Spring 1981

Transverse thermal response of a composite bridge

Michael J. Filla

TRANSVERSE THERMAL RESPONSE

OF A

COMPOSITE BRIDGE

BY

MICHAEL J. FILLA, 1955 -

A THESIS

Presented to the Faculty of the Graduate School of the

UNIVERSITY OF MISSOURI-ROLLA

In Partial Fulfillment of the Requirements for the Degree

MASTER OF SCIENCE IN CIVIL ENGINEERING

1981

Approved by

Joe H. Emanuel (Advisor) Wm. A. Andrews

Myron G. Parry

PUBLICATION THESIS OPTION

This thesis has been prepared in the style utilized by the Journal of the Structural Division, American Society of Civil Engineers. Pages iii, iv, and 1 through 36 will be presented for publication in that journal. Appendices A, B, and C have been added for purposes normal to thesis unity.

CIVIL ENGINEERING ABSTRACT

A two-span, composite-design, test structure was subjected to thermal loading. The resultant longitudinal and transverse response was correlated with that calculated by a prior theoretical procedure. The theoretical procedure was reconfirmed for longitudinal stress, and indicated within reasonable probability for transverse stress. No measurable diaphragm influence was observed.

TRANSVERSE THERMAL RESPONSE OF A COMPOSITE BRIDGE

By Jack H. Emanuel and Michael J. Filla

ABSTRACT

This experimental investigation was conducted to further substantiate a prior theoretical study of thermal stresses induced in composite girder bridge structures. The objectives of the study were to subject a two-span laboratory test structure to a steady-state thermal loading and (a) correlate the observed transverse thermal response with that calculated by the theoretical procedure, and (b) determine the effect of diaphragms on transverse thermal response. The theoretical procedure assumed the slab in some state between plane stress and plane strain (partially restrained) and the beam in plane stress.

The study (a) reconfirmed the validity of the theoretical procedure for prediction of the longitudinal behavior of a composite bridge structure; (b) indicated that, although not in close agreement with the observed values, the theoretical procedure provides transverse stresses within reasonable probability and of a magnitude to warrant consideration by the design engineer; and (c) revealed no measurable diaphragm influence on the test structure of the study.

KEYWORDS: Bridge decks; Bridge movements; Bridges (composite); Bridges (structural); Bridges (thermal stresses); Bridges (transverse response); Composite beams; Concrete (reinforced); Temperature distribution; Thermal coefficient of expansion; Thermal strains; Thermal stresses.

ACKNOWLEDGEMENTS

The author wishes to express his appreciation to Dr. Jack H. Emanuel, Chairman of the Graduate Committee, for his encouragement, advice, and patient editing of this manuscript. Thanks are also due to Drs. William A. Andrews and Myron G. Parry for serving on the Graduate Committee.

A special acknowledgement is given to Dr. J. Leroy Hulsey without whose help and encouragement this study could never have materialized.

Acknowledgement is also given to Dr. Peter G. Hansen for his advice and interest in the study, to Mr. Ken Haas for the design and fabrication of data recording equipment, to fellow graduate students for their valuable friendship, and to the author's parents for their continual support and encouragement.

The author is especially grateful to Miss Laurie Jean Armstrong for her patient typing of the drafts and the final copy of this manuscript.

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

LIST OF TABLES

TRANSVERSE THERMAL RESPONSE OF A COMPOSITE BRIDGE

By Jack H. Emanuel,[1] F. ASCE, and Michael J. Filla,[2] A. M. ASCE

INTRODUCTION

Field observations show that in many cases the supporting and expansion devices do not function as anticipated by the design engineer. Expansion devices which do not behave as expected are uneconomical and can cause damage to the structure. A knowledge of the probable magnitudes of thermal movement and stress would permit the designer to make a more rational selection of the types of bearings, expansion devices, and joint sealants, thus reducing maintenance costs (5,28). Consequently, the thermal response of bridges has been the subject of much interest in the past few years (2-4,6-14,16-21,23-32).

The studies conducted in Australia and Europe have been primarily concerned with concrete box-girder structures rather than with concrete-steel bridges of composite design. The heat transfer analysis involved in the two types of construction differs very little, but the determination of strains and stresses is greatly dissimilar.

A structure composed of a homogenous and isotropic material will develop thermal stresses when subjected to a nonlinear temperature distribution. The problem becomes more complex in a composite-girder

---

[1] Prof. of Civ. Engrg., Univ. of Missouri-Rolla, Mo.

[2] Graduate Teaching Asst., Univ. of Missouri-Rolla, Rolla, Mo.

structure because of the difference in the coefficients of thermal expansion of the concrete and the steel, and the types and magnitude of restraints imposed on the deforming structure. The longitudinal thermal response of bridges when subjected to vertical temperature gradients has been the subject of rather extensive studies (2,3,7,9, 10,16,17,21,26,29,31). However, significant transverse stresses may also be induced by restraint of transverse hogging (negative curvature) and axial movement. In a recently completed study at the University of Canterbury at Christchurch, New Zealand (29), the effect of cracking on the transverse thermal response of a multiple box-girder bridge and a double T-bridge was investigated. No experimental studies of the transverse thermal response of a composite-girder structure have been reported in the literature.

In a study conducted at the University of Missouri-Rolla (7,17), thermally induced stresses in composite-girder structures were investigated from a theoretical standpoint. During a subsequent study at UMR, (9,10,31) a composite-design laboratory structure was constructed and subjected to steady-state thermal loading, and the experimental results were correlated with values calculated by the procedure developed during the theoretical study. Further experimental substantiation of the theoretical study is desired toward development of rational design criteria.

This and a concurrent study (21) were initiated to extend the areas of experimental-theoretical correlation, utilizing the test structure of the prior UMR study. The objective of this study was to subject the laboratory test structure to a steady-state thermal loading and (a) correlate the observed transverse thermal response of the structure with calculated values obtained from the theoretical procedure, and (b) determine the effect of diaphragms on transverse thermal response.

LABORATORY TEST STRUCTURE

The test structure was constructed in the structural laboratory of the Engineering Research Laboratory at the University of Missouri-Rolla for a prior study (9,10,31). The structure was an adequate rather than a true model of a 15 ft - 15 ft (4.6 m - 4.6 m) two-span continuous composite-design bridge 45-in. (114-cm) wide with no skew. A curved steel plate and pintle bearing was used at the pier, and integral abutments were used at the ends.

Abutments.—Integral stub abutments with flexible pilings were modeled by the test structure. Modeling considerations included a desire for a constant soil modulus, noninterference of the container on the soil medium and pile interaction, and a reasonable piling-to-superstructure stiffness ratio.

A uniform sand was placed in 7-ft wide by 3-ft deep by 6-ft high (2.1-m x 0.9m x 1.8-m) abutment containers at a uniform density of 104 lb/ft^3 (1674 kg/m^3). The pilings were three steel bars 72 in. (183 cm) long with a 5-in. x 0.5-in. (127-mm x 13-mm) cross section buried 66 in. (168 cm) in the sand beneath each stringer. A 6-in. x 0.5-in. (152-mm x 13-mm) steel plate pile cap was welded across the tops of the piles to provide a rigid connection to the superstructure.

Pier and Bearings.—The pier was modeled by three standard pipe sections 2 in. (5 cm) in diameter by 76.5 in. (194 cm) long; one beneath each stringer. The pipe sections were rigidly attached to the floor, simulating a cantilever beam, in accordance with the fact that in the field most piers have a relative point of fixity.

Stringer plates were welded to the bottom flange of each stringer and bolted to the steel-plate abutment pile caps, providing an integral connection which transferred moment, rotation, and longitudinal displacement. The pier bearing consisted of curved steel plates bolted to the stringer plates and resting on a machined bearing plate supported by the pier. Chamfered pintles were used to prevent lateral displacement, and tapered holes in the curved steel rocker plates allowed rotation.

<u>Stringers</u> <u>and</u> <u>Slab</u>.—A M6 x 4.4 (15-cm x 64-N/m) wide-flange section was used for the three stringers. The stringers were spaced 20 in. (51 cm) on center, and stud shear connectors were welded to the top flanges in accordance with standard AASHTO procedure. As built, ten sets of C4 x 5.4 (10-cm x 79-N/m) channels were used for diaphragms to provide lateral bracing. The stringer and diaphragm layout is shown in Fig. 1.

The slab-to-stringer stiffness ratio limited the slab depth to 1.5 in. (38 mm). Two layers of 16-gauge 2.63-in. (68-mm) longitudinal by 2-in. (51-mm) transverse galvanized welded wire mesh, placed 0.25 in. (6 mm) and 1.25 in. (32 mm), respectively, from the top of the slab provided reinforcement and temperature steel in the longitudinal and transverse directions. The mix design was based on the basis of laboratory trial batches. The concrete had a maximum limestone aggregate size of 0.375 in. (110 mm). A masonry blend sand was used for the fine aggregate and the air content of the concrete was 6.3 percent. At the time of construction, four years prior to this investigation, the 38-day compressive strength averaged 4400 psi (30 320 kPa) for the cylinders tested.

INSTRUMENTATION

Instrumentation was installed at selected points on the structure to record temperatures, strains, and displacements. To achieve this, thermistors, electric resistance strain gages, and dial indicators were used. Data recording equipment consisted of four 10 channel Automation Industries Model SB-1 switch and balance units connected to an Automation Industries Model P-350 strain indicator, a 10 channel Baldwin-Lima-Hamilton Model 225 switch and balance unit connected to a Baldwin-Lima-Hamilton Model 120C strain indicator and a 100 channel thermistor stepping unit connected to a digital voltmeter (Dana Model 5400).

Three types of carbon-steel temperature-compensated SR-4 strain gages were used. The first type, Micro-Strain Model 6C-2x2-120 w/L and located in the cantilever reference bars, was utilized from a prior study (9,10,31). These gages had a gage factor of 2.05, resistance of 120 ohms, and a grid size of ¼ x ¼ in. (6.4 x 6.4 mm). The second type, Baldwin-Lima-Hamilton Model FAE-25-12-56 EWL, was used on the stringer and diaphragms. These gages had a gage factor of 2.05, resistance of 120 ohms, and a grid size of 1/4 x 1/8 in. (6.4 x 3.2 mm). The third type of strain gages was Micro-Measurements Model EA-06-10 CBE-120 with a gage factor of 2.045, resistance of 120 ohms, and a grid size of 1 x 1/4 in. (25.4 x 6.4 mm). The Micro-Measurements strain gages were used on the slab. When measuring strains in a concrete structure it is ordinarily desirable to use a strain gage of sufficient gage length to span several pieces of aggregate in order to measure the representative strain in the structure (22); thus a 1-in. gage length was chosen for the strain measurements on the slab.

The strain gages on the slab and stringer were mounted using Micro-Measurements M-Bond AE-10 two part epoxy. This epoxy exhibits essentially creep-free performance up to 200° F (93° C) when cured at room temperatures.

In order to develop a proper substrate for gage bonding, it was necessary to apply a leveling and sealing precoat of epoxy adhesive to the concrete. Before applying the precoat, surface irregularities were removed by disc sanding. The surface was then cleaned and the adhesive was worked into any voids, and leveled to form a smooth surface. When the adhesive was completely cured, it was abraded until the base material began to be exposed. Following this the epoxy surface was cleaned and prepared conventionally.

Fenwal Uni-Curve No. VVA 33J1 thermistors were selected for the temperature sensors. These thermistors are epoxy encapsulated temperature sensitive resistors with a maximum spherical diameter of 0.095 in. (2.4 mm), resistance tolerance of $\pm 0.4^{\circ}$ F (0.22° C) over a range of 30 - 175° F (-1.1 - 79° C). Actual temperature values were obtained from observed values by a computer reduction utilizing logarithmic equations.

A two-part metal-filled epoxy was used to attach the thermistors to their base locations in order to provide better heat conduction from the base material to the thermistor. A 100-channel stepping unit interfaced the thermistor leads to a digital voltmeter. Observed values were hand recorded.

