.1
BTM GIRD	15.7	9.4	-1.9	-46.9	-39.1

SPAN 12 NORTH GIRDER

	0656-0800	0656-0900	0656-1000	0656-1100	0656-1200
TOP SLAB	46.1	104.4	144.4	212.5	210.6
5	44.7	92.6	125.5	186.7	173.9
6	43.9	85.2	113.7	170.6	150.9
7	39.7	49.8	57.2	93.3	40.8
8	36.9	26.2	19.5	41.8	-32.7
BTM GIRD	36.4	21.8	12.4	32.1	-46.5

SPAN 13 SOUTH GIRDER

	0656-0800	0656-0900	0656-1000	0656-1100	0656-1200
TOP SLAB	69.0	151.3	221.3	243.8	215.4
13	65.8	134.8	195.1	201.9	178.9
14	63.9	124.5	178.7	175.7	156.1
15	54.4	75.0	100.0	49.8	46.7
16	48.0	42.0	47.6	-34.0	-26.3
BTM GIRD	46.9	35.8	37.8	-49.8	-40.0

SPAN 13 NORTH GIRDER

	0656-0800	0656-0900	0656-1000	0656-1100	0656-1200
TOP SLAB	60.2	100.3	166.5	223.2	201.3
17	53.2	87.3	141.8	181.7	167.7
18	48.9	79.2	126.3	155.9	146.8
19	27.6	39.4	50.7	29.0	44.0
20	13.7	13.4	1.3	-53.9	-23.2
BTM GIRD	11.1	8.5	-8.0	-69.4	-35.8

(Sign Convention: Tension stress position)

TABLE 11 Continued

SPAN 12 SOUTH GIRDER	0656-1302	0656-1401	0656-1458	0656-1559
TOP SLAB	133.9	170.4	124.3	79.9
1	102.9	134.1	98.9	62.0
2	83.5	111.4	83.0	50.9
3	-9.7	2.6	6.6	-2.7
4	-71.8	-70.0	-44.3	-38.5
BTM GIRD	-83.5	-83.6	-53.8	-45.2

SPAN 12 NORTH GIRDER	0656-1302	0656-1401	0656-1458	0656-1559
TOP SLAB	223.3	193.2	148.0	88.7
5	175.8	153.0	117.2	68.3
6	146.1	127.8	97.9	55.5
7	3.5	7.1	5.5	-5.6
8	-91.6	-73.4	-56.2	-46.4
BTM GIRD	-109.4	-88.5	-67.7	-54.1

SPAN 13 SOUTH GIRDER	0656-1302	0656-1401	0656-1458	0656-1559
TOP SLAB	281.0	222.4	259.6	237.6
13	233.6	190.1	221.1	204.3
14	203.9	170.0	197.1	183.5
15	61.6	73.4	81.6	83.6
16	-33.3	9.0	4.7	17.1
BTM GIRD	-51.1	-3.1	-9.8	4.6

SPAN 13 NORTH GIRDER	0656-1302	0656-1401	0656-1458	0656-1559
TOP SLAB	230.0	217.0	172.1	138.3
17	184.3	173.3	136.8	108.6
18	155.8	146.1	114.8	90.1
19	15.9	12.5	6.6	-.8
20	-75.5	-74.8	-64.0	-60.1
BTM GRID	-92.6	-91.1	-77.2	-71.2

(Sign Convention: Tension stress positive)

TABLE 12 WESTMONT VIADUCT - CHANGE IN GIRDER STRESS (psi)
August 10, 1980 to February 7, 1981

SPAN 10 SOUTH GIRDER

	A817-3817	A817-1116	A817-0207	B817-1116	B817-0207	1116-0207
TOP SLAB	77.4	14192.7	9473.5	14115.2	9396.1	-4719.2
1	93.2	12287.4	8-26.9	12194.2	8133.7	-4060.5
2	107.0	10620.4	7136.2	10513.4	7023.1	-3483.2
3	155.3	4785.6	3318.5	4630.3	3163.1	-1467.2
4	186.9	975.2	825.3	788.3	633.4	-149.9
BTM GIRD	192.8	260.7	357.8	67.9	165.0	97.1

SPAN 10 NORTH GIRDER

	A817-3817	A817-1116	A817-0207	B817-1116	B817-0207	1116-0207
TOP SLAB	-24.9	7575.6	4049.8	7600.4	4074.7	-3525.8
5	-3.9	7257.5	3706.5	7261.4	3710.4	-3551.0
6	14.4	6979.2	3406.1	6964.8	3391.7	-3573.1
7	75.8	6044.9	2397.6	5969.1	2321.8	-3647.3
8	117.7	5408.8	1711.0	5291.1	1593.3	-3697.8
BTM GIRD	125.5	5289.5	1582.2	5164.0	1456.7	-3707.3

SPAN 11 SOUTH GIRDER

	A817-3817	A817-1116	A817-0207	B817-1116	B817-0207	1116-0207
TOP SLAB	2522.9	11959.8	10565.8	9436.9	8042.9	-1394.0
13	1276.9	9966.7	8964.5	8689.8	7687.6	-1002.3
14	186.7	8222.9	7563.3	8036.2	7376.6	-659.5
15	-3706.9	1994.7	2559.3	5701.6	6266.1	564.5
16	-6198.8	-1991.3	-643.4	4207.5	5555.4	1347.9
BTM GIRD	-6666.0	-2738.7	-1243.8	3929.3	5422.2	1494.8

SPAN 11 NORTH GIRDER

	A817-3817	A817-1116	A817-0207	B817-1116	B817-0207	1116-0207
TOP SLAB	36.8	-	-	-	-	-
17	37.9	-	-	-	-	-
18	39.0	-	-	-	-	-
19	42.4	-	-	-	-	-
20	44.7	-	-	-	-	-
BTM GIRD	45.2	-	-	-	-	-

(Sign Convention: Tension stress positive)

