Thermal Effects in Concrete Bridge Superstructures

ROY A. IMBSEN and DAVID E. VANDERSHAF

ABSTRACT

The findings of a current research project entitled Thermal Effects in Concrete Bridge Superstructures are highlighted. The research project is sponsored by NCHRP. A brief discussion of the mechanisms of heat transfer as they relate to bridge structures is presented, including the thermal coefficients of various types of different concrete composed of different aggregates. Also included are brief discussions on mean effective bridge temperatures, temperature differentials, and the response analysis for nonlinear temperature gradients. Two case studies of longitudinal thermal response selected from those conducted in the research project are included in this paper. The thermal gradients used in New Zealand, England, and Ontario, Canada, plus those recommended by the Post-Tensioning Institute, are included in the response analysis. In addition, specific recommendations for improving the U.S. design provisions for thermal effects are included.

Traditionally, bridges have been designed to resist only the overall longitudinal movement arising from temperature strain. However, with the recent changes in bridge types, it has become apparent that temperature differentials also exist in bridge superstructures. These temperature differentials cause stresses that should be included in the design procedures. Although the current AASHTO specifications include probable temperature ranges of mean temperature conditions that affect expansion and contraction of concrete bridge superstructures, there is no recommendation for temperature differentials that may occur in individual superstructure sections.

The purpose of this paper is to highlight some of the findings in a current research project, Thermal Effects in Concrete Bridge Superstructures (1), which reviews and evaluates the various procedures that have been proposed for use in considering thermal effects on bridge superstructures. The ultimate goal of the research is to upgrade the current AASHTO code for thermal effects.

There are only a few published accounts of concrete bridges damaged by differential temperature effects. In 1981 Zichner (2) described the fundamentals for determining temperature effects in concrete bridges and indicated that cracks such as those shown in Figure 1 were observed during thorough inspections of several different bridges. The cracks, located in the bottom slabs and girder stems of these box-girder bridges, resulted in part from temperature differences that existed within the individual bridge superstructures.

Recently in Colorado, cracking was experienced in the webs and bottom deck soffits of four cantilever, segmental, prestressed bridges. Two of the bridges are approximately 747 ft long, the third is about 516 ft long, and the fourth is about 449 ft long. The three longer bridges have four spans, whereas the shorter one has three. The three-span bridge, shown schematically in Figure 2, exhibited the greatest amount of cracking (3). The crack patterns on the single-cell bottom deck soffit and webs are also shown in Figure 2. The largest crack width reported is approximately 0.13 in.

HEAT TRANSFER BY RADIATION

Heat transfer by radiation is generally considered to be the most important contribution of heat energy exchange on concrete bridge superstructures. During the daylight hours when the structure is exposed to the sun, especially during the warm summer months, a net gain of heat energy occurs throughout the depth of the structure, primarily as a result of solar radiation impinging on the surfaces of the structure. Conversely, primarily as a result of reradiation to the surrounding environment of the heat energy stored in the structure, a net loss of heat energy occurs during the night. During the summer the temperature in the top surface of the bridge deck is warmer than the soffit, which results in a positive gradient. A negative gradient develops on typical winter nights when the top surface is cooler than the soffit.

THERMAL PROPERTIES OF CONCRETE

The thermal coefficient of expansion for concrete is greatly dependent on its aggregate type and mix proportions (4-7). The cement paste of normal concrete usually has a higher thermal coefficient of expansion than the aggregate in the mix; however, because the aggregate occupies about 75 percent of the volume, it is the thermal expansion characteristics of the aggregate that dictate the anticipated volumetric change during a given temperature change.

Most codes specify an average thermal coefficient of 0.000011 to 0.000012 per degree Celsius (about 0.000006 per degree Fahrenheit) for reinforced concrete. The actual coefficients from laboratory tests on concrete samples (Table 1) vary by as much as 22 percent above and 64 percent below the higher value, depending on the aggregate type.

MEAN TEMPERATURES

Mean or effective bridge temperatures are associated with the long-term (seasonal) movements of a bridge. Bridge codes typically provide detailed guidelines for the calculation of overall longitudinal movements by specifying a range of temperatures that depend on the geographical location of the bridge and the structure type. The specified range of effective temperatures represents the average range to be considered in design. At times, a given range of effective temperatures may have to be adjusted to compensate for unusual conditions, such as frost pockets or sheltered, low-lying areas.

Emerson (8) defines the effective temperature of a bridge as that temperature that governs the longitudinal movement of the bridge deck. The effective temperature may be derived by performing a calculation that includes both the product of the areas between isotherms and their mean temperatures divided by the total area of cross section of the deck. Emerson (9) and Black et al. (10) have correlated the extreme values of effective bridge temperatures

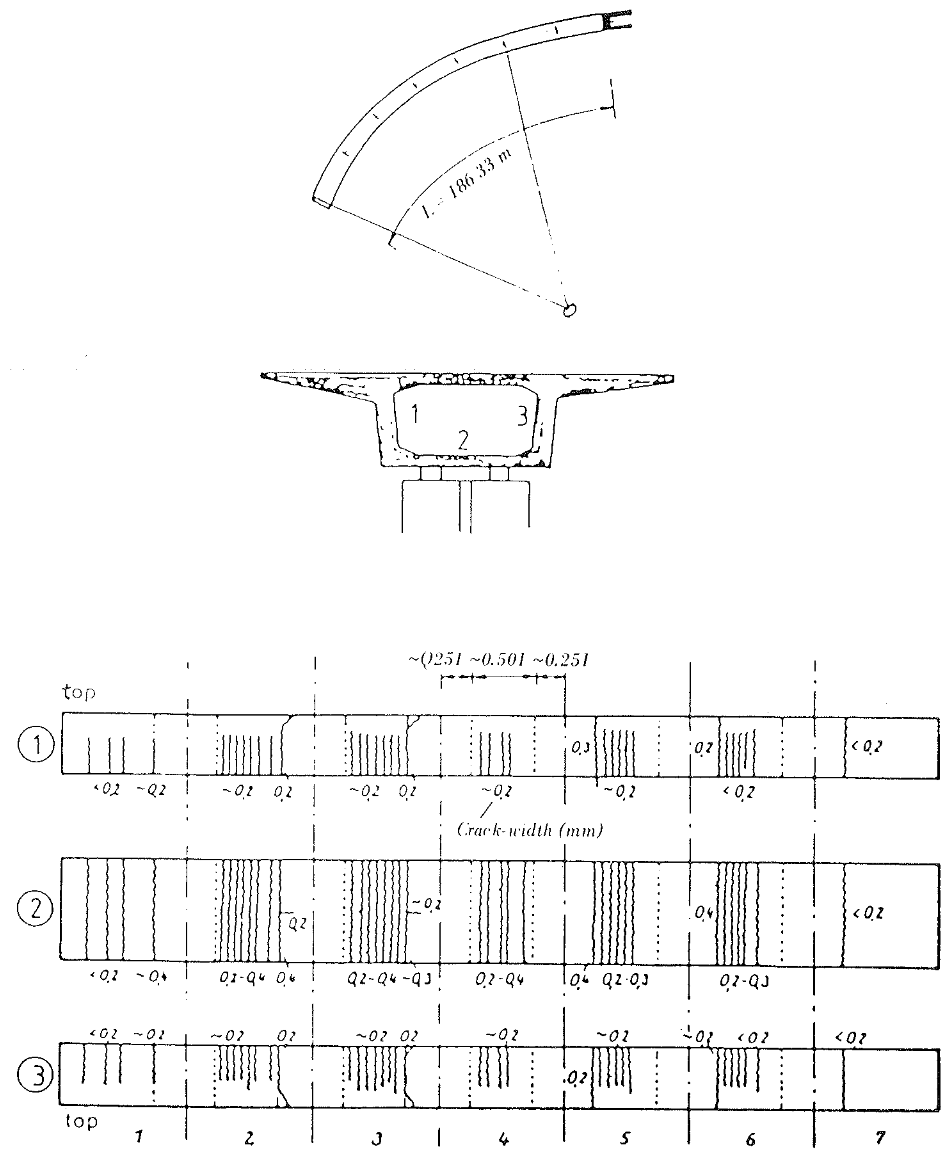


FIGURE 1 Cracks in a multispan box-girder bridge.

with shade temperatures. Emerson correlated shade temperatures with the temperatures obtained from structures instrumented with thermocouples, whereas Black et al. correlated shade temperatures with bridge movements to obtain the extreme values of effective bridge temperatures.

