1972

Thermal stresses and movements in bridges

Joseph Charles Reynolds

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THERMAL STRESSES AND MOVEMENTS IN BRIDGES

BY

JOSEPH CHARLES REYNOLDS, 1948-

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

1972

Approved by

(Advisor)

PUBLICATION THESIS OPTION

This thesis has been prepared in the style utilized for publications of the American Society of Civil Engineers. Pages 1-46 will be presented for publication in the Journal of the Structural Division of the ASCE.

ABSTRACT

THERMAL STRESSES AND MOVEMENTS IN BRIDGES

By Joseph Charles Reynolds, A. M. ASCE

A state of the art regarding the thermal behavior of bridges and considerations to be given to the resultant thermal effects is presented. The results of studies related to the thermal effects on bridges are reviewed. Studies attempting to relate environmental factors to bridge temperatures, and subsequently to bridge movements and stresses, indicate that the task is extremely complex. Some correlation has been made between equations predicting bridge temperatures and movements based on weather bureau records. However, further research is needed to evaluate the effects of factors other than temperature, such as creep, shrinkage, and humidity on bridge movements and stresses.

KEY WORDS: bibliographies; bridges (decks); bridges (structures); composite construction; concrete bridges; strains; stresses; structural engineering; temperature.

ACKNOWLEDGEMENT

Appreciation is expressed to my thesis advisor, Jack H. Emanuel, Associate Professor of Civil Engineering, for his encouragement and assistance during the preparation of this paper.

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THERMAL STRESSES AND MOVEMENTS IN BRIDGES

By Joseph Charles Reynolds,[1] A. M. ASCE

INTRODUCTION

Uncertainties as to the magnitudes and effects of thermally induced stresses and/or movements in bridges are of major concern to bridge design engineers. Thermal effects on bridges are caused by both the short term daily temperature changes and the more lengthy seasonal temperature changes. There are many factors involved in the longitudinal deformation of a bridge, but among the major factors to be considered are the thermally induced movements, along with creep and shrinkage early in the life of the structure (30)[2]. Therefore, thermal stresses and movements should be considered in the design process. However, due to the lack of a rational design criteria, the design engineer can not be certain that a structure is both the safest and most economical. Consequently, many uneconomical structures result due to either the higher initial costs of over-design or the higher maintenance costs of under-design.

A survey by Ekberg and Emanuel (23) was made to determine, among other things, the problems designers have in dealing with thermal stresses and movements in bridges. From the replies of 40 state highway departments and 96 consulting engineers, it was found that thermal

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[2]Numerals in parentheses refer to corresponding items in Appendix I.-Bibliography.

effects were considered more frequently for steel bridges than for concrete bridges. The design engineers expressed a desire for a rational method of design for thermal effects. They also indicated an interest in information about actual movements of different types of bridges and about actual temperature distributions due to variation and duration of ambient temperatures.

Current design practice generally is to ignore thermal effects in both simple span and continuous bridges and assume that movements at expansion devices relieve any induced stresses. However, non-uniform temperature distributions throughout the depth of the bridge may create local stresses of considerable magnitude. There are many uncertainties as to how to calculate the thermal stresses induced by a non-linear temperature distribution.

The need for more research in the area of thermal behavior of bridges was expressed editorially in 1960 (53). Especially emphasized were four concepts: 1) thermal stresses have caused damage to many structures due to neglect in design of the thermal effects; 2) adequate provision for thermal strains could be made in design similarly as provisions are made for dead and live loads; 3) the primary consideration should always be safety, but a close second is good engineering to prevent structural damage economically; and 4) rather than always providing for free movement due to temperature change it might be more economical to design the structure to resist thermally induced stresses.

Since 1960 much work has been done in the area of thermal strains. Several bridges have been observed by Zuk (76), each during a one-year period, to determine some basis for predicting the temperature distribution which would result from a set of given conditions. Daily and yearly temperature distributions were recorded. Air temperature, wind, humidity, intensity of solar radiation, and type of material all affect the temperature distribution. For example, in a composite steel and concrete bridge, the concrete deck heats up from the top down through the slab due to solar radiation, while the steel is shaded most of the day and thus will maintain a temperature approximately equal to the ambient temperature. Thus, a non-linear temperature distribution is established and thermal stresses and/or strains result. Seasonal temperature changes cause bigger movements than daily changes. One difficulty in the study of thermal effects is that either strain or stress, or both strain and stress, may result (41). Thermal strain is present without stress if the material is free to expand. Thermal stresses are present without strain if the material is completely restrained from movement. A combination of strain and stress is usually present because the materials are never completely free to move or completely restrained. Complicating the study of thermal strains is the presence of strain due to creep, shrinkage, humidity, moisture swelling, and change in vehicular loading.

Expansion devices which do not behave as expected (e.g., frozen rockers) are uneconomical and can cause damage to the structure. A knowledge of the 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.

Reducing the number of expansion joints in a structure will decrease the amount of joint maintenance required as well as the noise and roughness experienced by motorists at these joints (54). The remaining joints will be somewhat larger, so the importance of knowing what movements to expect is emphasized.

A continuous concrete bridge deck on roller bearings shortens due to concrete shrinkage, and to thermal contraction due to a drop in temperature (69,68). A continuous prestressed-concrete bridge will also shorten due to elastic shortening and plastic flow induced by prestressing. Since the deck generally shortens by both ends moving inward, some point on the bridge deck will not move. This point--called the stagnation point--is located at the "center of gravity" of the horizontal stiffnesses of the deck supports. The location of the stagnation point can be varied to a certain extent by the type of bearings used. If the stagnation point is known, the movements of other portions of the deck are defined, and a better idea can be had of the movements (thermal and otherwise) to be expected at the various bearings. The horizontal forces, which are transmitted to the piers and abutments due to the movement of the bridge deck, can then be more realistically calculated (66).