The total longitudinal deck deflection and the vertical deflection at the midspans were recorded by using dial indicators with a least count of 0.001 in. (0.025 mm). The indicators for vertical deflection were mounted on wooden standards, whereas the indicators at the abutments were attached to metal channels that were rigidly attached to the sandbox frame.

INSTRUMENTATION ORIENTATION

Eight locations were chosen for the placement of instrumentation as shown in Fig. 2. Instrumentation groups 1, 3, 4, 6 and 8 were utilized from a prior study (9,10,31), and groups 3, 5, and 7 were placed for this study.

At locations 1 and 8, midway between the center and outside stringers, slab transducers (15) were placed in 4-in. (10-cm) wide by 9-in. (23-cm) long cantilever temperature-reference bars enclosed on the two sides and the end by 1/2-in. (13-mm) thick flexible styrofoam. Wire mesh was omitted in these bars so that the concrete could expand freely under unrestrained thermal expansion. The styrofoam produced essentially no resistance to small expansive movements and provided insulation between the boundaries. The cantilever temperature-reference bars were used to experimentally determine the thermal coefficient of expansion of the slab (15).

Instrumentation groups 2, 5, and 7 consisted of gage locations at the center stringer, and midway between the center and outside stringers. The gage locations are shown on a typical cross-section in Fig. 3. Each gage location consisted of a thermistor, and longitudinal and transverse strain gages.

Instrumentation groups 3, 4, and 6 were at sections along the center stringer. At these locations thermistors were placed at six points vertically through the deck slab, seven thermistors were evenly spaced down the stringer web, and two were attached to the bottom flange; one at the outer edge of the flange and the other directly beneath the web. An elevation of this instrumentation is shown in Fig. 4. Due to the uniformity of the heat flux over the length of the structure, it was assumed the temperature distributions at locations 2, 5, and 7 were equal to the recorded temperature distributions at locations 3, 4, and 6 respectively. The temperature distribution in the slab section between the stringers was calculated by assuming a straight line approximation from the top to the bottom of slab.

Dial indicators with a least count of 0.001 in. (0.025 mm) were used to measure the vertical deflection of the center stringer at locations 2 and 7 and the longitudinal deck displacements at each abutment. The total deck movement at the bearing elevation was obtained by summing the abutment displacements.

Two thermistors were also positioned at 2 and 12 in. (5 and 30 cm) above the top of the deck and two at 12 and 30 in. (30 and 76 cm) below the deck to give an indication of the still air temperature and thermal gradients around the bridge.

HEAT SOURCE

The test structure was thermally loaded by radiation heating rather than a constant temperature heat source, because it was simpler and approximated actual field conditions imposed by the sun. General Electric Model 250R40 250 watt infrared reflector heat lamps were selected for the heat source. These lamps emitted a radiation level that was partially absorbed by the deck and in turn heated the bridge structure. The lamps were placed in four rows along the length of the bridge and were spaced 12 in. (30 cm) center-to-center both longitudinally and transversely. Alternate rows were staggered 6 in. (15 cm) to provide a more uniform radiation level. The 12-in. (30 cm) spacing was selected for uniformity of heat distribution and also to provide a deck temperature of approximately 150^{0} F (66^{0} C). The bulb faces were placed 20 in. (51 cm) above the deck in accordance with the manufacturers recommendation for the distance of the lamp from the heated subject being at least 1.6 times the lamp spacing (1.6 x 12 = 19.2 in. [49 cm]) for uniform radiation distribution.

Five 240-volt Variac transformers were used to vary the thermal loading. The emitted radiation varied with the applied voltage. The 115-volt lamps were connected in series by pairs to split the voltage output of the Variac transformers. These pairs were then connected in parallel to complete a transformer string. Thus, the lamp input voltage could be varied up to 120 volts. The voltage drop through the wires was less than one percent, because the transformer leads were connected to the center of a bulb string. All leads and couplers consisted of 12 gage wire.

To obtain uniform heat flux, the outside circuits required a higher voltage input than the interior circuits because the overalap of radiant energy along the edges was not as pronounced as in the center. To check the uniformity of the heat flux, a heat receptor was fabricated from a 5 x 3 x 1 - in. (127 x 76 x 25 - mm) carbon steel plate painted flat black on one of the 5 x 3 - in. (127 x 76 - mm) surfaces. Thermistors were placed on both surfaces, and the plate was encased in styrofoam to prevent the loss of heat from the sides and limit the convection to the top and bottom surfaces. The painted side was exposed to the radiation and the opposite face to ambient air. The circuits were considered to be adjusted properly for the most uniform heat flux when the heat receptor indicated a constant temperature at all locations on the deck.

TESTING PROCEDURE

The laboratory was sealed before each testing cycle to eliminate any outside drafts. Heating and air return vents were sealed, door cracks taped, and outside openings covered with plastic. Thus, the only source of forced convection would be air currents caused by thermal gradients developing into cyclic drafts as a result of the laboratory's size.

Prior to the start of a test, strain gages and dial indicators were zeroed, and thermistor readings were recorded for use as the reference temperature of the structure at zero strain. The transformers were turned on and each circuit adjusted until a uniform heat flux was produced in the deck. Steady-state temperatures through the cross section were achieved after approximately ten hours of heating.

Recorded values included longitudinal strain, transverse strain, and temperatures at previously described points on the stringer and slab; strains at the base of the pier to determine the lateral movement of the top of the pier; lateral displacements at the abutments; vertical displacements at the gage locations; and ambient air temperatures above and below the slab.

After all data were recorded, the heat lamps were turned off, the structure allowed to cool to room temperature, and strain and thermistor readings taken for comparison of cyclic action and instrumentation drift.

THERMAL STRESSES

Analysis of thermal strains and stresses in an indeterminate structure is achieved by (a) removing redundants to obtain a determinate structure, (b) dividing the simple determinate structure into a number of constant-section segments and determining the thermally induced segment strains and stresses, (c) applying the redundants as loads and obtaining by conventional methods of analysis the resultant induced stresses and strains caused by the redundants, and (d) superimposing the thermally induced and the redundantly induced strains and stresses.

Emanuel and Hulsey (7) and Hulsey (17) developed a procedure for determining thermally induced strains and stresses to account for slab-beam interaction. The geometric and material segment properties are assumed to be constant along the segment length; the temperature profile through the depth of the cross section is assumed to be constant along the segment length; and the slab and stringer are assumed to form a composite section. The web and bottom flange of the beam is assumed to be in a state of plane stress, and the slab and top beam flange in a state of partial transverse restraint.

The theoretical stresses and strains presented in this report were obtained by utilizing a computer program--Soil-Structure Interaction Program (SSIP)--developed by Hulsey (17). The program was modified for this study by Filla (15) to calculate the strains and stresses in the slab section between the stringers.

DATA REDUCTION

Temperature.—Conversion of thermistor readings to temperature would generally be accomplished by the use of a manufacturer supplied ohm-^{o}C conversion graph or table. In this instance, the internal resistance of the 100 channel stepping unit required to interface the large number of thermistors precluded the reading of thermistor output in ohms. Thus, the output was read in millivolts; equations were developed for ohm-^{o}C conversion at 20^{o} F (11^{o} C) temperature increment ranges; and a computer program written and used for conversion of millivolts to ohms, ohms to ^{o}C, and ^{o}C to ^{o}F.

Strain.—Reduction of observed data obtained from the carbon-steel temperature-compensated SR-4 strain gages required correction for 1) apparent strain, 2) self-temperature-compensating (STC) mismatch, 3) compensated (nonindicated) thermal strain, and 4) resistance change of leadwires (15).

A computer program was developed and used for conversion of observed, as recorded, strain to actual thermally induced strain.

Initial and final dial indicator readings were reduced and combined to provide point deflections and overall structural movement.

RESULTS OF EXPERIMENTAL INVESTIGATION

Temperature Distribution.—As described previously, a steady-state thermal gradient was achieved by exposing the bridge deck to infrared radiation for a period of approximately ten hours. Consistent repeated test results were obtained. Temperature profiles taken along the length of the bridge fell within a 5^{0} F (2.8^{0} C) band, and profiles across the width of the bridge fell within a 4^{0} F (2.2^{0} C) band.

The temperature profiles for a typical test are shown in Fig. 5. These profiles were used as input for computer calculation of theoretical strains. As shown in Fig. 5, the temperature at the middle stringer varied from 159^{0} F (71^{0} C) at the top surface of the deck to 140^{0} F (60^{0} C) at the bottom of the deck and 114^{0} F (46^{0} C) at the bottom flange of the stringer. In the slab section between the stringers, the temperature varied from 159^{0} F (71^{0} C) at the top surface of the deck to 148^{0} F (64^{0} C) at the bottom of the deck. The bottom of the deck at the stringer was approximately 8^{0} F (-13^{0} C) cooler than the bottom of the deck midway between the stringers. This can be explained by the fact that steel has a high thermal conductivity and the stringer acts as a heat sink.

Ambient temperatures were 115^{0} F (46^{0} C) and 112^{0} F (44^{0} C) at 2 in. (5 cm) and 12 in. (30 cm), respectively, above the deck surface. Below the deck, ambient temperatures were 86^{0} F (29^{0} C) at 12 in. (30 cm) and 85^{0} F (29^{0} C) at 30 in. (76 cm).

<u>Strain Distribution</u>.—As previously discussed, the observed strains included apparent strain, STC mismatch, the effect of abutment and pier restraints, and resistance change in leadwires. The apparent strain is a function of temperature, and is usually assumed to be a linear function within certain temperature ranges. For the stringer and diaphragm gages, the correction ranged from zero at 75° F (24° C) to -65 micro strain at 150° F (66° C). The apparent strain for the gages on the slab ranged from zero at 75° F (24° C) to -42 micro strain at 160° F (70° C). Because of the instrumentation available and the large number of strain gages, a two-wire lead which requires a correction for resistance change of the leadwires was used (no such correction is needed for three-wire leads). Correction factors were experimentally determined for the resistance change of the leadwires for gages located at 1) the top of the deck, 2) the bottom of the deck, and 3) the bottom of the stringer (15).

The main objective of this study was to subject the laboratory test structure to a steady-state thermal loading and (a) correlate the observed transverse thermal response of the structure with calculated values obtained from the computer program, SSIP, and (b) determine the effect of diaphragms on transverse thermal response. Two groups of tests were conducted. For the first group of tests, hereafter referred to as Series One, the diaphragms were removed from the structure and the results correlated with the calculated values obtained from SSIP. The second group of tests, hereafter referred to as Series Two, were run with the diaphragms positioned and welded at the locations shown in Fig. 1. The Series Two results were then compared to the Series One results. Theoretical strains and stresses could not be computed for the Series Two tests because computer program SSIP cannot handle the out-of-plane springs required to model the effect of diaphragms.

From the data obtained from the cantilever sections of instrument groups 1 and 8, the coefficient of thermal expansion of the concrete was determined to be 4.1 x $10^{-6}/^{o}$ F (7.4 x $10^{-6}/^{o}$ C). The change in value from 3.5 x $10^{-6}/^{o}$ F (6.3 x $10^{-6}/^{o}$ C) as determined in the prior investigation reflects the age effect (approximately 4 years) and other factors influencing the thermal coefficient of expansion (1,15).

The experimental temperature profiles were used with the previously described procedure of Emanuel and Hulsey (7) and the computer program, SSIP, developed by Hulsey (17) to obtain theoretical strains and stresses. The following material properties were used to calculate the theoretical values:

Young's Modulus--Steel..........29.0 x 10^{6} psi (20.0 x 10^{7} kPa)

Young's Modulus--Concrete........ 4.5 x 10^{6} psi (3.1 x 10^{7} kPa)

Poisson's Ratio--Steel........... 0.3

Poisson's Ratio--Concrete........ 0.18

Coef. of Thermal Exp.--Steel..... 6.5 x $10^{-6}/^{o}$ F (11.7 x $10^{-6}/^{o}$ C)

Coef. of Thermal Exp.--Concrete.. 4.1 x $10^{-6}/^{o}$ F (7.4 x $10^{-6}/^{o}$ C)

To permit correlation with the prior and concurrent tests, the observed and calculated strains in both the longitudinal and transverse directions are compared in the following.