TEMPERATURE DIFFERENTIALS

Because of the unsatisfactory thermal conductivity of concrete, the diurnal temperature effects on a concrete bridge superstructure usually produce temperature gradients. Large, positive temperature gradients occur during days with high solar radiation, clear skies, a large range of ambient temperatures, and a light wind. Negative temperature gradients develop during periods associated with night and winter conditions. The temperature gradients that form in a given structure are governed by heat flow through the body and are a function of the density, specific heat, and thermal conductivity of the concrete.

One-dimensional heat flow in the vertical direction is generally considered to be sufficiently accurate to conduct most analyses of bridge superstructures. Researchers have conducted both analytical and experimental studies to determine temperature differentials that occur in bridge superstructures. The research efforts that led to the development of the codes in New Zealand, England, and Ontario, Canada, are briefly discussed.

New Zealand

Priestley (11,12) analyzed the effects of several assumed thermal gradients and compared the results with measured data available at the time. One of the assumed thermal gradients consisted of a linear temperature distribution through the top deck slab, as proposed by Maher (13), and which was supported by measured temperatures from three bridges located in the British Isles (14,15). Other assumed thermal gradients included the temperature distribution proposed by the Ministry of Works of New Zealand, and distributions in which temperatures vary with depth

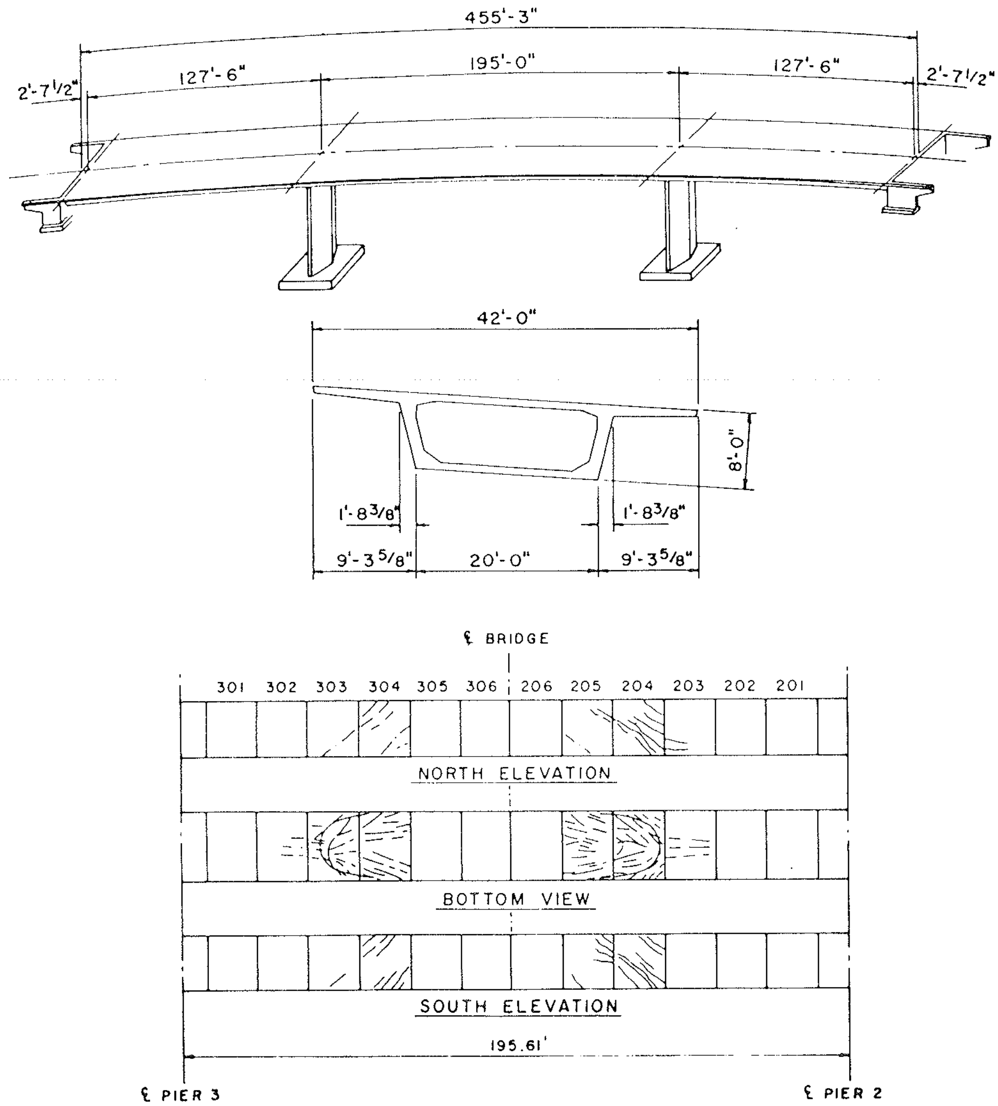


FIGURE 2 Miller Creek Bridge in Colorado.

TABLE 1 Thermal Coefficient of Concrete (-0.000001 per degree Celsius)

Aggregate Type	PCA (6)	Emerson	Browne (5)	Ontario
Quartzite		12.7	11.7-14.6	12.8
Quartz	11.9		9.0-13.2	
Sandstone	11.7	11.7	9.2-13.3	11.7
Gravel	10.8	13.2	9.0-13.7	13.1
Granite	9.5	9.6	8.1-10.3	9.5
Dolerite		9.6		9.5
Basalt	8.6		7.9-10.4	
Marble			4.4- 7.4	
Limestone	6.8	7.3	4.3-10.3	7.4

as second-degree, fourth-degree, and sixth-degree parabolas. The sixth-degree parabola was found to be in satisfactory agreement with measured data, and its use was recommended for superstructure depths between 1200 and 1500 mm (47 and 59 in.).

In 1976 Priestley (15,16) proposed a revised temperature distribution that consisted of three individual parts, as shown in Figure 3. In the first part, temperatures are assumed to decrease nonlinearly from a maximum at the top surface of the deck slab to a minimum at a depth of 1200 mm. The nonlinear variation is represented by a fifth-degree parabola. The second part of the revised distribution applies only to a deck slab over an enclosed cell of a box girder, in which case temperatures are assumed to decrease linearly. The third and final part of the revised distribution assumes a linear variation of temperatures over the bottom 200 mm (8 in.) of the cross section.

Priestley also found that the maximum temperature at the top of the concrete deck slab would decrease linearly with the thickness of bituminous overlay because of the insulating properties of this material.