This paper seeks to acquaint the reader with the state of the art regarding thermal behavior of bridges and considerations to be given to the resultant thermal effects. The results of studies related to the thermal effects on bridges are reviewed and grouped for continuity and clarity. The relationship of the ambient temperature to the bridge temperature, and the relationship of both ambient and bridge temperatures to thermal stresses and movements are discussed. Some of the types of supporting and expansion devices used to provide for thermal movements are also presented. The current code requirements regarding thermal effects on bridges of both the United States (AASHO-1969) and Germany (DIN-1967) are discussed.

AIR TEMPERATURE

The records of air temperature can be an important and useful tool to the design engineer in estimating the effects of temperature on a structure. Air temperature as well as other weather data such as humidity, wind, and amount of sunlight are regularly recorded for almost all regions of the world. By interpreting the local weather data, suitable provision can be made in the design to resist or allow for the movements induced by temperature changes.

There are two basic temperature cycles (9). The daily cycle usually begins with a low temperature being attained just before sunrise. The sun's appearance causes a steady rise in temperature until the daily peak temperature is reached, usually in mid-afternoon a few hours before sunset, and then air temperatures drop more rapidly to a low reached prior to sunrise the next morning. The basic daily temperature cycle may be altered by the presence of clouds shading the area or releasing some form of precipitation. This can result in a sudden drop of temperature. New air masses moving into the locality from a cooler or warmer region may also mask the usual daily temperature cycle. The yearly temperature cycle results from the changes in position and distance of the earth relative to the sun. Both of these temperature cycles are important to the design engineer. The daily cycle provides quick temperature variations through the different parts of the structure while the yearly cycle induces the greatest overall movements. In an attempt to establish the range of bridge temperature and movement for which a bridge should be designed, Emerson (25) used meteorological records showing the absolute and the average maximum and minimum temperatures in Britain. She found that the current British standards did not provide temperature ranges appropriate to any area of Britain, and that the minimum temperatures listed were too high. Other British researchers (18,58) have also used weather bureau records in an attempt to define the response of bridges to changes in their environment. Berwanger (9) used the records of Winnipeg International Airport to help select three possible temperature distributions in a bridge. Steward (54) used the temperatures recorded by government weather stations located near bridge sites in calculating the apparent coefficients of thermal expansion of those bridges.

BRIDGE TEMPERATURE

The bottom elements of a bridge will ordinarily have the same temperature as that of the air (76). The upper elements and the exterior beams will vary in temperature depending upon the amount of solar radiation received, the wind, and the amount and type of precipitation (9). The top of the slab is warmer than the bottom of the bridge when the sun shines on the exposed deck. The top will cool faster than the girders when a rain or snow storm first begins. A uniform temperature can exist just before sunrise when the air temperature has remained nearly constant for several hours. Thus, a variety of temperature distributions are possible throughout the depth of a bridge.

Several different temperature distributions have been used by researchers. Berwanger found that three temperature distributions were typical of those most likely to occur in a composite concrete-deck and steel-girder bridge. The bottom of the slab was assumed to be at the temperature of the steel girder. The first case involved a uniform temperature throughout the slab. Cases two and three had a non-linear temperature variation in the slab, one with the top of the slab warmer than the bottom and one with it cooler. Maher (41) assumed a linear temperature distribution in the top slab of a box section in his study of continuous, prestressed-concrete bridges built of hollow concrete box sections.

This assumption was based on observations of the Medway and Western Avenue Extension bridges which showed an approximately linear temperature differential of 26°F and 21°F, respectively, through the top slab of the box section. The sides and bottom of the box section were assumed to be at a constant temperature, with only the top slab of the box section having a linear temperature distribution.

Zuk (72) used various linear thermal gradients in developing an elastic theory for determining the stresses and strains in a composite concrete-slab and steel-beam section. Later (76), he broadened his theory to cover any temperature function in the composite section. Liu and Zuk (40) worked with four types of prestressed-concrete members: 1) prestressed beam with straight tendons; 2) prestressed beam with draped tendons; 3) prestressed beam with straight tendons, composite with a concrete slab; and 4) prestressed beam with draped tendons, composite with a concrete slab. They extended Zuk's earlier work with composite members (72) to these four cases of prestressed members, and showed example problems using four different temperature distributions for each type of member. The shear and the moment at the interface between the beam and slab and the force in the tendon were all determined from the solution of three simultaneous equations.

Mean bridge temperatures as well as temperatures at various points in a bridge have been recorded by several researchers. Menzies (44) measured the mean temperature of the Moat Street Flyover in Britain by lowering a thermometer into a tube partially filled with mercury which had been embedded vertically to a depth of five inches in the concrete slab. He related the air temperature to the temperature at mid-depth of the concrete slab by the following equation:

$$0.156 d\theta_c/dt + \theta_c = \theta_a \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where t = time, in hours; θ_c = concrete temperature, in °C; and θ_a = air temperature, in °C. Wroth (67) used the interior air temperature, as measured in the center compartment of the spine beam, as a guide to the concrete temperature in his studies of the thermal movements of the 2054-ft Hammersmith Flyover. Emerson (25) used a mean bridge temperature and the shaded air temperature in her attempt to correlate temperature with the movements of three bridges in Southern England: the Medway Bridge (70% concrete beam and slab, 30% variable depth concrete box); the Hammersmith Flyover (concrete box spine beam); and the Beachley Viaduct/Wye Bridge (steel box).