Longitudinal Strains for Series One Tests.—Longitudinal strains for repeated tests fell within a narrow bandwidth similar to that observed for the temperature profiles. The strain profiles showed negative curvature (lengthening of top deck fibers greater than that of bottom flange fibers) at the midspan locations and indicated positive curvature at the pier. These relationships are compatible with the temperature profiles (the top of the section warmer than the bottom). The observed and theoretical longitudinal strains were in reasonable agreement except at the bottom of the slab next to the stringer where the observed strains were 30 to 35 percent greater than the theoretical values, and compared more closely with the theoretical strain within the flange width (slab-flange interface). This large difference in correlation is believed to result from the heat-sink action of the stringer and the assumption in the theoretical procedure that the thermal gradient varies in the vertical direction only, i.e.; the temperature distribution in the slab is constant on any given horizontal plane a) within the width of the flange or b) outside (beyond) the flange; resulting in the same theoretical value for any slab location beyond the stringer flanges. As previously discussed, the temperature at the bottom of the slab was 8^{0} F (-13^{0} C) higher midway between the stringers than adjacent to the stringer flange. Also, local shear stresses develop near the edge of the top flange due to the abrupt change in normal stresses, and the theoretical procedure neglects the stress concentrations (St. Venant's principle) at the slab-flange interface.

Potential sources of experimental error are faulty gages, inaccurate instrumentation, or error in conversion from recorded to compensated strains. Erroneous theoretical values would also result from incorrect material properties. To insure unrestrained movement, no steel was placed in the cantilever reference bars used to determine the thermal coefficient of expansion of the concrete deck.

Reinforcing steel in concrete would produce a larger effective coefficient of thermal expansion. For this study, input of an assumed larger coefficient of thermal expansion could also result in theoretical longitudinal strains greater than the observed values.

The temperature distribution of this study was essentially identical to the maximum power level of the prior study by Wisch (9,10,31) and that of the concurrent study (21). This provided a data bank for comparison of the effect of the change in modulus of elasticity and the coefficient of thermal expansion of the concrete deck. A comparison of theoretical longitudinal stresses obtained in this and the prior study is shown by Filla (15).

There was no differential strain at the base of the pier, which indicates that no longitudinal displacement occurred at the bearing elevation of the pier thus resulting in symmetrical longitudinal displacements about the center of the structure. This symmetrical action was substantiated by the dial indicator readings at each abutment, which varied from 0.038 in. (0.097 cm) to 0.042 in. (0.107 cm) as compared to a theoretical value of 0.045 in. (0.115 cm). The deflections recorded at the midspans were erratic and no conclusions could be drawn based on this data.

Transverse Strains for Series One Tests.—Transverse strains for repeated tests fell within a narrow bandwidth similar to that observed for the temperature and the longitudinal-strain profiles. Typical Series One experimental and theoretical transverse strains at the Midspans and Pier are presented in Table 1. When constructed, the test structure was not instrumented for determination of transverse strains. For this study it was feasible only to instrument the top and bottom of the slab midway between stringers, the top of the slab directly above the stringer web, and the bottom of the slab adjacent to the top stringer flange. Thus, the experimental curvature at the slab-flange interface could not be determined. However, the theoretical procedure yields positive curvature (lengthening of bottom deck fibers greater than that of top deck fibers) of the slab at this location. For the slab section midway between the stringers, the strains presented in Table 1 show positive curvature; not compatible with the temperature profiles or the curvatures calculated by the theoretical procedure.

Experimental and theoretical transverse strains were in reasonable agreement (10 to 15 percent) except at the top of the slab over the stringer web and at the bottom of the slab adjacent to the flange. However, the experimental and theoretical values indicated conflicting slab curvatures midway between stringers. As with the longitudinal strains, the transverse strains at the bottom of the slab adjacent to the stringer flange were 30 percent greater than the theoretical values. As for the longitudinal strains, this difference is believed due to heat-sink action, to the temperature used to calculate the theoretical strain, and to stress concentrations at the slab-flange interface. The transverse strains at the top of the slab directly above the stringer web were approximately 33 percent larger than the theoretical values. The theoretical procedure takes into account transverse interface compatibility (strains and curvatures) between the slab and the beam flange. However, although the (longitudinal) row of shear connectors directly above the stringer web provided longitudinal strain compatibility, it is improbable that slab-friction alone provided transverse compatibility, and experimental strains could vary greatly from the theoretical strain.

For the slab segment midway between the stringers, the observed transverse strains varied approximately 10 to 20 percent from the theoretical values. Moreover, as stated earlier, the experimental strains (greater at the bottom of the slab than at the top) indicate positive curvature, whereas the theoretical procedure gives equal values for the strain at the top and at the bottom of the slab.

The following describes the attempts made to correlate more closely the indicated experimental and theoretical curvatures:

1. The theoretical procedure uncouples the slab segment within the stringer flange from the slab segment beyond the flange, the slab segment within the flange from the stringer flange, and also the top stringer flange from the top of the web in order to account for compatibility in the transverse direction. These component members and their subsequent compatibility forces require a solution for eight unknowns. By discounting the transverse restraint between the flange and the slab (discussed above) the unknowns were reduced from eight to six. A negligible change in the theoretical strains resulted from this simplification.

2. The theoretical procedure also assumes equal stringer deflections and that the stringers do not rotate torsionally contrary to probable response. As a check of the reasonableness of the observed strains, subtracting the observed strain from the thermal strain for an unrestrained condition ($\alpha \cdot \Delta T$) gives the magnitude of restraint strain and an indication of the combined axial and torsional forces required of the exterior stringers. However, for this study, this

procedure resulted in improbable stringer action and the conclusion that the curvature calculated from the observed strains was unreasonable. For the instrumentation used and the 10 to 12-hr test duration time, a drift of 10 to 15 micro-strain is to be expected. Drift is also a possible source of error in the experimental determination of the wire temperature correction factor. Had the strains at the bottom of the slab been 20 to 30 micro-strain smaller, the correlation of the experimental and theoretical curvatures would be acceptable.

The theoretical procedure uses the interaction of longitudinal, transverse, and vertical strains and Poisson's ratio in determination of stress. A prediction of stress based on the experimental observations is not possible because no vertical strains were measured. However, because of the close correlation of the experimental and theoretical strains in the longitudinal direction, the theoretical longitudinal stresses shown in Table 2 are believed valid; and, within the rationale given for the differences between the experimental and theoretical transverse strains, the theoretical transverse stresses are believed of reasonable magnitude and are also presented in Table 2.

At the stringer, the theoretical transverse stress at the top of the slab above the stringer web ranged from 2.7 to 3 times the longitudinal stress, and at the slab-flange interface the transverse stress was approximately 1.5 times the longitudinal stress. For the slab segment between the stringers, the transverse stress at the top of the slab was approximately 1.3 times the longitudinal stress, and at the bottom of the slab the transverse and longitudinal stresses were essentially the same.

Series Two Tests.—The Series Two Tests were run with the diaphragms welded at the locations shown in Fig. 1. There was no measurable difference between the observed strains of the Series Two Tests and those of the Series One Tests. The recorded diaphragm strains were small and erratic.

CONCLUSIONS

This investigation was initiated as an intermediate study to verify a theoretical procedure developed by Emanuel and Hulsey (7) and Hulsey (17) for analysis of composite-girder bridge structures supported by flexible substructures and subjected to environmental loadings. The main objective of this study was to subject the laboratory test structure to a steady-state thermal loading and (a) correlate the observed transverse thermal response of the structure with calculated values obtained from the computer program, SSIP, and (b) determine the effect of diaphragms on transverse thermal response. Comparisons were made between the experimental longitudinal and transverse strains and the theoretical values computed by using the experimental temperature distributions and computer program SSIP.

Based on the correlation of consistent readings of the longitudinal strains the following conclusions were reached:

1. The observed and theoretical longitudinal strains were in reasonable agreement. Resultant stresses in the test structure, which are functions of longitudinal, transverse, and vertical strains, can be expected to be similar to the theoretical values.

2. The correlation confirmed that the theoretical procedure is adequate for a reasonable prediction of the longitudinal behavior of

composite-girder bridge structures subjected to thermal loading.

Based on the rationale given for the differences in the transverse strains, it is believed that the values of strain and stress given by the theoretical procedure are reasonable, and that thermally induced stresses in the transverse direction are of a magnitude to warrant consideration by the design engineer. The transverse stresses were as much as three times the longitudinal stress and up to 12 percent of the allowable compressive stress in the concrete. No measurable diaphragm influence on the test structure was observed.

RECOMMENDATIONS FOR FURTHER STUDY

During the course of any investigation, many questions emerge as a result of the research. It often happens that many of these questions are beyond the scope of the study and remain unanswered. Such was the case during this investigation.

Further investigation of the transverse thermal response of composite bridge structures would aid the development of a rational design procedure to account for thermal behaviour. The following, especially, should be explored.

1. Investigation of differential stringer displacement and torsional rotation.

2. A more thorough study of the effects of diaphragms and supports on transverse action (both experimental and theoretical).

3. A study of the effect of shear connectors on deck-slab interface forces and stresses in the transverse direction.

4. A study of the effect of cracks in the slab on the transverse thermal response of a composite bridge structure.

Other studies, which should provide information of value to bridge engineers and those in related fields were suggested in prior studies (9,10,17,21,31).

APPENDIX I.--REFERENCES

1. Berwanger, C., and Sarkar, A. F., "Effect of Temperature and Age on Thermal Expansion and Modulus of Elasticity of Concrete," *Behaviour of Concrete Under Temperature Extremes*, American Concrete Institute Special Publication SP-39, 1973, pp. 1-22.

2. Berwanger, C., and Symki, Y., "Thermal Stresses in Steel-Concrete Composite Bridges," *Canadian Journal of Civil Engineering*, Vol. 2, No. 1, 1975, pp. 66-84.

3. Berwanger, C., and Symko, Y., "Finite-Element Solutions for Thermal Stresses in Steel-Concrete Composite Bridges," *Experimental Mechanics*, Vol. 16, No. 5, May, 1976, pp. 168-175.

4. Black, W., Moss, D. S., and Emerson, M., "Bridge Temperatures Derived from Measurement of Movement," *TRRL Report LR 748*, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, 1976.

5. Ekberg, C. E., and Emanuel, J. H., "Problems of Bridge Supporting and Expansion Devices and an Experimental Comparison of the Dynamic Behavior of Rigid and Elastomeric Bearings," *Final Report, Project* 547-S, Engineering Research Institute, Iowa State University, Ames, Iowa, Aug., 1967.

6. Emanuel, J. H., et al., "An Investigation of Design Criteria for Stresses Induced by Semi-Integral End Bents: Phase I--Feasibility Study," *Missouri Cooperative Highway Research Program Final Report* 72-9, University of Missouri-Rolla, Rolla, Mo., 1973.

7. Emanuel, J. H., and Hulsey, J. L., "Thermal Stresses and Deformations in Nonprismatic Indeterminate Composite Bridges," Tranportation Research Record 607, Transportation Research Board, National Academy of Sciences, Washington, D.C., 1976, pp. 4-6.

8. Emanuel, J. H., and Hulsey, J. L., "Temperature Distribution in Composite Bridges," Journal of the Structural Division, ASCE, Vol. 104, No. ST1, Paper 13474, Jan., 1978, pp. 65-78.

9. Emanuel, J. H., and Wisch, D. J., "Thermal Stresses Induced in a Composite Model Bridge Structure," Missouri Cooperative Highway Research Program Final Report 75-2, University of Missouri-Rolla, Rolla, Mo., 1977.

10. Emanuel, J. H., and Wisch, D. J., "Thermal Response of a Continuous Two-Span Composite Bridge Structure," Transportation Research Record 711, Transportation Research Board, National Academy of Sciences, Washington, D.C., 1979, pp. 40-46.

11. Emerson, M., "The Calculation of the Distribution of Temperature in Bridges," TRRL Report LR 561, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, 1973.