Great Britain

In 1973 Emerson (8,17) described a method for calculating the one-dimensional heat flow within a concrete-slab bridge by using an iterative, finite-difference solution scheme. The method relates the bridge temperature to solar radiation, ambient air temperature, and wind speed. The model of the structure in this case is composed of several layers, and a starting time is assumed, at which point

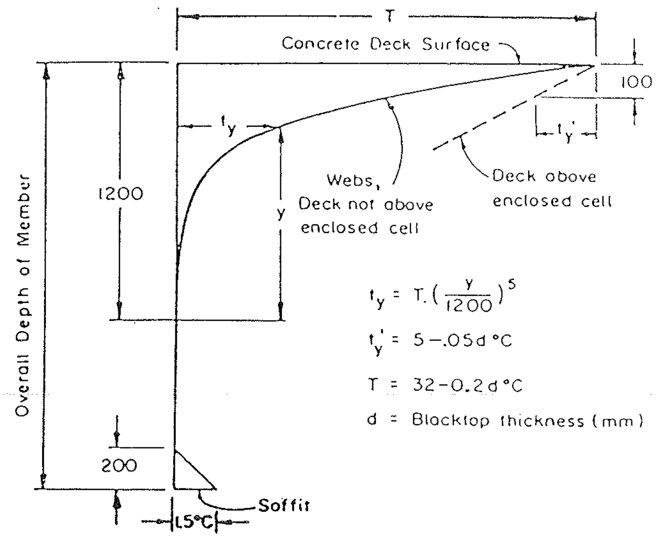


FIGURE 3 Temperature difference from the New Zealand design brief.

the equations governing the boundary conditions are applied.

The assumption of boundary conditions requires an estimation of the times at which the nonlinear differential distribution is at a minimum. It is further assumed that the temperature throughout the structure is a constant at this time. Emerson (8) estimated that the beginning time for concrete bridges was 0800 hours for the heating phase and 1600 hours for the cooling phase. By using these input parameters, a nonlinear differential temperature distribution was computed at 15-min intervals until a maximum gradient was reached at approximately 1500 hours. Temperatures predicted by the model correlated well with measured prototype summer and winter temperature distributions. The current British Standard BS 5400 (18) has been updated to include the temperature distributions determined in these research efforts, as shown in Figure 4.

Ontario

In 1975 Radolli and Green (19) developed a one-dimensional heat-flow analysis similar to the one used by Emerson (8). Although acknowledging that the assumption of one-dimensional heat flow is not technically correct, they cited comparisons that indicated that satisfactory correlations exist between observed and predicted temperature gradients obtained from a one-dimensional heat-flow analysis. They were able to use this approach to develop simplified formulas for use in design.

Comparisons between the British standards (18), Maher (13), the New Zealand Ministry of Works (20), and Priestley's sixth-degree parabola to an I-girder indicate that the resulting stresses are strongly dependent on the temperature differences and temperature gradients. Comparisons between the gradients proposed by Leonhardt et al. (21), Priestley, Maher, and the one-dimensional heat flow were presented for varying superstructure depths. The results were decomposed into continuity and self-equilibrating stresses. Radolli and Green proposed that simple design formulas be used for designs that do not require an understanding of the temperature gradient.

RESPONSE ANALYSIS

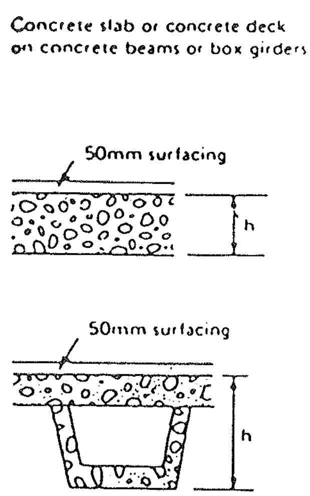
Having selected a given temperature gradient or loading, the bridge designer is next faced with performing the response analysis. There are several ways to accomplish this; the two most useful methods follow.

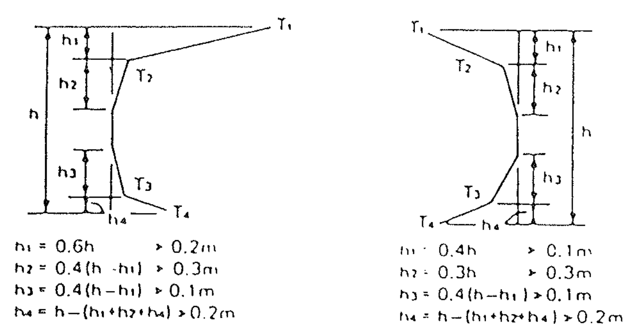
General Method

The general method procedure, shown in Figure 5a, involves separating the arbitrary gradient into three components: axial, bending, and residual. It is first necessary to solve these three problems, and then to superimpose the resulting three stress distributions, as shown in Figure 5b.

Equivalent Prestress Method

The equivalent prestress method procedure, which is much easier to apply than the preceding one, is based on an analogy between thermal strains and prestress strains. In this method the stresses from a completely restrained structure are superimposed with the stresses that result from the removal of the restraints, as shown in Figure 5c. Removal of the restraint is accomplished by dividing the restrained stress field into a series of equivalent negative prestress forces, as shown in Figure 5d.





h metres	T_1 (°Centigrade)	T_2	T_3	T_4
0.2	9.5	0.5	0.2	0.5
0.4	13.5	0.75	0.5	2.5
0.6	15.0	1.0	1.0	4.0
0.8	16.0	1.75	1.5	5.0
≥ 0.9	16.0	2.0	1.5	5.0

h metres	T_1 (°Centigrade)	T_2	T_3	T_4
0.2	3.5	0.5	0.25	1.5
0.4	6.5	0.75	0.5	3.5
0.6	8.5	1.0	1.5	5.0
0.8	10.0	1.5	2.5	6.5
≥ 0.9	10.5	2.0	3.5	7.0

FIGURE 4 Temperature difference from the British Code of Practice, BS 5400.

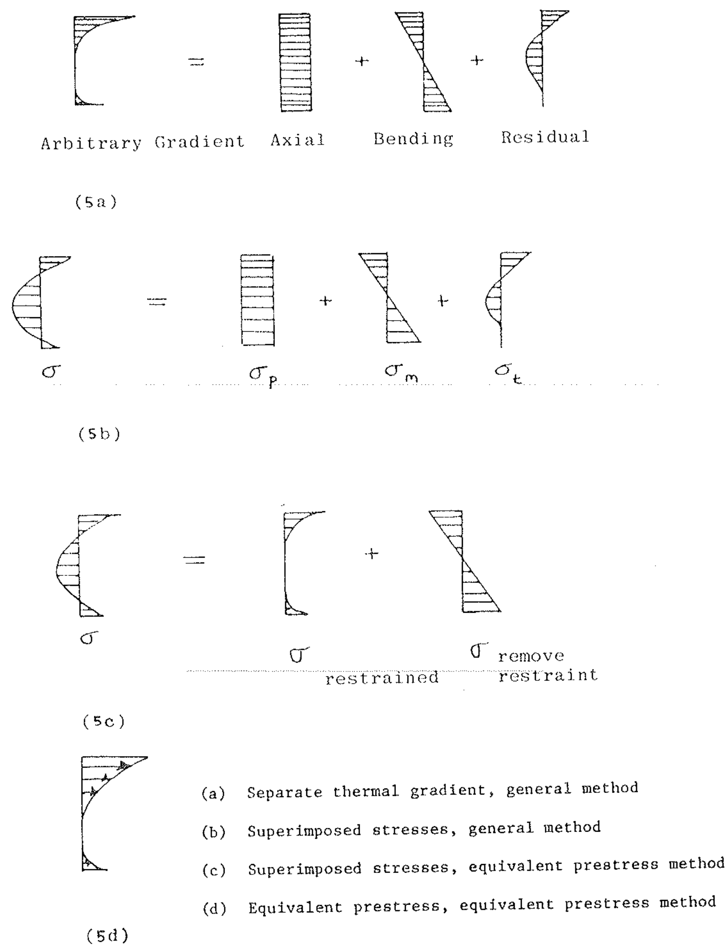


FIGURE 5 Response analyses.

Most of the case studies presented in the following sections were analyzed by this method.

ALTERNATIVE APPROACHES

The bridge design codes of different countries provide the designer with many different approaches to help account for thermal effects. The codes basically differ in the refinement used to determine the meteorological conditions at proposed bridge sites, the types of thermal loadings considered, and the methods used to accommodate these thermal loadings.

