THERMAL GRADIENTS AND THEIR EFFECTS ON SEGMENTAL CONCRETE BOX GIRDER BRIDGES

by

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The thermal gradients in segmental concrete box girder bridge structures are not fully known as a result of variations in location, daily temperature changes, radiant load, wind speed, surface reflectiveness, box shape, and other variables. These variables make the prediction of thermal gradients difficult. In a continuous structure thermal gradient stresses caused by longitudinal moments can equal those of live loads. The number of segmental concrete box structures studied for temperature gradients is fairly low.

An on-site instrumentation program to measure the thermal gradients in a post-tensioned segmental box girder was conducted at the U.S. 183 Segmental Bridge in Austin, Texas. Thermocouples were installed in the bridge superstructure to measure the concrete temperature in the box and wings. Strain gauge devices were developed and placed at several locations in the superstructure to document the effects of temperature gradients on the superstructure. On-site data recording equipment continually acquired concrete temperatures and strains over a ten month period beginning in March of 1995.

Data concerning temperature gradients and thermal induced strains through the superstructure depth, across the webs and slabs of the box are presented. These temperatures and strains are analyzed for their affect on the superstructure. Calculations are made to compare predicted strains to measured strains. Comparisons are made from design thermal gradients to those measured and recommendations are made concerning design gradients for hollow prestressed concrete bridge superstructures.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1

INTRODUCTION

1.1 GENERAL

The effects of annual fluctuation in the average temperature in concrete bridges have been well understood by designers for many years. Bridge designers have accommodated this effect by providing expansion joints, sliding or flexible bearings, and flexible substructures.

A second type of thermal effect occurs in most box girder bridges. The AASHTO Guide Specification [6] requires that it be considered in the design of segmental concrete bridges. This effect occurs as the result of a thermal gradient in the structure. This gradient develops throughout the depth of the girder and to some degree through the slab and webs of the girder. Problems have occurred in some early segmental concrete bridge structures as a result of totally ignoring thermal gradients. Concrete cracking in the following structures has been attributed to thermal gradients: the Newmark Viaduct in New Zealand, the Fourth Danube Bridge in Vienna, and The Jagst Bridge in Untergreisheim [1]. In addition four cast-in-place segmental prestressed concrete box girder bridges in Colorado experienced cracking in the webs and bottom deck soffits which was attributed to thermal gradients [2]. Csagoly and Bollman of the Florida Department of Transportation reported significant opening of segmental bridge dry joints due to thermal gradients in several bridges studied in the Florida Keys [3].

There is unity among the structural engineering community with regard to existence of thermal gradients in all bridge type structures. There is much less agreement, however, regarding the magnitude and shape of the thermal gradient to be applied to a structure as a loading condition. Clearly predicting the magnitude of the thermal gradient is difficult. This prediction is dependent not just on climatological conditions, but also upon the material properties and shape of the structure. In San Antonio Texas, on the Downtown "Y" segmental viaduct, the measurement of the maximum positive temperature gradient reached a magnitude of 15.5^{o} C during an extensive time period of measurement [3]. In a full scale segmental box girder erected at Penn State University, researchers Hoffman, McClure and West measured a maximum positive temperature gradient of 26.7^{o} C [1]. Large temperature gradients in continuous structures can produce substantial positive bending moments.

Many different thermal gradient loadings have been used around the world for concrete bridge structures. In Continental Europe 10^{o} C to 15^{o} C magnitude positive linear temperature gradients are used for design loadings [2]. New Zealand uses a maximum magnitude of 32^{o} C for their non-linear temperature gradients for design loadings [2]. Australia uses a maximum magnitude of 24^{o} C for their non-linear temperature gradients for design loadings [2]. There is little agreement among the world's bridge design codes as to the magnitude or shape of positive or negative temperature gradients.

In the US the 1985 NCHRP Report 276 "Thermal Effects in Concrete Bridge Superstructures" [2] proposed positive non-linear design gradients (deck warmer than webs) based on the 1983 report by Potgieter and Gamble (See Figure 1.1) [4]. In their study, weather station data collected in the US was combined with the results of a finite difference one-dimensional heat flow program to predict extreme positive design thermal gradients. Negative non-linear design gradients in this NCHRP report were based on the British Standard BS 5400 [5]. These proposed positive and negative design gradients were specified for four geographic regions with different maximum values for each region as well as for varying deck conditions (concrete, thin asphalt or thick asphalt surfaces). *The 1989 AASHTO Guide Specification for the Design and Construction of Segmental Concrete Bridges* [6] (to be referred to for the remainder of this report as the *AASHTO Guide Specification*) adopted these recommendations (somewhat under protest) for positive and negative design gradients.

These non-linear gradient values were not well received by all bridge designers in the US. The non-linearity of these gradients and their application to the typical flanged box shapes add to the complexity of calculations performed for design of these type structures. The amount of longitudinal post-tensioning required by design for these gradients on a continuous structure can be as large as the post-tensioning required for the design of live loads. Clouding the issue of design gradients in bridge superstructures is the fact that AASHTO currently has two separate design specifications which coexist with the previously mentioned segmental design guide. These two additional design specifications are the *AASHTO Standard Bridge Specification* [7] and the *AASHTO Load and Resistance Factor Design Specification for Bridges* [8]. The Standard Bridge Specification only *recommends* the inclusion of the thermal gradient loadings in design of conventional stringer and deck bridge type structures whereas the other two AASHTO specifications *require* the inclusion of this loading in the design of the bridge types they cover. If included in the design of one bridge but not in the design of another type, the results could yield an unfair economical advantage to the bridge type for which the thermal loading was not required.

Figure 1.1 Typical positive temperature gradient (for Zone 2 of the US - See Figure 2.2, 2.3 and Table 2.1).

The *AASHTO Guide Specification* recommends in its commentary that future field research be undertaken to verify the design gradients. The Instrumentation Study of Precast Segmental Box Girder Bridges on US 183 in Austin, which is introduced in the next section, included instrumentation to verify the thermal gradients that actually exist in these type structures. Instrumentation was also installed to verify to some degree the structural response of the structure to these thermal gradients.

1.2 U.S. 183 PROJECT DETAILS

Instrumentation of Precast Segmental Box Girder Bridges on US 183 in Austin is a study funded by the Texas Department of Transportation in joint cooperation with the Federal Highway Administration. This field study of the US 183 Precast Segmental Box Girder Bridge is in very close proximity to the Ferguson Structural Engineering Laboratory in Austin. The segmental viaduct location is shown in Figure 1.2 vicinity map. As a result of this proximity, transportation of staff and equipment as well as monitoring of data required little mobilization effort. This proximity also allowed for quick reaction time to instrumentation schedule changes and problems as they occurred during construction.

Figure 1.2 Vicinity Map
(after Bonzon [10])

The US 183 Segmental Project consists of approximately 6 centerline miles of elevated twin freeway and its associated ramps. All of the main freeway structure and most of the ramp structures are traditional segmental trapezoidal concrete box girder spans built using typical span-by-span construction. One of the ramp spans is a continuous segmental concrete box structure built by the balanced cantilever method.

The typical spans are not fully continuous as has often been typical in segmental construction. Rather the otherwise simple spans are joined by a thin slab at the deck in place of having an expansion joint or full continuity. These spans are intended to act or function basically as simple spans with the thin closure acting only as an inexpensive (“poorboy”) joint for live load partial continuity. The post-tensioning in these spans is a mix of internal straight strand tendons and draped external strand tendons. This mix of simple spans and straight internal tendons allows for very fast and efficient segmental construction. Simple spans effectively eliminate the restraint stresses caused by thermal gradients in continuous structures. These restraint stresses add to the total stress and thus offset much of the advantage gained by the use of continuous structures.

The US 183 study was similar to a companion study of the San Antonio “Y” box girder project, conducted by Roberts, Breen, and Kreger at the University of Texas in 1993 [3]. A great deal of effort went into evaluating and developing field instrumentation for this study by Arrϑllaga [9]. Very workable and dependable systems were developed for measurements of deflections, tendon forces, friction losses, thermal gradients and joint movements. These systems were installed in the San Antonio “Y” project in 1991 and 1992 and were monitored on a regular basis over several years. The information gained on that study aided in the selection of monitoring equipment and devices to be used on the US 183 Instrumentation Project. Specific areas of study in the US 183 Instrumentation Project were:

- prestress losses in external tendons
- diffusion of post-tensioning forces
- temperature gradients and their effects
- temporary post-tensioning efficiency and joint behavior
- behavior of the cast-in-place and precast segmental substructures
- behavior of the partially continuous unit
- strains and cracking in anchor zones and deviators

- balanced cantilever construction of the IH 35 - US 183 connector

It was intended that the information gathered from this project would be used to clarify or revise AASHTO provisions that had to be based on “best judgment” in the absence of data. In addition this information can lead to a reduction in the uncertainties in the design of these type structures and provide for more durable, easier to construct and lower cost bridges in the future.

1.3 FACTORS AFFECTING GRADIENT

Both vertical and horizontal components of temperature variations in a concrete box girder are influenced by multiple factors. The principal factors affecting this variation are shown in Fig. 1.3 and include:

- Climatological: solar radiation, air temperature, and wind speed (Climatological factors are estimated based on geographical parameters, such as latitude, longitude, altitude.)
- Geometric: cross-sectional geometry, overlay thickness, and orientation of bridge.
- Material properties: thermal conductivity, color, density, specific heat, and absorptivity.

A maximum positive gradient (deck warmer than webs) occurs when several days of generally cool weather is followed by clear warm days with intense solar radiation and light winds. A maximum negative gradient (webs warmer than deck) occurs when several days of warm weather is followed by a severe cold front with rain cooling the deck. The generally low conductivity of concrete helps to produce large gradients since incoming heat input cannot be transferred quickly to other parts of the cross section.

Figure 1.3 Factors affecting thermal gradients.

1.4 FACTORS AFFECTING STRUCTURAL RESPONSE

The structural response to a temperature gradient is affected by two factors:

- Shape of the gradient. (Linear or Non-linear)
- The static determinacy of the structure.

A statically determinant beam subjected to positive linear gradient will elongate and deflect upwards, but will have no temperature gradient-induced stress (see Figure 1.4).