Zuk studied the temperatures and strains of two bridges: one a 66-ft composite concrete-deck and steel-stringer bridge, and the other a 36-ft reinforced-concrete bridge. A 24-point automatic recorder was used to record the temperatures hourly at various points throughout the depth and width of each bridge. All strain measurements were taken under dead load and constant slab moisture conditions in an attempt to separate the thermal strains from those which might result from other causes. Temperatures were recorded for a one-year period for the 66-ft composite bridge (76). Then a one-inch thick coating of sprayed foam urethane insulation was applied to the middle 20 ft of the bridge and temperatures were recorded for an additional nine months. Typical temperature distributions were shown for both cases and the effect of the insulation was discussed. The purpose of the insulation was to reduce the number of freeze-thaw cycles experienced by the bridge deck. This was accomplished by the heat flow up from the insulated beams into the slab which heated up the slab a few degrees warmer than it would have been without the insulation. One problem noted was the possibility of moisture retention by the insulation causing corrosion at the tension cracks (76). Temperatures were recorded for a one-year period for the 36-ft reinforced-concrete bridge, and some typical values including the thermal contours through the beam and slab were presented (78). As expected, noticable differences were found between the exterior beams exposed to the sun and the shaded interior beams.

Barber (6) developed and tested an equation relating weather factors to the maximum pavement temperature. The coefficients may be modified in order to use the equation with different pavements and in different regions of the country. Zuk (76) developed coefficients for use in Barber's equation to calculate the maximum bridge surface temperature for two conditions. For a normal concrete deck in the Middle Atlantic States the maximum surface temperature in degrees Fahrenheit is:

$$T_m = T_a + 0.18L + 0.667(0.50T_r + 0.054L) \quad (2)$$

in which T_m = the maximum surface temperature, in °F; T_a = the average daily temperature, in °F; T_r = the daily range in air temperature, in °F; and L = the solar radiation received on a horizontal surface, in g-cal/cm^2/day.

The maximum surface temperature of a bitumen covered deck would be:

$$T_m = T_a + 0.027L + 0.65(0.50T_r + 0.081L) \quad (3)$$

The constants in both equations would vary depending on the local conditions. The Langleys of solar radiation "L" may be determined from U.S. Weather Bureau maps or may be measured with an inexpensive pyrheliometer.

An approximate equation for the maximum differential between the top and bottom temperatures of a composite steel and concrete bridge was also developed by Zuk (76):

$$\Delta T_m = T_m - T_a - \lambda T_r \quad \text{.......................................(4)}$$

where λ is the factor indicating the phase lag between the maximum surface temperature and the maximum ambient temperature. For the Middle Atlantic States a lag factor of one-fourth was found to be appropriate for the summer and a value of one-half for the winter. An equation was also presented showing the temperature distribution throughout the slab depth. Results obtained from these equations were compared with the results of field tests conducted on the 66-ft uninsulated composite bridge. As expected, the maximum surface temperature of a bitumen covered deck was approximately 15°F higher than the normal grey-white concrete deck for sunny summer afternoons. Calculated and measured temperatures as well as the maximum temperature differential between the top and bottom of the bridge checked very closely. Comparisons of the recorded temperature variations through the thickness of the slab with the calculated values showed fair agreement. The higher temperatures measured on the bitumen covered pavements should be noted by designers because many concrete bridge decks will eventually receive a new wearing surface; possibly bituminous. No relief of stresses should be expected from the possible insulation value of the bituminous surfacing. In Southern England a thickness of two inches of bituminous surfacing was required before the insulation of the surfacing began to match the greater heat absorption of the darker surface (32). Other tests (23) indicated that a three-inch layer of bituminous surfacing on a seven-inch pavement slab reduced the maximum surface temperature of the slab by one-third.

Emerson (25) found that in England the maximum range of mean bridge temperatures was equal to the range of shade temperatures for concrete bridges. Steel box-section bridges had a temperature range from 6°F below the minimum temperature to about 1.5 times the maximum temperature in degrees Celsius.

During the one-year period of observation of the 66-ft composite bridge, Zuk (76) measured a high temperature of 123°F on top of the concrete slab with an air temperature of 97°F, and a low temperature of -6°F at the bottom of the steel girder with an equal air temperature. The maximum temperature differential for an interior beam on the 66-ft uninsulated bridge was 37°F. An exterior beam had a 42°F difference. Temperature differentials for the insulated bridge were up to 25 percent greater than those which occurred before the insulation was applied. This increase was due to the slower transfer of heat from the air to the insulated beam.

Naruoka, et al., (28) conducted temperature tests on a composite concrete slab and steel girder bridge in Japan. Temperatures of 122°F were observed at the top surface of the two-inch thick asphalt surfacing with an air temperature of 93°F. The concrete slab reached its highest temperature about two hours later (4:30 p.m.) with the top and bottom temperatures being 108°F and 91°F, respectively. Both temperatures were higher than the air temperatures at the top and bottom of the slab.

For simply supported structures, when the top of the slab is warmer than the bottom, the stresses developed will be opposite to those stresses caused by live and dead loads. However, when the top of the slab is cooler than the bottom, the stresses induced by the temperature difference will be additive to those resulting from live and dead loads.