The codes in most countries are similar in that they contain provisions for some sort of thermal gradient that produces a corresponding stress distribution; this corresponding stress distribution is then grouped with other stresses (e.g., dead load, live load, prestress) so as to modify the design. This usually results in an increase in the prestress force, sometimes by a relatively large amount.

However, researchers in Switzerland and Germany are currently developing a new approach based on the concept that partial prestressing is sufficient to accommodate thermal gradient stresses and strains. This design procedure leaves the prestress force unchanged, and relies instead on mild steel reinforcement to resist the thermal stresses. This, at least in part, explains why the large number of existing prestressed concrete bridges that were designed and built without consideration given to thermal gradient effects have continued to perform satisfactorily for so many years of service.

The minimum design requirements for bridges in the United States are governed by the AASHTO design specifications. In certain cases additional, more detailed design criteria may be used. With respect to temperature, the design recommendations of the Post-Tensioning Institute (PTI) are often used in the design of prestressed-concrete bridges. Several individual states have also developed their own design procedures for considering thermal effects, which in most cases are similar to those recommended by PTI. A detailed description of the procedures, compiled from a survey of the states, is contained in the NCHRP report (1).

CASE STUDIES

Longitudinal and transverse temperature effects were applied to a selected group of U.S. bridges as part of the NCRHP research project. Four thermal gradients were used to study the longitudinal effects, whereas two thermal gradients were used to study the transverse effects.

Four thermal gradients (Figure 6) were selected for the case studies on longitudinal temperature effects. These gradients were selected because they are representative of those currently being used. They include those specified in the New Zealand, British, and Ontario codes. In addition, the gradient recommended by PTI was included because it is somewhat representative of the gradients currently used in the United States. A summary of the bridges included in the NCHRP project on case studies for

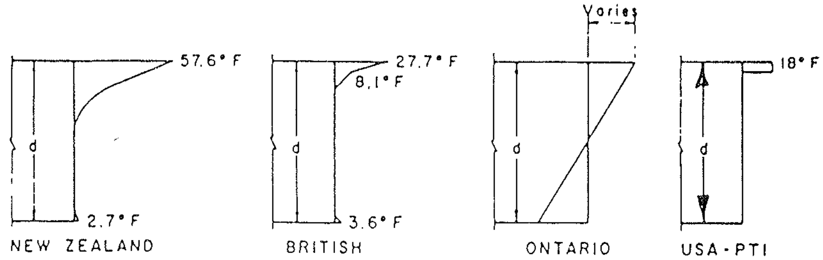


FIGURE 6 Thermal gradients used for case studies.

TABLE 2 Summary of Bridges Included in Case Studies for Longitudinal Temperature Effects

Case No.	Name and Location	Superstructure	Substructure	Length	Depth	Depth/Span Ratio	No. of Spans	No. of Hinges	No. of Frames	Comment
1L	Colorado River Bridge, California and Arizona	Precast, prestressed I-girder	Pier wall	565 ft, 6 in.	6 ft, 3 in.	0.056	5	0	1	Representative I-girder
2L	West Silver Eagle Road overhead, California	Cast-in-place pre-stressed box girder	Double column	750 ft, 0 in.	5 ft, 0 in.	0.037	6	0	1	Representative box girder
3L	Turkey Run Creek Bridge, Indiana	Precast, prestressed segmental box girder	Single column	317 ft, 0 in.	9 ft, 0 in.	0.057	2	0	1	Segmental cantilever
4L	Kishwaukee River Bridge, Illinois	Precast, prestressed segmental box girder	Single column	1,096 ft, 0 in.	11 ft, 8 in.	0.047	5	0	1	Segmental cantilever
5L	Columbia River Bridge, Washington	Cast-in-place pre-stressed segmental box girder	Single column	1,870 ft, 0 in.	9 ft, 0 in.[a] 24 ft, 0 in.[b]	0.053	5	0	1	Segmental and haunched cantilever
6L	West Silver Eagle Road overhead (falsework), California	Cast-in-place pre-stressed box girder	Double column	750 ft, 0 in.	5 ft, 0 in.	0.037	6	1	2	Falsework loading
7L	East Connector, California	Cast-in-place rein-forced box girder	Single column	1,104 ft, 0 in.	6 ft, 0 in.	0.055	11	2	3	Multiframe
8L	Miller Creek F-11-AK, Colorado	Precast, prestressed segments and box girder	Single column	445 ft, 3 in.	8 ft, 0 in.	0.041	3	0	1	Case history

[a]Minimum. [b]Maximum.

longitudinal effects is given in Table 2. The results of two of these case studies (case numbers 5L and 8L) are included in this paper. The applicable portions of the codes are those that pertain to the positive gradients that occur during the day when there is high solar radiation.

Plots of top and bottom fiber stress versus the distance longitudinally along the bridge are presented in this paper for both of the case studies. In addition, section stresses are included at selected points along the bridge to show the stress variations at different depths.

The plots show that maximum fiber stresses usually occur at the pier or column supports adjacent to the abutments. The plots also show that changes in superstructure cross sections caused by flares in the bottom slab and girder stems or in the haunched superstructure cause significant changes in fiber stresses.

The bridges analyzed were assumed to have a coefficient of thermal expansion of 0.000006 per degree Fahrenheit, uncracked section properties, and no reductions in thermal gradients for surfacing, elevation, and so forth.

Case 5L: Columbia River Bridge

The Columbia River Bridge was included in the case studies because it is a major structure and because it uses a customized and optimized cross section. Another reason for selecting this bridge is to evaluate the effect of the pronounced variation in the depth of its structure caused by the haunches at the interior supports. This bridge was designed according to the Washington State criteria on thermal effects. For longitudinal thermal effects of box girders, the criteria specify a temperature increase of 20°F in the top slab. The stresses caused by this increase in temperature are combined with dead load for a service loading. In addition, another service load condition, which results from one-half the temperature gradient (i.e., 10°F), is combined with the dead load and full live load.

This structure, shown schematically in Figure 7, is a five-span, 1,870-ft-long bridge.

Figures 8 and 9 are plots of top and bottom fiber stresses, respectively, along the bridge centerline. The Ontario code was not considered in this case because it was not apparent how this code should be applied to nonprismatic sections.

Figures 10 and 11 show plots of the variation in stresses, with section depth for the three gradients considered for 9- and 24-ft-deep sections, respectively.

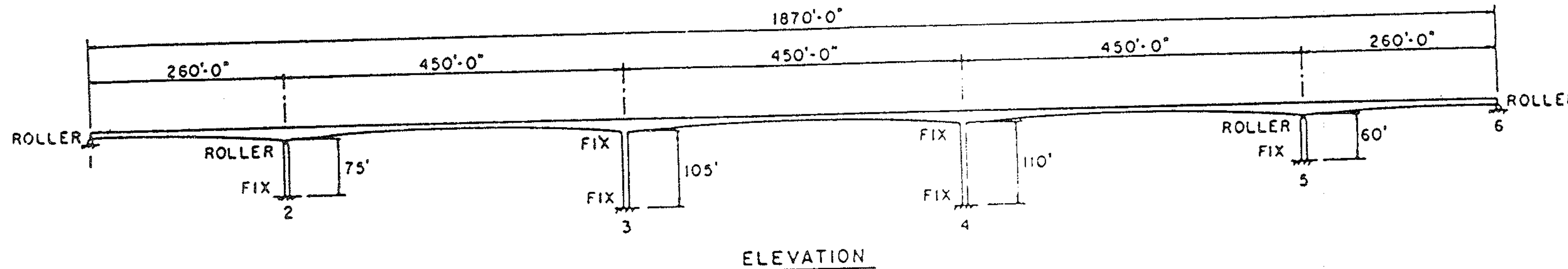
Case 8L: Miller Creek Bridge

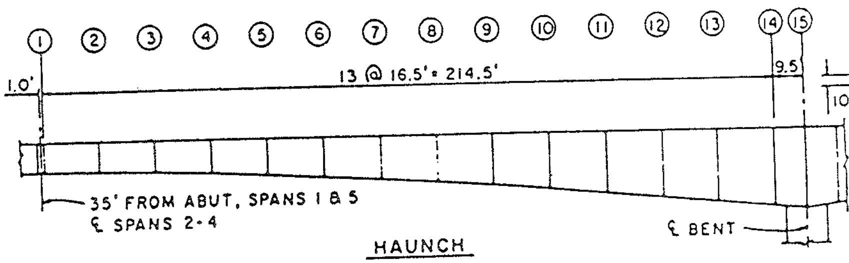
The Miller Creek Bridge and several other bridges in the same area developed severe cracking problems shortly after completion of construction. Because thermal gradient effects are suspected as being one of the causes of this distress, this bridge was included in the case studies.