Figure 1.4 Linear gradient applied to a statically determinant beam.
(after Roberts[3])

A non-linear gradient applied to a statically determinant beam will develop internal stresses to maintain static equilibrium (the sum of all forces on any given free-body must be equal to zero to remain stationary). These stresses are referred to as self-equilibrating stresses. These self-equilibrating stresses are the result of the difference between the strains the structure wants to develop and the strains it is forced to develop to keep plane sections plane and meet the support boundary conditions.

The magnitude of these self-equilibrating stresses can be found by considering a fully restrained beam subjected to the non-linear temperature gradient shown in Figure 1.5.

Figure 1.5 Non-linear gradient on restrained beam
(after Roberts [3])

The variables used in this disscussion are as defined below:

y = distance from the center of gravity of the cross-section,

$T(y)$ = temperature at a depth y,

$b(y)$ = net section width at a depth y,

E = modulus of elasticity,

α = coefficient of thermal expansion.

$\sigma_{se}(y)$ = self-equilibrating stress at depth y,

A = cross-sectional area,

I = moment of inertia of the section.

The stress in the member due to full restraint of elongation and rotation is equal to:

$$\sigma_{temp}(y) = E\,\alpha T(y)$$

where

$\sigma_{temp}(y)$ is compressive if the temperature gradient is positive (warming)

The restraining axial force, P, can be calculated as follows:

$$P = \int_Y E\,\alpha\,T(y)\,b(y)\,dy$$

The restraining force is compressive if the temperature is positive (warming). The restraining moment acting on the section is:

$$M = \int_Y E\,\alpha\,T(y)\,b(y)\,y\,dy$$

Note for the case of the unrestrained beam (determinant beam) in Figure 1.5, the restraint forces M and P must be removed. The remaining stresses in the beam (see Figure 1.6), the self-equilibrating stresses, are the sum of the fully restrained beam stresses and the stresses due to released axial and bending restraint forces and can be written as follows:

$$\sigma_{se}(y) = E\,\alpha T(y) - P/A - My/I$$

Temperature gradients applied to an indeterminate (continuous) beam, either linear or non-linear, will result in secondary (restraint) moments in the beam as reactions develop at the supports. The positive gradient will cause the beam to camber up over its entire length, yet the dead weight of the structure will generally keep the beam upon the supports even though there is no restraint mechanism, thus producing the secondary moments (see Figure 1.7)

Figure 1.6 Non-linear gradient on restrained beam
(after Roberts [3])

Figure 1.7 Non-linear temperature gradient applied to indeterminate beam
(after Roberts [3])

Solutions to thermal gradient problems are quite tedious for indeterminate structures with non-linear temperature gradients. The shape of the modern winged trapezoidal box employed in concrete segmental construction today requires the use of complex programs to solve for the internal stresses. Choosing an appropriate thermal gradient without guidance from code provisions can be very complex. Indeterminate reactions are general fairly easy to evaluate with modern frame programs yet structures with three dimensional curves can be challenging when trying to determine support reactions, moments, and torsional moments.

1.5 OBJECTIVES

The objectives for this study are:

- To evaluate and compare the thermal gradient findings of previous studies in relationship to this study.
- To measure the shape and magnitude of the peak positive and negative temperature gradients in this structure for the time period of the study and to compare them to the current AASHTO models.
- To measure the magnitude of the self-equilibrating strains and compare those strains to those from an analytical model with the same temperature gradient loading imposed upon it.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

The following reviews are summaries of previous field or laboratory investigations of thermal gradients.

2.2 POTGIETER AND GAMBLE [4]

This study became the basis for the design recommendations of the NCHRP 276 Report. This study used a one-dimensional heat flow model in conjunction with a finite difference analytical method to quantify the magnitude of temperature differences at extreme conditions. The program that was used accounted for convection, conduction, radiation, and incorporated variables such as heat transfer and weather conditions.

Two days of field measurements were gathered at the Kishwaukee River Bridge to confirm their analytical program results. Their data from field conditions were very similar to the data from the computer model.

Weather data were collected from 26 solar radiation collecting weather stations around the US. Extreme conditions collected allowed the determination of extreme positive gradients. See Figure 2.1 for weather station locations and the maximum positive gradients. These gradients (in degrees Fahrenheit) shown in Figure 2.1 are those predicted by the computer model for unsurfaced bridges and the number of days they are expected to occur each year. Some of the conclusions of this study were:

1. Further study is needed to be able to predict the percentage of thermal loading, which can realistically be applied to the structure.
2. Thermal stresses are mainly related to the service load limit states and they have little effect on the factored live and dead load ultimate structural capacity.
3. A large number of field studies are necessary to evaluate the validity of the computer model's predicted bridge response to thermal loadings especially in light of the multitude of bridge types being utilized today.

Figure 2.1 SOLMET Stations and peak positive temperature differences (after Roberts, et al [3]).

2.3 NCHRP 276 REPORT [2]

This report was a comprehensive overview of thermal effects in concrete bridges specifically focused on developing guidelines for inclusion into the AASHTO bridge design specifications. This report reviewed the apparent thermal cracking problems with then current concrete bridges around the world citing several examples. This report also reviewed the then current (1985) design code provisions for 11 countries around the world. Of the 11 countries reviewed only four (Australia, England, New Zealand, and the US) required the use of a non-linear positive gradient. Denmark, France, Germany, and Sweden recommended a linear gradient which varied from 5 ^{o}C to 15 ^{o}C. Canada, Italy, and Japan did not then specify a positive gradient. England and the US specified the use of a negative non-linear gradient. Denmark, Germany, and Sweden then specified a linear negative gradient. The rest of the countries reviewed specified no negative gradient at all. This report gave examples of methods to find temperature-induced stresses and provided sample calculations for those methods.

Appendix A of the NCHRP 276 Report gives the design guidelines which were later adopted by AASHTO. These guidelines rely heavily upon the research work performed by Potgieter and Gamble [4]. These guidelines divided the US into four radiation intensity zones (see Figure 2.2). The non-linear positive and negative gradients recommended for each of these four zones varied in magnitude based on information about the magnitude of solar radiation measured in locations around the US (see Figure 2.3 and Table 2.1 for positive gradient recommendations). The negative non-linear temperature gradient recommended was taken from the British Standard 5400 [5] (see Figure 2.4 and Table 2.2). This recommended negative gradient had no analytical basis nor did it have field research data to substantiate it. In addition this report recommended design values for the concrete coefficient of thermal expansion based on different aggregate types (see Table 2.3).

Figure 2.2: Proposed maximum solar radiation zones (after Imbsen, et al [2])

Figure 2.3: Recommended positive vertical temperature gradient (after Imbsen, et al [2]).

Table 2.1: Recommended temperature coefficient magnitudes for positive gradients (after Imbsen, et al[2]).

Figure 2.4: Recommended negative vertical temperature gradient (after Imbsen, et al [2]).

Table 2.2: Recommended temperature coefficient magnitudes for negative gradients (after Imbsen, et al[2]).

Aggregate Type	Thermal Coefficient of Concrete (0.000001 per ^{o}C)
Quartzite	12.8
Quartz	11.5
Sandstone	11.7
Gravel	12.4
Granite	9.5
Dolemite	9.5
Basalt	9.0
Marble	4.3 to 7.4
Limestone	7.2

Table 2.3: Recommended design values for concrete coefficient of thermal expansion (after Imbsen et al[2]).

2.4 HOFFMAN, McCLURE AND WEST [1]

In this study researchers at Penn State University built a full scale segmental box girder bridge for testing and instrumentation. Of particular interest was the study performed on thermal gradients and the structure's response to those gradients on this simple span bridge. Thermocouples as illustrated in Figure 2.5 were installed and measured along with deflections and horizontal support movements eighteen times daily from October 25, 1978 to October 16, 1979. Figure 2.6 shows the peak positive and negative gradients measured in this study. The main conclusions for this part of the study were that the heat flow is nominally two-dimensional (vertical not along the bridge axis) and that the predominant input for upward deflections is solar radiation.

Figure 2.5: Thermocouple layout for Hoffman, McClure and West test bridge.

Figure 2.6: Maximum recorded gradients for Hoffman, McClure and West study.

2.5 AASHTO SEGMENTAL GUIDE SPECIFICATION [6]REQUIREMENTS

The 1989 AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges had the following provisions for the thermal gradients:

> **7.4.4 Differential Temperature**
>
> Positive and negative differential superstructure temperature gradients shall be considered in accordance with Appendix A of National Cooperative Highway Research Program Report 276 "Thermal Effects in Concrete Bridge Superstructures" [2].

This Specification also amends the AASHTO Standard Specifications [7] load cases by adding the following service load condition:

$$(DL + SDL + EL) + \gamma_E E + B + SF + R + S + (DT) \qquad (2.8)$$

All loads shown in parentheses the above equation are the loads defined and required by the AASHTO Segmental Specification. Those not in parentheses are defined and required by the AASHTO Standard Specification.

DL = structure dead load	B = buoyancy
SDL = superimposed dead load	SF = stream flow pressure
EL = erection load	R = rib shortening
E = earth pressure	S = shrinkage
γ_E = earth pressure coefficient	DT = thermal gradient loading

There is an additional loading requirement for load combinations that include full live load with impact which allows DT to be reduced by 50 percent. The temperature loading T in the standard specification is redefined in the segmental specification as:

$$T = (TRF + DT) \qquad (2.9)$$

where TRF is equal to the original temperature loading in the standard specification.

2.6 AASHTO LRFD SPECIFICATION [8]REQUIREMENTS

The temperature gradient effects of the 1994 edition of the AASHTO LRFD Bridge Design Specifications are analyzed under six different load combinations as shown in Table 2.4. The load factor Y_{TG} is to be specifically chosen differently for each project based on the type of structure being designed and the limit state being investigated. The commentary suggests that for strength limit states a lower factor for Y_{TG} could be used. The AASHTO LRFD Code

Table 2.4: Excerpt from the AASHTO LRFD Specification [8] "Table 3.4.1-1 - Load Combinations and Load Factors"

Figure 2.7: AASHTO LRFD Specification [8] vertical positive gradient shape.