THERMAL STRESSES AND MOVEMENTS

Uniform temperature changes in a homogeneous and isotropic material cause axial deformation. A varying temperature distribution through the bridge produces flexural deformation. In composite bridges the concrete deck is anchored to the steel girders by shear connectors. Theoretically, there is no movement between the steel-girder and concrete-deck at the interface. Thus the differing coefficients of thermal expansion of the steel and concrete will create additional stresses as the two materials try to match the movements of each other (9).

Internal thermal stresses are normally affected more by large temperature differentials than by the large overall temperature changes between summer and winter which will cause general expansion or contraction of the bridge (76).

There are two options available to the designer who is considering the effects of temperature differentials on a bridge. A sufficient number and size of expansion joints can be provided to accommodate the thermal movements, or the structure can be designed to resist the stresses induced when the materials are restrained from movement. The first method is commonly used for simple span bridges and the second is often used for continuous span bridges, but either method can be used with either type structure. In any case the designer needs to know what thermal stresses and/or movements will occur. Several types of bridges have been studied to determine the movements that occur with varying temperatures.

The Hammersmith Flyover (67) is a precast, prestressed-concrete, continuous four-lane viaduct, 2054-ft long between abutments, with

16 spans, mostly of 140 ft. The main structural element is a 26-ft wide continuous hollow-spine beam supported on central columns. Each column is supported on a pair of roller bearings which provide a 10-in. movement range. All of the longitudinal expansion movement of the suspended superstructure occurs at one expansion joint. The designers provided for 14-in. of movement at the expansion joint--10.7-in. for temperature and humidity movements; and 3.3-in. for creep and shrinkage. In the design, a coefficient of thermal expansion of $6 \times 10^{-6}/°F$ and a bridge temperature range of 60°F were used to predict thermal movement of the concrete. Extensive measurements were made on this bridge to determine the actual effects of temperature, humidity, and residual creep and shrinkage. The air temperature inside the box-section structure was used as an indicator of the concrete temperature. The movements measured at one of the pier columns closely followed the temperature changes with no discernible time lag. A linear relationship between these movements and the interior air temperature was shown in a plot of three days' readings. A 50°F temperature range was observed in the structure. The thermal movement was 8.2-in., which corresponds to a coefficient of thermal expansion of $6.7 \times 10^{-6}/°F$ if the effects of humidity are neglected.

Black and Adams (13) observed the magnitude of movements of five bridges in Great Britain--each of different size and type. In each case, the five-day mean bridge temperature (calculated from the movement at a joint and an assumed coefficient of thermal expansion) and the mean ambient temperature recorded at the nearest government recording station were quite comparable. Thus the probable range of five-day mean movements could be calculated from the published five-day mean ambient temperature. The impetus for this investigation came from the trend toward the construction of more bridges with longer spans (100 to 200 ft) which increased the problems with joints and bearings due to the greater movements experienced by these bridges. The effectiveness of various joints and bearings in accommodating the thermal movements was discussed. However, the authors did not, at this stage in their survey, make any specific recommendations as to the most suitable joints and bearings to use under specific circumstances.

Berks (8) found that improvements in technology in Britain allowed for more efficient use of materials in bridges, resulting in lighter weight structures with longer spans. The longer spans lead to proportionately larger movements at the expansion joints. Thermal movement was found to be the primary one to consider. Annual movement cycles were 1/2 in. and 3/4 in./100 ft of span for concrete and steel bridges, respectively.

A theoretical study of thermal stresses in a composite steel and concrete section was published by Zuk (72). An elastic theory was developed for determining the stresses and strains due to various linear thermal gradient patterns. In the analysis, the beam and slab were separated to determine the stresses in each, and then recombined in accordance with boundary and compatibility conditions of no slip at the interface and equal curvature of the two parts at the interface. The stresses (both axial and lateral) can be determined in any part of a composite section comprised of a concrete slab and a steel beam by the following equations:

SLAB:

$$f_{xs(y)} = F/(2ap) + 3y_s(Fa - Q)/(2a^3p) \text{ ....(5)}$$

$$f_{zs(y)} = mf_{xs(y)} - c_sE_sT_{(y)} \text{ ....(6)}$$

where 2a = slab thickness, in in.; c = coefficient of thermal expansion, in in./in./°F; E = modulus of elasticity, in psi; F = interface shear, in lb; f_{xs} = longitudinal slab stress (+f = tension), in psi; f_{zs} = transverse slab stress, in psi; m = Poisson's ratio; p = slab width, in in.; Q = interface couple, in in.-lb; T = temperature change, in °F; y_s = distance measured from the mid-thickness of the slab (+ down), in in.

BEAM:

$$f_{xb(y)} = -F/A + y_b(-Fd_1 - Q)/I \text{ ....(7)}$$

where A = area of beam, in sq in.; d_1 = distance from centroidal axis to the top interface, in in.; f_{xb} = longitudinal stress in beam, in psi; I = moment of inertia about centroidal axis of beam, in in.^{4}; y_b = distance measured from the mid-thickness of the slab (+ down), in in. Some values calculated with these equations, such as longitudinal stresses in the beam of up to 24,000 psi, exceeded the limits of standard specifications. These theoretical values would, of course, be modified somewhat by the actions of creep, slip, and local plastic yielding which are neglected in the theoretical development.