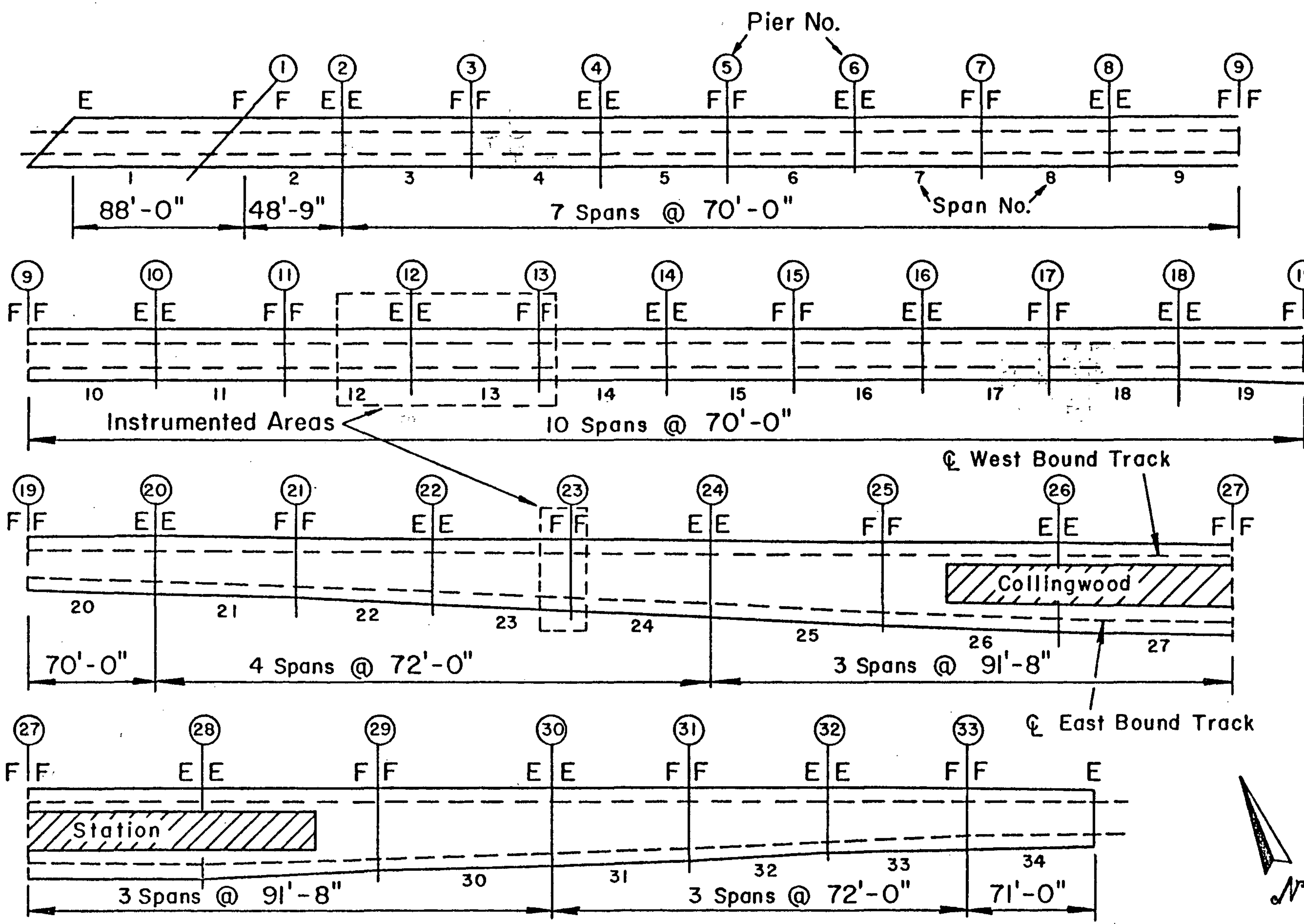


Fig. 1 Locational Instrumented Areas - Collingswood Viaduct

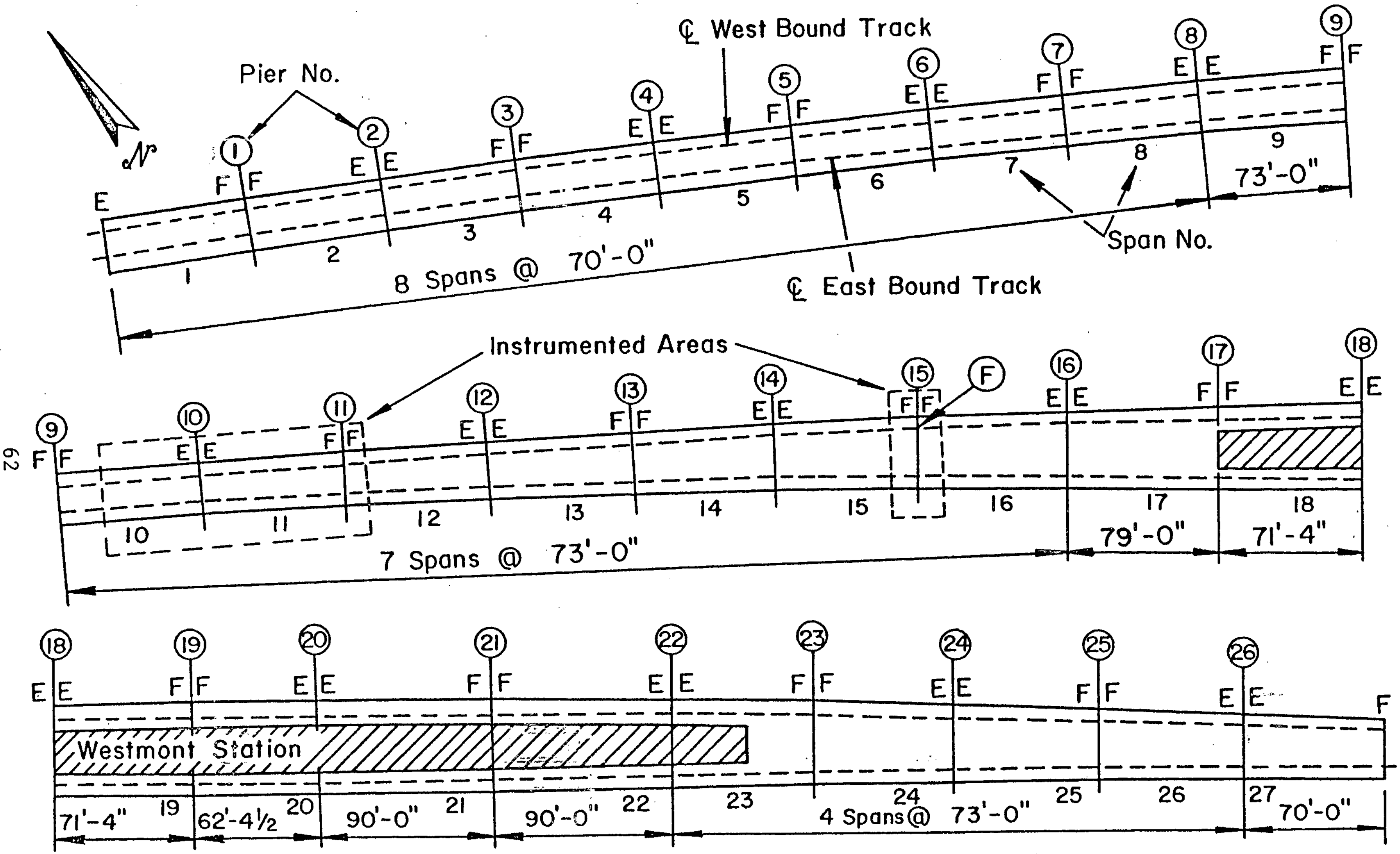


Fig. 2 Locations of Instrumented Areas - Westmont Viaduct

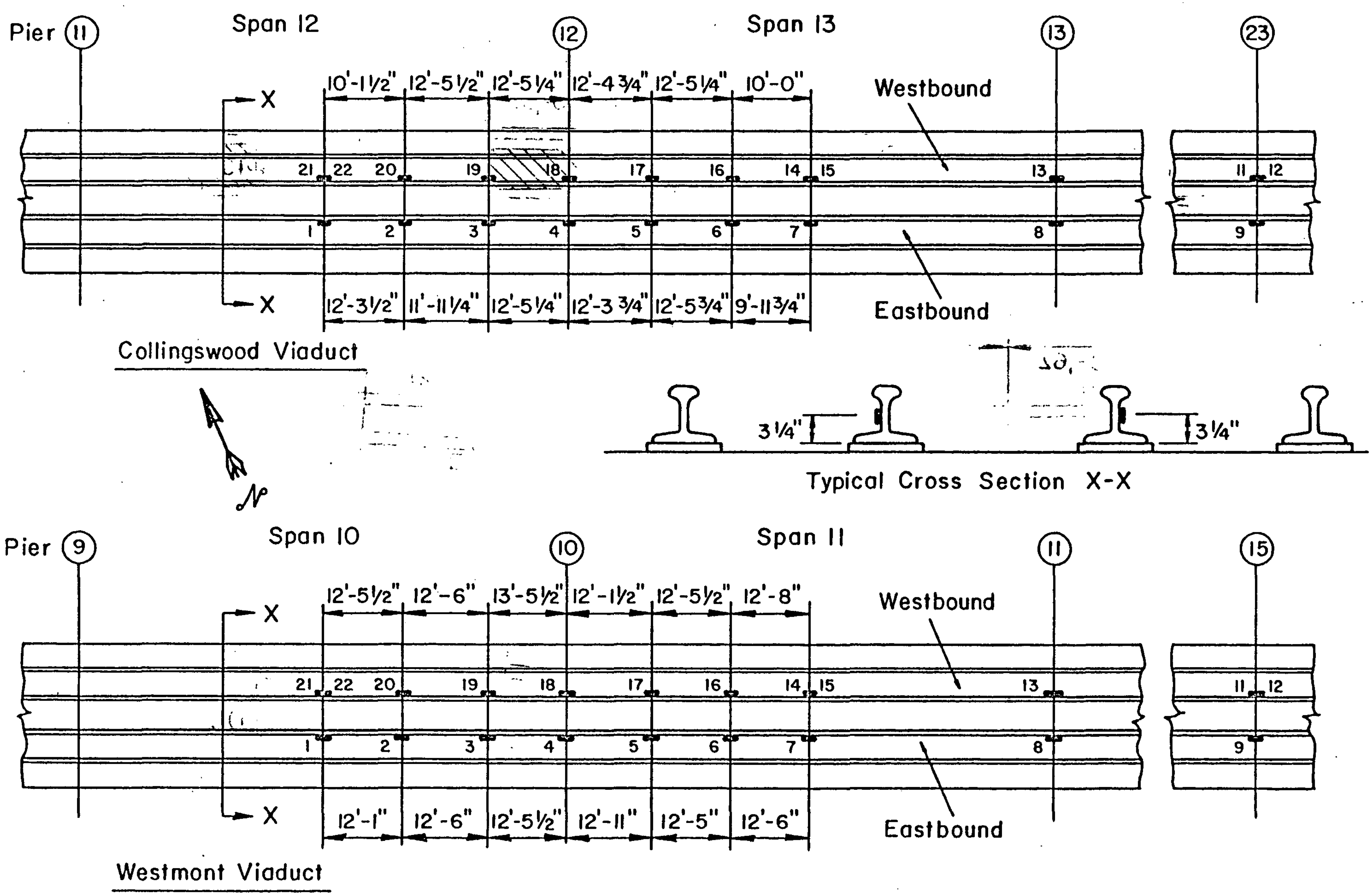
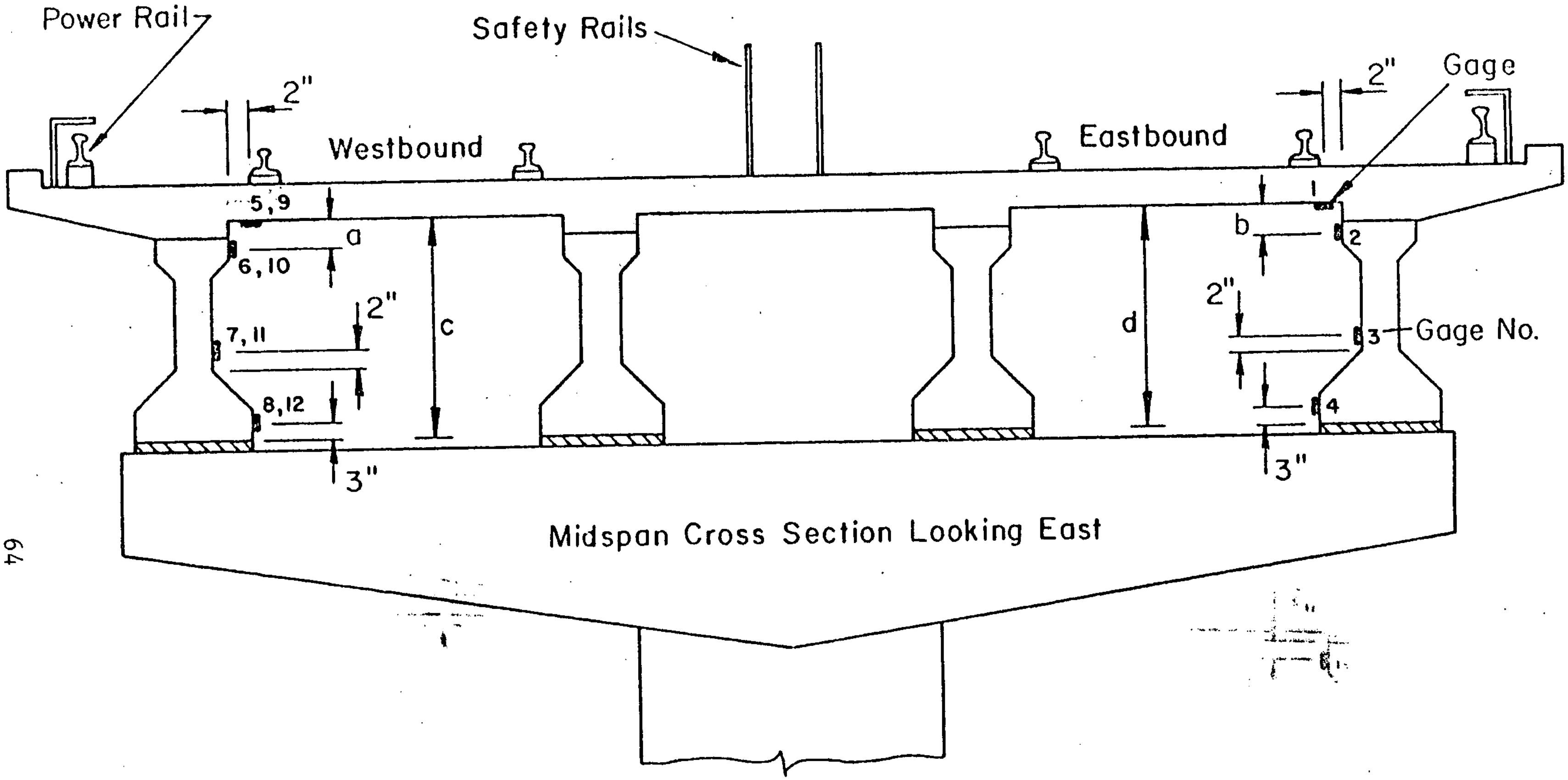
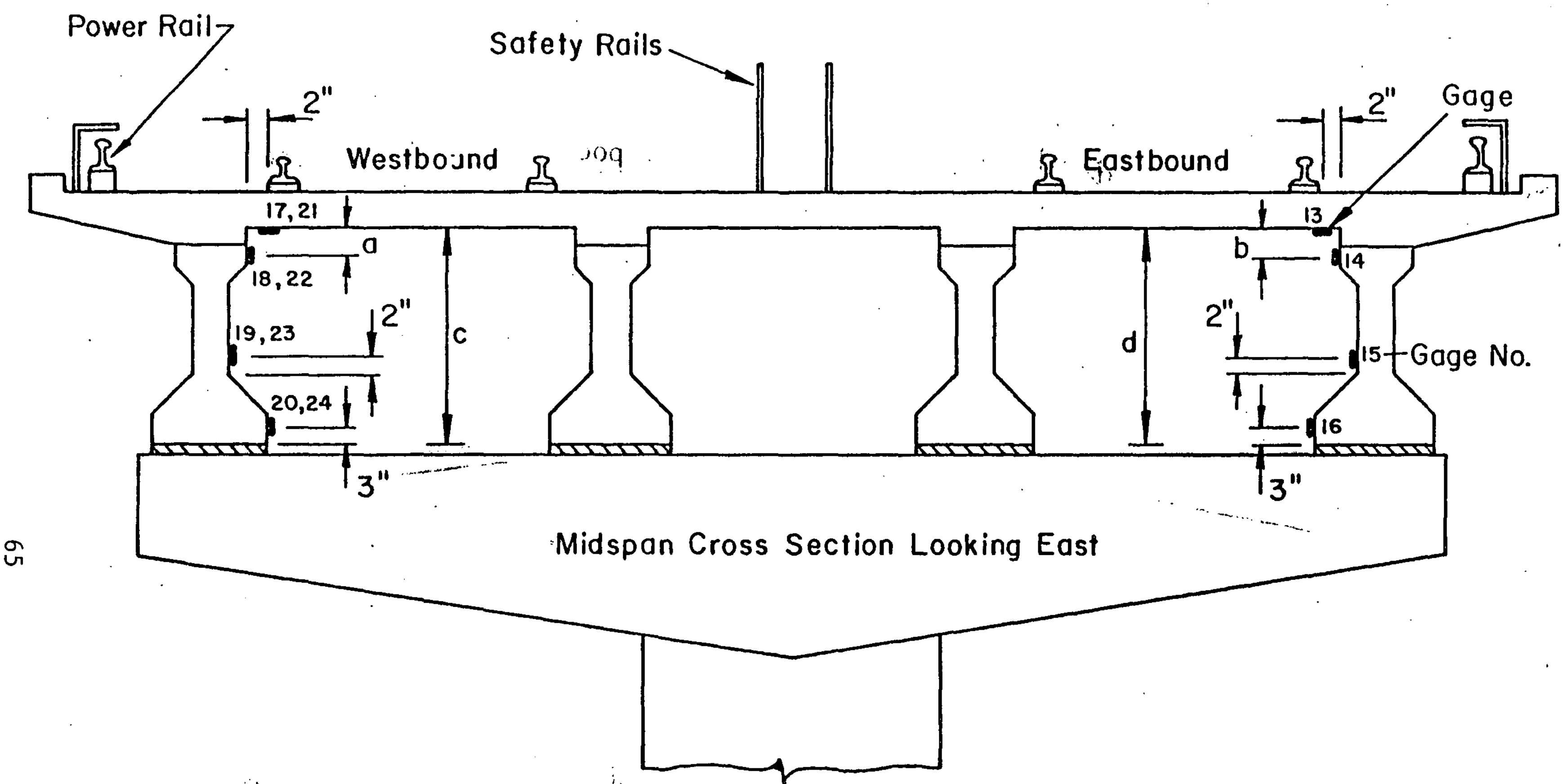