Table 2.5: AASHTO LRFD Specification [8] positive gradient magnitude values.

specifies a positive gradient shape based on the NCHRP Report 276 [2]. The Solar radiation zones are the same as recommended in Report 276, however the shape of the positive temperature gradient has been modified to a simpler shape (See Figure 2.7 and Table 2.5). Dimension "A" shown in Figure 2.7 is specified as:

- 300mm (for concrete superstructures which are 400mm in depth or greater)
- Superstructure depth - 100 mm (for concrete superstructures which are less than 400mm in depth)
- 300mm (for steel superstructures, where t is equal to the concrete deck thickness)

The negative gradient is specified as the positive gradient shape values shown in Table 2.5 multiplied by -0.5. The temperature specified for T_3 is 0.0 unless a site specific study is done to determine appropriate values and shall not exceed 2.8 ^{o}C for positive gradients and -1.4 ^{o}C for negative gradients.

2.7 AASHTO GUIDE SPECIFICATION [7]REQUIREMENTS

The 1989 AASHTO Guide Specification for Highway Bridges only very vaguely requires the consideration of thermal effects in any bridge design. Section 3.16. requires that bridge designs consider the stresses and movements caused by temperature change. It does not give any specific requirements for thermal gradients to be considered.

2.8 ROBERTS - WOLLMANN [3]

This study was a field investigation into the behavior of a segmental concrete box girder built by the span-by-span method. The specific areas of research in this study were:

- Prestress losses in external tendons
- Transverse diffusion of external post-tensioning forces
- Temperature gradients and their effects
- Joint efficiency
- Anchorage zone and deviator behavior

This study will only focus on the research in the area of thermal gradients and its effects on the structure.

One idea of interest that Roberts[3] attributed originally to research by Priestley is that at service level, loading thermal gradients will produce a thermal deformation and a corresponding proportional thermal force due to linear elastic behavior. However, at ultimate load levels the factored thermal deformation is added to the deformations produced by other factored loads. The force generated by the thermal loading at this factored load level is far less significant as compared to the service loading, because of the greatly reduced stiffness of the structure at this increased load level due to cracking and localized yielding. (See Figure 2.8).

Figure 2.8 Thermal effects at service load and ultimate load
(after Roberts, et al [3]).

The thermal study consisted of the installation of thermocouple and concrete strain measuring devices with a data recording system. Thermocouples were installed in the webs and the top and bottom slabs of two spans with different cross-sections. The thermocouple layouts for these two cross-sections are shown in Figure 2.9. These thermocouples were installed prior to segment casting and were monitored every 30 minutes from July 25, 1992, to July of 1993. In addition to thermal monitoring, on four occasions concrete strains and structure deflections were monitored over the period of a day.

Maximum positive thermal gradients reported by Roberts for the section were 50% of the design magnitude specified (with no asphalt topping) by the AASHTO segmental specification. The same section with 50mm of asphalt topping was reported as 78% of the maximum design gradient. The maximum negative gradient reported was 65% of the design gradient for the untopped condition and 50% for the section after the addition of 50mm of asphalt topping.

Figure 2.9: Thermocouple layout for Roberts [3]San Antonio "Y" bridges.

2.9 AROCKIASMY AND REDDY [11]

This report is an analytical study of temperatures in two existing segmental box bridges with field measurement devices recording temperatures and strains in those two bridges.

A finite element thermal analysis was conducted on two bridges for summer and winter temperature patterns. Prevailing conditions such as: ambient air temperature, turbidity factors, latitude, longitude, altitude, and surface azimuth angle were input into FETAB (Finite Element Temperature Analysis of Bridges) for analysis. The predicted temperatures and strains from the analysis were compared with data collected from the two bridges in the field. The field data was collected during two periods of the year. The first period was July-August 1991. The second was January-February 1992. The instrumentation used to collect the data was retrofitted into the existing bridges. The field data collected was then compared to the analytical data derived from the finite element analysis.

Some of the important conclusions from this research included the following:

- Temperature variations along the top flange (deck) are not constant but are symmetrical about the vertical axis of the cross-section .
- Higher wind speeds lower the maximum temperature of the top flange.
- Predicted temperatures agreed with field data only during short periods of the day.
- Solar Radiation has a large influence on temperature gradients within the top flange.
- Critical stress levels and distribution vary largely in design guidelines from different countries around the world. New Zealand's code specification yields the largest design stress values. Only the PCI/PTI code yields web tensile stresses comparable to New Zealand's code.
- The maximum measured positive temperature gradient was 25.6 ^{o}C as compared to 22.8 ^{o}C which is the design gradient recommended for this bridge from NCHRP Report 276.

An important recommendation from this study is:

US bridge design requirements should be expanded to include the effects of thermal gradients.

2.10 BRANCO AND MENDES [12]

This paper presents a numerical method for analysis of temperature gradients in concrete bridges utilizing the famous Fourier equation. This analysis method involves the use of a finite element program to obtain a solution for specific structures. This method is not really practical for most bridge designers. However, for specific structures in which this type of analysis is warranted this method may be useful.

In this paper the authors give methods to account for thermal irradiation, heat absorption from short wave solar radiation, and convection heat transfer. The authors present correlation between field data and their example analyses. In addition the authors present a method to account for geometrical differences and climatic conditions. The authors conclude that this is a useful tool for special bridge studies to evaluate thermal effects.

2.11 PENTAS, AVENT, GOPU, AND REBELLO [13]

This study investigated the thermal distribution and movements of a prestressed concrete girder bridge located on US 190 over the Atchafalaya River at Krotz Springs, Louisiana. The objective of the study was to determine the thermal characteristics and temperature distributions through the depth of the bridge slab and girder.

The bridge was instrumented and then monitored over a 2 year period approximately once a month for 12 or 24 hours. This 12 or 24 hour monitoring alternated on a month-to-month basis. The location of the thermocouples is shown in Figure 2.10. The peak positive temperature gradients recorded appear to be about 13 ^{o}C from the graphs presented. The peak negative temperature gradients recorded appear to be about 4 ^{o}C from the graphs presented. The graphs of the recorded temperature gradients are shown in Figure 2.11.

Figure 2.10: Thermocouple locations for US 190 Atchafalaya River Bridge.
(after Pentas et al[13]).

Figure 2.11 : Temperature distributions through the depth of the section for the US 190 Atchafalaya River Bridge.
(after Pentas et al[13]).

The authors of this paper then developed a graph of bridge temperature gradient (positive) as a function of ambient temperature. This graph was developed with a relatively small set of data (16 days or partial days from a one year period) which did not appear to have the circumstances necessary for developing peak positive temperature gradients (i.e. cold nights followed by days of intense solar radiation).

The authors draw many conclusions from this limited data. It appears that although the gradients presented in this study are consistent with the ambient temperature history the limited data set limits the validity of the conclusions.

2.12 ANDRES [14]

This study was a companion investigation during the first phase of the field study of the US 183 Instrumentation Project. The main focus of Andres' study was researching the flow of forces in an unusually-shaped concrete pier. Andres also investigated the temperature trends and resulting induced stresses in the same pier. One very interesting model which Andres presents in her study investigates the concept of a large concrete pier as somewhat analogous to a bronze-encased steel bolt undergoing a uniform temperature change as shown in Figure 2.12. Both materials respond differently to the uniform temperature change because of their differing coefficients of expansion. Andres compared this analogy to that of the pier's, where there are large temperature differences between the "shell" and "core" concrete (see Figure 2.13).

Figure 2.12: Bronze-encased steel bolt undergoing a uniform temperature change.
(after Bonzon[10])

Figure 2.13: Concrete pier model undergoing a differential temperature change.
(after Bonzon[10])

2.13 BONZON [10]

This study was also a companion investigation during the second phase of the field study of the US 183 Instrumentation Project. The main focus of Bonzon's study was researching the thermal gradients, post-tensioning strains and thermal-induced strains in a hollow post-tensioned segmentally constructed concrete pier. Some of the important conclusions of this study are:

- Pier temperatures fluctuate much more at the outer surfaces of the cross-section than the interior surfaces.
- Solar radiation produces the largest and most rapid changes in temperature at the outer faces of the pier cross-section. Air temperature changes are a secondary mechanism of heat transfer.
- The temperature distribution for hollow box bridge piers was more two-dimensional than the normal one-dimensional temperature gradient experienced in a hollow bridge superstructure.

CHAPTER 3

INSTRUMENTATION DEVELOPMENT

3.1 INTRODUCTION

This chapter describes the instrumentation chosen for measurement of thermal gradients and their effects in the US 183 mainline superstructure. A full description of the superstructure span location selection process is also included.

3.1.1 Instrumentation

The determination of the thermal gradients and the structure's response to the thermal gradient was accomplished by measuring the temperatures and concrete strains at various locations throughout the box cross-section.

Particular components and data collection systems were chosen based on information from a previous study by Arrϑllaga [4] and from the previous study by Roberts [3].

3.1.2 Instrumented Span

The span chosen for instrumentation was selected several months prior to the start of superstructure casting operations in order to prepare instrumentation to be included in the casting of the chosen segments. The important criteria for selecting the instrumented span was:

- beginning the research early in the construction sequence
- accessibility of the span
- length of the span
- location of the expansion joints
- location of the substructure study

Span D-5 was the final choice for the thermal gradient study. This span was the fifth to be erected and met the other important criteria. The first few spans erected were avoided in order to minimize the usual start-up construction problems. The location of span D-5 is shown in Figure 3.1. In the following sections all instrumentation devices are shown and discussed.

Figure 3.1: Span D-5 Location.

3.2 CONCRETE STRAIN DEVICES

Concrete strain measuring devices were manufactured in the Ferguson Structural Engineering Laboratory. These strain devices provided an electrical signal which could be read by a continuously operating computer based data logger. This allowed data collection to be achieved without anyone present. In addition the strain device could be incorporated into the interior of the concrete as opposed to being mounted on the surface as with other traditional concrete strain measuring devices. The surface type concrete strain gages are often damaged. Prefabrication of the devices under laboratory conditions would allow for a greater percentage of gages to be working in the field. Laboratory fabrication also allowed for pre-testing of each gage to insure accuracy.

3.2.1 Background

Detailed information about concrete strain measuring devices can be found in a study by Arréllaga [9]. The concrete strain devices used in this study are a modification of a device used in research by Stone and are similar to a modified Mustran cell as described by Arréllaga [9].

3.2.2 Description

The concrete strain device consists of a small electrical resistance strain gage bonded to a small steel rod which is threaded on each end to allow for nuts and washers to be placed on each end. A plastic sleeve is provided so that only the end nuts and washers bond to the concrete. This strain device is embedded in the concrete at time of casting. This device is shown in Figure 3.2.