Liu and Zuk (40) extended Zuk's earlier work (72) with composite sections to prestressed-concrete sections. Interface shears and moments were calculated using the more complex equations developed for four different cases of prestressed members. Shears and moments up to

30,000 lb and 120,000 in.-lb were calculated with a 25°F temperature differential through the prestressed slab and beam. The calculated concrete stresses were within allowable levels. However, stresses in the slab in a direction transverse to the beams varied from 1000 psi in tension to 800 psi in compression; they could cause cracking in the slab if only the minimum reinforcement required by AASHO was used in that direction.

Berwanger (9) developed equations which more fully considered the factors affecting thermal stresses found in composite reinforced concrete-slab and steel-beam bridges. In calculating the induced thermal stresses, he took into account the temperature differentials throughout the depth of the bridge and the difference between the coefficients of thermal expansion of the steel beams and concrete slab, as well as the different coefficients of thermal expansion of the reinforcing steel and the concrete slab.

Houk (33) conducted tests on small unreinforced beams to study the effects of volume changes thermally induced in massive concrete structures at locations of high restraint. Tensile and compressive strains were measured at the outer fibers of plain concrete beams. He found the magnitudes of these thermal stress-strain values to be significant near test failure. Strains developed by the temperature changes would approach those measured at failure of the beam in loading. Expansion joints which do not provide sufficient room for expansion can be locations of high restraint similar to the conditions studied by Houk.

Steward (54) studied the movements at 231 expansion joints in 80 bridges throughout California during a three-year period. The use of expansion joints at the abutments was found to be very significant to the total bridge movement. Increases in the movement per unit length for structures with expansion joints at the abutments varied from 31 percent in the valley areas of California to 58 percent in the mountains (California uses the movement per unit length criteria in designing for thermal movement of highway bridges). In comparing actual movements with the design values, it was found that several concrete structures experienced both greater movements and smaller temperature ranges than specified in design. The type of expansion bearing used had no significant effect on the thermal joint movements. Thermal coefficients of expansion of $5.3 \times 10^{-6}/°F$ and $6.5 \times 10^{-6}/°F$ were calculated for concrete and steel, respectively, for the structures with the greatest movements. The values recorded on other structures would have approached these values if their movements had not been restricted by joints filled with debris or constructed too narrow to allow free movement of the structure. The thermal movement of box girder structures was less than that of other concrete structures due to the insulating effects of the air trapped inside the box section. Movements at uniformly spaced expansion joints on long structures were not necessarily the same. Design values for movement of concrete structures under various climatic conditions were also given.

Zuk (77) presented a simple empirical formula intended for use as a design check of thermal stresses in simply supported composite highway bridges. Based on a series of field tests on various bridges in the 50 to 70 ft range, the formula relates the thermal stresses at the bottom of the girder to the temperature difference between the top and bottom of the slab and the depth of the bridge as follows:

$$f_b = 2500T_s/h \qquad (8)$$

in which f_b = the thermal stress in the bottom flange of the girder (+ equals tension), in psi; T_s = the temperature difference between the top and bottom of the slab (top temperature less the bottom temperature), in °F; h = total depth of the bridge, in inches.

It was recommended that this equation should be verified for different regions of the country to determine what variations might be needed for different geographic areas.

End movements of four bridges were studied by Zuk (79,80), each for a one-year period, to accumulate data for use in possible correlation of air temperatures and bridge movements. The data showed that there was a general relationship between ambient temperatures and the end movements of the bridges, but that other factors such as creep, shrinkage, restraints caused by abutments and bearings, and loading all affect bridge movements. A summary of the apparent coefficients of expansion for the bridges, top and bottom, for both winter and summer and the yearly extremes was also presented. It was noted that the values are widely scattered. No specific reason for this was given but possible variables were listed, such as creep, shrinkage, volume change due to moisture, and dead and live loads. As a guide for design, Zuk recommended that the magnitude of end movements be assumed as double that which would be due to temperature alone.

Three types of investigations were conducted by Wah and Kirksey (60): theoretical; experimental laboratory model; and field tests on a newly constructed bridge (De Zavala overpass, San Antonio, Texas). A series of equations were developed to calculate the thermal effects on a beam-slab bridge. One set of equations was related to the thermal expansion of beam-slab bridges, and another set was related to the flexure of beam-slab bridges.

The equations developed for deflection of the slab and beam were similar to those for deflection due to lateral loading except that the thermal loads may vary throughout the thickness of the beam and slab as well as on the surface. The generalized equations may be used for any temperature variation. A computer program was developed for use in the solution of the series of equations.

An 8-ft by 6-ft by 7-in. thick slab model was constructed to approximate both the mathematical model and the highway bridge slab to be used in the field tests. However, due to its stiffness, the resultant deflections were too small, even for large thermal gradients. The model was abandoned and replaced by an 8-ft by 6-ft by 3-in. thick concrete slab with two layers of welded steel fabric serving as reinforcement--one near the top surface and one near the bottom. The model was heated by banks of infrared bulbs.

Creep effects were found to have a great influence on the behavior of the heated slab. The slab arched up when the heat was first applied due to the top heating up more quickly than the bottom of the slab. However, as time progressed the upward deflection decreased, and in some cases actually reversed, even though the temperature difference between the top and the bottom of the slab remained nearly the same. The creep deflections tended to offset thermal deflections when the top of the slab was warmer than the bottom. The thermal stresses calculated from strains measured in the model were quite high but not necessarily accurate due to the unknown effects of creep.