12. Emerson, M., "Extreme Values of Bridge Temperatures for Design Purposes," TRRL Report LR 744, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, 1976.

13. Emerson, M., "Temperature Differences in Bridges: Basis of Design Requirements," TRRL Report LR 765, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, 1977.

14. Emerson, M., "Temperature in Bridges During the Hot Summer of 1976," TRRL Report LR 783, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, 1977.

15. Filla, M. J., "Transverse Thermal Response of a Composite Bridge," thesis presented to the University of Missouri-Rolla at Rolla, Missouri, in 1981, in partial fulfillment of the requirements for the degree of Master of Science.

16. Hambley, E. C., "Temperature Distributions and Stresses in Concrete Bridges," Structural Engineering, Vol. 56A, No. 5, May, 1978, pp. 143-148.

17. Hulsey, J. L., "Environmental Effects on Composite-Grider Bridge Structures," thesis presented to the University of Missouri-Rolla, at Rolla, Mo., in 1976, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

18. Hulsey, J. L., and Emanuel, J. H., "Finite Element Modeling of Climatically Induced Heat Flow," In Numerical Methods for Differential Equations and Simulation (A. W. Bennett and R. Vichnevetsky, eds.), North-Holland, Amsterdam, 1978, pp. 111-114.

19. Hunt, B., and Cooke, N., "Thermal Calculations for Bridge Design," Journal of the Structural Division, ASCE, Vol. 101, No. 9, Sep., 1975, pp. 1763-1781.

20. Lanigan, A. G., "The Temperature Response of Concrete Box Girder Bridges," School of Engineering Report No. 94, thesis presented to the University of Auckland, at Auckland, New Zealand, in 1973, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

21. Lewis, D. B., "Abutment-Thermal Interaction of a Composite Bridge," thesis presented to the University of Missouri-Rolla at Rolla, Mo., in 1980, in partial fulfillment of the requirements for the degree of Master of Science.

22. Micro-Measurements, Measurements Group, "Strain Gage Selection Criteria, Procedures, Recommendations," Tech Note TN-132, author, Romulus, MI, 1976.

23. Priestly, M. J. N., "Design Thermal Gradients for Concrete Bridges," New Zealand Engineering, Vol. 31, No. 9, Sep., 1976, pp. 213-219.

24. Priestly, M. J. N., "Design of Concrete Bridges for Temperature Gradients," Journal of the American Concrete Institute, Vol. 75, No. 5, May, 1978, pp. 209-217.

25. Radolli, M., and Green, R., "Thermal Stresses in Concrete Bridge Superstructures under Summer Conditions," Transportation Research Record 547, Transportation Research Board, National Academy of Sciences, Washington, D.C., 1975, pp. 23-26.

26. Rahman, F., and George, K. P., "Thermal Stress Analysis of Continuous Skew Bridge," Journal of the Structural Division, ASCE, Vol. 105, No. ST7, July, 1979, pp. 1525-1541.

27. Rambhai, P., "Thermal Effects in Concrete Box Girder Bridges," School of Engineering Report No. 139, University of Auckland, Auckland, New Zealand, 1976.

28. Reynolds, J. D., and Emanuel, J. H., "Thermal Stresses and Movements in Bridges," Journal of the Structural Division, ASCE, Vol. 100, No. ST1, Proc. Paper 10275, Jan., 1974, pp. 63-78.

29. Thurston, S. J., "Thermal Stresses in Concrete Structures," Research Report No. 78-21, (Civil Engineering), thesis presented to the University of Canterbury, at Christchurch, New Zealand, in 1978, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

30. Wah, T., and Kirksey, R., "Thermal Characteristics of Highway Bridges," Highway Research Board, National Academy of Sciences, Washington, D.C., 1969.

31. Wisch, D. J., "Thermal Stresses Induced in a Model Composite Bridge Structure," thesis presented to the University of Missouri-Rolla, at Rolla, Mo., in 1977, in partial fulfillment of the requirements for the degree of Master of Science.

32. Zuk, W., "Thermal and Shrinkage Stresses in Composite Beams," Journal of the American Concrete Institute, Vol. 58, No. 3, Sept., 1961, pp. 327-340.

LIST OF CAPTIONS

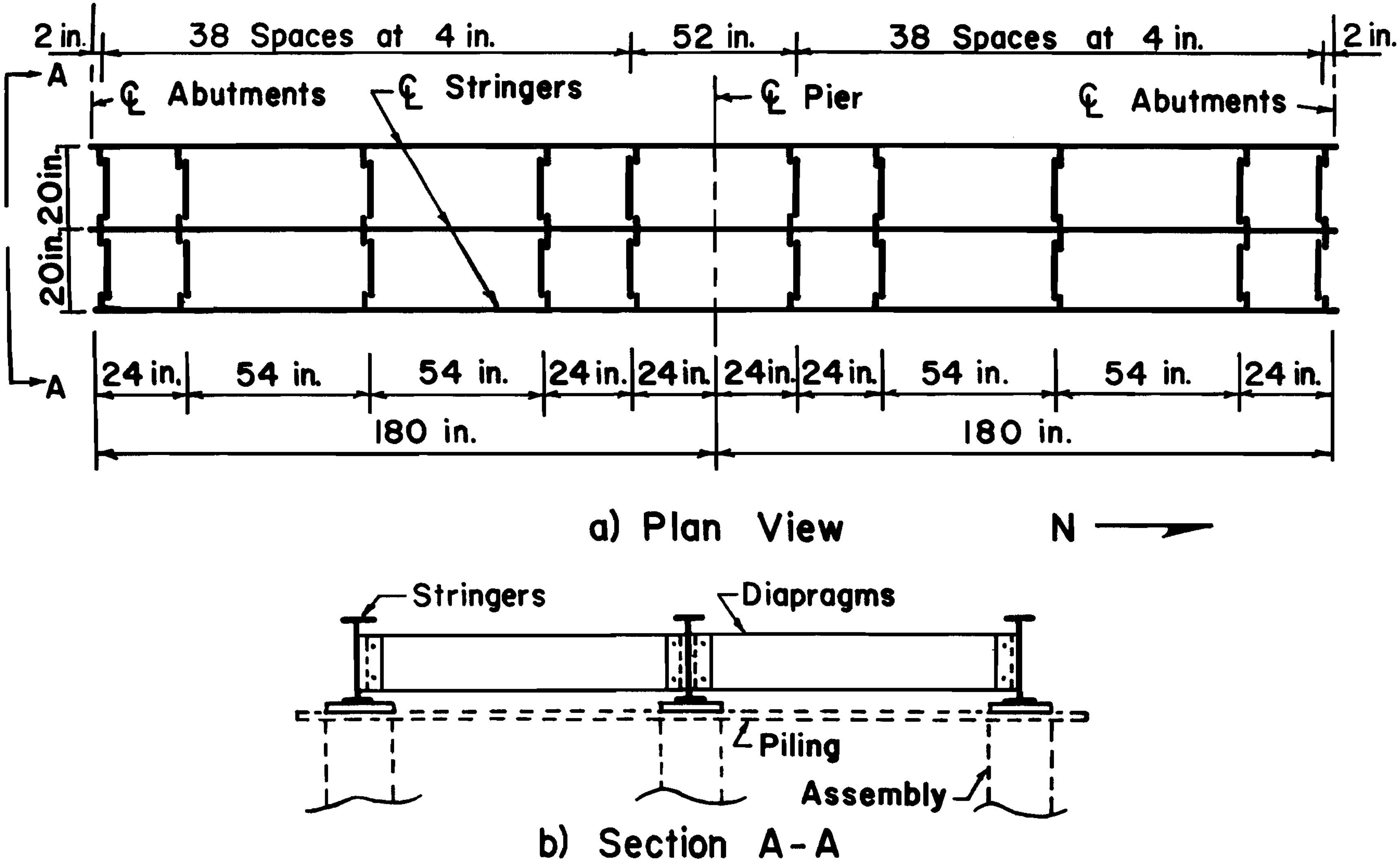
2 in.
38 Spaces at 4 in.
52 in.
38 Spaces at 4 in.
2 in.
A
℄ Abutments
℄ Stringers
℄ Pier
℄ Abutments
20in.
20in.
24 in.
54 in.
54 in.
24 in.
24 in.
24 in.
24 in.
54 in.
54 in.
24 in.
180 in.
180 in.
A
a) Plan View
N
Stringers
Diapragms
Piling
Assembly
b) Section A-A

Figure 1.

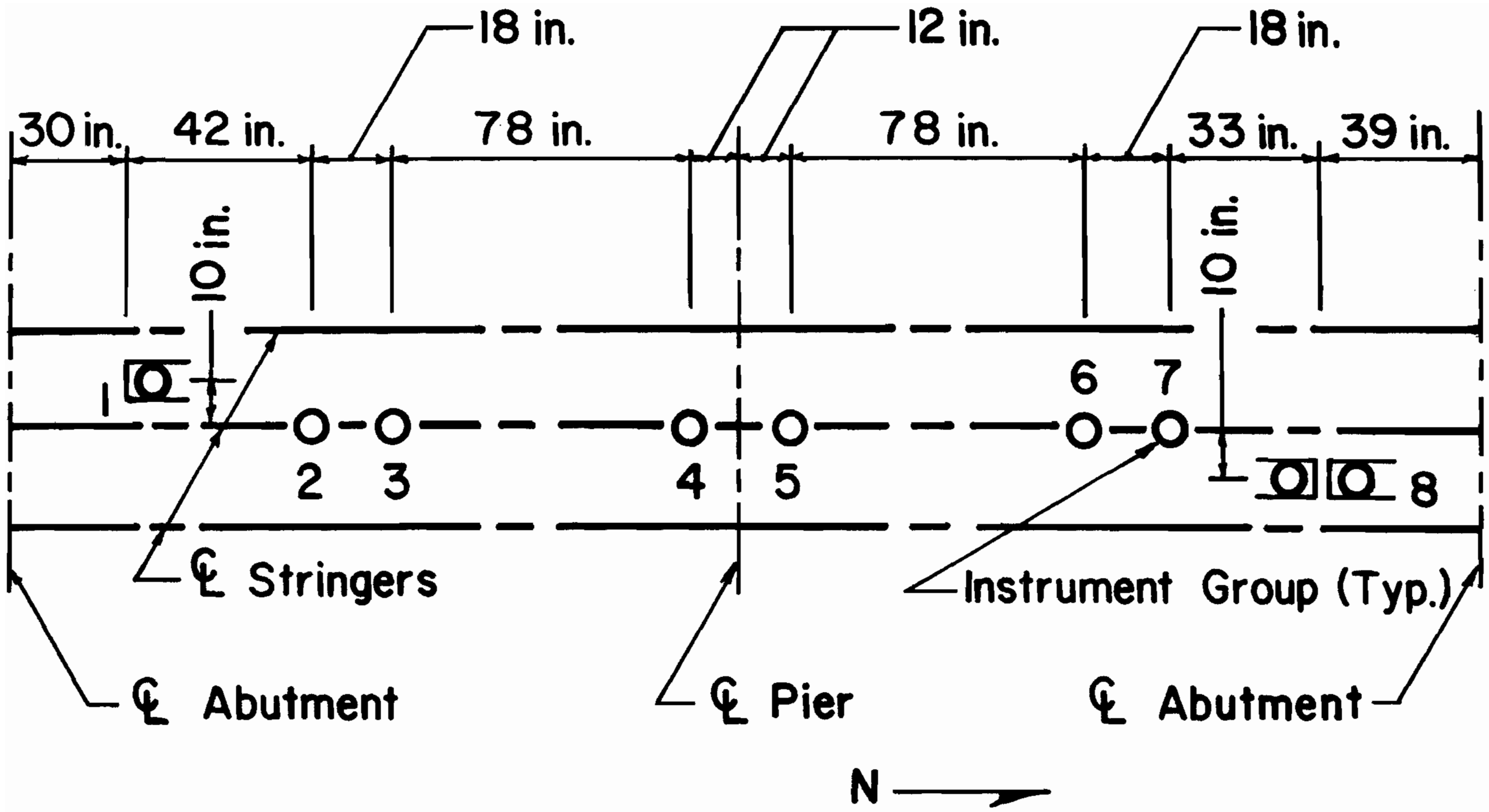
18 in.
12 in.
18 in.
30 in.
42 in.
78 in.
78 in.
33 in.
39 in.
10 in.
10 in.
1
2
3
4
5
6
7
8
℄ Stringers
Instrument Group (Typ.)
℄ Abutment
℄ Pier
℄ Abutment
N

Figure 2.