The cold rolled steel rod is a 4.75mm diameter rod approximately 235mm long and is threaded approximately 20mm on each end. The effective gage length between washers is 203mm. The strain gage is a small 350 ohm electrical resistance strain gage. The strain gauge selected for this device compensates for thermal changes in the gauge thus eliminating another variable eliminate from data recorded. This strain gage was bonded to the center of the steel rod. An acrylic coating was applied to the strain gage after the electrical leads were installed and isolated from the steel rod. Approximately 165mm of heat shrink plastic tubing was slipped over the strain gage, leads, and steel rod. The nuts were #10x32 USC and the washers were approximately 25mm in diameter. The nuts and washers were placed on each end and tightened to yield the 203mm gage length. The heat shrink tubing was shrunk to hold wire leads in place and protect the strain gage. The tubing serves an additional purpose of debonding the steel bar to accurately measure average strain over the gage length. Finally a coating of epoxy was applied to the ends of the heat shrink tubing for waterproofing. The final assembly was tested in a loading ram for soundness and accuracy.

Electronic strains gauges are basically small resistors. When strains occur in this type of resistor a change in its resistance occurs. The strain vs. resistance change of the resistors is known. These resistors are bonded to the material to which the desired strain is to be measured. A current is then applied to a closed loop containing the resistor. The resulting voltage drop is recorded and subsequent changes in voltage drop can indicate strains in the material. The data aquisition system measures this drop by applying an excitation voltage through a wheatstone bridge circuit comprised of several 350 ohm resistors added to the data aquisition system. Figure 3.3 shows a schematic for these electronic strain gages.

3.3 THERMOCOUPLES

All measurement of temperatures in the concrete were achieved using thermocouples. These very simple instruments are very reliable and accurate even under the worst of conditions. Thermocouples installed in the Downtown San Antonio "Y" Project by Roberts [3] are still functioning five years after their installation.

Figure 3.2: Concrete strain gage.
(after Bonzon[10])

Figure 3.3: An electronic stain gage.
(after Bonzon[10])

3.3.1 Background

When two wires composed of dissimilar metals are connected at both ends and one of the ends is heated, a continuous current flows in the thermoelectric circuit. Thomas Seebeck made this discovery in 1821. If this circuit is broken at the center, the net open circuit voltage is a function of junction temperature and the composition of the two metals (see Figure 3.4). All dissimilar metals exhibit this effect. The voltage change across the broken circuit cannot be measured directly with a voltmeter since the voltmeter leads create an additional thermoelectric circuit. The data acquisition system chosen can measure the net open circuit voltage by compensating for the additional circuitry and providing another way to measure the external reference temperature at the unit.

3.3.1 Description

The thermocouple chosen for this application is the type T wire. The two dissimilar metals used in this type thermocouple are copper and constantan. This type thermocouple was selected for the almost linear temperature vs. voltage curve in the range of temperatures in the structure and for its compatibility with the concrete environment. The wire length was predetermined and cut. The end was twisted and soldiered. The other end was wired directly to the multiplexer.

Figure 3.4: Thermocouples

3.3 DATA ACQUISITION SYSTEM

All gauges in Segment D5-9 were connected to a data acquisition system located inside of span D5. This system was preassembled and tested in the laboratory prior to installation.

The data recorder used for collection of data on this project was a Campbell 21X Data Logger. This data logger was connected to several Campbell AM416 Multiplexers that functioned as switching relays to allow as many as 128 devices to be linked to one data logger. The 21X memory capacity required researchers to upload the data from the multiplexer every 10 days.

The Cambell equipment was powered by a 12 volt marine use deep-cycle battery. With alternate batteries the equipment could be powered for about 6 months between battery charges. The researchers had to access the inside of the box girder from the end span and crawl through 4 spans or climb a 9m extension ladder into a small access hole located on the bottom slab of the box. The data acquisition system was housed in a painted plywood box locked in an expanded metal housing which was bolted to the web of the box (See Figure 3.5).

Figure 3.5: Data recording equipment inside of Span D5.

CHAPTER 4

INSTRUMENTATION IMPLEMENTATION AND SURVEILLANCE

4.1 THERMOCOUPLE LOCATIONS

The selection of thermocouple locations was based on consideration of two dimensional thermal gradients in the box rather than three dimensional thermal gradients. Two previous studies (Roberts - Wollmann and Hoffman, McClure and West) suggested that emphasis be given to the vertical and horizontal gradients since they are larger and more important to the behavior of the structure than the longitudinal gradients. Therefore a large number of thermocouples located at one segment in the box would be more beneficial to the determination of vertical gradients than an equivalent number of thermocouples spread at various points along the box. The midspan segment of a span was chosen for its distance from any box cross-sectional deviations and their possible influence. Figure 4.1 shows the general location of the instrumentation in span D5 for this study.

One aspect of thermal gradients that appears to often have been ignored by design specifications and by designers is the effect of non-linear thermal gradients across the slabs and webs which affects the transverse design of the slabs and webs. This area of research seems to have been ignored in previous studies. At several locations in the top slab, bottom slab and webs, thermocouples were placed across the slab to record these gradients.

There were several problems in selecting the number of thermocouples to be used and their locations. Large numbers of thermocouples would reduce the amount of time the data collection system could store data without overwriting. In addition selection of critical locations within the box cross-section would ultimately control the number of thermocouples selected. A balance between the number of desired thermocouples and practical limits would have to be achieved. Redundancy was not considered critical to the selection process because of the past performance history of the type of thermocouples which had been selected.

Figure 4.1: General location of all instrumentation in this study.

The final choice was 35 Type T (copper vs. constantan) thermocouples. These thermocouples were installed in one vertical plane through the midspan segment at varying locations in the top slab, bottom slab, and webs to try to obtain as much data as possible. There was some overlap in locations for data verification and in case of thermocouple failure. These locations are shown in Figure 4.2.

Figure 4.2 Thermocouple Locations

4.2 TRANSVERSE STRAIN GAUGE LOCATIONS

Transverse strain gauge devices were needed to determine the magnitude of strains occurring as the result of non-linear thermal gradients across the depth of the slabs and webs. These devices were located in a vertical plane approximately 152mm from the plane of thermocouples in the same midspan segment. These devices were placed at expected peak transverse stress locations (resulting from the thermal gradients) in the webs and slabs of the box. The locations of these devices are shown in Figure 4.3

Figure 4.3 Transverse Strain Gauge Locations

4.3 LONGITUDINAL STRAIN GAUGE LOCATIONS

Longitudinal strain gauge devices were needed to determine the magnitude of strains occurring as the result of non-linear thermal gradients through the depth of the superstructure. These devices were located in the same vertical plane as the thermocouples in the same midspan segment. These devices were placed at various locations throughout the web and slabs of the box to provide a representation of the longitudinal self-equilibrating strains resulting from the non-linear thermal gradients in the webs and slabs of the box. The locations of these devices are shown in Figure 4.4.

Figure 4.4: Longitudinal Concrete Strain Gauge Locations

4.4 DATA ACQUISITION SYSTEM

The central data acquisition system was mounted in a self-contained box in the void of the box in span D5. This location allowed for moderately easy access to obtain data and was well protected from the elements. The data was downloaded directly into an Intel based laptop computer. The downloading of data was easy and quick. The data acquisition was very reliable. Figure 4.5 shows the installed data acquisition system.

4.4 INSTALLATION

Installation of the thermocouples and strain gauge devices was accomplished in several stages. Careful planning and forethought went into the many months of preparations required for the total data collection system to work in place. The contract for the bridge contractor required his cooperation in providing access for the research team to the casting of segments and piers as well as the erection of the segments and casting of the joints.

Figure 4.5: Installed data acquisition system in-place in segment D5-9

The first stage of installation was the preliminary system selection described in Chapter 3. This involved selection of the thermocouples, concrete strain measuring devices, wiring, and data collection system. Additionally, planning of device location, data acquisition location, and conduit locations was accomplished. The total system for data collection was pre-assembled in the lab and tested for correct device location, programming accuracy, and proper device function.

Field installation started in the casting yard. The contractor was informed of the segment selected for instrumentation. They pre-assembled the reinforcing cage and set it aside for instrument installation several days prior to casting of the segment. The pre-assembled strain gauge devices with wiring and the thermocouple wiring assemblies were attached to the reinforcing cage (cage). The wires were carefully routed and attached to the cage to prevent damage during casting. The wires were all routed to aluminum ducts which exited the top of the bottom slab into the interior of the box. The additional wire length provided for connection to the data collection system was temporarily protected in plastic containers. The contractor then set the cage into the casting bed and adjusted the cage for proper clearances. The distances to each device were recorded after placement and checked again prior to casting. Adjustments were made if needed to device locations. The concrete was placed in the forms and cured. The segment was removed from the forms the next morning. Figure 4.6 shows the reinforcing cage during gauge installation prior to casting.

Figure 4.6: Gauge installation on Segment D5-9 reinforcing cage prior to casting.

Each device was tested after curing to insure proper functioning. Devices not working were noted. This segment was stored from September 26, 1994 to the last week of March 1995 when it was delivered to the construction site. Span D-5 was erected and the completed span set on it's bearings on March 30, 1995 at 2:24 p.m. The data recording systems had been pre-assembled and connected to the instruments. From that point on continuous readings have been taken on the thermocouples and strain gauge devices. There have been short lapses in the data collected when the data collection system reached it's maximum storage capacity and overwrote data.

4.5 WEATHER CONDITIONS

Weather information was provided by the Local Climatological Data Monthly Summary.

4.6 COEFFICIENT OF THERMAL EXPANSION

The three concrete beam molds were made from concrete sampled during casting of segment D5-9. The molds were cured under similar conditions to the segment. Demec extensometer points were attached to the four sides of the representative concrete beams in order to determine strain changes during thermal cycling. On August 7-8, 1995 the samples were thermally cycled to determine the thermal coefficient of expansion. The concrete beam specimens were placed in the heating chamber at 48.89^{o}C and 36% relative humidity for approximately 70 hours and then measured by the extensometer. The specimens were then placed in the cooling chamber at 4.44^{o}C and 90% relative humidity for approximately 18 hours. After 18 hours in the cooling chamber they were measured by the extensometer. The microstrain change over the temperature range and coefficient of thermal expansion was calculated (See Table 4.1). The average coefficient of thermal expansion for this segment was 9.79 x 10^{-6} m/m/^{o}C. For the remainder of this document the coefficient of thermal expansion used in calculations will be 1.0 x 10^{-5} m/m/^{o}C.