This structure, shown schematically in Figure 12, is a three-span, 455-ft-long bridge.

Figures 13 and 14 are plots of top and bottom fiber stresses, respectively, along the bridge centerline.

The Miller Creek Bridge is of special interest because the structure developed relatively severe cracking in the bottom flange and girder stems at approximately the one-quarter and three-quarter





SECTION	D (ft)	y_b (ft)	A (ft²)	I (ft⁴)
1	9.00	6.00	121.32	1309.84
2	9.19	6.13	121.90	1375.68
3	9.60	6.41	123.23	1524.75
4	10.18	6.79	125.11	1751.09
5	10.90	7.18	129.14	2132.84
6	11.77	7.50	136.75	2775.83
7	12.76	7.85	145.60	3619.63
8	13.87	8.22	155.75	4706.64
9	15.10	8.65	166.35	6060.87
10	16.44	9.13	177.24	7720.86
11	17.89	9.69	187.65	9705.54
12	19.44	10.29	198.45	12109.05
13	21.10	10.95	209.06	14971.47
14	22.85	11.66	219.50	18331.48
15	24.00	11.70	223.10	18540.00

SUPERSTRUCTURE f'c = 5000 psi
SUBSTRUCTURE f'c = 4000 psi

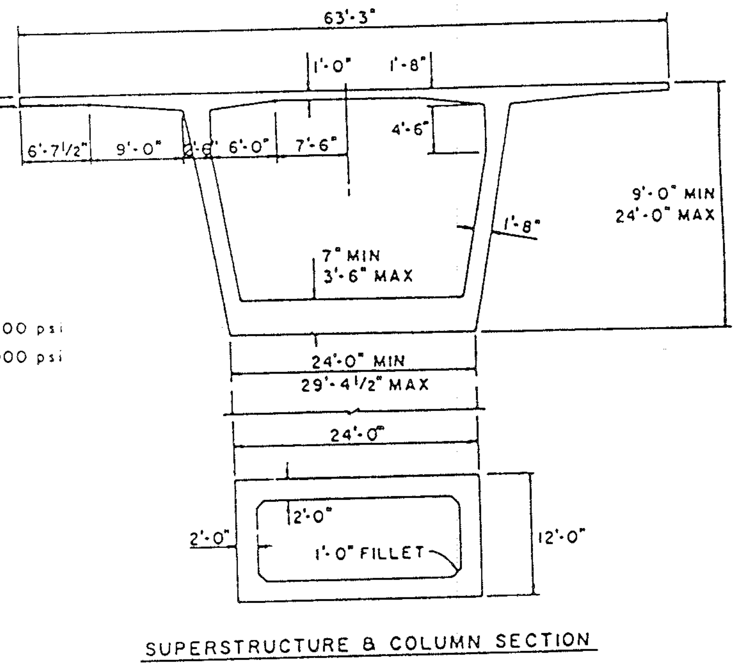


FIGURE 7 Case 5L: Columbia River Bridge superstructures and substructure details used for analysis.

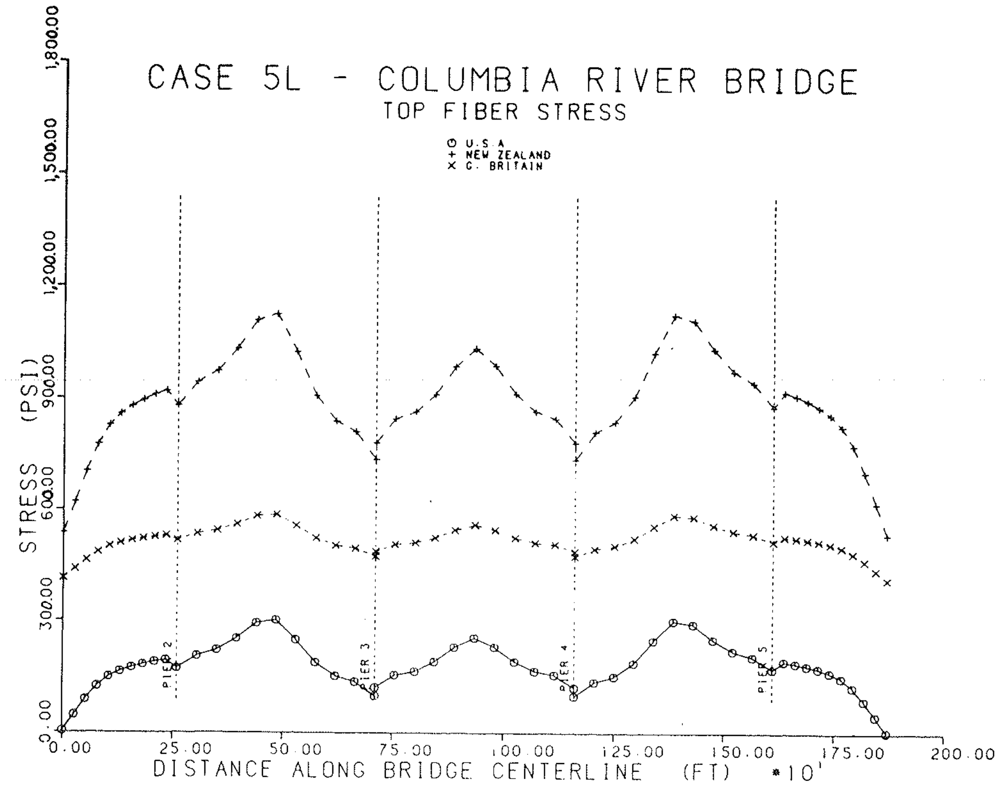


FIGURE 8 Case 5L: longitudinal variations in top fiber stresses along a girder centerline caused by positive temperature gradients.