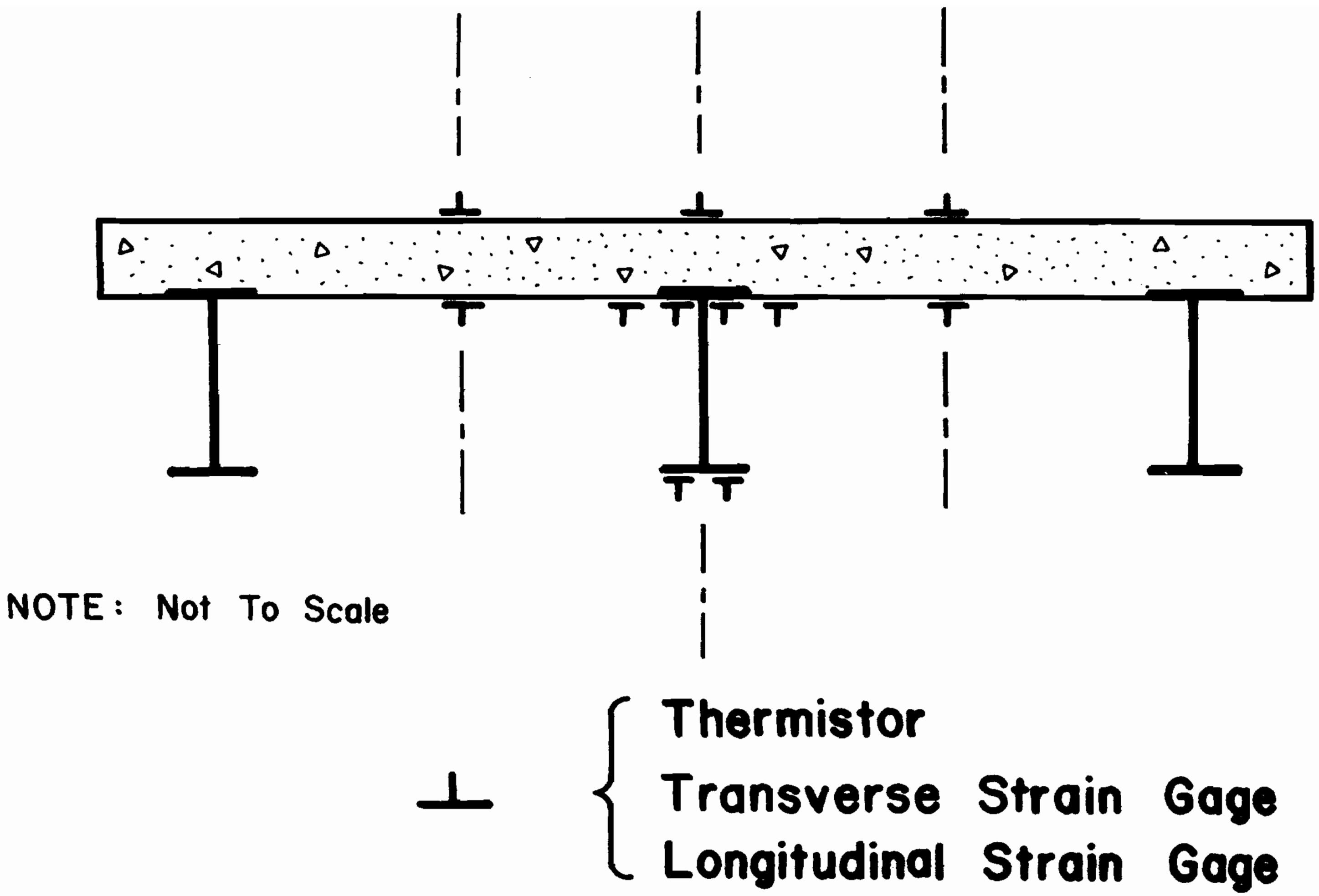
NOTE: Not To Scale
Thermistor
Transverse Strain Gage
Longitudinal Strain Gage

Figure 3.

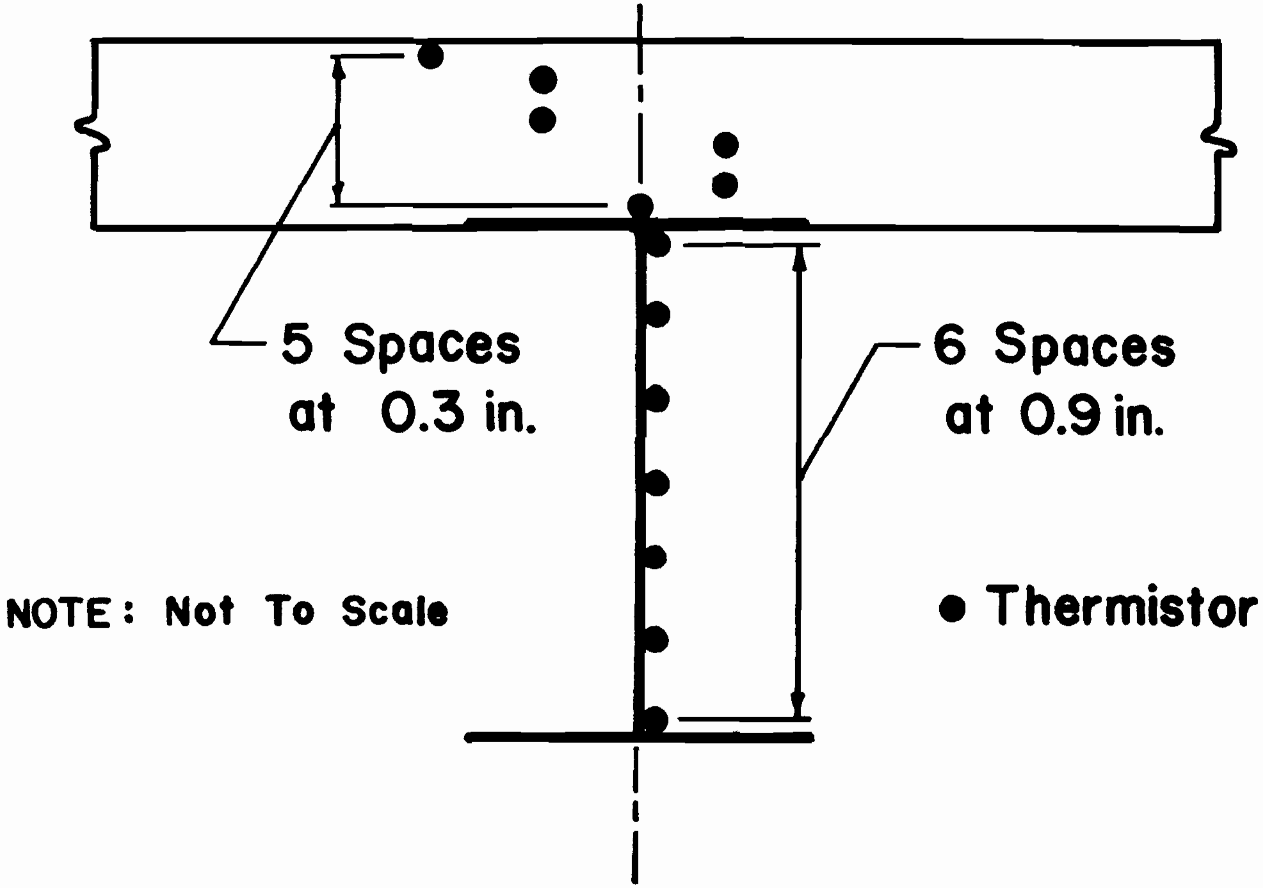
5 Spaces
at 0.3 in.
6 Spaces
at 0.9 in.
NOTE: Not To Scale
● Thermistor

Figure 4.

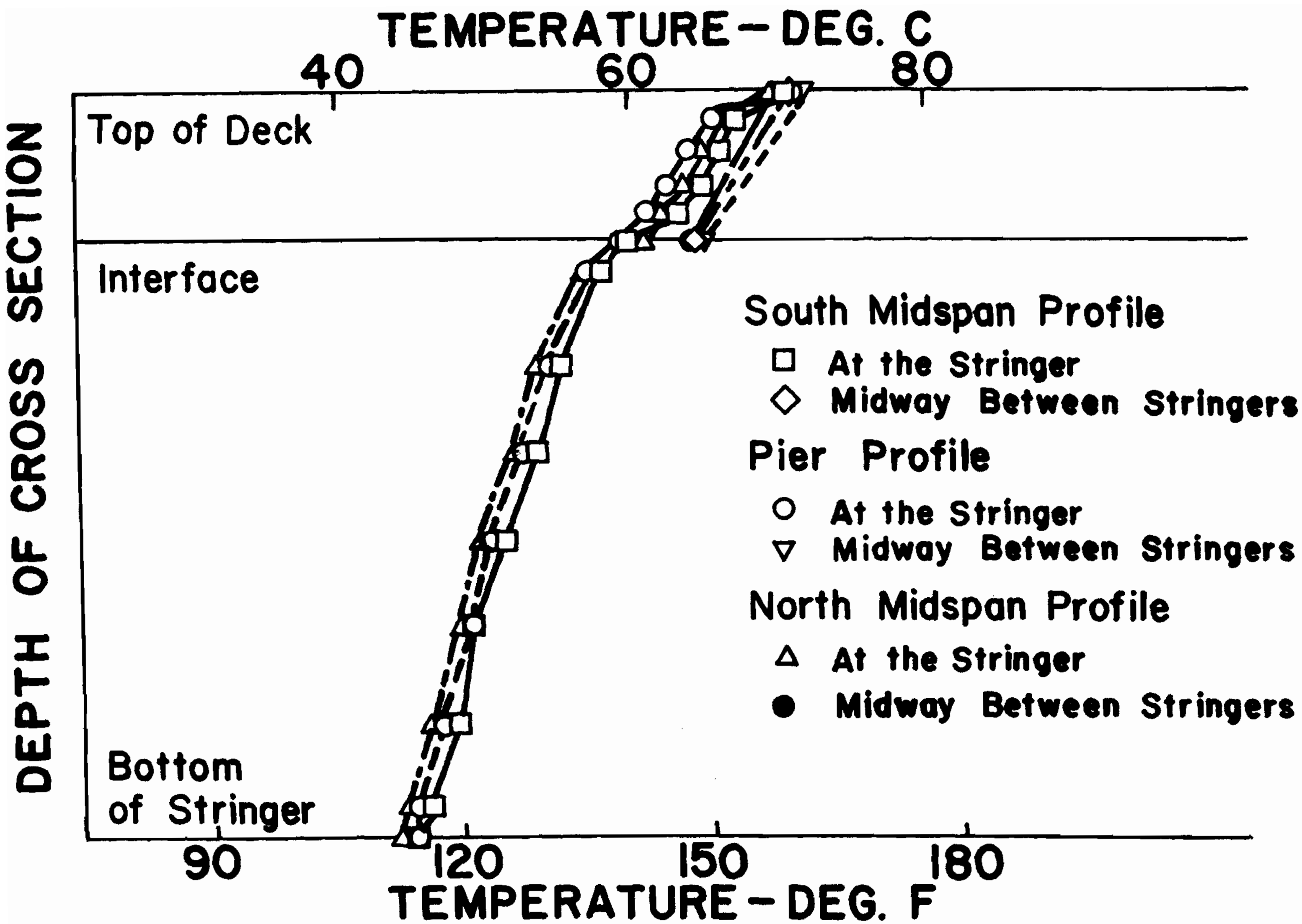
TEMPERATURE - DEG. C
40
60
80
Top of Deck
Interface
Bottom of Stringer
DEPTH OF CROSS SECTION
South Midspan Profile
At the Stringer
Midway Between Stringers
Pier Profile
At the Stringer
Midway Between Stringers
North Midspan Profile
At the Stringer
Midway Between Stringers
90
120
150
180
TEMPERATURE - DEG. F

Figure 5.