Specimen No. 1

Side #	Demec Gauge Reading at Temp. °C		ΔStrain	Coef. of Ther. Exp.
	4.44	48.89	m/m	(m/m/°C)
Initial Zero	671	669		
1	549	663	0.000456	0.00001026
2	978	1084	0.000424	0.00000954
3	840	950	0.000440	0.00000990
4	528	634	0.000424	0.00000954
Final Zero	671	669		
			Average Value of α =	0.00000981

Specimen No. 2				
Side #	Demec Gauge Reading at Temp. °C		ΔStrain	Coef. of Ther. Exp.
	4.44	48.89	m/m	(m/m/°C)
Initial Zero	671	669		
1	738	849	0.000444	0.00000999
2	716	826	0.000440	0.00000990
3	921	1035	0.000456	0.00001026
4	496	605	0.000436	0.00000981
Final Zero	671	668		
			Average Value of α =	0.00000999

Specimen No. 3				
Side #	Demec Gauge Reading at Temp. °C		ΔStrain	Coef. of Ther. Exp.
	4.44	48.89	m/m	(m/m/°C)
Initial Zero	670	669		
1	237	345	0.000432	0.00000972
2	850	962	0.000448	0.00001080
3	852	948	0.000384	0.00000864
4	644	753	0.000436	0.00000981
Final Zero	670	669		
			Average Value of α =	0.00000974

Table 4.1: Coefficient of Thermal Expansion test results for span D5-9 concrete beam specimens.

CHAPTER 5

DATA PRESENTATION

5.1 GENERAL DATA INFORMATION

Temperatures for thermocouples and concrete strains were electronically recorded hourly during an approximately ten month period from March 22, 1995 through January 15, 1996. Only short lapses in continuous data collection occurred due to limitations in the data collection equipment and data retrieval. A small amount of data was recorded while the segment containing the thermocouples was stored in the casting yard.

5.2 POSITIVE THERMAL GRADIENTS THROUGH THE BOX DEPTH

Positive thermal gradients in concrete box girders of a given shape and material are most affected by daily temperature fluctuations, solar radiation, and wind speed. Typical winged trapezoidal box girders experience well-defined positive temperature gradients during days of large solar radiation. This structure is no exception. The top slab of the girder experiences large temperature swings while the protected bottom slab tends to remain unaffected. Peak positive temperature gradients tend to occur in the spring when nighttime temperatures are still relatively cool, daytime solar radiation is high, and wind speeds are relatively low. These gradients appear to be largest when longer periods of cooler ambient temperatures are followed by the large solar radiation days.

Figure 5.1 shows the temperature as a function of time for three selected thermocouples in the 48 hour period during the 10 months of observation in which the maximum positive temperature gradient occurred. During this time the surface of the structure was untopped. These temperatures illustrate the large temperature changes of the top of web versus the middle of the web and bottom of web.

The maximum positive temperature gradient occurred on May 20, 1995 at approximately 4:00 PM. Figure 5.2 shows the positive gradient that occurred at that time as

Figure 5.1 Web Temperatures at Selected Locations in the Right and Left Web During the 48 Hour Period of Maximum Positive Temperature Gradient.
May 20, 1995

Figure 5.2 Peak Positive Temperature Gradients through the Box Depth.
May 20, 1995

measured along the centerline of the left and right web of the structure. The shape of the gradient is typical for the larger positive temperature gradients. Notice the largest gradient recorded in the left web is approximately 16.4^o C. However this temperature is recorded 38mm below the surface of the top slab and would be higher if recorded at the surface of the top slab.

5.3 NEGATIVE THERMAL GRADIENTS THROUGH THE BOX DEPTH

Negative thermal gradients in concrete box girders of a given shape and material are most affected by daily temperature fluctuations, concrete material, and wind speed. Typical winged trapezoidal box girders experience well-defined negative temperature gradients during days of large temperature drops accompanied by high wind speeds and or rain which usually occurs with passage of cold fronts. This structure is no exception. As shown in Figure 5.3, the top slab of the girder experiences large downward temperature swings while the relatively protected webs and bottom slab (only the outside of the box receives cooling winds across the surface) tend to remain less affected. Peak negative temperature gradients tend to occur in the fall through spring when downward temperatures swings are largest. These gradients appear to be largest when longer periods of moderate to warm ambient temperatures are followed by the passage of large cold fronts. These gradients tend to peak in the early morning hours prior to significant solar radiation.

Figure 5.3 shows the temperature as a function of time for three selected thermocouples in the 48 hour period during the months of observations in which the maximum negative temperature gradient occurred. These temperatures illustrate the temperature lag of the middle of the web versus the top of the web and bottom of the web. The rise in temperature of the top of web thermocouple is related to the daily solar radiation.

The maximum negative temperature gradient occurred on January 18, 1996 at approximately 7:00 AM. Figure 5.4 shows the negative gradient that occurred at that time along the centerline of the left and right webs of the structure. The shape of the gradient is typical for the larger negative temperature gradients. Notice the largest negative temperature gradient recorded in the left web is approximately 6.8^o C. However this temperature is recorded 38mm below the surface of the top slab and would be higher if recorded at the surface of the top

Figure 5.3 Web Temperatures at Selected Locations in the Right and Left Web During the 48 Hour Period of Maximum Negative Temperature Gradient.
January 18, 1996

Figure 5.4 Peak Negative Temperature Gradients through the Box Depth.
January 18, 1996

slab. Note also that positive temperature difference values of 1.5^{o} C were measured at 1500mm above the bottom soffet. The NCHRP gradient of Figure 2.4 does not show any positive values.

5.4 WEB AND SLAB GRADIENTS

The webs and slabs of the superstructure experience non-linear thermal gradients both positive and negative under the same conditions that the superstructure as a whole does. These gradients cause equilibrium stresses and strains as well as restraint moments. These gradients from the same days as the maximum positive and negative gradients are presented in the following sections.

5.4.1 Positive Temperature Gradients through Slabs and Webs

The peak positive temperature gradient through a slab or web occurs in the top slab. As shown in Figure 5.5, that peak gradient is 17.5 ^{o}C which occurred in the middle of the top slab. For each location there are only three points of data with the outer gauges at least 25mm or more below the surface. The surface temperatures would be significantly higher. However, only the data recovered is presented here. Figure 5.5 shows these gradients that occurred at 4:00 PM on May 20, 1995. These gradients are adjusted to illustrate differences assuming zero at the center of web or slab for illustration purposes.

Figure 5.5: Peak positive temperature gradients through slabs and webs.
May 20, 1995

5.4.2 Negative Temperature Gradients through Slabs and Webs

The peak negative temperature gradient through the slab or web occurs to the far right side of the interior top slab. That peak gradient is 11.0 ^{o}C. For each location there are only three points of data with the outer gauges at least 25mm or more below the surface. If the surface temperatures were recorded the gradients would most likely be somewhat higher. Figure 5.6 shows the gradients that occurred at 7:00 AM on January 18, 1996. These gradients are adjusted to illustrate differences assuming zero at the center of web or slab for illustration purposes. Please note that gauge number T33 which unfortunately is at the center of the top slab at the location of the highest negative gradient was not working on this day and that only the interior and exterior gauges were recorded at this location.

Figure 5.6: Peak negative temperature gradients through slabs and webs.
January 18, 1996

5.5 TRANSVERSE CONCRETE STRAINS

Transverse concrete strain gauges in segment D5-9 are located in a vertical plane approximately 152mm from the plane of thermocouples in the same midspan segment. These devices were placed at expected peak transverse stress locations (resulting from the thermal gradients) in the webs and slabs of the box. For more detailed information on the locations of these devices see Chapter 4.

Loading from thermal gradients in the transverse direction of concrete box bridges is often ignored. In general thermal gradients are large only through the depth of the top slab. These top slab thermal gradients should have the largest effect on transverse strains and design.

Figure 5.7 shows a plot of hourly temperature and strain changes on May 20, 1995, of concrete strain devices C3 and C4 located in the top slab at the web top slab intersection. Device C3 is located near the top of the slab and device C4 is located near the bottom of the slab. Thermocouple devices T34 and T35 are located in the nearby parallel plane of devices at the same slab location. Device T34 is located near the top of the slab and device T35 is located near the bottom of the slab. It should be noted that the electronic strain gauges selected for this project compensated for most of the strain effects due to temperature change in the concrete. That is, thermal strains from concrete contraction or expansion are eliminated by the gauge itself. All strain measurements reflect strains due to externally applied forces or restraint forces only. Notice from the graph that strain changes occur as the temperature rises or falls at device locations. The maximum compressive strain that occurs is approximately 10 percent of the maximum compressive strain allowed in a linear stress/strain design for post-tensioned concrete.

Figure 5.7: Temperature and strain changes on May 20, 1995
Top slab at intersection of right web.

Figure 5.8 shows a plot of hourly temperature and strain changes on May 20, 1995, of concrete strain devices C5 and C6 located in the top of the right web at the web/top slab intersection. Device C5 is located near the inside of the web and device C6 is located near the outside of the web. Thermocouple devices T9, T10, and T11 are located in the nearby parallel plane of devices at the same web location. Device T9 is located near the center of the web. Device T10 is located near the inside of the web. Device T11 is located near the outside of the web. The strain changes correspond well with the changes in temperature. As expected the strain changes are much smaller in the webs than the top slab resulting from the smaller temperature changes in the web.

Figure 5.8: Temperature and strain changes on May 20, 1995
Top of right web at intersection of top slab.

Figure 5.9 shows a plot of hourly temperature and strain changes on May 20, 1995, of concrete strain devices C7 and C8 located near the middle of the right web. Device C7 is located near the inside of the web and device C8 is located near the outside of the web. Thermocouple devices T12, T13, and T14 are the closest thermocouple devices located in the nearby parallel plane of devices located slightly closer to the bottom of the web. Device T12 is located near the center of the web. Device T13 is located near the inside of the web. Device T14 is located near the outside of the web. The strain changes correspond well with the changes in temperature. As before, the expected strain changes are much smaller in the webs than in the top slab resulting from the smaller temperature changes in the web.