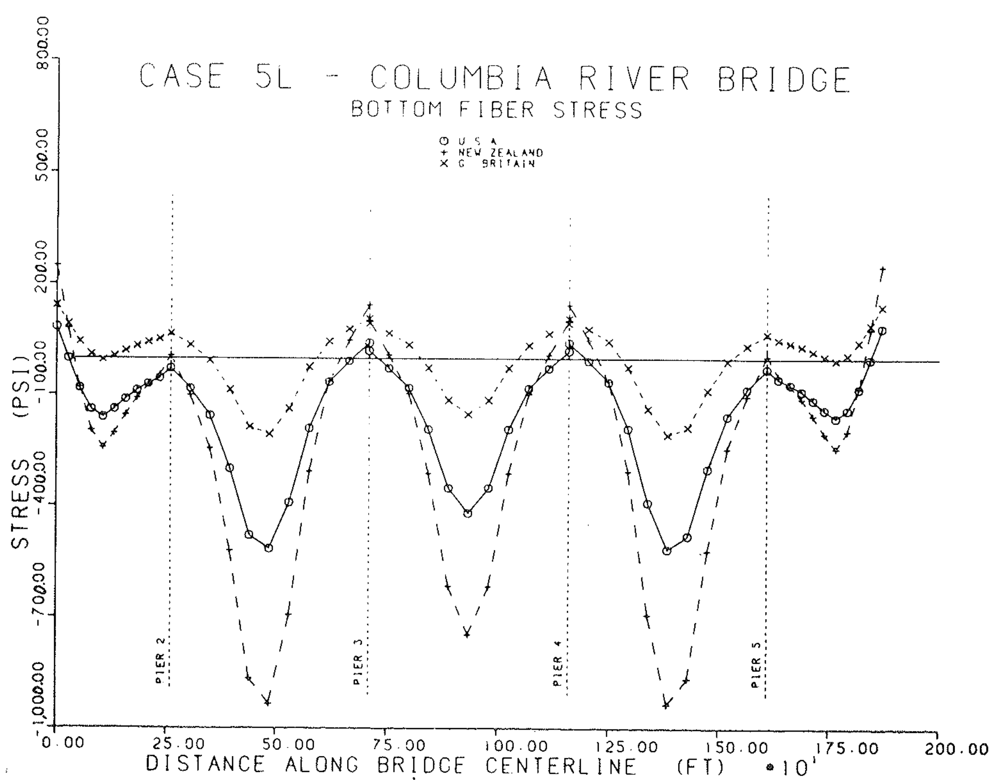


FIGURE 9 Case 5L: longitudinal variation in bottom fiber stresses along a girder centerline caused by positive temperature gradients.

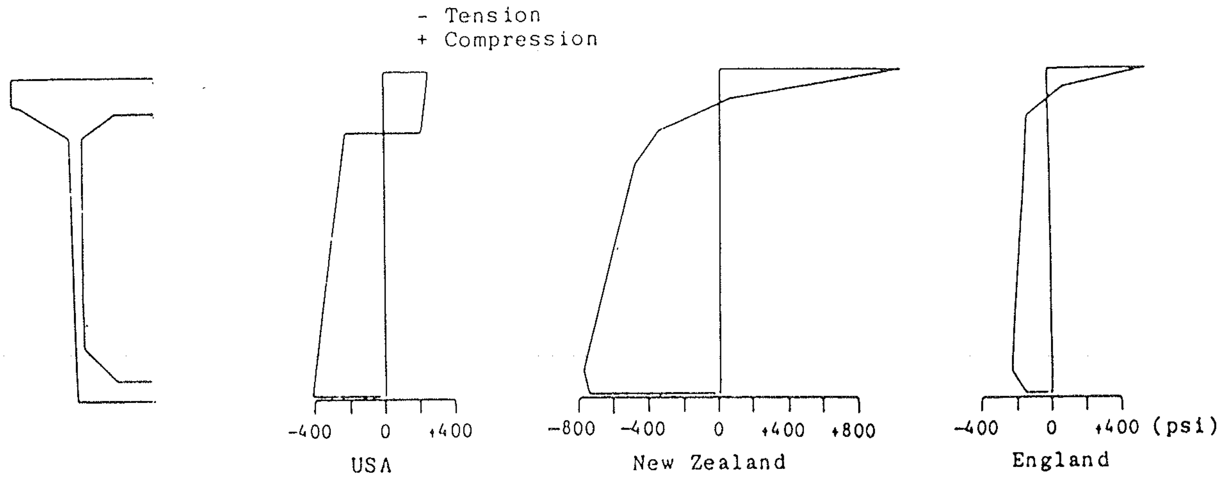


FIGURE 10 Case 5L, 9-ft deep cross section: variation in girder cross-section stresses with depth caused by positive temperature gradients.

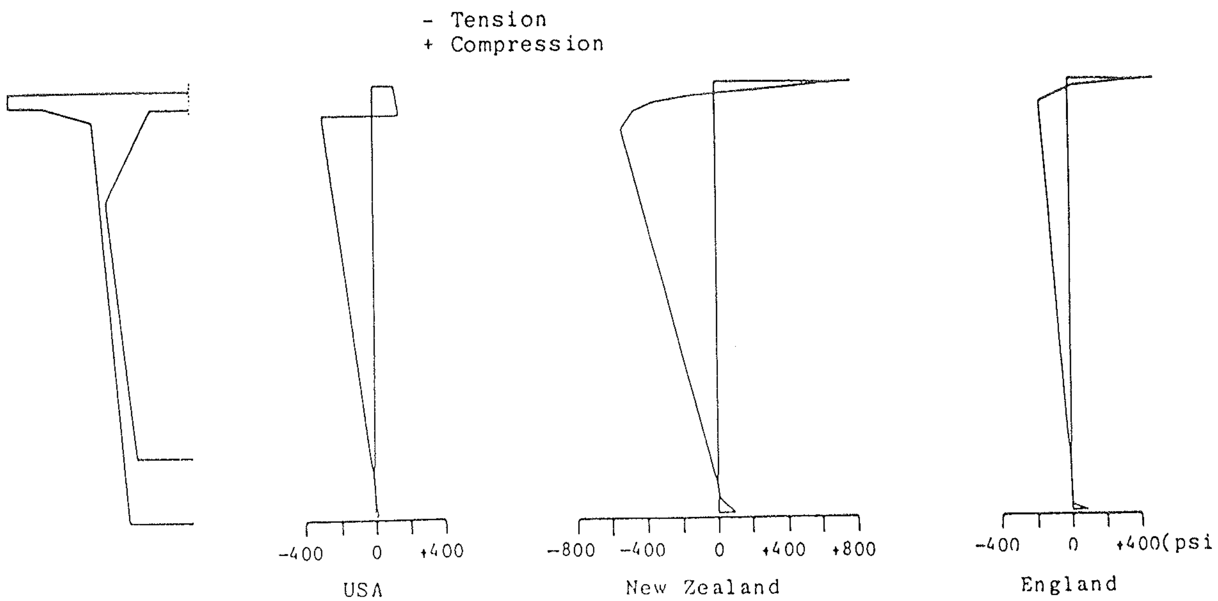


FIGURE 11 Case 5L, 24-ft deep cross section: variation in girder cross-section stresses with depth caused by positive temperature gradients.

points of span 2 (see Figure 2) a short time after completion of construction. Thermal gradient effects are thought to have been a significant contributing factor to this distress. The crack widths, in fact, were observed to be opening and closing on a daily basis, generally correlating reasonably well with daily temperature fluctuations.

This structure was constructed by the segmental, balanced-cantilever method. Prestress tendons are typically placed in the bottom slab within the center portion of the span to resist positive bending moments that can result from creep after the cantilevered portions of the superstructure are tied together. In this bridge these tendons terminated in the vicinity of the cracking. Several other similar bridges constructed nearby at about the same time have developed similar cracking patterns.

A plot that shows dead-, prestress-, and U.S. thermal-load stresses is shown in Figure 15. This plot shows that tensile stresses occur in the bottom fiber at the same location where the cracks developed in the Miller Creek Bridge.

Based on the details of construction and the characteristics of the observed cracking, it is postulated that the reasons for the cracking appear to be some combination of the following:

1. Greater inelastic redistribution of stress (i.e., increase of positive moment from creep) than anticipated,
2. Stress concentrations in the prestress anchorage zone, and
3. Thermal gradient stresses.