Fig. 3 Instrumentation of Rails - Collingswood and Westmont Viaducts



	Span	a	b	c	d
Collingswood	12	5"	5"	48"	48"
Westmont	10	7"	7"	49½"	50½"

Fig. 4 Instrumentation of Deck and Girders - Collingswood Viaduct Span 12 and Westmont Viaduct Span 10



Span		a	b	c	d
Collingswood	13	5"	5"	48½"	48"
Westmont	11	7"	7"	49"	51"

Fig. 5 Instrumentation of Deck and Girders - Collingswood Viaduct Span 13 and Westmont Viaduct Span 11

Fig. 6 Completed Installation of a Strain Gage to a Rail

Fig. 7 Strain Gage Installation near the Bottom of a Prestressed Girder

Fig. 8 Installation of Wiring to a Terminal Box

Fig. 9 View of Switch Box, the Two Indicators and the Two Cables leading to the Terminal Boxes on the Safety Rail and on Top of Pier

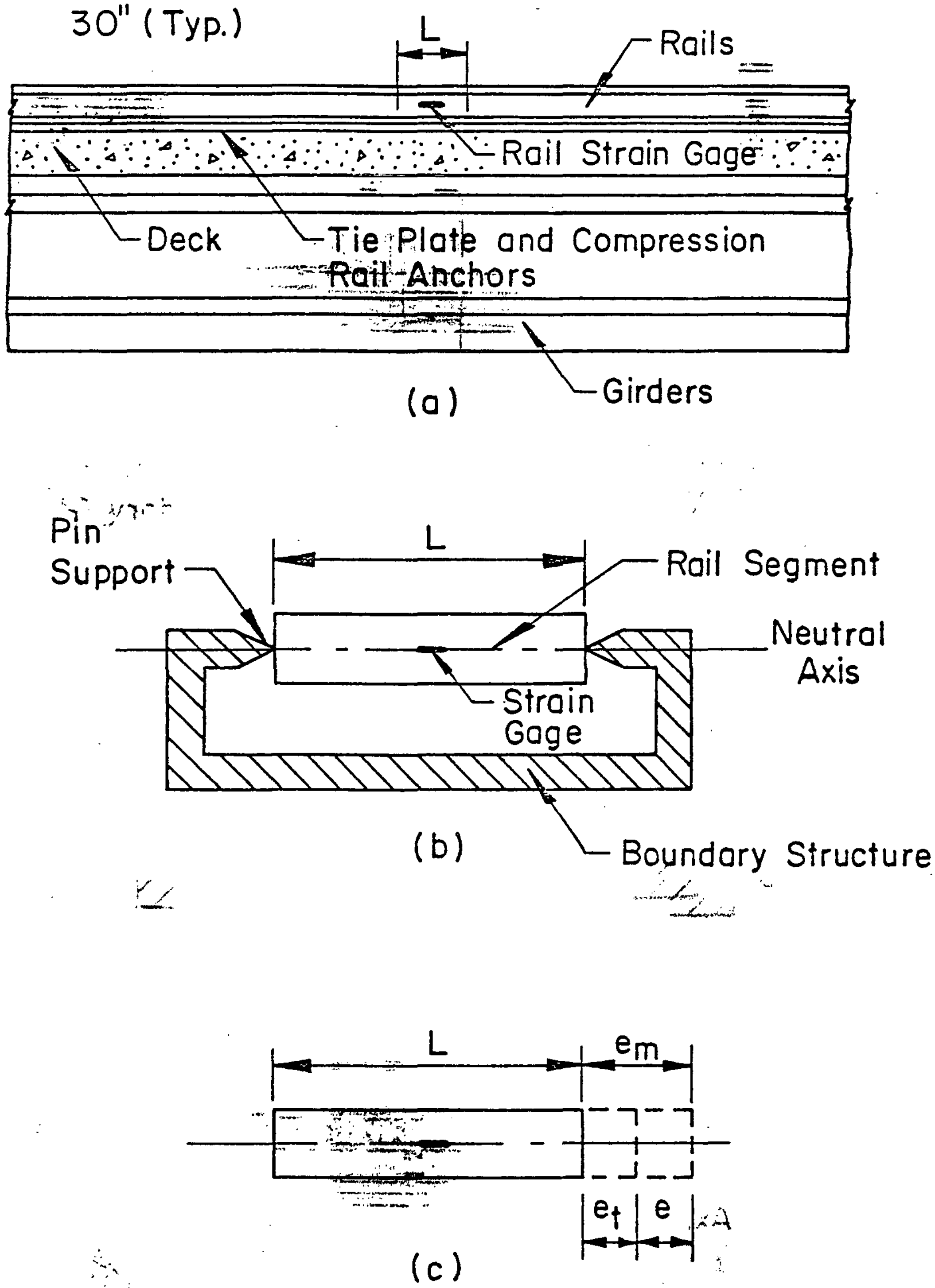


Fig. 10 Simplified Analytical Model used to Compute Change of Stress in Rail

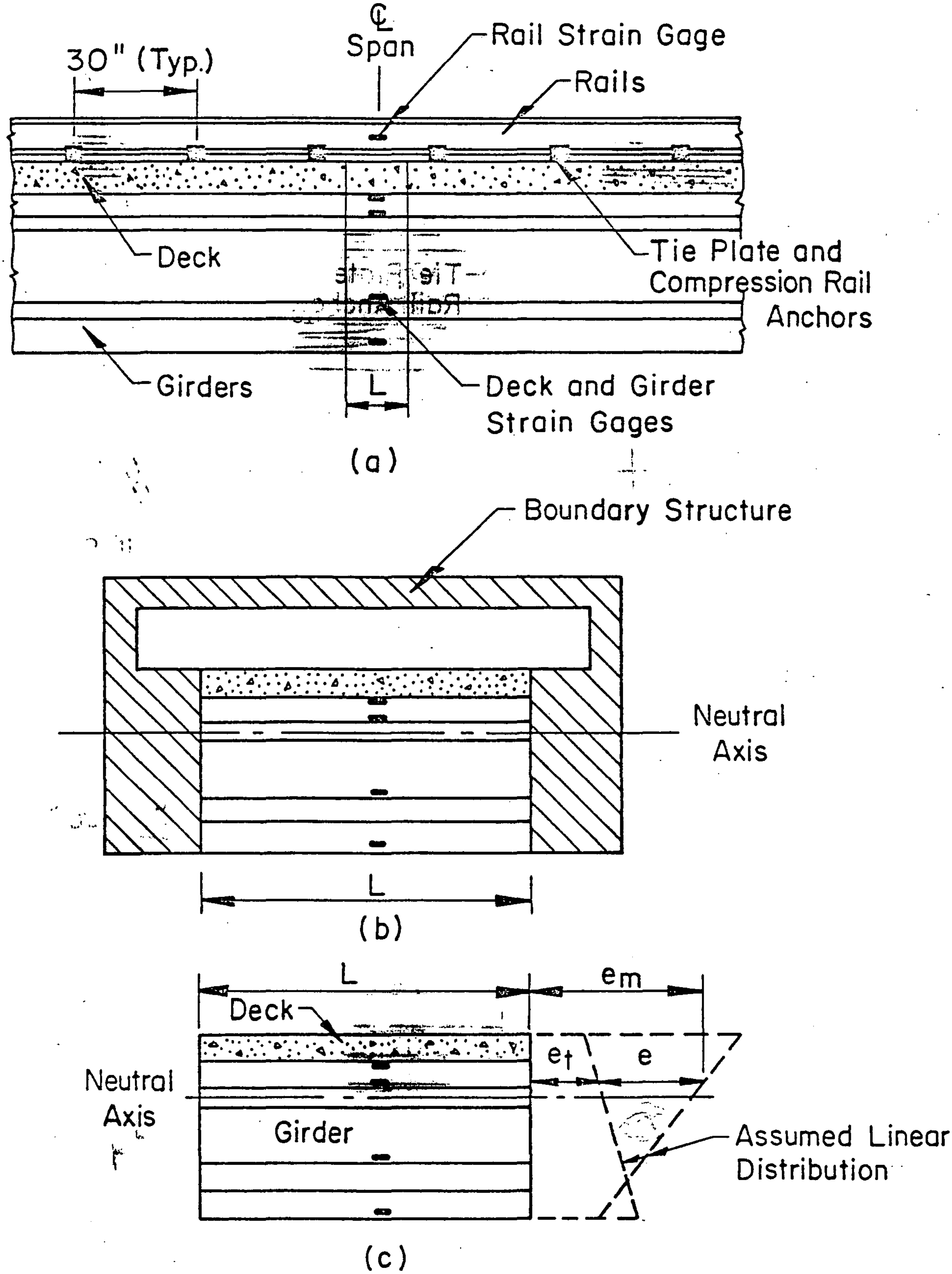


Fig. 11 Simplified Analytical Model used to Compute Change of Stress in Girder and Deck