Figure 5.9: Temperature and strain changes on May 20, 1995
Near the middle of the right web.

Figure 5.10 shows a plot of hourly temperature and strain changes on May 20, 1995, of concrete strain devices C9 and C10 located near the bottom of the right web. Device C9 is located near the inside of the web and device C10 is located near the outside of the web. Device C9 was obviously not properly functioning on the day this information was recorded. Thermocouple devices T12, T13, and T14 are located in the nearby parallel plane of devices at the same web location. Device T12 is located near the center of the web. Device T13 is located near the inside of the web. Device T14 is located near the outside of the web. The strain changes correspond well with the changes in temperature. As before, the expected strain changes are much smaller in the webs than the top slab resulting from the smaller temperature changes in the web.

Figure 5.10: Temperature and strain changes on May 20, 1995

Near the bottom of the right web.

Figure 5.11 shows a plot of hourly temperature and strain changes on May 20, 1995, of concrete strain devices C11 and C12 located in the bottom slab at the intersection with the right web. Device C11 is located near the top of the bottom slab and device C12 is located near the bottom of the bottom slab. Thermocouple devices T15, T16, and T17 are located in the nearby parallel plane of devices at the same web location. Device T15 is located near the center of the bottom slab. Device T16 is located near the top of the bottom slab. Device T17 is located near the bottom of the bottom slab. The strain changes correspond well with the changes in temperature. As before, the expected strain changes are much smaller in the webs than the top slab resulting from the smaller temperature changes in the web.

Figure 5.11: Temperature and strain changes on May 20, 1995
Near the intersection of the bottom slab and right web.

Figure 5.12 shows a plot of hourly temperature and strain changes on May 20, 1995, of concrete strain devices C13 and C14 located at the midpoint of the bottom slab. Device C13 is located near the top of the bottom slab and device C14 is located near the bottom of the bottom slab. Thermocouple device T18 is located in the nearby parallel plane of devices at the same web location. Device T18 is located near the center of the bottom slab. The strain changes correspond well with the changes in temperature. The strain changes indicate a net compression in the bottom slab. These strains will be compared to traditional analysis in Chapter 6 to see if this is realistic.

Figure 5.12: Temperature and strain changes on May 20, 1995

At the middle of the bottom slab.

5.6 LONGITUDINAL CONCRETE STRAINS

Longitudinal concrete strain gauges located in segment D5-9 are located in a vertical plane approximately 152mm from the plane of thermocouples in the same midspan segment These devices were placed at intervals down the web and across the top slab to verify predicted strains resulting from the thermal gradients through the structure depth. For the purpose of this study only results from devices C18, C22, C23, and C24 will be presented here. For more detailed information on the locations of these devices see Chapter 4.

Figure 5.13 shows a plot of hourly temperature and strain changes on May 20, 1995, of concrete strain devices C18, C22, C23, and C24 located vertically along the outside of the right web. Device C18 is located near the top surface of the structure in the plane of the web. Device C22 is located near the top quarterpoint of the web. Device C23 is located near the bottom quarterpoint of the web. Device C24 is located near the bottom slab/web intersection. Thermocouple device T5 is near the location of device C18. Thermocouple T9 is located near device C22. Thermocouple T12 is located near device C23. Thermocouple T15 is located near device C24. The strain changes correspond well with the changes in temperature. As anticipated, the largest strain changes occur in the top and bottom slabs. These strains are relatively small. Since the structure is simply supported at this point in time, strains due to restraint cannot occur. These strains will be compared to a traditional analysis in Chapter 6.

Figure 5.13: Temperature and strain changes on May 20, 1995
Through the depth of the right web.

CHAPTER 6

ANALYTICAL STRESS MODEL

6.1 LONGITUDINAL ANALYSIS

Results of a traditional longitudinal analysis in which temperature gradients are input as the basic actions will be compared to data measured by the concrete strain gauge devices in order to determine if traditional hand methods yield reasonable results for structural response to real temperature gradients.

6.1.1 Applied Positive Temperature Gradient

The maximum positive temperature gradient which was observed during the period of this study occurred on May 20, 1995 at approximately 4:00PM. This gradient however cannot be directly applied as a loading to an analytical model. The difference in temperatures at any point over a given time period is the loading that needs to be applied. Ideally one would like to start with a period of zero gradient. However, this is impractical under real conditions. Since the measured strains and temperatures are monitored on a continuous hourly basis, a time period in which the temperature gradient is very minimal is a reasonable starting point. The time period selected for the period of least temperature gradient is 8:00AM which occurred 8 hours prior to the measured peak positive gradient. Temperatures occurring at 4:00PM will be subtracted from those occurring at 8:00AM on the above mentioned date. The difference will then be applied as a loading on the model. One will be able to compare the strains predicted by the model to the strain changes occurring over the 8 hour time period. The applied temperature gradient is shown graphically in Figure 6.1.

6.1.2 Model

The model used for longitudinal analysis is presented in Chapter 1 and was used by Imbsen, et al [2] and Roberts [3]. This analysis is used to determine the superstructure longitudinal self-equilibrating stresses and strains which are induced by the thermal gradients in the structure. This method is commonly used during superstructure design to locate potential areas of tensile stress. Example calculations for the superstructure can be found in the Appendix. All calculations are for 1/2 of the total structure (Forces, moments, I, A, etc.). Please note that in the example, the summation of moments is taken about the bottom extreme fiber of the structure and then resolved into a moment about the centroid.

6.1.3 Longitudinal Self-equilibrating Stresses and Strains

Figure 6.1 shows the temperature and stress distributions calculated for the aforementioned time period on May 20, 1995. This temperature change is applied to a fully restrained superstructure. The restraint axial force and moment are then calculated. These restraints are resolved into stress distributions and subtracted from the initial temperature stress distribution. The resultant is the self-equilibrating forces shown in Figure 6.1.

Strain change results from the analysis and the measured changes for each gauge are compared below:

Gauge Number	Analysis strain change($1.0x10^{-6}$)	Measured strain change($1.0x10^{-6}$)
C18	15	35
C22	-62	-34
C23	29	19
C24	39	21

All the measured strain changes are in the correct direction. With the exception of gauge C18 the measured values are lower than analysis values. One can see from the analysis stress distribution that gauge C18 falls on a very steep gradient of the stress diagram. If the gauge was moved up 25mm during the concrete pour the traditional analysis shows the strain would be approximately 35 microstrain. The other three measured values vary from 54% to 65% of the analysis values.

Figure 6.1: Calculation of self-equilibrating forces induced by the maximum positive temperature gradient, May 20, 1995

6.2 TRANSVERSE ANALYSIS

The transverse design and analysis of segmental concrete box structures are often very complex. The analysis typically requires the use of finite element programs because of the complex distribution of post-tensioning stresses due to tendon spacing and changing cross-section due to internal diaphragms. In addition wheel loadings occurring on a two dimensional plane complicate the analysis. The AASHTO Segmental Code [6] does not specifically require that the loading due to temperature gradients be applied to the transverse analysis of the box. With all of the other complexities in the transverse analysis the thermal gradient loading is often overlooked by designers. Results of a traditional analysis will be compared to data measured by the concrete strain gauge devices to determine if traditional methods yield reasonable results for structural response for real temperature gradients.

6.2.1 Applied Positive Temperature Gradient

The maximum positive temperature gradient which was observed during the period of this study occurred on May 20, 1995 at approximately 4:00PM. As discussed previously in this chapter, the temperature gradient cannot be directly applied as a loading to an analytical model. The temperature differential from the time period discussed in Section 6.1.1 on May 20, 1995 will be applied in the analytical model. One will be able to compare the strains predicted by the model to the strain changes occurring over the 8 hour time period. The applied temperature gradient is shown graphically on Figure 6.2.

Figure 6.2: Transverse Positive Temperature Gradients

6.2.2 Model

The model used for this transverse analysis is similar to that used for longitudinal analysis in Section 6.1. The main difference will be that since there are only three thermocouple measurements in the depth of each member (slab or web) then a three point non-linear gradient will be applied to each section and the resulting restraint forces for each discrete section (slab or web) will be determined. These restraint forces will then be applied to a 2-dimensional plane frame model comprised of the global assembly of transverse box elements. This analysis is used to determine the superstructure transverse stresses and strains which are induced by the thermal gradients in the structure.

6.2.3 Transverse Self-equilibrating Stresses and Strains

The temperature and stress distributions were calculated for each discrete element for the aforementioned time period, May 20, 1995. These temperature changes were applied to each fully restrained discrete element of the box. The restraint axial forces and moments are then calculated. These restraints are applied to the frame model. The results of the frame program are reduced to stresses. The model for the frame program is shown in Figure 6.3. The transverse concrete strain gauge device locations are shown in Figure 6.4. The final stresses and strains from the traditional analysis are shown in Table 6.1. In Table 6.2, the strains from the analysis are compared against the strain changes recorded from concrete strain gauge devices on May 20, 1995.

Figure 6.3: Transverse stress-strain analysis frame model.

Figure 6.4: Transverse concrete strain gauge device locations.

MEMBER	NODE	RESTRAINT STRESSES		S.E. STRESSES	
NUMBER	NUMBER	GAUGE LOCATION		GAUGE LOCATION	
		OUTSIDE	INSIDE	OUTSIDE	INSIDE
		MPa	MPa	MPa	MPa
1	1	-0.484	0.680	-0.160	-0.170
	2	-0.484	0.680	-0.160	-0.170
2	2	-0.368	0.515	-0.190	-0.110
	3	-0.716	0.863	-0.190	-0.110
3	3	-0.499	0.550	-0.190	-0.270
	4	-1.016	1.067	-0.190	-0.270
4	4	-1.016	1.067	-0.190	-0.270
	5	-1.533	1.583	-0.190	-0.270
5	5	-0.985	0.898	0.000	0.000
	6	-1.200	1.113	0.000	0.000
6	6	-4.564	4.350	0.320	-0.230
	7	-4.564	4.350	0.320	-0.230
7	7	-4.564	4.350	0.310	0.490
	8	-4.564	4.350	0.310	0.490

TOTAL TRANSVERSE TEMPERATURE GRADIENT ANALYSIS STRESS AND CORRESPONDING STRAIN					
MEMBER	NODE	FINAL STRESS		FINAL STRAINS	
NUMBER	NUMBER	GAUGE LOCATION		GAUGE LOCATION	
		OUTSIDE	INSIDE	OUTSIDE	INSIDE
		MPa	MPa	MICROSTRAIN	MICROSTRAIN
1	1	-0.644	0.510	-15	12
	2	-0.644	0.510	-15	12
2	2	-0.558	0.405	-13	9
	3	-0.906	0.753	-21	18
3	3	-0.689	0.280	-16	7
	4	-1.206	0.797	-28	19
4	4	-1.206	0.797	-28	19
	5	-1.723	1.313	-40	31
5	5	-0.985	0.898	-23	21
	6	-1.200	1.113	-28	26
6	6	-4.244	4.120	-99	96
	7	-4.244	4.120	-99	96
7	7	-4.254	4.840	-99	113
	8	-4.254	4.840	-99	113

Table 6.1: Stresses and strains from transverse temperature gradient analysis.