Although the primary cause of the distress cannot be precisely determined, it appears that the inelastic redistribution of stress that results in an increase of positive moment (reason 1) is probably the most important single factor. Local tension stresses caused by prestress anchorages and thermal

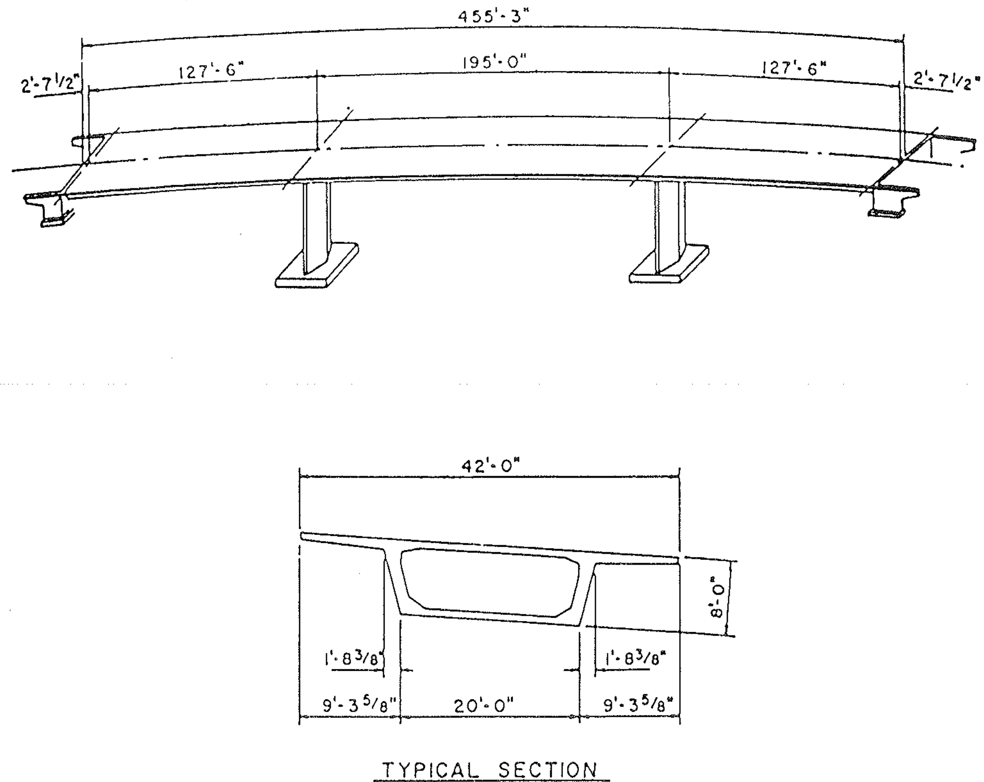


FIGURE 12 Miller Creek Bridge superstructure and substructure details used for analysis.

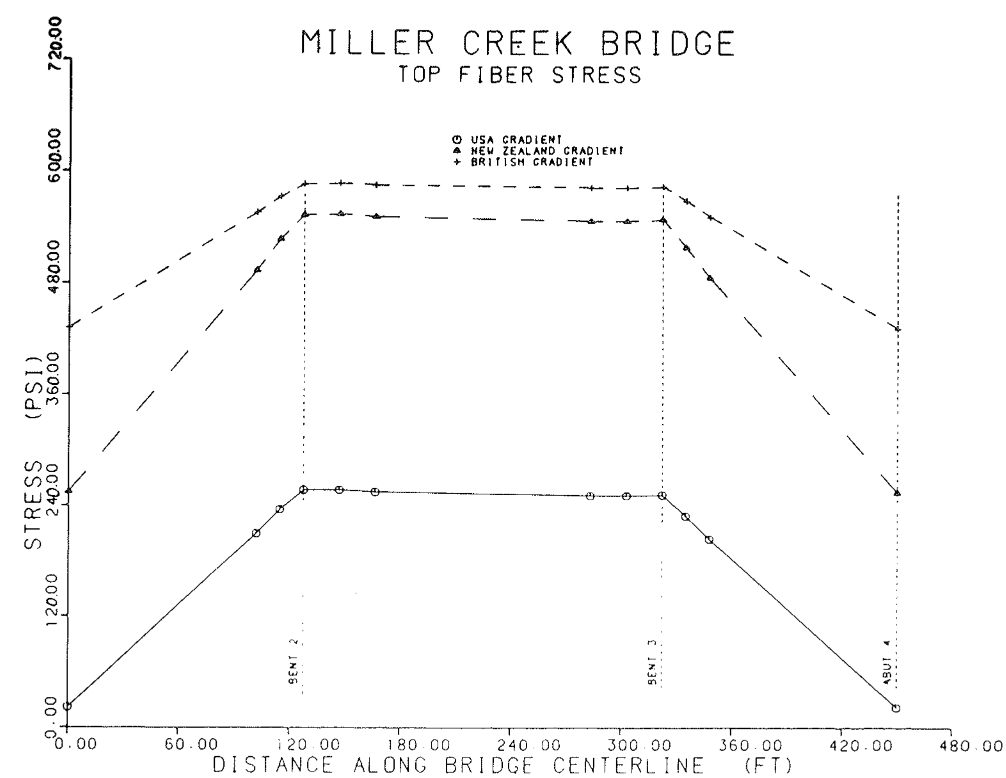


FIGURE 13 Case 8L: longitudinal variation in top fiber stresses along a girder centerline caused by positive temperature gradients.

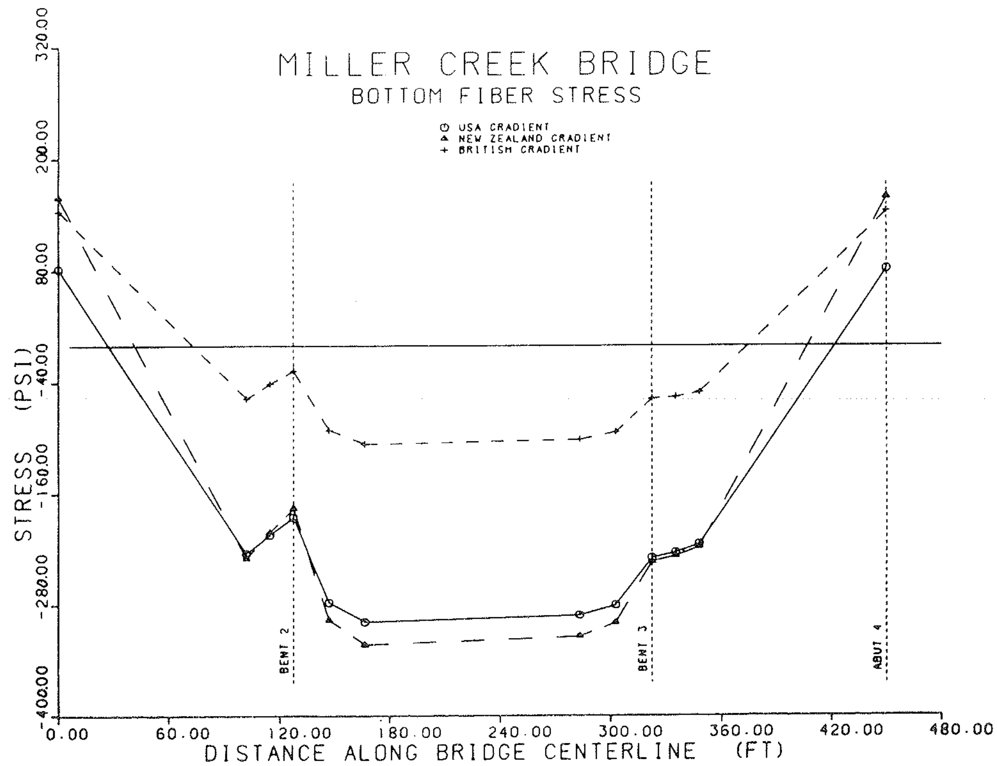


FIGURE 14 Case 8L: longitudinal variation in bottom fiber stresses along a girder centerline caused by positive temperature gradients.