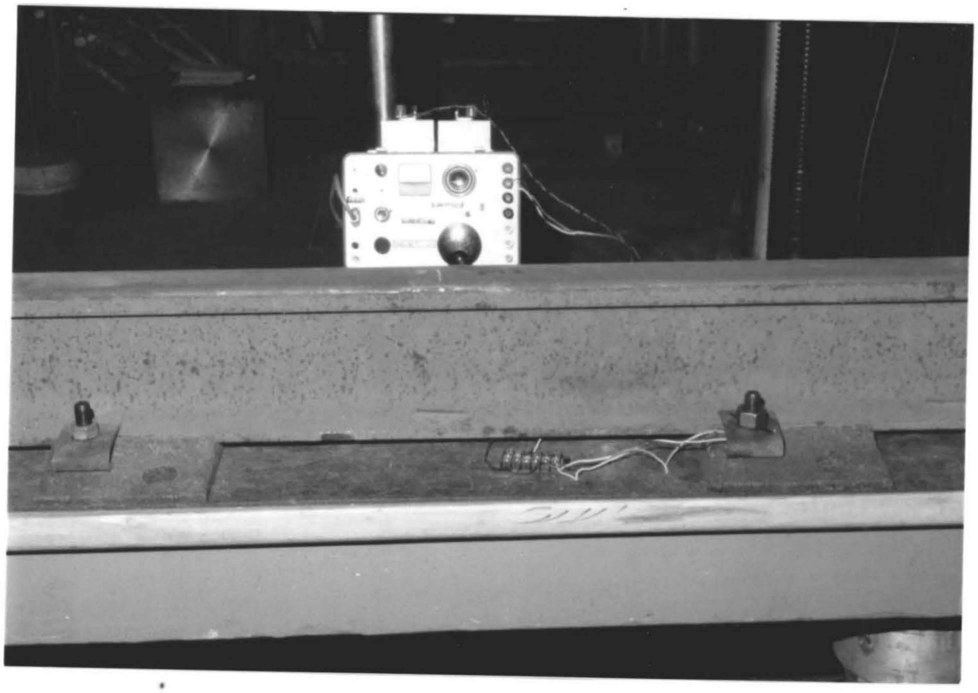


Fig. 12 View of Test Setup used to Determine Bolt Face vs Turn-of-the-Nut

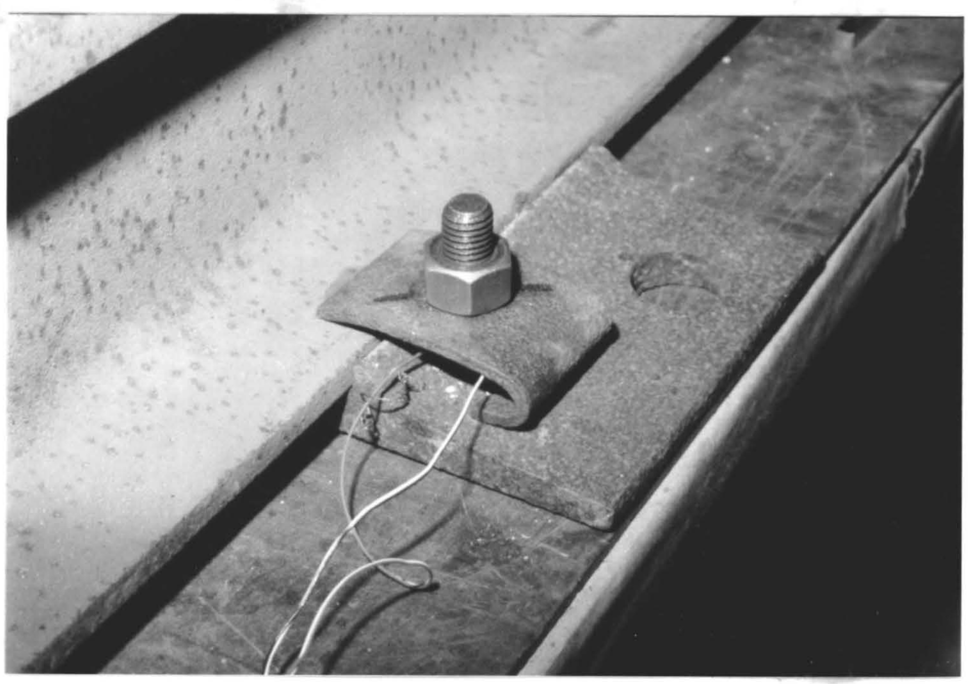


Fig. 13 View of the Test Location Shown on the Right Side of Fig. 12

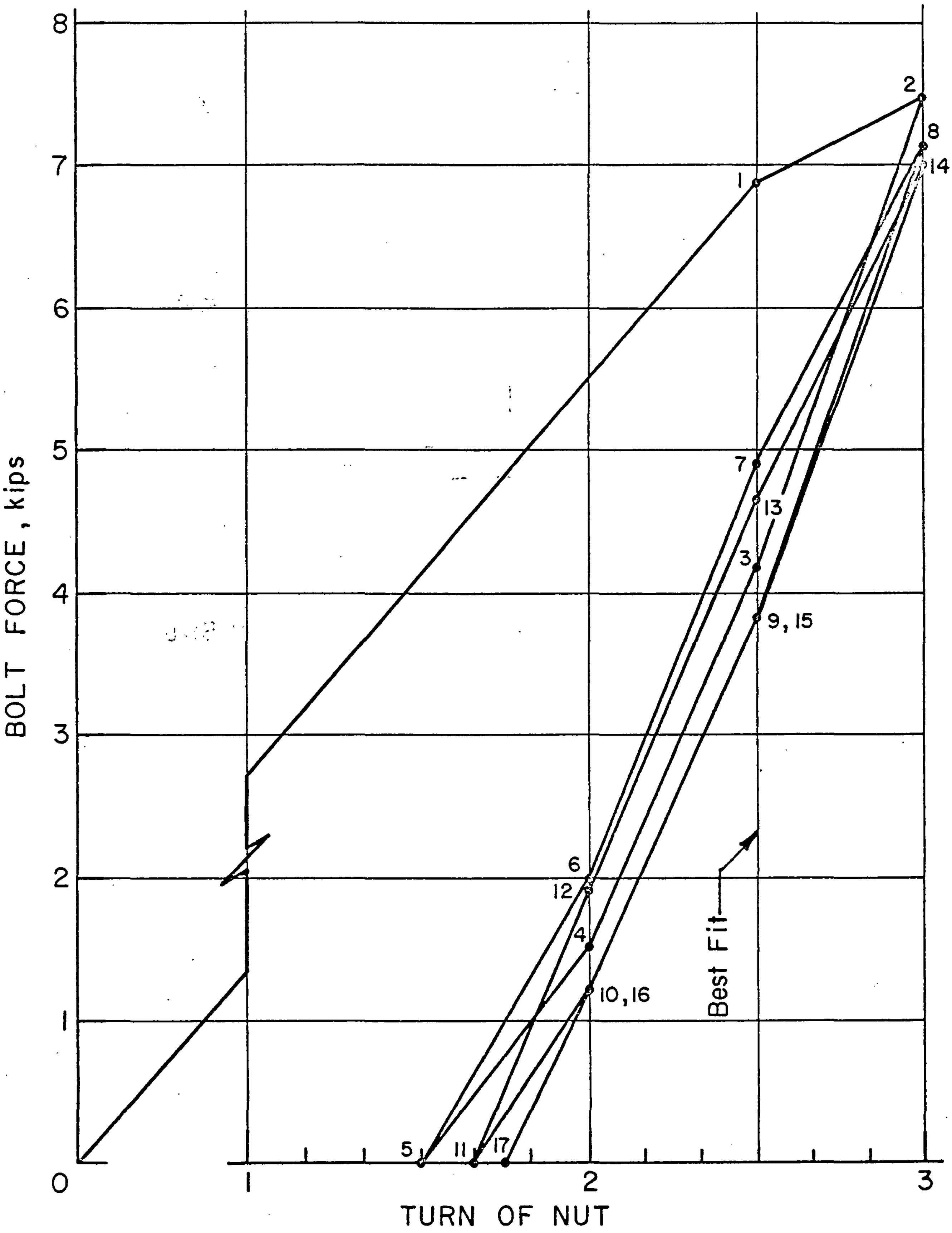


Fig. 14 Test 1 Results - Bolt Force vs Turn-of-the-Nut

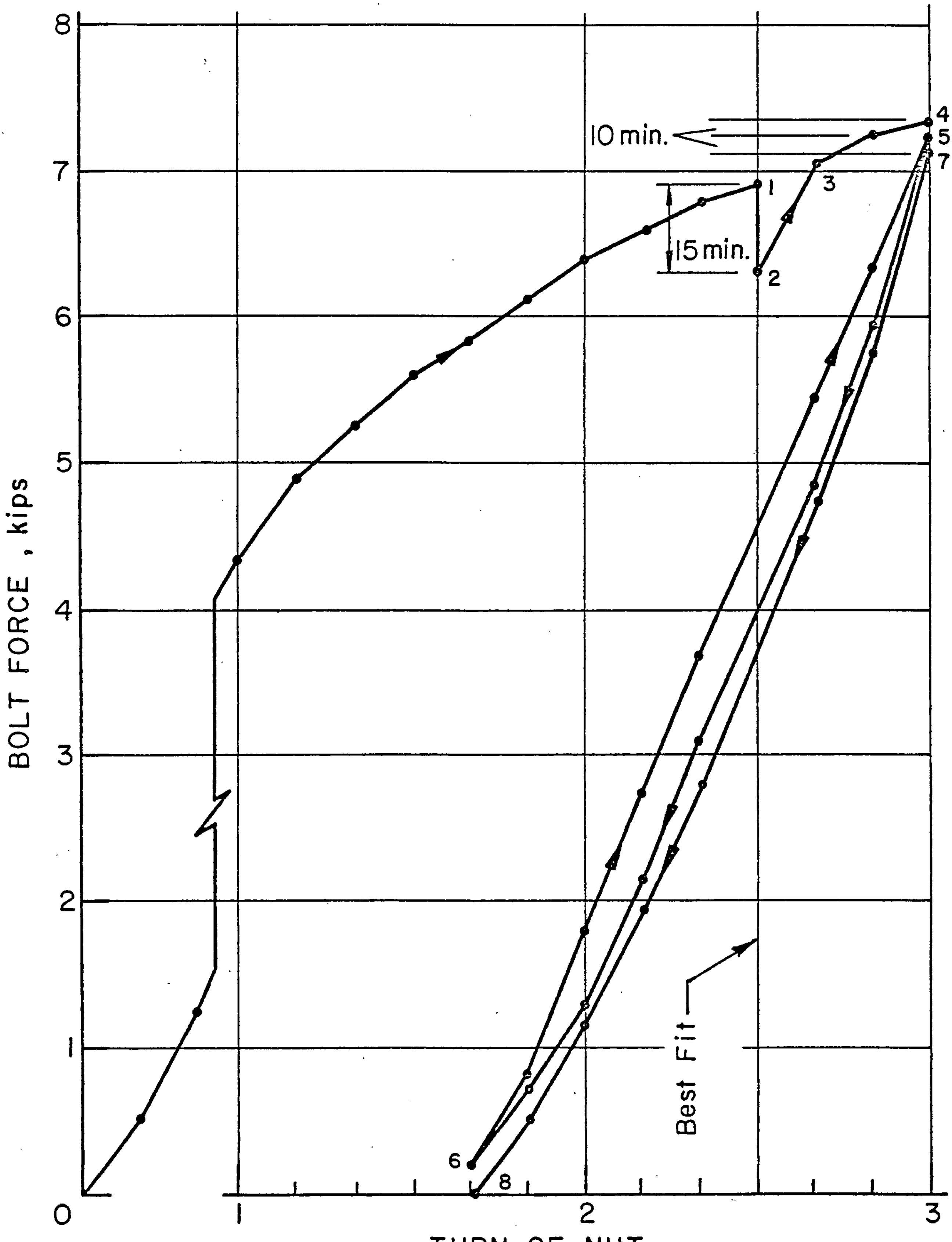


Fig. 15 Test 2 Results - Bolt Force vs Turn-of-the-Nut