RESULTS FROM TRANSVERSE TEMPERATURE GRADIENT ANALYSIS			
GAUGE	STRAIN FROM	STRAIN RECORDED AT GAUGE LOCATION	
LOCATION	ANALYSIS	NEAR OUTSIDE	NEAR INSIDE
	*	OF BOX	OF BOX
	MICROSTRAIN	MICROSTRAIN	MICROSTRAIN
C14	-15	-10	
C13	12		**
C12	-14	-12	
C11	11		18
C10	-18		-9
C9	-12	**	
C8	-28		-16
C7	19	5	
C6	-32		-32
C5	26	8	
C4	61	39	
C3	-63		-36
C2	113	**	
C1	-99		**

** Values are averaged at corners of box.*
*** Gauges not working on day of peak stress*

Table 6.2: Comparison of strain from transverse analysis and gauge recordings.

The strains from the analysis compare fairly well with the recorded strain changes. All the strains are in the correct direction. The maximum variance from the analysis strains when compared to the recorded strains is 74%. The average variance when using the absolute values is 46.7%. Nine of the ten strains recorded are smaller than the analysis values. Four of the transverse concrete strain gauge devices were not working on this date. It is unfortunate that two of the four gauges which were not working were at the location of peak predicted strain (midspan of the top slab).

As expected the transverse element most affected by the large positive thermal gradients in the structure is the top slab. The predicted stress from the positive thermal gradient alone in the top slab is about 29% of the current allowable compressive stress for a 42MPa ultimate strength post-tensioned segmental concrete. The peak predicted and recorded stress in the web and bottom slab is less than 5% of the allowable compressive stress for a 42MPa ultimate strength post-tensioned segmental concrete. From the analysis it became apparent that the axial restraint of the box upon the top slab and upon the total thermal stress or strain is very small. The large majority of stress and or strain is the result of the restraint (secondary) moment.

CHAPTER 7

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

7.1 SUMMARY

This project is only part of one phase of field instrumentation and monitoring of the new U.S. 183 segmental post-tensioned concrete viaduct in Austin, Texas. This portion of research specifically focuses on the thermal gradients and the structural response of those gradients experienced by the mainlane bridge structure. This structure is a segmentally-constructed, post-tensioned, concrete, hollow box bridge. The segment being studied is part of a bridge unit that is partially continuous over three spans in its final configuration. In the initial phase of construction the segment is within a span that is simply supported. In this portion of the research concrete strain gauge devices and thermocouples were installed in one segment of the mainlane superstructure to monitor the thermal gradients in the slabs and webs and vertically through the depth of the box structure This portion of this phase of the mainlane structure instrumentation reports data collected from the span's erection in March of 1995 to February of 1996.

Other phases of research on this project not included in this report are additional mainlane superstructure span, mainlane cast-in-place concrete pier [14], segmentally constructed, post-tensioned, concrete, hollow box bridge pier [10], and balanced-cantilever ramp superstructure span instrumentation.

A set of 35 thermocouples were installed in one vertical plane through the midspan segment at varying locations in the top slab, bottom slab, and webs to try to obtain as much data as possible. The thermocouples were very reliable and provide very good data concerning the temperature variations throughout the webs, slabs, and depth of the box.

A set of 14 concrete strain gauge devices were placed in a vertical plane approximately 152mm from the plane of thermocouples in the same midspan segment. These devices were oriented with their primary axis in the transverse direction. Transversely placed concrete strain gauge devices were needed to determine the magnitude of strains occurring as the result of non-linear thermal gradients across the depth of the slabs and webs. These devices were placed at expected peak transverse stress locations (resulting from the thermal gradients) in the webs and slabs of the box. These concrete strain gauge devices work fairly well. A few were damaged in casting of the segment and some deteriorated with time. The gauges did record valuable data in establishing the strains as a result of transverse structural response to thermal gradients, primarily through the top slab.

A set of 15 concrete strain gauge devices were located in the same vertical plane as the thermocouples in the same midspan segment. These strain gauge devices with their primary axis oriented longitudinally were needed to determine the magnitude of strains occurring as the result of non-linear thermal gradients through the depth of the superstructure. Portions of these devices were also used to determine diffusion of stresses in the structure in another phase of research (not included in this report). Four of the 15 devices were placed at various locations throughout the web and slabs of the box to provide a representation of the longitudinal self-equilibrating strains resulting from the non-linear thermal gradients in the webs and slabs of the box.

A detailed description of location for all instrumentation included in this report is given in Chapter 4.

7.1.1 Positive Thermal Gradients Through The Depth of The Box

The temperature fluctuations occurring through the depth of the structure exhibit classic behavior. Peak positive temperature gradients tend to occur in the spring when nighttime temperatures are still relatively cool, daytime solar radiation is high, and wind speeds are relatively low. A positive gradient occurs when the top surface is warmer than the rest of the box. These gradients appear to be largest when longer periods of cooler ambient temperatures are followed by the large solar radiation days. The largest gradients were recorded between 4:00PM and 5:00PM. The general temperature fluctuations followed the daily ambient air temperature changes with some lag due to concrete's large thermal mass. The top slab is influenced largely by solar radiation during the daytime hours. Solar radiation accounted for the largest daily temperature fluctuations in any part of the segment instrumented. The poor thermal conductivity of concrete limited the temperature fluctuations of the concrete near the interior of the box. In addition the shade of the wings of the box prevented large temperature fluctuations in the webs and bottom slabs of the box due to solar radiation.

The data from the thermocouples was analyzed and the peak positive temperature gradient was determined for the time period of the study. The maximum positive temperature gradient occurred on May 20, 1995 at approximately 4:00 PM. The largest positive gradient that occurred at that time as measured along the centerline of the left web was approximately 16.4^{o} C. However this temperature was recorded 38mm below the surface of the top slab and would be higher if recorded at the surface of the top slab.

The positive thermal gradients recorded from the structure on the aforementioned date and the current AASHTO Segmental Design Guide [6] recommended design thermal gradient (Zone 2 - no topping) are compared in Figure 7.1. As can be seen from the graph the match of recorded data is close to the current design gradient. Most differences are very minor (usually less than 1^{o}C), except there is a noticeable difference near the bottom soffit. The design gradient is about 3^{o}C larger (warmer) at the bottom than the recorded values. The values recorded close to the top surface fall very near the design gradient.

Figure 7.1: Comparison of maximum recorded positive thermal gradient to the AASHTO Segmental Design Guide[6] design gradient (Zone 2 - no topping).

There is a proposed revision to the AASHTO Segmental Design Guide [6] positive thermal design gradient. The proposed revision includes the positive and negative thermal design gradients and loading combinations. The positive thermal gradients recorded from the structure on May 20, 1995 and the AASHTO Segmental Design Guide [6] newly proposed design thermal gradient (Zone 2 - no topping) are compared in Figure 7.2. As can be seen from the graph the match of recorded data is not very close to the newly proposed positive thermal design gradient. There is a noticeable difference from about 500mm above the bottom soffit to the deck. In this area the difference varies from about a 1^{o}C to as much as 10^{o}C larger (warmer) value recorded than the design gradient. In addition there is a noticeable difference in peak values. Using this design gradient would be unconservative for a continuous structure.

Figure 7.2: Comparison of maximum recorded positive thermal gradient to the AASHTO Segmental Design Guide[6] newly proposed design gradient (Zone 2 - no topping).

In general the self-equilibrating stresses are low compared to other stresses from applied loadings in the structure. In a continuous multi-span bridge, stresses from thermal gradients are largely caused by restraint stresses occurring as the result of continuity (see Chapter 1 for further information regarding restraint stresses). In addition there is usually little axial restraint in bridges of this type. The largest factor in determining the effects of thermal gradients is to look at the restraint moments produced from the model presented in Chapter 1 and Appendix A. In Table 7.1 restraint moments and restraint axial forces are presented for the peak thermal gradient recorded in the structure and are compared against the restraint moments and restraint axial forces which would be present under the design axial load for both the current and newly proposed AASHTO Segmental Bridge Design Guide [6]. One can clearly see that the current AASTHO positive thermal gradient (Zone 2 - untopped deck) more closely represents the values calculated from data recorded in the structure. The proposed AASHTO design gradient would be unconservative to use in this structure if it was fully continuous. The current AASHTO design gradient would be adequate for use in this structure if it was fully continuous. It is clear that the forces calculated from the current AASHTO positive thermal design gradient are much closer to the forces calculated from the recorded data than those calculated from the proposed positive thermal design gradient.

COMPARISON OF RESTRAINT MOMENTS AND AXIAL LOADS FOR POSITIVE THERMAL GRADIENTS		
Loading Type	**Axial Restraint Force***	**Restraint Moment****
	MN	**MN-m**
Data Recorded from Structure	**25.26**	**10.94**
Current AASHTO Design Grad.	**25.16**	**12.24**
Proposed AASHTO Design Grad.	**9.40**	**5.28**

* Positive values indicate compression.
** Positive values indicate compression in the top fiber.

Table 7.1 Comparison of restraint forces calculated from Design Thermal Gradient to those calculated from data recorded in structure.