The De Zavala Overpass is a 46-ft span, simply supported bridge with 14 pan-type reinforced concrete beams as supporting girders. During its construction 377 thermocouples and 14 concrete embedment gages were installed in the bridge. Temperatures were recorded hourly for two days in August and one 22-hour period during a storm in December, 1967. Stresses, computed from the measured strains, were shown for the two days in August. Creep would not be as significant a factor in these stresses as for the model because lower temperature differentials existed for the field tests. There was a significant difference between the measured bridge deflections and the deflections calculated from the mathematical model. This reportedly was due primarily to the deviation of the bridge from the theoretical model and the presence of different boundary conditions.

Tensile stresses up to 1500 psi were measured in the concrete at the top of the slab. These stresses would be offset by the compressive stresses ordinarily found in the top of the slab due to dead and live loads. The tensile stresses in the bottom of the slab ranged up to 950 psi. These stresses would be additive with those of dead and live loads.

The results indicated that an attempt to completely explain the thermal behavior of bridges would require the solution of several sets of lengthy equations. Creep, if included, would tend to further increase the complexity of the equations.

CURRENT SPECIFICATIONS

The 1969 AASHO Standard Specifications for Highway Bridges governs the design of highway bridges in the United States. Two sections of the code are especially pertinent to this study (4, p. 25, 56).

1.2.15--THERMAL FORCES

> Provision shall be made for stresses or movements resulting from variations in temperature. The rise and fall in temperature shall be fixed for the locality in which the structure is to be constructed and shall be figured from an assumed temperature at the time of erection. Due consideration shall be given to the lag between air temperature and the interior temperature of massive concrete members or structures.
>
> The range of temperature shall generally be as follows:
>
> Metal Structures
>
> Moderate climate, from 0 to 120 F.
> Cold climate, from -30 to 120 F.

Concrete Structures	Temperature rise	Temperature fall
Moderate climate ...	30 F.	40 F.
Cold climate	35 F.	45 F.

1.5.4--EXPANSION

> In general, provision for temperature changes shall be made in all simple spans having a clear length in excess of 40 feet.
>
> In continuous bridges, provision shall be made in the design to resist thermal stresses induced or means shall be provided for movement caused by temperature changes.
>
> Expansion not otherwise provided for shall be provided by means of hinged columns, rockers, sliding plates or other devices.

A minimum amount of reinforcement must be provided in all directions to resist the formation of temperature and shrinkage cracks. An allowance of 1-1/4 in. per 100 ft of steel bridge is to be provided for thermal movements. There is no set provision for movement in concrete bridges, perhaps because the concrete does not react as quickly to the heating and cooling effects of weather changes. The larger mass and higher specific heat of a concrete bridge as compared to a steel bridge cause it to be less affected by short-term temperature extremes (27). Temperature variations throughout the depth of the structure are not mentioned as a condition to be investigated during design.

The German Standards (November, 1967) provide for an assumed bridge erection temperature of 50°F (35). Normal temperature variation for composite steel and concrete bridges is ± 54°F. This is a reduction from the 1963 standards which provided for a 58.5°F temperature variance. A variance of ± 54°F is to be used in calculating the thermal movements at bearings and expansion joints for reinforced-concrete or prestressed-concrete bridges. A non-uniform temperature of ± 27°F for steel and ± 9°F for concrete bridges is to be considered in design. A 9°F reduction is allowed if the dimensions of the concrete members exceed a thickness of 27.6 in., or are insulated in some manner.

A survey of United States and foreign bridge codes by Zuk (79) disclosed that only the 1965 (now 1969) AASHO specifications make a specific statement concerning actual movement (1-1/4 in. per 100 ft for steel bridges). Others, of course, have provisions which refer indirectly to the need to consider the effects of temperature.

In another discussion of code provisions for thermal effects, Zuk (77, p. 10) stated:

> Germany's code (DIN 1078, 1958) for temperature effects in composite construction is as follows:
>
> A. Indeterminate structure: a straight line variation of ± 15°C shall be assumed between the top of concrete slab and bottom of steel girder.

B. Statically determinate structures: the thermal effect shall be allowed for by an additional shrinkage of 10 x 10^{-5}. These stresses shall be combined with live load stresses as follows:
(a) Full live load + half temperature difference.
(b) Full temperature variation and live load reduced by 1% per meter of span to 40 meters span, then constant 40% reduction.

C. Shearing forces due to temperature difference may be distributed as a triangular shearing force diagram at the end of the girder with a length equal to the effective slab width. (In association with this shearing force the code also specifies the use of heavy end anchorages tieing the slab and beam together at their interface.)

Austria, Sweden, and Japan are the only other countries with a thermal stress provision in their codes, and these codes are all essentially based on the German code. The United States has no direct provision, although Section 1.2.15 of the 1961 AASHO Standard Specifications for Highway Bridges states: "Provision shall be made for stresses or movements resulting from variations in temperature."

EXPANSION JOINTS

Close attention must be paid to the proper design of expansion joints if they are to allow for the smooth passage of vehicles, while not restricting the free movement of the bridge. The joints are intended to reduce restraints on the structure which would otherwise be present due to differences in the deformation of the overall system of the structure. Koster (35) separated expansion joints into five categories: 1) simple joints with straight edges to the gap; 2) toothed joints with interlocking fingers; 3) simple sliding plate joints, which span and cover the gap; 4) elaborate sliding plate joints which have additional devices to help span the gap resulting from larger displacements; and 5) composite expansion joint systems, which have a deformable section, such as neoprene, spanning the gap. Gortz, Agnew, and Palmer (30) have developed joints for large (up to 73 in.) small (under 2 in.), and medium (1/2 to 10 in.) movement ranges. Berks has developed four joints for various ranges of movement based on the following design criteria (8, p. 439):

1) Conformity to the...[standard codes].
2) A life equal to that of the bridge structure.
3) Ease of installation.
4) Ease of access for inspection and maintenance.
5) Watertightness [or provision for directing water and grit away].
6) Ability to resist loading and to transfer loads so that they are absorbed without overstressing the assembly or causing damage to the structure of the bridge.
7) No restriction on the movement of the bridge due to temperature variation, creep or shrinkage.
8) Noise and vibration free surfaces.
9) A high inbuilt safety factor.
10) The incorporation of a seal which is durable, resists puncture and abrasion and performs adequately at all temperatures.