TABLE 1

TRANSVERSE STRAINS

Location (1)		South Midspan (2)	Pier (3)	North Midspan (4)
Top of Slab Above Stringer Web	(μ in./in.)	315[a] (239)[b]	301 (241)	306 (239)
Bottom of Slab Adjacent to Stringer	(μ in./in.)	379 (299)	374 (299)	383 (299)
Top of Slab Midway Between Stringers	(μ in./in.)	294 (299)	285 (299)	277 (299)
Bottom of Slab Midway Between Stringers	(μ in./in.)	333 (299)	346 (299)	347 (299)

[a]Experimental Strains.

[b]Theoretical Strains.

TABLE 2

THEORETICAL STRESSES

Location (1)		Longitudinal (2)	Transverse (3)
MIDSPANS			
Top of slab at the stringer	(psi)	-139	-378
	(kPa)	(-959)	(-2608)
Bottom of slab at the stringer	(psi)	379	583
	(kPa)	(2616)	(4022)
Top of slab midway between stringers	(psi)	-128	-166
	(kPa)	(-884)	(-1147)
Bottom of slab midway between stringers	(psi)	175	166
	(kPa)	(1208)	(1147)
PIER			
Top of slab at the stringer	(psi)	-197	-378
	(kPa)	(-1358)	(-2608)
Bottom of slab at the stringer	(psi)	377	583
	(kPa)	(2601)	(4022)
Top of slab midway between stringers	(psi)	-128	-166
	(kPa)	(-884)	(-1147)
Bottom of slab midway between stringers	(psi)	175	166
	(kPa)	(1208)	(1147)

VITA

Michael John Filla was born on July 15, 1955, at Washington, Missouri. He received his primary and secondary education at St. Francis Borgia Schools in Washington, Missouri. In August, 1973, he entered the University of Missouri-Rolla in pursuit of a degree in Civil Engineering which he received in December, 1977.

He worked as an Associate Engineer in the Structural/Civil Department for Brown & Root, Inc. at Houston, Texas, for the period January, 1978, to August, 1978.

Since August, 1978, he has been enrolled in the Graduate School of the University of Missouri-Rolla and has held a graduate teaching assistantship. He is an Engineer-in-Training in the State of Missouri, a member of Chi Epsilon, Tau Beta Pi, Phi Kappa Phi, and an Associate Member of the American Society of Civil Engineers.

APPENDIX A

EXPERIMENTAL DETERMINATION OF THE THERMAL COEFFICIENT OF EXPANSION OF THE CONCRETE DECK

APPENDIX A

EXPERIMENTAL DETERMINATION OF THE THERMAL COEFFICIENT OF EXPANSION OF THE CONCRETE DECK

Experimental evaluation of thermally induced stresses first requires an accurate determination of the effective coefficient of thermal expansion of the component materials of the structure. The following method was used to determine the thermal coefficient of expansion of the concrete deck.

A cantilever beam will uniformly strain without stress if unrestrained and subjected to a uniformly distributed temperature change. The thermal coefficient of expansion of the material will be the unit strain divided by the temperature change. Thus, cantilever reference bars were placed in the slab at locations 1 and 8, midway between the center stringer and an outside stringer, as shown in Fig. 1. Plan and elevation views of the small slab cantilever reference bar at location 1 are shown in Fig. 2, and plan and elevation views for the bars at location 8 are shown in Figs. 3 and 4, respectively. These reference bars were enclosed on three sides by ½-in. (13-mm) thick flexible styrofoam. Wire mesh was not used in these sections so that the concrete could expand freely as a result of thermal change and unrestrained expansion could be obtained. The styrofoam produced essentially no resistance to small expansive movements, and provided insulation between the boundaries to prevent change of the thermal gradients by the air space. The thermal gradients and temperatures obtained were then representative of any point through the deck between the stringers.