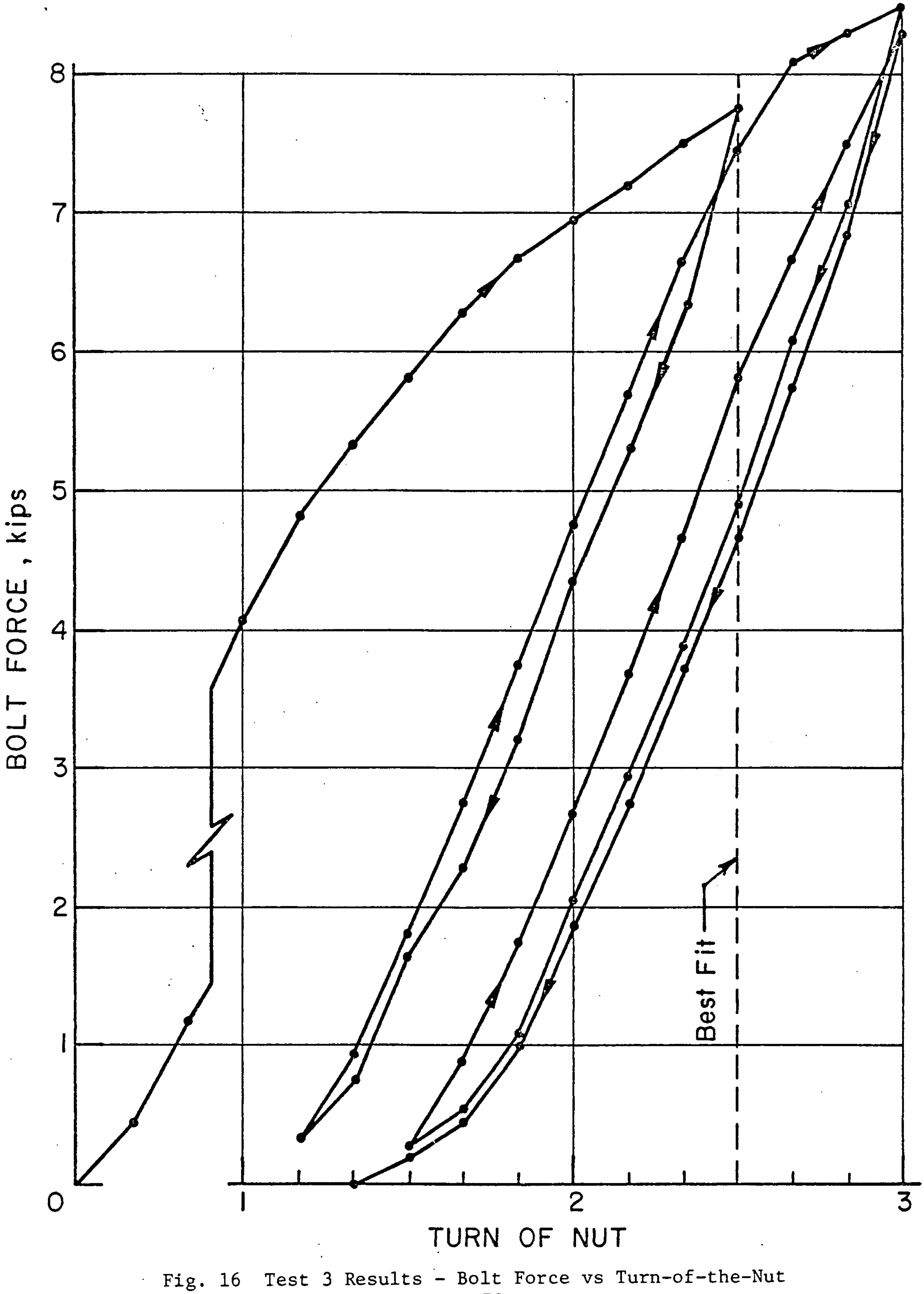


Fig. 16 Test 3 Results - Bolt Force vs Turn-of-the-Nut

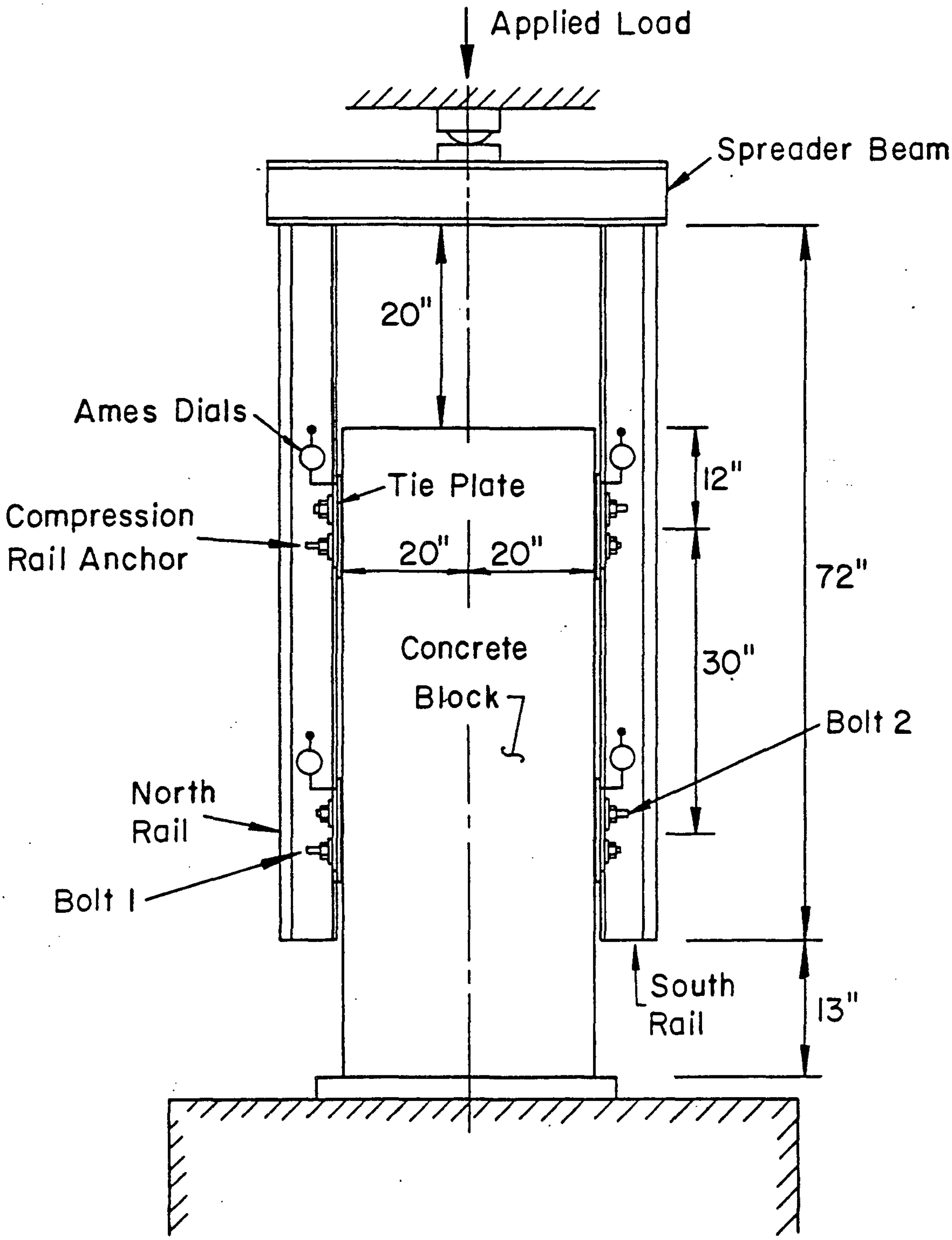


Fig. 17 Schematic of Test Setup and Instrumentation - Compression Rail Anchor Restraint Tests

Fig. 18 Compression Rail Anchor Restraint Test in Progress

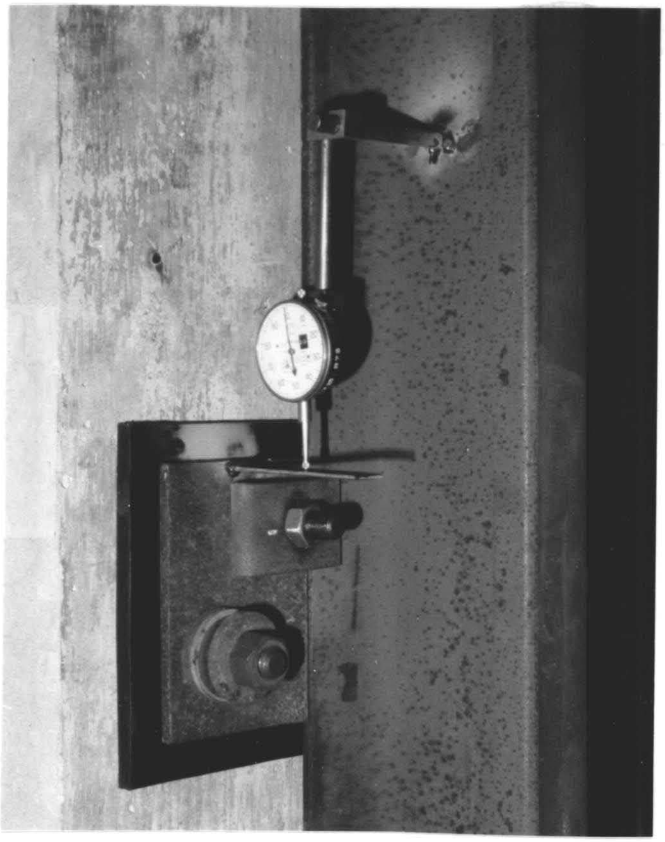


Fig. 19 Close-Up View of Bolt 2 Location (Fig. 17)