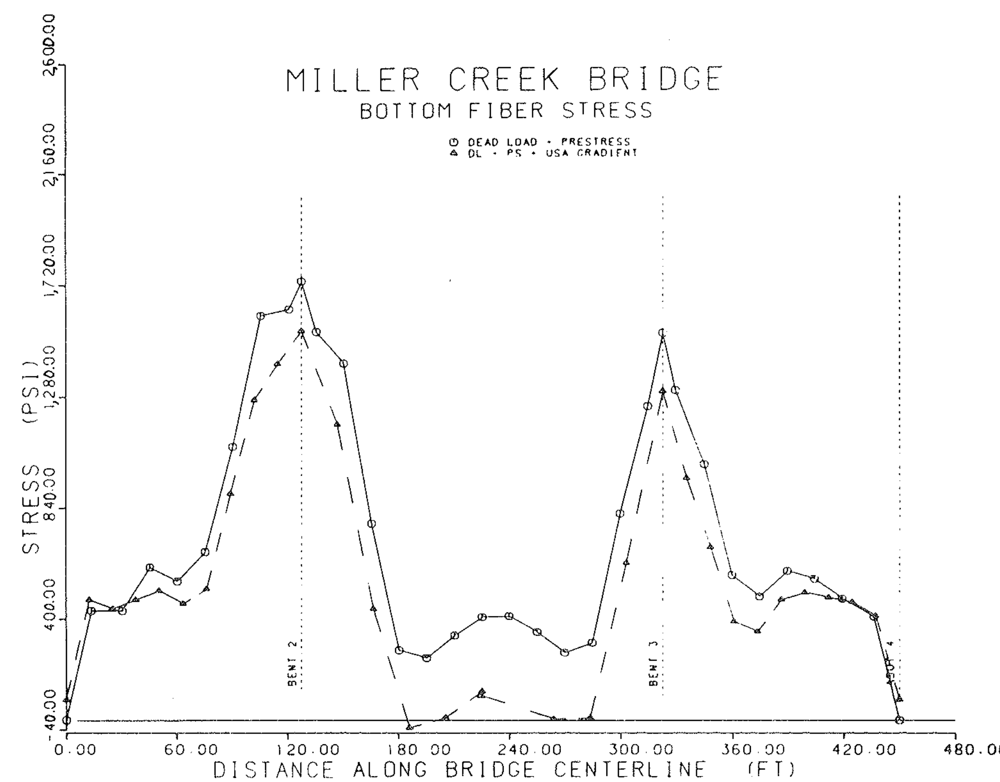


FIGURE 15 Case 8L: longitudinal variation in bottom fiber stresses along a girder centerline caused by combined dead load, positive temperature gradient, and prestressing.

gradient stresses, although significant, are probably of secondary importance.

The cracking probably could have been avoided by extending the bottom slab prestress tendons in the zones of high bottom-fiber compression, thus anchoring them much closer to bents 2 and 3.

In summary, it appears that although the thermal gradient stresses do contribute to the cracking problem, they are not the basic cause of it.

CONCLUSIONS AND RECOMMENDATIONS

Several case studies were conducted to investigate the effects of various assumed thermal gradients on the longitudinal and transverse fiber stresses induced in various types of concrete bridge superstructures. The following conclusions about the nature of thermal effects on concrete bridges are drawn from an assessment of the results of these case studies.

1. Although fiber stresses induced in different bridges by any single thermal gradient may vary in magnitude, the stress patterns are generally similar.
2. Cross-sectional changes such as bottom slab flares and haunches have a significant effect on the fiber stresses induced by thermal gradients.
3. The calculated fiber stresses were sensitive to the type of temperature gradient assumed. Although some gradients produced similar, extreme fiber stresses, the differences in fiber stresses between the extreme fibers within the girder section were significant. In many cases this difference was sufficient to affect the requirements for longitudinal reinforcement. This could be a contributing factor in some cases where concrete cracking has been observed.

Based on the results from these case studies and a review of the design approaches used in other countries, it is apparent that U.S. bridge design requirements should be expanded to include the effects of a thermal gradient. The current AASHTO procedures for considering longitudinal movement are basically adequate, although it would be desirable to include a more accurate method for determining seasonal variations in temperature. In light of these observations, the following recommendations are made for improved thermal design procedures in the United States.

1. Maps that reflect climatic variation, similar to maps used in the British design standard and the Ontario bridge code, should be included in the AASHTO specifications. Data contained in these maps can be used to develop the much-needed methods for determining structure temperatures.
2. An AASHTO guide specification should be developed to provide for the effects of thermal gradients. Because there is little or no evidence of distress caused by a temperature gradient in conventionally reinforced-concrete bridges, this guide specification should be directed toward the larger, prestressed-concrete bridges, which past experience has demonstrated are more likely to develop problems from temperature differentials. This guide specification should include the following:
    a. Methods for determining both longitudinal and transverse stresses;
    b. A nonlinear temperature gradient with a shape similar to that used in the British design standard; actual temperature differentials will vary;
    c. An option to accommodate the gradient stresses by increasing the prestress force or by determining the amount of auxiliary mild reinforcement by the partial prestressing concept;
    d. A map that indicates the maximum probable solar radiation in different geographical areas to help determine the maximum temperature differential for a given bridge site; and
    e. Transverse analysis consideration of the effects of a temperature differential caused by solar radiation on the top deck slab.
3. In developing a method for designing bridges for thermal effects, there are several points that must be kept in mind. These points, which relate to the current state of the art and design practices, are as follows:
    a. The format of the design specifications should be general enough to include advances in the state of the art as they are developed and allow for extension of the procedures to other bridge types in the future,
    b. The procedures should provide for maximum simplicity without sacrificing significant accuracy,
    c. The need for future training to implement the procedures must be considered, and
    d. In light of the limited number of cases in which temperature-induced distress has been observed, there is a potential problem with designer acceptance of elaborate thermal design procedures.

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Publication of this paper sponsored by Committee on Concrete Bridges.

Fatigue Behavior of Welded Wrought-Iron Bridge Hangers

PETER B. KEATING, JOHN W. FISHER, BEN T. YEN, and WILLIAM J. FRANK

ABSTRACT

The behavior of fatigue crack growth and fatigue strength of welded lap splice wrought-iron hangers in a railroad bridge was studied. The original wrought-iron hangers were lap spliced with steel plates for the purpose of tightening the members. Field inspection revealed cracks in the welded lap splices. Examination of simulated test joints and cracked hanger splices in the laboratory indicated that fatigue cracks would develop from weld deposits at the cut of the wrought-iron bar, propagate into the steel splice plate, and cause failure. Fatigue cracks could also propagate into the wrought-iron bars but would be arrested by the slag (iron silicate) stringers. Breaking of the wrought-iron bar only occurred when the applied stress was quite high in comparison with the yield point. Measured live-load stresses in the actual bridge member were relatively low. Evaluation of traffic and load records indicates that the effective live-load stresses would be well below the fatigue strengths of these spliced joints. No imminent problem of fatigue failure is expected in the hangers.

Before the refinement of the steel-making process, wrought iron was widely used as the principle structural material in bridge construction. It was used in many railroad structures during the period from the late 1800s to the early 1900s. Although the material properties of wrought iron have been well known since the beginning of its use, the welded fatigue behavior has never been adequately quantified. Thus a study on the fatigue behavior of welded wrought-iron splice plate repairs on Norfolk and Western Railway Bridge No. 651 in Hannibal, Missouri, is presented. Welded repairs are known to result in low fatigue strength details with steel components. Because the structural members were wrought iron with steel reinforcement, it was desired to evaluate the seriousness of the resulting welded details and to assess the degree of cumulative damage that may have occurred.

BRIDGE DESCRIPTION

The Norfolk and Western Railway Bridge was originally built for the Wabash Railroad in 1888 by the Detroit Iron and Bridge Works. The bridge spans the Mississippi River with seven truss spans and one continuous swing span for a total length of 1,580 ft (Figure 1). It carries a single track and has a truss spacing of 18 ft. The bridge members are con-