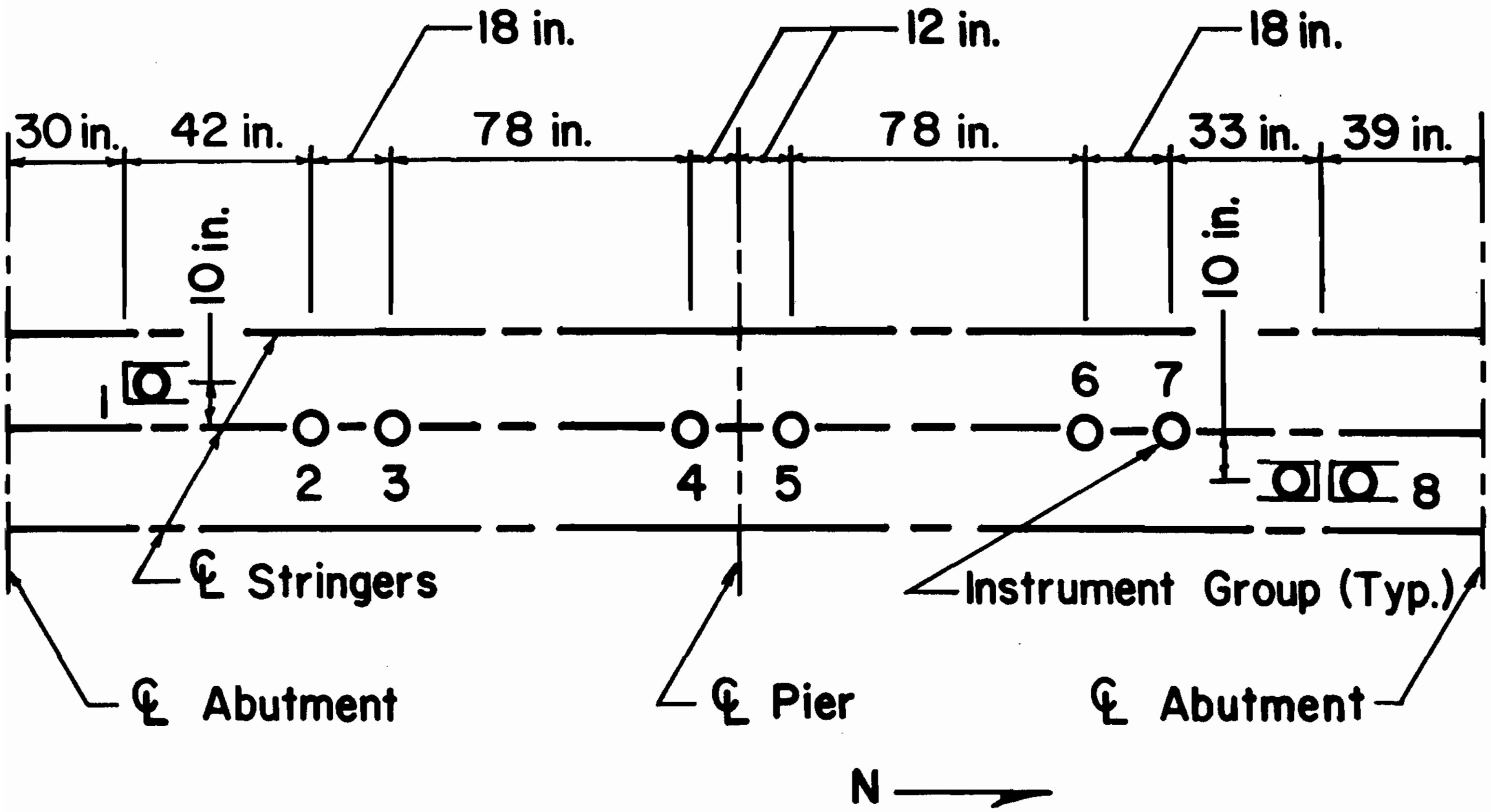

Figure 1. Plan View of Deck Instrumentation

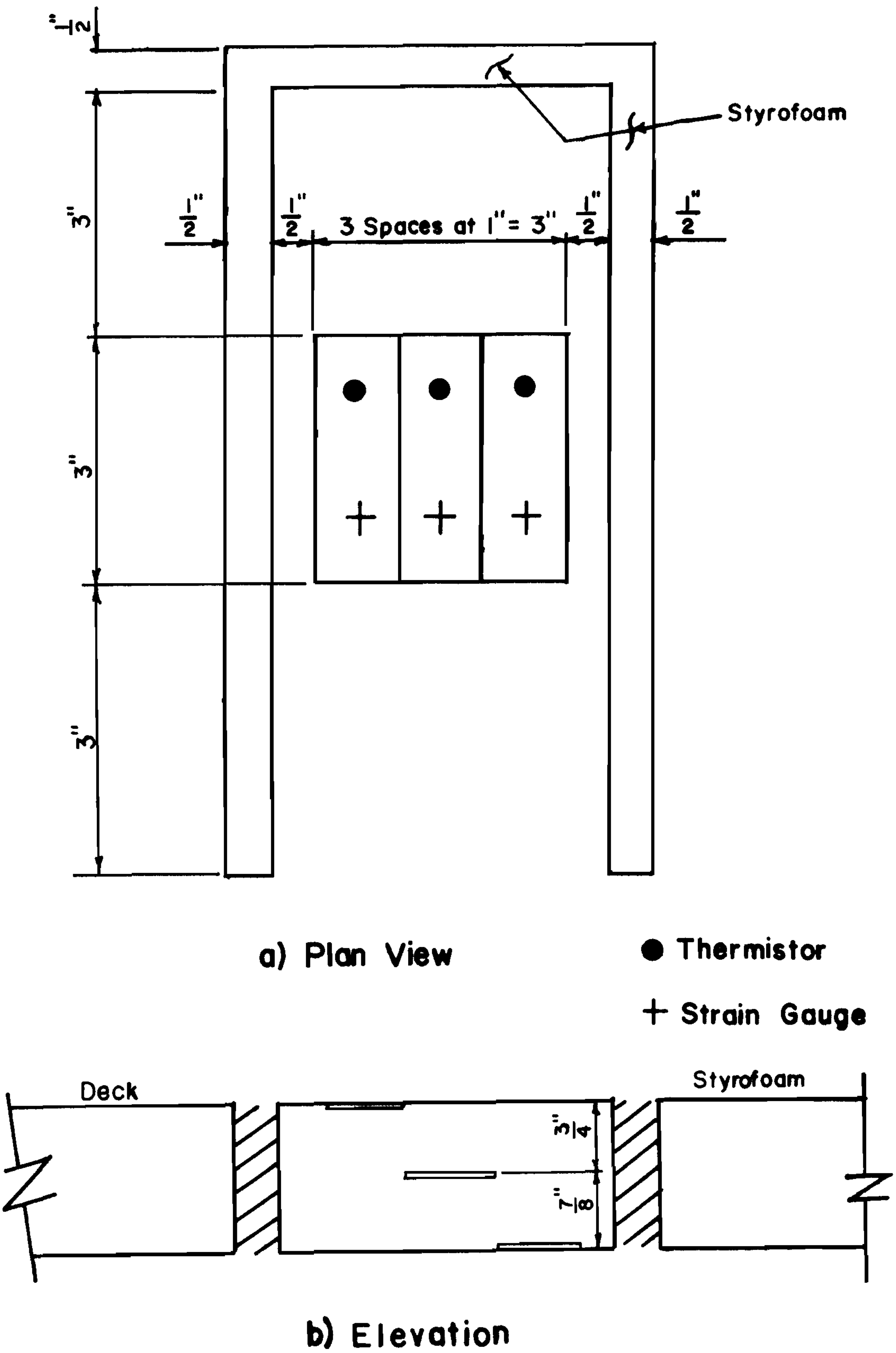

Figure 2. Reference Bar Instrumentation at Location 1

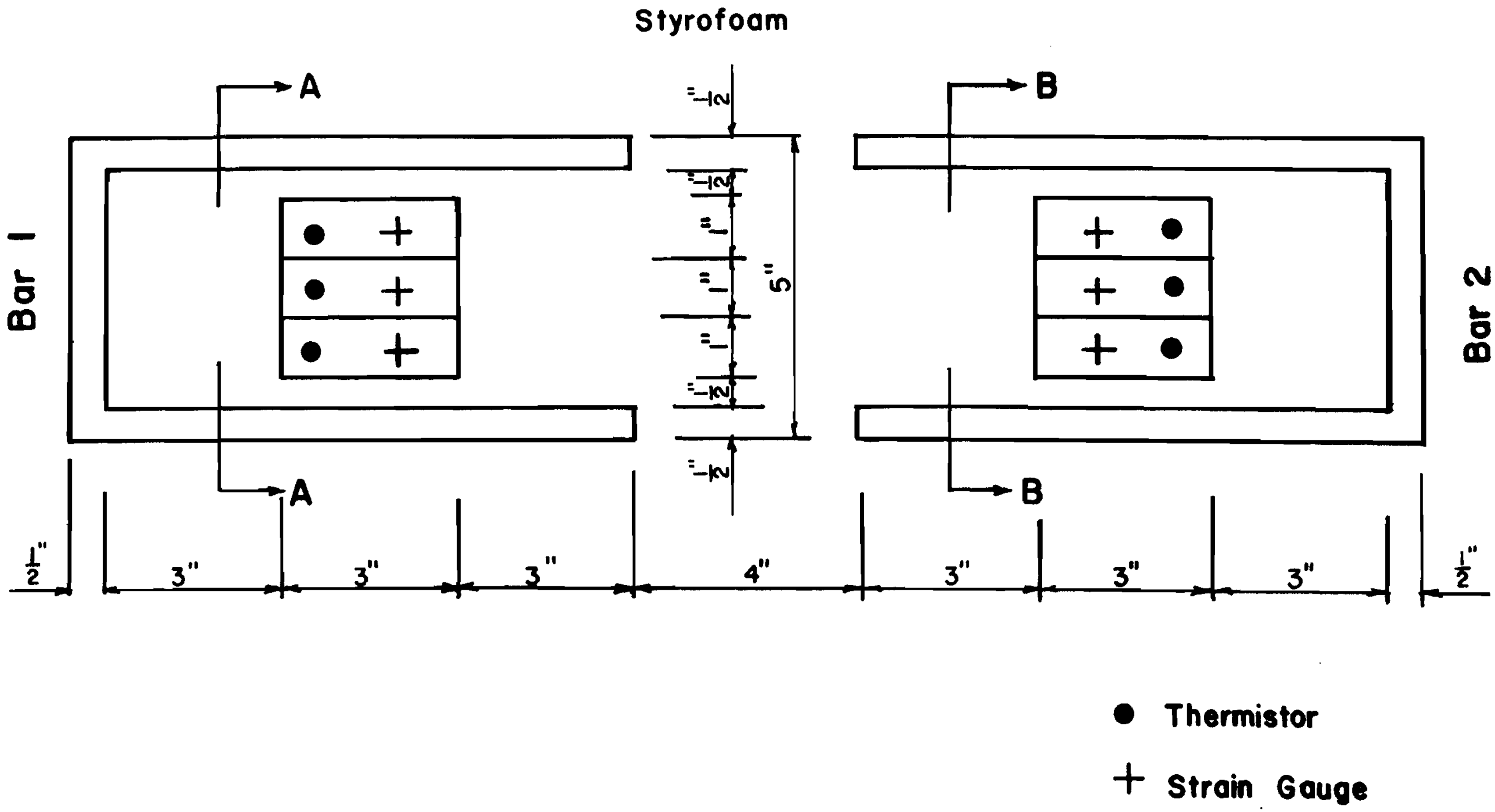

Figure 3. Plan View of Reference Bar Instrumentation at Location 8

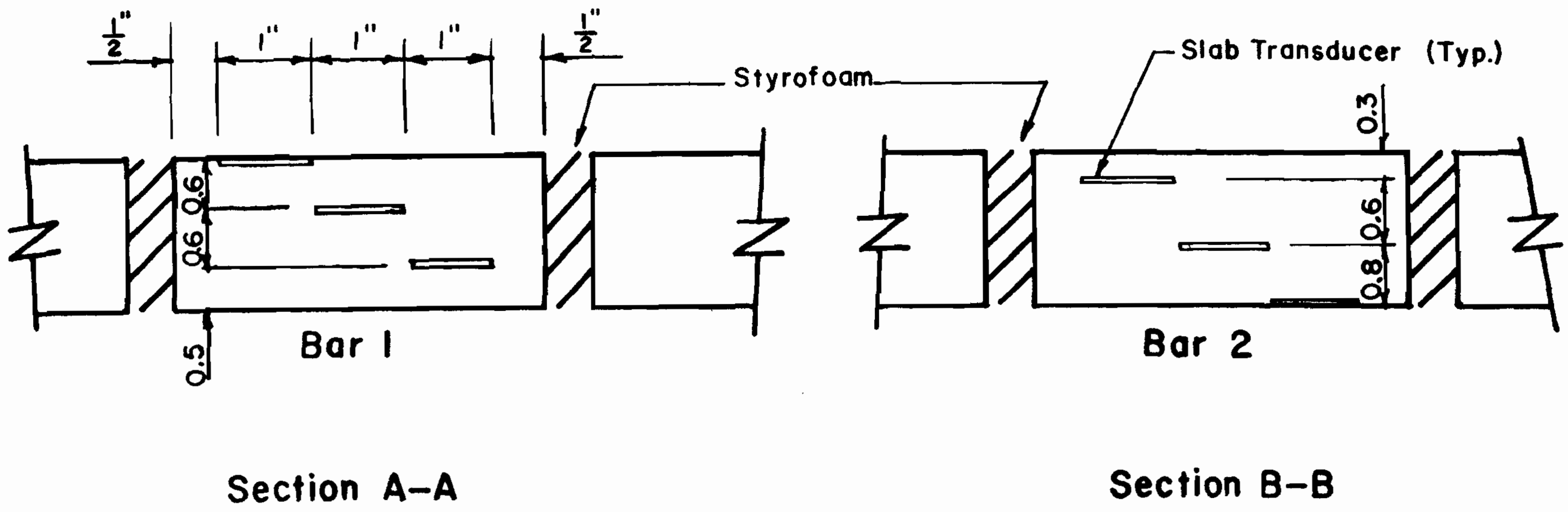

Figure 4. Sections of Reference Bars at Location 8

A slab transducer consisting of a glass microscope slide, strain gauge, and thermistor is shown in Fig. 5. As described previously, the strain gauges were Micro-Strain model 6C-2x2-120 w/L and the thermistors were Fenwal Uni-Curve No. VVA 33J1. A glass microscope slide was chosen because the thermal conductivity of the slides and the concrete are quite similar. The mechanical properties of the glass and the concrete matched very well except for Young's modulus.

The thermistors were connected to a digital voltmeter (Dann Model 5400) through a 100-channel thermistor stepping unit. The resistance of each thermistor was indicated by the volt-ohm meter. The temperature was determined for each thermistor by the use of the temperature-resistance graph furnished by the manufacturer. Strains were measured by the means of a Baldwin-Lima-Hamilton Model 120C strain indicator. A Baldwin-Lima-Hamilton Model 225 10 channel switch and balance unit was used to interface the strain gages and strain indicators.

At the start of each test the temperature and the initial strains were recorded and the heat lamps were turned on. After steady state temperatures were obtained, temperatures and strains were again recorded. The lamps were turned off and the bridge allowed to return to room temperature, after which the temperature and strains were again recorded for comparison with initial values.

No change in strain should be observed in a self-temperature-compensating strain gage subjected to a temperature change when freely suspended or when mounted to an unrestrained specimen of the material for which the gage was compensated and subjected to a uniform thermal gradient. However, a change in strain will occur. Thus, it is obvious that other factors are involved. The primary factors are

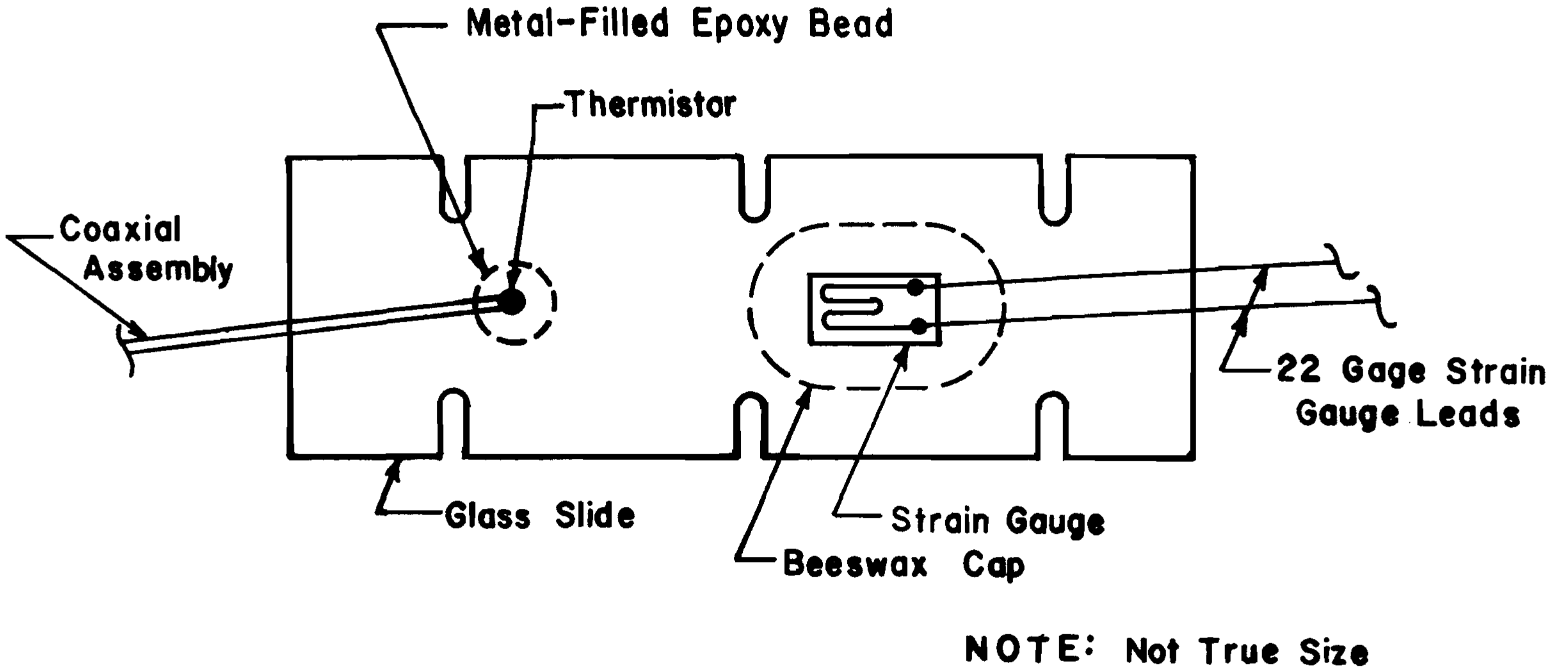

Figure 5. Slab Transducer

apparent strain and self-temperature-compensating (STC) mismatch.

The resistivity of an electric resistance strain gage, either unrestrained or strained, is also a function of gage temperature. Thus, a change in gage temperature, either externally or internally induced, will generally cause a resistance change in the gage and an indicated change in strain, which is referred to as apparent strain to distinguish it from strain caused by an applied stress. The magnitude of this apparent strain may be calculated by using the correction equation furnished by the manufacturer for each particular gage lot, in this instance:

for $T < 100^{\circ}$ F

$$\varepsilon_{app(st)} = 0 \tag{1}$$

for 100° F $< T < 200^{\circ}$ F

$$\varepsilon_{app(st)} = -(T - 100) \tag{2}$$

wherein $\varepsilon_{app(st)}$ is the apparent strain correction for the type of steel (1018) for which the gage was compensated in micro in./in. and T is the temperature of the gage in $^{\circ}$ F at the time of strain reading.