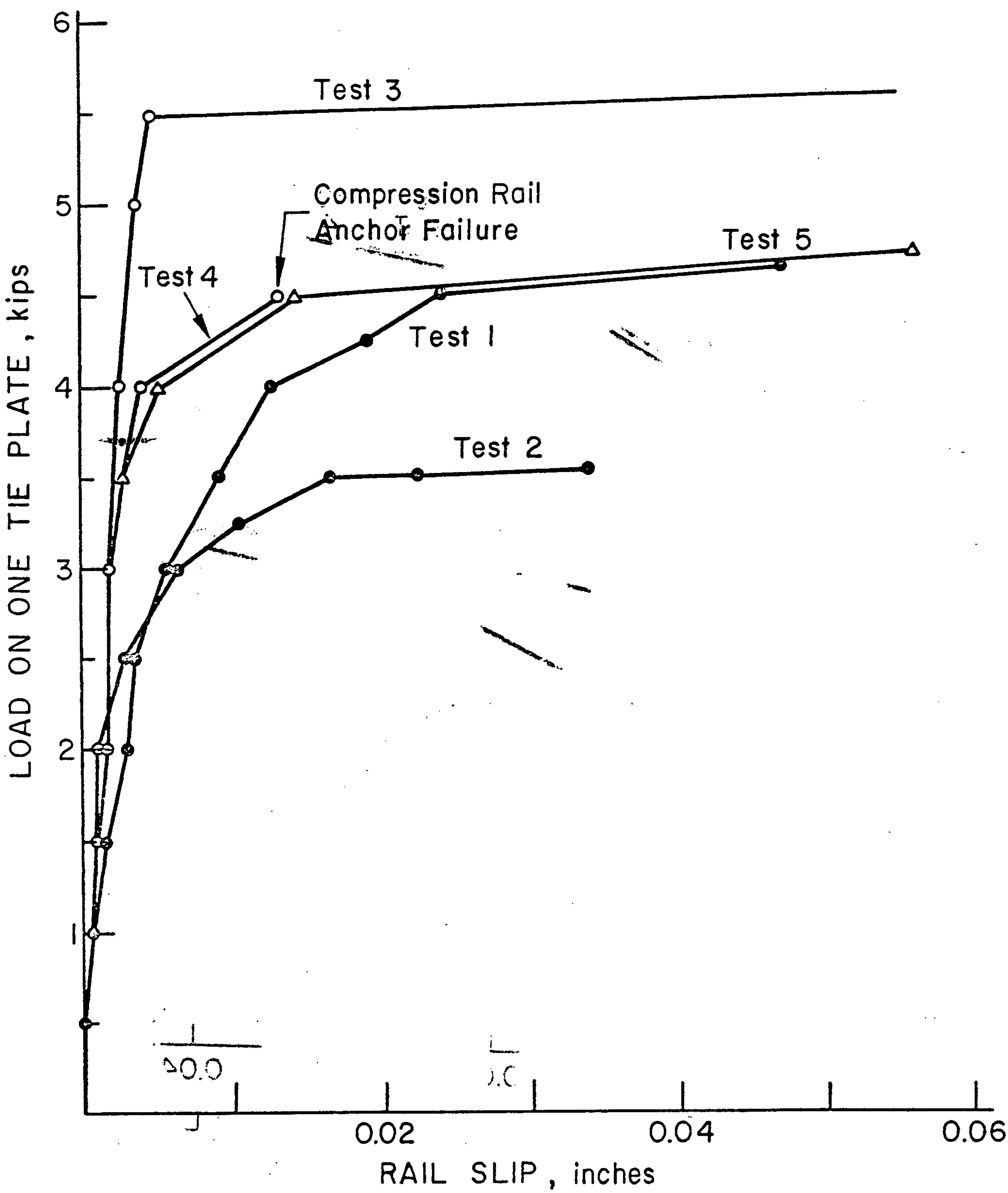


Fig. 20 Results of February 18 Compression Rail Anchor Restraint Tests

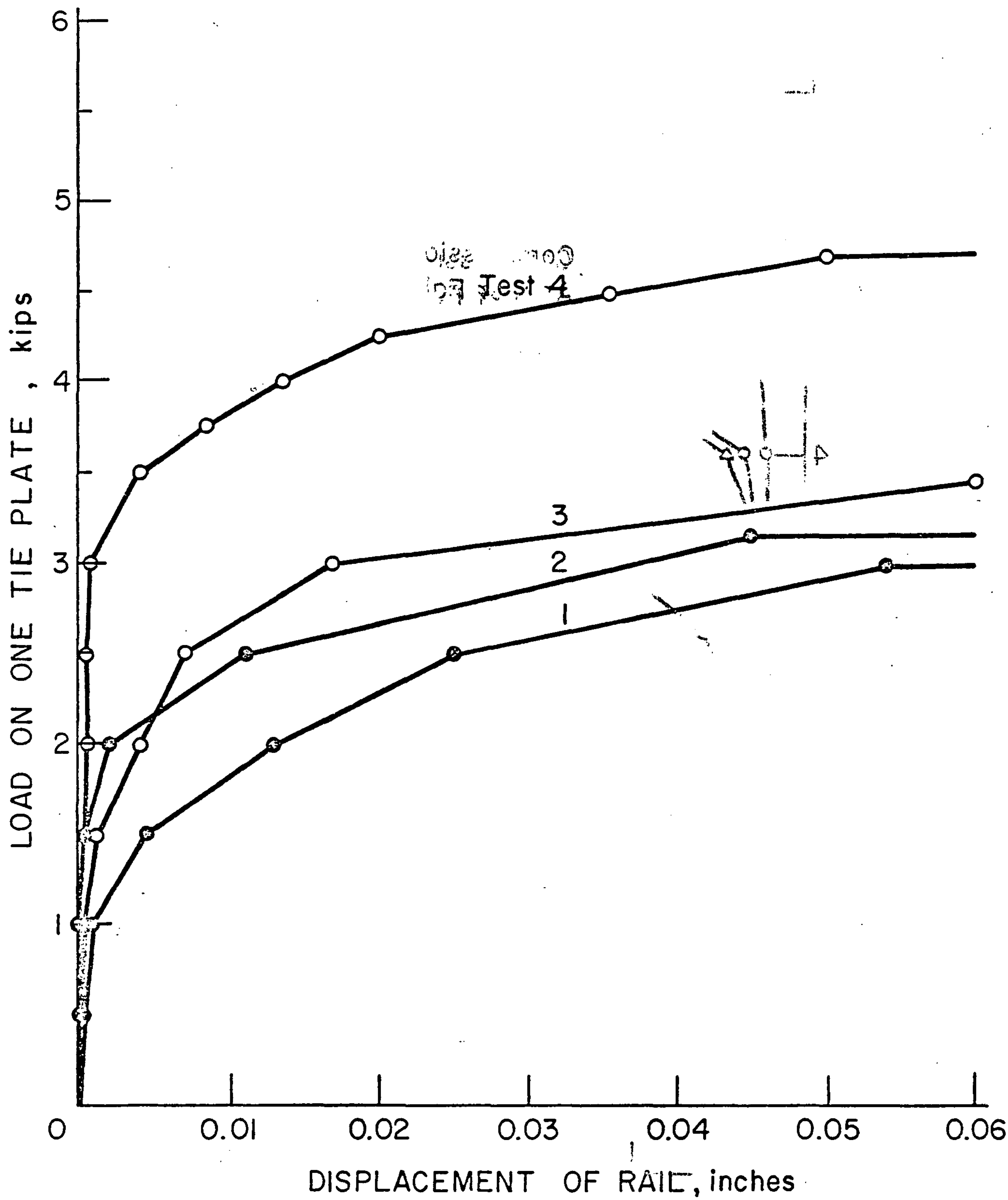


Fig. 21 Results of April 14 Compression Rail Anchor Restraint Tests

8. REFERENCES

1. Gannett, Fleming, Corddry and Carpenter, Inc.
   "PROPOSAL - To Provide Engineering Services for the Inspection, Investigation, and Evaluation of the Collingswood and Westmont Viaducts," Gannett, Fleming, Corddry and Carpenter, Inc., PO Box 1963, Harrisburg, Pennsylvania 17105, June 1979.

2. Lavanchy, D.
   "Concrete Railroad Viaducts with Maintenance Free Track Support System," a Thesis presented to the graduate committee of Lehigh University in candidacy for the degree of Master of Science in Civil Engineering, Lehigh University, Bethlehem, Pennsylvania 18015, December 1980.

3. Gannett, Fleming, Corddry and Carpenter, Inc. and Frederick R. Harris, Inc., "Viaduct Anchorage Assemblies - Trackwork Type "V" and "V-1"," DRPA Southern New Jersey Rapid Transit System, Haddonfield-Kirkwood Line, Dwgs. Sheet 52 of 67, September 1966.

```
      PROGRAM ONE(INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
      INTEGER E(12),GN(10)
C     THIS PROGRAM CALCULATES THE CHANGES IN RAIL STRESSES
C     FOR THE COLLINGSWOOD VIADUCT FROM AUG. 10, 1980 TO
C     FEB. 7, 1981
C      E = CALENDAR DATES
C      GN = GAGE NUMBER
C     READ IN AND PRINT OUT HEADINGS
      READ(5,100)E
  100 FORMAT(12A4)
      WRITE(6,110)
  110 FORMAT(1H1,3X,≠COLLINGSWOOD VIADUCT - RAIL STRESS - ≠
     *,≠AUG 10/80 TO FEB 7/81≠,///)
      WRITE(6,120)E
  120 FORMAT(4X,6(A4,1H-,A4,1X))
      WRITE(6,130)
  130 FORMAT(4X,6(9H---------,1X,),/)
C     READ IN GAGE NUMBERS AND CALCULATE CHANGES IN RAIL
C     STRESSES
C      TEMP. GAGE 22 - USE WITH STRAIN GAGES 1, 2, 3, 19, 20
C      AND 21
C      TEMP. GAGES 15 AND 22 - AVERAGES THE TWO GAGE
C      REASINGS AND USE WITH STRAIN GAGES 4 AND 18
C      TEMP. GAGE 15 - USE WITH STRAIN GAGES 5, 6, 7, 8, 13,
C      14, 16 AND 17
C      TEMP. GAGE 12 - USE WITH STRAIN GAGES 9 AND 11
      DO 1 I=1,4
      READ(5,200)N,GN
  200 FORMAT(11I2)
    1 CALL RST(N,GN)
      STOP
      END

      SUBROUTINE RST(N,GN)
      DIMENSION T(8),S(10,8),ST(10,5),GN(10)
C      T = RAIL TEMPERATURE GAGE READING
C      S = RAIL STRAIN GAGE READING
C      ST = CHANGE IN RAIL STRESS
C      ALP = COEF. OF EXPANSION OF STEEL
C      EE = ELASTIC MODULUS
      ALP=0.0000065
      EE=29500000.
C     READ IN RAIL TEMPERATURES, STRAIN GAGE READINGS
      READ(5,90)T
      READ(5,90)((S(I,J),J=1,8),I=1,N)
   90 FORMAT(8F10.0)
```

```
C     CALCULATE AVERAGE OF RAIL TEMPERATURES AND AVERAGE OF
C     RAIL STRAIN GAGE READINGS
      DO 4 I=1,4
      II=I*2
      IJ=II-1
    4 T(I)=(T(II)+T(IJ))/2.
      DO 1 I=1,N
      DO 1 J=1,4
      II=J*2
      IJ=II-1
    1 S(I,J)=(S(I,II)+S(I,IJ))/2.
C     COMPUTE CHANGES IN RAIL STRESSES - ONE STRAIN GAGE AT
C     A TIME
      DO 3 I=1,N
      M=0
      DO 2 K=1,3
      KK=0
      DO 2 L=K,3
      KK=L+1
      DELT=T(KK)-T(K)
      DELS=(S(I,KK)-S(I,K))*0.000001
      M=M+1
    2 ST(I,M)=(DELS-ALP*DELT)*EE
    3 WRITE(6,91)GN(I),(ST(I,M),M=1,6)
   91 FORMAT(/,2X,I2,6(F9.1,1X))
      RETURN
      END
```