From a survey of deck expansion joints in Britain, Kerensky (34) concluded that edge plates at expansion joints must be solidly bedded and pulled down with considerable force (12 tons/ft run) throughout their length. The French recommended that the bolts holding down the sealing devices be prestressed to seven tons per foot of joint (62).

Lally and Milek (39) reported that costs of expansion joints increase rapidly as they become more complex. For example, using the cost of a simple armored expansion joint as a base, sliding plate expansion joints are twice as expensive, while finger expansion joints are ten times as expensive.

Some design engineers eliminate expansion joints entirely by anchoring the slab to flexible stub abutments and piers. This has become generally accepted and is being used for both steel and concrete superstructures, with lengths of 400 ft and greater (23). However, opinions vary as to how to determine and provide for the stresses induced and as to whether or not an allowance for such stresses is necessary.

JOINT SEALANTS

The task of today's joint sealants has been compared to that of last century's roofs on covered bridges (16). Bridge joint sealants are needed to protect the various structural elements such as bearings, structural steel, and supporting concrete, from water, grit, and deicing chemicals.

Joint sealants were examined from a structural approach by Dreher (22). Rather than merely using the slot as a mold for the joint sealer, his objective was to form a bridge between the two elements of the deck. Hinged levers, arched springs, and other more elaborate devices were considered. All of these types would be in compression, and thus would not rely on a bond with the sides of the gap to maintain a good seal.

Cook (21) found that the properties of high bond strength, extensibility, and excellent short-term memory possessed by polysulfide sealants made them very desirable for use. Additional study is required to determine the effects of work hardening, aging, and weathering on the sealant.

Tons (57) listed five factors which require consideration in joint sealant design: 1) characteristics of the joint, including type of pavement material; 2) properties of sealant to be used; 3) properties and condition of the sealant-joint interface; 4) quality of workmanship; and 5) type of service to which the sealed joint is to be subjected.

Preformed elastomeric joint seals were found to be the most effective by Gunderson (31). Movement ratings for various preformed elastomeric joint seals were given. A specification for evaluating the quality of elastomeric joint sealers for bridges was proposed by Kozlov (37).

One recent system is a modular joint sealing system which is prefabricated at the factory and placed in the deck before the concrete is placed (64,65). Movements of up to 18 inches have been accommodated to date by this system.

Criteria for evaluating the sealing performance capability of any given material were developed by Watson (62). Such a material must: 1) be capable of responding to many types of movement; 2) seal out foreign incompressibles and harsh chemicals; 3) seal out free water and assist in moving it to the proper channels; 4) be capable of absorbing various types and ranges of movement within itself without being extruded from the joint opening; 5) survive the wear and impact of traffic, maintenance vehicles, and climatic conditions; 6) perform in temperature extremes; 7) have a long service life, at least equal to that of the deck surfacing; 8) be accessible for inspection and maintenance, and provide for adjustments required by changes in deck length, pavement pressures, or abutment settlements; 9) provide good riding qualities, skid resistance if wider than eight inches, and structural support for traffic if gap is wider than four inches. The sealing system should be equally effective at the juncture of curb and pavement. Photographs illustrating the problems that result from poor joint sealing were shown (61,62). Such problems as salt brine deterioration of the concrete, staining of the concrete, accumulation of incompressible materials in the joints leading to crushing of the concrete, and corrosion of the reinforcing bars occur regularly at bridges with poor joint sealants.

BRIDGE BEARINGS

Fairbanks (28, p. 73) defined a bridge bearing as: "a seat for a girder or truss that provides the necessary reaction to prevent vertical movements of the structure through a uniform transfer of load to the foundation."

Bridge bearings should: 1) give adequate support; 2) permit movement and rotation only in the direction and magnitudes designed for originally; 3) have a consistent performance with age; and 4) be either easily replaceable or have a life equal to that of the structure (13). Four variables affecting the designer's choice of bearings are: 1) cost; 2) loading; 3) available headroom; and 4) resistance to movement required of the bearing.

Early bearings were simply bronze or cast iron sliding plates which were generally unsatisfactory due to high initial friction coefficients and the difficulty in maintaining proper lubrication. Roller bearings and rockers were later developments, but the need to provide for movement and rotation in both the longitudinal and transverse directions led them to become bulky and unwielding. Nevin (46) suggested two solutions to this problem: rubber bearings for structures with small loads and movements, and the sliding plate technique for structures with movements greater than those which a rubber bearing can accommodate. Polytetrafluoroethylene (PTFE), commonly known as Teflon, has a low friction coefficient and a complete absence of the stick-slip property which reduced the value of previous sliding plates. Kerensky (34) found that coefficients of friction of two percent or less could be expected with Teflon coated plates under suitable conditions. Unlike most materials, Teflon materials experience a reduced coefficient of friction with an increase in load (24).