STC mismatch results when a strain gage is mounted on a material other than that used in obtaining the data for development of the apparent strain correction equation. In this investigation STC mismatch would occur because of the difference in the coefficients of thermal expansion of the concrete deck and 1018 steel.

For a concrete cantilever beam subjected to thermal loading, the true strain in the beam is given by the equation:

$$\varepsilon_{true} = \alpha_{con} \cdot \Delta T \tag{3}$$

where ε_{true} is the true strain due to a thermal gradient in micro in./in.; α_{con} is the thermal coefficient of the concrete deck; and ΔT is the change in temperature in ^{o} F.

If this true strain is being recorded by a self-temperature-compensating strain gage, the true strain in a cantilever beam is given by the equation:

$$\varepsilon_{true} = \varepsilon_{read} - \varepsilon_{app(st)} + (6.7 - \alpha_{con})\Delta T + (\alpha_{con} \cdot \Delta T) \tag{4}$$

where ε_{read} is the strain recorded in micro in./in.; $\varepsilon_{app(st)}$ is the apparent strain correction for the type of steel (1018) for which the gage was compensated in micro in./in.; the numerical value, 6.7, is the thermal coefficient of the 1018 steel for which the gage was compensated; α_{con} is the thermal coefficient of the concrete deck; and ΔT is the change in temperature ^{o} F.

Equating Eq. 3 and Eq. 4, and rearranging terms:

$$\alpha_{con} = \frac{-\varepsilon_{app(st)} + \varepsilon_{read}}{\Delta T} + 6.7 \tag{5}$$

in which the terms are defined in Eq. 4.

The step-wise development of the results of the ten tests summarized in Table I is illustrated for test number 8 in the following.

Step 1. Strain gages balanced (to zero) and initial temperature of 73^{o} F (23^{o} C) recorded.

Step 2. Steady state temperature and strains recorded.

Step 3. The change in temperature, ΔT, and the change in strain, ε_{read}, calculated (i.e., final reading minus initial reading).

Step 4. The apparent strain correction calculated for the 1018 steel for which the gage was compensated, using Eq. 2.

TABLE I

EXPERIMENTAL DATA

Test (1)	Strain Read (2)	ε_{app} (3)	ΔT (4)	α (5)
1	-188	-42.9	57.7	4.19
2	-198	-43.1	56.1	3.94
3	-199	-40.2	54.8	3.80
4	-190	-41.9	59.3	4.20
5	-193	-42.7	55.5	3.99
6	-191	-41.9	61.9	4.30
7	-176	-37.5	54.8	4.17
8	-186	-41.6	54.6	4.06
9	-192	-40.1	54.9	3.93
10	-185	-39.0	56.4	4.11

Step 5. Knowing the apparent strain correction for the steel (1018) for which the gage was compensated and its coefficient of thermal expansion (6.7), the actual strain recorded (ε_{read}), and the change in temperature (ΔT), the coefficient of thermal expansion for the concrete deck is calculated, using Eq. 5, as being 4.06 x 10^{-6} in./in.^{o} F, i.e.,

$$\alpha_{con} = \frac{-(41.86) + (-186)}{54.6} + 6.7$$

$$\alpha_{con} = 4.06 \ \mu \text{ in./in.}^{o} \text{ F}$$

The results of the ten tests shown in Table I were averaged and the thermal coefficient of expansion for the concrete deck was taken as 4.1 x 10^{-6} in./in.^{o} F.

APPENDIX B

DATA REDUCTION OF CARBON-STEEL TEMPERATURE-COMPENSATED SR-4 STRAIN GAGES

APPENDIX B

DATA REDUCTION OF CARBON-STEEL TEMPERATURE-COMPENSATED SR-4 STRAIN GAGES

As described previously, reduction of observed data obtained from the carbon-steel temperature-compensated SR-4 strain gages required correction for 1) apparent strain, 2) self-temperature-compensating (STC) mismatch, 3) compensated (nonindicated) thermal strain, and 4) resistance change of leadwires.

Theoretically a steel-temperature-compensated gage attached to an unrestrained steel specimen should indicate no strain when subjecte to a temperature change (1,2). However, changes in the electrical resistance properties of the gage caused by external temperature chang internal heating, and small differences in the material of the gage and of the specimen, will produce an indicated strain. The electrical resistance-apparent strain relationship is shown on graphs furnished by the gage manufacturer for data reduction.

STC mismatch is the indicated thermal strain produced by the difference in thermal coefficients of expansion when a self-temperature-compensated gage is mounted on an unrestrained specimen having a thermal coefficient of expansion other than that for which the gage is compensated.

Compensated, or nonindicated, thermal strain is the unit thermal strain which would be induced in an unrestrained specimen subjected to a temperature change, i.e., $\alpha \cdot \Delta T$, the product of the thermal coefficient of expansion and the change in temperature.

4. The test structure was allowed to cool for comparison of cyclic action and instrumentation drift.

The final correction factors were determined by repeating the above procedure and averaging the results. The following correction factors were used to reduce the experimental data:

Top of the Deck . . . . 192 micro strain

Bottom of the Deck. . . . . 55 micro strain

Bottom of the Stringer. . . . . 33 micro strain

The resistance change of the leadwires due to an increase in temperature is reported by the indicator as a tensile strain; thus the correction factors were subtracted from the observed, recorded strain.

REFERENCES

1. Lewis, D. B., "Abutment-Thermal Interaction of a Composite Bridge," thesis presented to the University of Missouri-Rolla, at Rolla, Missouri, in 1980, in partial fulfillment of the requirements for the degree of Master of Science.
2. Micro-Measurements Division, Measurements Group, "Temperature-Induced Apparent Strain and Gage Factor Variation in Strain Gages," Tech Note TN-128-3, author, Raliegh, N.C., 1976.

APPENDIX C

THEORETICAL STRESSES

APPENDIX C

THEORETICAL STRESSES

The objective of this study was to correlate experimental and theoretical transverse strains induced in a composite bridge structure by steady-state thermal loading. The correlation of experimental and theoretical longitudinal strains is discussed in the results of the experimental investigation. However, the page limitation restrictions prevented inclusion of the numerical values of the longitudinal theoretical stresses.

Determination of stress in the theoretical procedure utilizes the interaction of longitudinal, transverse, and vertical strains and Poisson's ratio, with longitudinal strains being the major parameter. In this study, it was feasible to instrument only for longitudinal and transverse strains. Thus, a prediction of stress based strictly on experimental observation is not possible. However, because the experimental and theoretical longitudinal strains correlate closely, theoretical stresses calculated from the observed temperature profile are believed to be valid.

As previously discussed, the power level for this study duplicated as closely as possible the maximum power level of the prior study (4) and the concurrent study (2). This provided a data bank for comparison of the effect of the change in modulus of elasticity and coefficient of thermal expansion from those of the prior study. Table II shows the material properties used for computing the theoretical stresses for both the present study and the prior study.

TABLE II

MATERIAL PROPERTIES

Property (1)	Value: Steel (2)	Value: Concrete (3)
YOUNG'S MODULUS		
Present Study	29.0×10^6 psi	4.5×10^6 psi
	(20.0×10^7 kPa)	(3.1×10^7 kPa)
Prior Study	30.0×10^6 psi	3.0×10^6 psi
	(20.7×10^7 kPa)	(20.7×10^6 kPa)
POISSON'S RATIO		
Present Study	0.30	0.18
Prior Study	0.30	0.20
THERMAL COEFFICIENT		
Present Study	$6.5 \times 10^{-6}/^{o}$ F	$4.1 \times 10^{-6}/^{o}$ F
	($11.7 \times 10^{-6}/^{o}$ C)	($7.4 \times 10^{-6}/^{o}$ C)
Prior Study	$6.5 \times 10^{-6}/^{o}$ F	$3.5 \times 10^{-6}/^{o}$ F
	($11.7 \times 10^{-6}/^{o}$ C)	($6.3 \times 10^{-6}/^{o}$ C)

It should be noted that the temperature profiles of this study and those of the prior study will not coincide. The surface temperature of the bridge deck was approximately 158^{o} F for this study which is 8^{o} F warmer than that of the prior study. For this study the strain gages and thermistors were mounted directly on the surface of the bridge deck, whereas in the prior study (4) the transducers were embedded just below the surface of the concrete deck. However, at interior elevations through a typical cross section of the bridge the temperature profiles are essentially identical.

The previous described procedure of Emanuel and Hulsey (1) and the computer program developed by Hulsey (3) were used to obtain the theoretical stresses. Three cases were analyzed: a) both the slab and the beam in plane stress, b) the slab in plane strain and the beam in plane stress, and c) the slab in some state between plane stress and plane strain (partially restrained) and the beam in plane stress.

The theoretical stresses are tabulated in Tables III and IV for the present investigation and for the prior study, respectively.

TABLE III

THEORETICAL STRESSES FOR PROPERTIES AT THE TIME OF THIS INVESTIGATION

Location (1)		Case a (2)	Case b (3)	Case c (4)
MIDSPANS				
Top of Slab	(psi)	-59	-118*	-110
	(kPa)	(-409)	(-814)	(-755)
Bottom of Slab	(psi)	162	128	254*
	(kPa)	(1113)	(881)	(1751)
Top of Stringer	(psi)	-2829	-1883	-3360*
	(kPa)	(-19 492)	(-12 974)	(-23 150)
Bottom of Stringer	(psi)	900	2598*	1648
	(kPa)	(6204)	(17 900)	(11 355)
PIER				
Top of Slab	(psi)	-67	-188*	-139
	(kPa)	(-460)	(-1295)	(-956)
Bottom of Slab	(psi)	161	125	253*
	(kPa)	(1111)	(864)	(1743)
Top of Stringer	(psi)	-2830	-1899	-3368*
	(kPa)	(19 499)	(-13 084)	(-23 206)
Bottom of Stringer	(psi)	1051	4316*	2484
	(kPa)	(7241)	(29 737)	(17 115)

*Maximum of the three cases.

TABLE IV

THEORETICAL STRESSES FOR PROPERTIES
OF THE PRIOR STUDY

Location (1)		Case a (2)	Case b (3)	Case c (4)
MIDSPANS				
Top of Slab	(psi)	9	-31	-47*
	(kPa)	(62)	(-214)	(-324)
Bottom of Slab	(psi)	140	119	264*
	(kPa)	(966)	(821)	(1822)
Top of Stringer	(psi)	-4380	-3290	-4890*
	(kPa)	(-30 222)	(-22 700)	(-33 740)
Bottom of Stringer	(psi)	1190	1910*	1250
	(kPa)	(8211)	(13 179)	(8625)
PIER				
Top of Slab	(psi)	11	-42	-44*
	(kPa)	(76)	(-290)	(-304)
Bottom of Slab	(psi)	140	114	264*
	(kPa)	(966)	(787)	(1822)
Top of Stringer	(psi)	-4380	-3340	-4880*
	(kPa)	(-30 222)	(-23 046)	(-33 672)
Bottom of Stringer	(psi)	1150	2940*	1150
	(kPa)	(7935)	(20 286)	(7435)

*Maximum of the three cases.

REFERENCES

1. Emanuel, J. H. and Hulsey, J. L., "Thermal Stresses and Deformations in Nonprismatic Indeterminate Composite Bridges," Transportation Research Record 607, Transportation Research Board, National Academy of Sciences, Washington, D.C., 1976, pp. 4-6.

2. Lewis, D. B., "Abutment-Thermal Interaction of a Composite Bridge," thesis presented to the University of Missouri-Rolla at Rolla, Mo., in 1980, in partial fulfillment of the requirements for the degree of Master of Science.

3. Hulsey, J. L., "Environmental Effects on Composite-Girder Bridge Structures," thesis presented to the University of Missouri-Rolla, at Rolla, Missouri, in 1976, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

4. Wisch, D. J., "Thermal Stresses Induced in a Model Composite Bridge Structure," thesis presented to the University of Missouri-Rolla, at Rolla, Missouri, in 1977, in partial fulfillment of the requirements for the degree of Master of Science.