APPENDIX B

```
      PROGRAM TWO  (INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
      INTEGER E(10),GN(10)
C     THIS PROGRAM CALCULATES THE CHANGES IN RAIL STRESSES
C     FOR THE COLLINGSWOOD VIADUCT ON SEP. 15, 1980
C      E = START TIMES
C      GN = GAGE NUMBER
C     READ IN AND PRINT OUT HEADINGS
      READ(5,100)E
  100 FORMAT(10A4)
      WRITE(6,110)
  110 FORMAT(1H1,3X,≠COLLINGSWOOD VIADUCT - RAIL STRESS - ≠
     *,≠SEPTEMBER 15, 1980≠,///)
      WRITE(6,120)E
  120 FORMAT(8X,5(A4,1H-,A4,1X))
      WRITE(6,130)
  130 FORMAT(8X,5(9H---------,1X,),/)
C     READ IN GAGE NUMBERS AND CALCULATE CHANGES IN RAIL
C     STRESSES
C      TEMP. GAGE 22 - USE WITH STRAIN GAGES 1, 2, 3, 19, 20
C      AND 21
C      TEMP. GAGES 15 AND 22 - AVERAGES THE TWO GAGE
C      REASINGS AND USE WITH STRAIN GAGES 4 AND 18
C      TEMP. GAGE 15 - USE WITH STRAIN GAGES 5, 6, 7, 8, 13,
C      14, 16 AND 17
C      TEMP. GAGE 12 - USE WITH STRAIN GAGES 9 AND 11
      DO 1 I=1,4
      READ(5,200)N,GN
  200 FORMAT(I1I2)
    1 CALL RST(N,GN)
      STOP
      END


      SUBROUTINE RST(N,GN)
      DIMENSION T(6),S(10,6),ST(10,5)
      INTEGER GN(10)
C      T = RAIL TEMPERATURE GAGE READING
C      S = RAIL STRAIN GAGE READING
C      ST = CHANGE IN RAIL STRESS
C      ALP = COEF. OF EXPANSION OF STEEL
C      EE = ELASTIC MODULUS
      ALP=0.0000065
      EE=29500000.
C     READ IN RAIL TEMPERATURES, STRAIN GAGE READINGS
      READ(5,90)T
      READ(5,90)((S(I,J),J=1,6),I=1,N)
   90 FORMAT(6F10.0)
```

```
C     COMPUTE CHANGES IN RAIL STRESSES - ONE STRAIN GAGE AT
C     A TIME
      DO 1 I=1,N
      M=0
      K=1
      KK=0
      DO 2 L=K,5
      KK=L+1
      DELT=T(KK)-T(K)
      DELS=(S(I,KK)-S(I,K))*0.000001
      M=M+1
    2 ST(I,M)=(DELS-ALP*DELT)*EE
    1 WRITE(6,91)GN(I),(ST(I,M),M=1,5)
   91 FORMAT(/,6X,I2,5(F9.1,1X))
      RETURN
      END
```

```
      PROGRAM THREE(INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
      DIMENSION T(4,8),S(4,8)
      INTEGER E(12)
C
C     THIS PROGRAM CALCULATES THE CHANGES IN GIRDER STRESSES
C     FOR THE COLLINGSWOOD VIADUCT FROM AUG. 10, 1980 TO
C     FEB. 7, 1981
C     THIS PROGRAM USES THE SAME MODEL AS THAT USED TO
C     CALCULATE THE RAIL STRESSES
C
C      E = CALENDAR DATES
C      GN = GAGE NUMBER
C
C     READ IN AND PRINT OUT HEADINGS
C
      WRITE(6,100)
  100 FORMAT(1H1,2X,≠COLLINGSWOOD VIADUCT - GIRDER STRESS -≠
     *,≠ AUG 10/80 TO FEB 7/81≠,///)
      READ(5,110)E
  110 FORMAT(12A4)
      WRITE(6,120)E
  120 FORMAT(4X,6(A4,1H-,A4,1X))
      WRITE(6,130)
  130 FORMAT(4X,6(9H---------,1X),//)
C
C     CALCULATE CHANGES IN GIRDER STRESSES - ONE GIRDER AT A
C     TIME
C
C      TEMP. GAGE  9 - USE WITH STRAIN GAGES  1 AND  5
C      TEMP. GAGE 10 - USE WITH STRAIN GAGES  2 AND  6
C      TEMP. GAGE 11 - USE WITH STRAIN GAGES  3 AND  7
C      TEMP. GAGE 12 - USE WITH STRAIN GAGES  4 AND  8
C      TEMP. GAGE 21 - USE WITH STRAIN GAGES 13 AND 17
C      TEMP. GAGE 22 - USE WITH STRAIN GAGES 14 AND 18
C      TEMP. GAGE 23 - USE WITH STRAIN GAGES 15 AND 19
C      TEMP. GAGE 24 - USE WITH STRAIN GAGES 16 AND 20
C
      DO 1 I=1,4
    1 CALL GST(GN)
      STOP
      END


      SUBROUTINE GST(GN)
      DIMENSION T(4,8),S(4,8),ST(4,6)
      INTEGER GN(4)
C
C      T = GIRDER TEMPERATURE GAGE READING
C      S = GIRDER STRAIN GAGE READING
C      ST = CHANGE IN GIRDER STRESS
C      GN = GAGE NUMBER
C      ALP = COEF. OF EXPANSION OF CONCRETE
C      EE = ELASTIC MODULUS
C
      ALP=0.000006
      EE=4030000.
C
C     READ IN GIRDER TEMPERATURES, STRAIN GAGE READINGS AND
C     GAGE NUMBERS
C
      READ(5,92)GN
   92 FORMAT(4I2)
      READ(5,90)  ((T(I,J),J=1,8),I=1,4)
      READ(5,90)  ((S(I,J),J=1,8),I=1,4)
   90 FORMAT(8F10.0)
```

```
C      CALCULATE AVERAGE OF GIRDER TEMPERATURES AND AVERAGE
C      OF GIRDER STRAIN GAGE READINGS
       DO 1 I=1,4
       DO 1 J=1,4
       II=J*2
       IJ=II-1
       T(I,J)=(T(I,II)+T(I,IJ))/2.
     1 S(I,J)=(S(I,II)+S(I,IJ))/2.
C      COMPUTE CHANGES IN GIRDER STRESSES - ONE STRAIN GAGE
C      AT A TIME
       DO 2 I=1,4
       M=0
       DO 3 K=1,3
       KK=0
       DO 3 L=K,3
       KK=L+1
       DELT=T(I,KK)-T(I,K)
       DELS=(S(I,KK)-S(I,K))*0.000001
       M=M+1
     3 ST(I,M)=(DELS-ALP*DELT)*EE
     2 WRITE(6,91)GN(I),(ST(I,M),M=1,6)
    91 FORMAT(2X,I2,6(F9.1,1X),/)
       RETURN
       END
```

APPENDIX D

```
      PROGRAM FOUR (INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
      DIMENSION T(4,6),S(4,6)
      INTEGER E(10),GN(4)

C     THIS PROGRAM CALCULATES THE CHANGES IN GIRDER STRESSES
C     FOR THE COLLINGSWOOD VIADUCT ON SEP. 15, 1980
C     THIS PROGRAM USES THE SAME MODEL AS THAT USED TO
C     CALCULATE THE RAIL STRESSES

C      E = START TIMES
C      GN = GAGE NUMBER

C     READ IN AND PRINT OUT HEADINGS

      READ(5,100)E
  100 FORMAT(10A4)
      WRITE(6,110)
  110 FORMAT(1H1,3X,≠COLLINGSWOOD VIADUCT - GIRDER STRESS -≠
     *,≠ SEPTEMBER 15, 1980≠,///)
      WRITE(6,120)E
  120 FORMAT(8X,5(A4,1H-,A4,1X))
      WRITE(6,130)
  130 FORMAT(8X, 5(9H---------,1X),/)

C     READ IN GAGE NUMBERS AND CALCULATE CHANGES IN GIRDER
C     STRESSES - ONE GIRDER AT A TIME

C      TEMP. GAGE  9 - USE WITH STRAIN GAGES  1 AND  5
C      TEMP. GAGE 10 - USE WITH STRAIN GAGES  2 AND  6
C      TEMP. GAGE 11 - USE WITH STRAIN GAGES  3 AND  7
C      TEMP. GAGE 12 - USE WITH STRAIN GAGES  4 AND  8
C      TEMP. GAGE 21 - USE WITH STRAIN GAGES 13 AND 17
C      TEMP. GAGE 22 - USE WITH STRAIN GAGES 14 AND 18
C      TEMP. GAGE 23 - USE WITH STRAIN GAGES 15 AND 19
C      TEMP. GAGE 24 - USE WITH STRAIN GAGES 16 AND 20

      DO 1 I=1,4
      READ(5,140)GN
  140 FORMAT(4I2)
    1 CALL GST(GN)
      STOP
      END


      SUBROUTINE GST(GN)
      DIMENSION T(4,6),S(4,6),ST(4,5)
      INTEGER GN(4)

C      T = GIRDER TEMPERATURE GAGE READING
C      S = GIRDER STRAIN GAGE READING
C      ST = CHANGE IN GIRDER STRESS
C      GN = GAGE NUMBER
C      ALP = COEF. OF EXPANSION OF CONCRETE
C      EE = ELASTIC MODULUS

      ALP=0.000006
      EE=4030000.

C     READ IN GIRDER TEMPERATURES, STRAIN GAGE READINGS

      READ(5,90) ((T(I,J),J=1,6),I=1,4)
      READ(5,90) ((S(I,J),J=1,6),I=1,4)
   90 FORMAT(6F10.0)
```

```
C     COMPUTE CHANGES IN GIRDER STRESSES - ONE STRAIN GAGE
C     AT A TIME
      DO 1 I=1,4
      M=0
      K=1
      KK=0
      DO 2 L=K,5
      KK=L+1
      DELT=T(I,KK)-T(I,K)
      DELS=(S(I,KK)-S(I,K))*0.000001
      M=M+1
    2 ST(I,M)=(DELS-ALP*DELT)*EE
    1 WRITE(6,92)GN(I),(ST(I,M),M=1,5)
   92 FORMAT(/,5X,I2,5(F9.1,1X))
      RETURN
      END
```