Elastomeric bearings can absorb individual vertical, horizontal, and rotational movements, as well as any combination of the three (52). They can be designed to resist large vertical loads with little deformation, and yet, be soft enough to allow horizontal thermal movements (5). These bearings have proven satisfactory and economical for use with concrete and steel bridges (28). Their low cost, freedom from maintenance, and low profile has led to their wide use as bridge bearings (20). In a theoretical analysis, Zuk calculated that the dynamic stresses in a bridge supported on elastomeric bearings would be significantly lower (15%) than for conventional rigid bearings (73). The study indicated that the use of elastomeric bearings might provide an additional savings over the cost difference between them and metal bearings, in that less increase in design strength of the structure need be made for impact. McKeel and Kinnier (43) found from field tests that little advantage of reduced stresses would be gained from the use of elastomeric bearings under normal loading conditions and compared these results with both the theoretical study of Zuk (73) and the experimental model study of Emanuel and Ekberg (24).

Pare (49, p. 39) reported that: "neoprene bearings have performed extremely well over the past ten years [1957-1967] and all installations built during this period appear to be continuing their load carrying function with little or no deterioration. Elastomer bearings can be designed for any degree of vertical or horizontal restraint desired.

The flexibility of design and material is unique in the opportunity it affords the designer in controlling the load distribution to the substructure."

FUTURE NEEDS

There is a great need for a simple, rational approach to the problem of determining the effects of environmental changes on a structure. Additional studies seeking to relate air temperatures with bridge temperatures are needed. Further research is needed to determine actual bridge temperature distributions in order to predict the thermal stresses more reliably. The relative magnitudes of stresses caused by factors other than temperature, such as creep, shrinkage and humidity needs to be examined.

The magnitude of stresses induced in a bridge whose superstructure is tied to flexible stub abutments and piers should be determined so they can be included in the design. Continuing field observations will allow the design engineer to evaluate both the benefits and liabilities of using flexible substructures to replace more conventional expansion and bearing devices. There is some hesitancy among bridge design engineers to utilize elastomeric bearing pads due to the lack of long-term testing. The ten-year study reported herein regarding the successful use of elastomeric bearings would be a good base upon which to add further reports of the behavior of such bearings.

SUMMARY

There is a continuous need for the development of semi-empirical formulas suitable for use by design engineers in dealing with thermal effects on bridges. Previous investigations have sought to somehow relate environmental conditions to bridge temperatures and thus to stresses and movements. Bridge movements have been related to air temperature data published by the weather bureau. Relatively good results have been obtained in predicting maximum surface temperatures of the bridge deck and maximum temperature differentials between the top and bottom of the bridge based on weather data. Test results and the equations presented indicated that surface temperatures of asphalt-surfaced bridge decks would be as much as 15°F warmer than normal grey-white concrete decks when exposed to the summer sun. While temperature differentials through the bridge depth as great as 42°F were measured, in general, temperature differentials in the range of 20 to 30°F can be expected.

It is difficult to separate stresses thermally induced in a material from those stresses caused by creep, shrinkage, and plastic yielding. Interface slip in composite members also alters theoretical stress patterns. The large number of variables involved make any attempt to compare results between different bridges very difficult.

Studies of air temperatures and other environmental factors as they affect bridge temperatures indicate that the task of predicting the actual temperature distribution in a bridge is extremely complex. A uniform temperature distribution, used by some researchers, will

occur only under special conditions, such as during the winter when clouds screen the bridge deck from the sun's effects. The use of a mean bridge temperature neglects the bending stresses developed when the warmer parts of the bridge expand more than the cooler parts. A more accurate view of the stress levels in a bridge is found by using a non-linear temperature distribution through the bridge depth. Composite steel and concrete bridges usually have a non-uniform temperature distribution through the concrete slab, while the steel beam is at the temperature of the bottom of the slab. A linear temperature distribution through the top slab is often assumed for concrete box sections; the sides and bottom of the box have been found to have approximately constant temperature.

The cost of expansion devices rises rapidly as they become more complex. Use of flexible stub abutments and piers eliminates these costs. Also it removes the possibility that failure of the expansive devices to work properly will cause damage to the structure.

APPENDIX I.-BIBLIOGRAPHY

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APPENDIX II.-NOTATION

The following symbols are used in this paper:

A = area of beam, sq in.;

$2a$ = slab thickness, in in.;

c = coefficient of thermal expansion, in in./in./°F;

d_1 = distance from centroidal axis to the top interface, in in.;

E = modulus of elasticity, in psi;

F = interface shear, in lb;

f_b = thermal stress in bottom flange of the girder, in psi;

f_{xb} = longitudinal stress in beam (+f = tension), in psi;

f_{xs} = longitudinal stress in slab, in psi;

f_{zs} = transverse stress in slab, in psi;

h = total depth of bridge, in in.;

I = moment of inertia, in in.^{4};

L = solar radiation received on a horizontal surface, in g-cal./cm^2/day;

m = Poisson's ratio;

p = slab width, in in.;

Q = interface couple, in in.-lb;

T = temperature change, in °F;

T_a = average daily temperature, in °F;

T_m = maximum surface temperature, in °F;

T_r = daily range in temperature, in °F;

T_s = temperature difference between the top and bottom of the slab (top temperature less the bottom temperature), in °F;

t = time, in hours;

y_b = distance measured from centroidal axis of beam (+ up), in in.;

y_s = distance measured from the mid-thickness the slab (+ down), in in.;

θ_a = air temperature, in °C;

θ_c = concrete temperature, in °C; and

λ = lag factor.

VITA

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