APPENDIX E

```
      PROGRAM FIVE (INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
      DIMENSION T(4,8),S(4,8),X(4)
C     THIS PROGRAM CALCULATES THE CHANGES IN GIRDER STRESSES
C     FOR THE COLLINGSWOOD VIADUCT FROM AUG. 10, 1980 TO
C     FEB. 7, 1981
C     THIS PROGRAM LINEARIZES THE OUTPUT TO PROGRAMS THREE
C     AND FOUR AND EXTENDS THE RESULTS TO THE TOP OF SLAB
C     AND BOTTOM OF GIRDER.
C      E = CALENDAR DATES
C      GN = GAGE NUMBER
C      T = GIRDER TEMPERATURE GAGE READING
C      S = GIRDER STRAIN GAGE READING
C      X = DISTANCE FROM THE GAGE TO TOP OF SLAB
C      D = DISTANCE FROM TOP OF SLAB TO BOTTOM OF GIRDER
C     READ IN AND PRINT OUT HEADINGS
C      P12 = SPAN 12     P13 = SPAN 13     GD = GIRDER
C      SOU = SOUTH      NOR = NORTH
      READ(5,94)P12,P13,NOR,SOU,GD
   94 FORMAT(2A7,2A5,A6)
      WRITE(6,100)
  100 FORMAT(1H1,2X,≠COLLINGSWOOD VIADUCT - GIRDER STRESS -≠
     *,≠ AUG 10/80 TO FEB 7/81≠,//)
C     CALCULATE CHANGES IN GIRDER STRESSES - ONE GIRDER AT A
C     TIME
C      TEMP. GAGE  9 - USE WITH STRAIN GAGES  1 AND  5
C      TEMP. GAGE 10 - USE WITH STRAIN GAGES  2 AND  6
C      TEMP. GAGE 11 - USE WITH STRAIN GAGES  3 AND  7
C      TEMP. GAGE 12 - USE WITH STRAIN GAGES  4 AND  8
C      TEMP. GAGE 21 - USE WITH STRAIN GAGES 13 AND 17
C      TEMP. GAGE 22 - USE WITH STRAIN GAGES 14 AND 18
C      TEMP. GAGE 23 - USE WITH STRAIN GAGES 15 AND 19
C      TEMP. GAGE 24 - USE WITH STRAIN GAGES 16 AND 20
      WRITE(6,200)P12,SOU,GD
      CALL GST(T,S,X,D)
      WRITE(6,200)P12,NOR,GD
      CALL GST(T,S,X,D)
      WRITE(6,200)P13,SOU,GD
      CALL GST(T,S,X,D)
      WRITE(6,200)P13,NOR,GD
      CALL GST(T,S,X,D)
  200 FORMAT(15X,A10,2A7,/)
      STOP
      END
```

```
      SUBROUTINE GST(T,S,X,D)
      DIMENSION T(4,8),S(4,8),X(4),STY(4),ST(4,6),STT(6),
     *STB(6)
      INTEGER E(12),GN(4)
      READ(5,90)E
   90 FORMAT(12A4)
C      ST = CHANGE IN GIRDER STRESS
C      GN = GAGE NUMBER
C      ALP = COEF. OF EXPANSION OF CONCRETE
C      EE = ELASTIC MODULUS
      ALP=0.000006
      EE=4030000.
C     READ IN GIRDER TEMPERATURES, STRAIN GAGE READINGS,
C     GAGE NUMBERS AND LOCATIONS OF STRAIN GAGES
      READ(5,91)((T(I,J),J=1,8),I=1,4)
      READ(5,91)((S(I,J),J=1,8),I=1,4)
   91 FORMAT(8F10.0)
      READ(5,92)(X(I),I=1,4),D
   92 FORMAT(5F10.0)
      READ(5,94)GN
   94 FORMAT(4I2)
C     CALCULATE AVERAGE OF GIRDER TEMPERATURES AND AVERAGE
C     OF GIRDER STRAIN GAGE READINGS
      DO 1 I=1,4
      DO 1 J=1,4
      II=J*2
      IJ=II-1
      T(I,J)=(T(I,II)+T(I,IJ))/2.
    1 S(I,J)=(S(I,II)+S(I,IJ))/2.
C     COMPUTE CHANGES IN GIRDER STRESSES - ONE STRAIN GAGE
C     AT A TIME
      DO 3 I=1,4
      M=0
      DO 3 K=1,3
      KK=0
      DO 3 L=K,3
      KK=L+1
      DELT=T(I,KK)-T(I,K)
      DELS=(S(I,KK)-S(I,K))*0.000001
      M=M+1
    3 ST(I,M)=(DELS-ALP*DELT)*EE
```

```
C     LINEARIZE THE CHANGES OF STRESSES
      DO 4 J=1,5
      DO 2 I=1,4
    2 STY(I)=ST(I,J)
      N=4
      CALL LNFIT(N,X,STY,A1,A2)
      DO 5 I=1,4
    5 ST(I,J)=A1+A2*X(I)
      STT(J)=A1
    4 STB(J)=A1+A2*D
      WRITE(6,95)E
   95 FORMAT(4X,6(A4,1H-,A4,1X))
      WRITE(6,96)
   96 FORMAT(4X,6(9H---------,1X))
      WRITE(6,93)STT,(GN(I),(ST(I,J),J=1,6),I=1,4),ST3
   93 FORMAT(2X,≠TOP≠,/,1X,≠SLAB≠,F8.1,5F10.1,//,4(3X,I2,
     *F8.1,5F10.1,/),2X,≠BTM≠,/,1X,≠GIRD≠,F8.1,5F10.1,//)
      RETURN
      END


      SUBROUTINE LNFIT(N,X,Y,A,B)
C     LINEAR LEAST SQUARES APPROXIMATION
      DIMENSION X(10),Y(10)
      S1=0.
      S2=0.
      S3=0.
      S4=0.
      DO 1 I=1,N
      S1=X(I)+S1
      S2=X(I)**2+S2
      S3=Y(I)+S3
    1 S4=X(I)*Y(I)+S4
      D=N*S2-S1**2
      A=(S2*S3-S1*S4)/D
      B=(N*S4-S1*S3)/D
      RETURN
      END
```

APPENDIX F

```
      PROGRAM SIX   (INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
      DIMENSION T(4,6),S(4,6),X(4)
C
C     THIS PROGRAM CALCULATES THE CHANGES IN GIRDER STRESSES
C     FOR THE COLLINGSWOOD VIADUCT ON SEP. 15, 1980
C     THIS PROGRAM LINEARIZES THE OUTPUT TO PROGRAMS THREE
C     AND FOUR AND EXTENDS THE RESULTS TO THE TOP OF SLAB
C     AND BOTTOM OF GIRDER.
C
C      E = START TIMES
C      GN = GAGE NUMBER
C      T = GIRDER TEMPERATURE GAGE READING
C      S = GIRDER STRAIN GAGE READING
C      X = DISTANCE FROM THE GAGE TO TOP OF SLAB
C      D = DISTANCE FROM TOP OF SLAB TO BOTTOM OF GIRDER
C
C     READ IN AND PRINT OUT HEADINGS
C
C      P12 = SPAN 12      P13 = SPAN 13      GD = GIRDER
C      SOU = SOUTH      NOR = NORTH
C
      READ(5,100)P12,P13,NOR,SOU,GD
  100 FORMAT(2A7,2A5,A6)
      WRITE(6,110)
  110 FORMAT(1H1,3X,≠COLLINGSWOOD VIADUCT - GIRDER STRESS -≠
     *,≠ SEPTEMBER 15 1980≠,///)
C
C     CALCULATE CHANGES IN GIRDER STRESSES - ONE GIRDER AT A
C     TIME
C
C      TEMP. GAGE  9 - USE WITH STRAIN GAGES  1 AND  5
C      TEMP. GAGE 10 - USE WITH STRAIN GAGES  2 AND  6
C      TEMP. GAGE 11 - USE WITH STRAIN GAGES  3 AND  7
C      TEMP. GAGE 12 - USE WITH STRAIN GAGES  4 AND  8
C      TEMP. GAGE 21 - USE WITH STRAIN GAGES 13 AND 17
C      TEMP. GAGE 22 - USE WITH STRAIN GAGES 14 AND 18
C      TEMP. GAGE 23 - USE WITH STRAIN GAGES 15 AND 19
C      TEMP. GAGE 24 - USE WITH STRAIN GAGES 16 AND 20
C
      WRITE(6,120)P12,SOU,GD
      CALL GST(T,S,X,D)
      WRITE(6,120)P12,NOR,GD
      CALL GST(T,S,X,D)
      WRITE(6,120)P13,SOU,GD
      CALL GST(T,S,X,D)
      WRITE(6,120)P13,NOR,GD
      CALL GST(T,S,X,D)
  120 FORMAT(20X,A10,2A7,/)
      STOP
      END
```

```
      SUBROUTINE GST(T,S,X,D)
      DIMENSION T(4,6),S(4,6),X(4),STY(4),ST(4,5),STT(5),
     *STB(5)
      INTEGER E(10),GN(4)
      READ(5,90)E
   90 FORMAT(10A4)

C       GN = GAGE NUMBER
C       ST = CHANGE IN GIRDER STRESS
C       ALP = COEF. OF EXPANSION OF CONCRETE
C       EE = ELASTIC MODULUS

      ALP=0.000006
      EE=4030000.

C     READ IN GIRDER TEMPERATURES, STRAIN GAGE READINGS,
C     GAGE NUMBERS AND LOCATIONS OF STRAIN GAGES

      READ(5,91)GN
   91 FORMAT(4I2)
      READ(5,92)((T(I,J),J=1,6),I=1,4)
      READ(5,92)((S(I,J),J=1,6),I=1,4)
   92 FORMAT(6F10.0)
      READ(5,93)(X(I),I=1,4),D
   93 FORMAT(5F10.0)

C     COMPUTE CHANGES IN GIRDER STRESSES - ONE STRAIN GAGE
C     AT A TIME

      DO 1 I=1,4
      M=0
      K=1
      KK=0
      DO 1 L=K,5
      KK=L+1
      DELT=T(I,KK)-T(I,K)
      DELS= (S(I,KK)-S(I,K))*0.000001
      M=M+1
    1 ST(I,M)=(DELS-ALP*DELT)*EE

C     LINEARIZE THE CHANGES OF STRESSES

      DO 4 J=1,5
      DO 2 I=1,4
    2 STY(I)=ST(I,J)
      N=4
      CALL LNFIT(N,X,STY,A1,A2)
      DO 5 I=1,4
    5 ST(I,J)=A1+A2*X(I)
      STT(J)=A1
    4 STB(J)=A1+A2*D
      WRITE(6,98)E
   98 FORMAT(8X,5(A4,1H-,A4,1X))
      WRITE(6,99)
   99 FORMAT(8X,5(9H---------,1X))
      WRITE(6,100)STT,(GN(I),(ST(I,J),J=1,5),I=1,4),STB
  100 FORMAT(6X,≠TOP≠,/,5X,≠SLAB≠,F8.1,4F10.1,//,4(7X,I2,
     *F8.1,4F10.1,/),6X,≠BTM≠,/,5X,≠GIRD≠,F8.1,4F10.1,//)
      RETURN
      END
```

```
      SUBROUTINE LNFIT(N,X,Y,A,B)
      DIMENSION X(10),Y(10)
C     LINEAR LEAST SQUARES APPROXIMATION
      S1=0.
      S2=0.
      S3=0.
      S4=0.
      DO 1 I=1,N
      S1=X(I)+S1
      S2=X(I)**2+S2
      S3=Y(I)+S3
    1 S4=X(I)*Y(I)+S4
      D=N*S2-S1**2
      A=(S2*S3-S1*S4)/D
      B=(N*S4-S1*S3)/D
      RETURN
      END
```

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