7.1.2 Negative Thermal Gradients Through The Depth of The Box

Negative thermal gradients in concrete box girders of a given shape and material are most affected by daily temperature fluctuations, concrete material, and wind speed. In negative thermal gradients the top of the structure is colder than the rest of the structure. Typical winged box girders experience well-defined negative temperature gradients during days of large temperature drops accompanied by high wind speeds and or rain which usually occur with passage of cold fronts. This structure is no exception. The top slab of the girder experiences large downward temperature swings while the relatively protected webs and bottom slab (only the outside of the box receives cooling winds across the surface) tend to remain less affected. Peak negative temperature gradients tend to occur in the fall through spring when downward temperatures swings are largest and when longer periods of moderate to warm ambient temperatures are followed by the passage of large cold fronts. These gradients tend to peak in the early morning hours near dawn prior to significant solar radiation.

The data from the thermocouples was analyzed and the peak negative temperature gradient were determined for the time period of the study. The maximum negative temperature gradient occurred on January 18, 1996 at approximately 7:00 AM. The largest negative gradient that occurred at that time as measured along the centerline of the left web is approximately 6.8^{o}C.

However this temperature is recorded 38mm below the surface of the top slab and would be higher if recorded at the surface of the top slab.

The negative thermal gradients recorded from the structure on the aforementioned date and the current AASHTO Segmental Design Guide [6] recommended design thermal gradient (Zone 2 - no topping) are compared in Figure 7.3. As can be seen from the graph the match of recorded data is relatively close to the current design gradient. There is a noticeable difference from about 1100mm to about 1900mm above the bottom soffit. In this area there is about a 1^{o}C to 2^{o}C larger (warmer) value recorded than the design gradient. In addition there is a small difference from about 250mm to about 900mm above the bottom soffit. The values recorded close to the top surface fall very near the design gradient.

Figure 7.3: Comparison of maximum recorded negative thermal gradient to the AASHTO Segmental Design Guide[6] design gradient (Zone 2 - no topping).

As mentioned earlier there is a proposed revision to the AASHTO Segmental Design Guide[6] negative thermal design gradient. The negative thermal gradients recorded from the structure on January 18, 1996 and the AASHTO Segmental Design Guide [6] newly proposed negative design thermal gradient (Zone 2 - no topping) are compared in Figure 7.4. As can be seen from the graph the match of recorded data is not very close to the newly proposed negative thermal design gradient. There is a noticeable difference everywhere except the 200mm nearest the deck.

Figure 7.4: Comparison of maximum recorded negative thermal gradient to the AASHTO Segmental Design Guide[6] newly proposed design gradient (Zone 2 - no topping).

As discussed earlier in this chapter self-equilibrating stresses in general are low when compared to other stresses from applied loadings to the structure. In a continuous multi-span bridge, stresses from thermal gradients are largely the result of restraint stresses occurring as the result of continuity. In addition since there is usually little axial restraint in bridges of this type, the largest factor in determining the effects of thermal gradients is the restraint moments produced from the model presented in Chapter 1 and Appendix A. In Table 7.2 restraint moments and restraint axial forces are presented for the peak negative thermal gradient recorded in the structure. The forces from the peak negative thermal gradient are compared against the restraint moments and restraint axial forces which would be present under the design axial load for both the current and newly proposed AASHTO Segmental Bridge Design Guide[6]. The current AASHTO negative thermal design gradient would be slightly unconservative to use in this structure if it were fully continuous. The proposed AASHTO negative thermal design gradient produces a larger restraint moment, yet because of its shape would produce unusual self-equilibrating stresses. It is unclear which model would more closely match the forces calculated from the recorded data. Both the existing and proposed AASHTO negative thermal design gradient would probably be an acceptable design gradient for this structure.

COMPARISON OF RESTRAINT MOMENTS AND AXIAL LOADS FOR NEGATIVE THERMAL GRADIENTS		
Loading Type	**Axial Restraint Force***	**Restraint Moment****
	MN	**MN-m**
Data Recorded from Structure	**-14.66**	**-3.90**
Current AASHTO Design Grad.	**-10.56**	**-2.98**
Proposed AASHTO Design Grad.	**-9.40**	**-5.36**

* Positive values indicate compression.
** Positive values indicate compression in the top fiber.

Table 7.2: Comparison of restraint forces calculated from Design Thermal Gradient to those calculated from data recorded in structure.

7.1.3 Longitudinal Strains From Positive Temperature Gradient

The hollow post-tensioned box girder bridge span in this study was simply supported during the time period of this study. Therefore the strains resulting from longitudinal temperature gradients are small when compared to those which would result from the structural restraint caused by continuity. The existence of self-equilibrium strains is important in helping to establish the structural response to thermal gradients. A more detailed explanation of self-equilibrating strains can be found in Chapter 1.

Strains from a traditional longitudinal analysis using hand methods (in which temperature gradients are input as the basic actions) were compared to strains measured by the concrete strain gauge devices. This comparison shows reasonably close results. Temperatures occurring at 4:00PM on May 20, 1995 were subtracted from those occurring at 8:00AM on May 20, 1995. The temperature difference was then applied as a loading to the model. One can compare the strains predicted by the model to the strain changes occurring over the 8 hour time period The model used for longitudinal analysis is presented in Chapter 1 and was used by Imbsen, et al [2] and Roberts [7]. This analysis is used to determine the superstructure longitudinal self-equilibrating stresses and strains which are induced by the thermal gradients in the structure. Strain changes from the analysis and the measured changes for each gauge are compared in Table 7.3:

Guage Number	Analysis strain change($1.0x10^{-6}$)	Measured strain change($1.0x10^{-6}$)
C18	15	35
C22	-62	-34
C23	29	19
C24	39	21

Table 7.3: Comparison of strain changes calculated from longitudinal hand analysis of applied Temperature changes to those from data recorded in structure.

All the measured strain changes are in the correct direction. With the exception of gauge C18 the measured values are lower than analysis values. The other three measured values vary from 54% to 65% of the analysis values. Self-equilibrium strains and stresses are generally small and are of little consequence generally to design of these type structures. Yet their existence is important in helping to establish the structural response to thermal gradients.

7.1.4 Transverse Strains From Positive Temperature Gradient

The transverse design and analysis of segmental concrete box structures are often very complex. The analysis typically requires the use of finite element programs because of the complex distribution of post-tensioning stresses due to tendon spacing and changing cross-section due to internal diaphragms. In addition wheel loadings occurring on a two dimensional plane complicate the analysis. The AASHTO Segmental Code [6] does not specifically require that the loading due to temperature gradients be applied to the transverse analysis of the box. With all of the other complexities in the transverse analysis the thermal gradient loading is often overlooked by designers. Results of a traditional analysis were compared to data measured by the concrete strain gauge devices. This traditional methods yielded reasonable results for structural response for real temperature gradients.

The maximum positive temperature gradient which was observed during the period of this study occurred on May 20, 1995 at approximately 4:00PM. Temperatures occurring at 4:00PM on May 20, 1995 were subtracted from those occurring at 8:00AM on May 20, 1995. The temperature difference was then applied as a loading to the model.

The model used for this transverse analysis is somewhat similar to that used for longitudinal analysis. The restraint forces from a three point non-linear gradient on each element of the box were applied to a 2-dimensional plane frame model comprised of the global assembly of transverse box elements. This analysis is used to determine the superstructure transverse stresses and strains which are induced by the thermal gradients in the structure. The final stresses and strains from the traditional analysis are shown in Table 6.1. The strains from the analysis are compared against the strain changes recorded from concrete strain gauge devices on May 20, 1995. Table 6.2 compares the predicted strains from the analysis to the strains recorded from the concrete strain gauge devices.

The strains from the analysis compare fairly well with the recorded strain changes. All the strains are in the correct direction. The maximum variance from the analysis strains when compared to the recorded strains is 74%. The average variance when using the absolute values is 46.7%. Nine of the ten strains recorded are smaller than the analysis values. Four of the transverse concrete strain gauge devices were not working on this date. It is unfortunate that two of the four gauges which were not working were at the location of peak predicted strain (midspan of the top slab).

As expected the transverse element most affected by the large positive thermal gradients in the structure is the top slab. The predicted stress from the positive thermal gradient alone in the top slab is about 29% of the current allowable compressive stress for a 42MPa ultimate strength post-tensioned segmental concrete. The peak predicted and recorded stress in the web and bottom slab is less than 5% of the allowable compressive stress for a 42MPa ultimate strength post-tensioned segmental concrete. From the analysis it became apparent that the axial restraint of the box upon the top slab and upon the total thermal stress or strain is very small. The large majority of stress and or strain is the result of the restraint (secondary) moment. It is clear that inclusion of thermal gradients in the design of the top slab is critical to its performance.

7.2 CONCLUSIONS

Several conclusions regarding thermal gradients can be drawn based on the instrumentation and analyses of the segmentally-constructed, post-tensioned, concrete, hollow box bridge span:

1. Solar radiation is by far the largest factor influencing the positive thermal gradients. It produces the largest quickest changes in deck temperatures.

2. The largest negative temperature gradient during the time period of this report showed significant cooling of the bottom slab (in addition to the top slab), with the webs experiencing the slowest cooling.

3. The maximum positive thermal gradient measured in the span were similar in shape and magnitude to the positive design thermal gradient of the *AASHTO Segmental Guide Specification* [6] and NCHRP Report 276 [2].

4. Restraint moments calculated for the maximum positive thermal gradient recorded in the structure were about 20% larger than those calculated for the positive design thermal gradient of the *AASHTO Segmental Guide Specification* [6] and NCHRP Report 276 [2].

5. The maximum negative thermal gradient measured in the span was similar in shape and magnitude to the negative design thermal gradient of the *AASHTO Segmental Guide Specification* [6] and NCHRP Report 276 [2].

6. Restraint moments calculated for the maximum negative thermal gradient recorded in the structure were 30% larger than those calculated for the negative design thermal gradient of the *AASHTO Segmental Guide Specification* [6] and NCHRP Report 276 [2].

7. The maximum positive temperature gradient in the top slab was similar in magnitude to the positive design thermal gradient of the *AASHTO Segmental Guide Specification* [6] and NCHRP Report 276 [2].

8. Due to the limited solar exposure the measured positive thermal gradients in webs and bottom slab of the structure were less than 10% of the design positive thermal gradient of the *AASHTO Segmental Guide Specification* [6] and NCHRP Report 276 [2]. In this structure the analysis indicated the stresses in the web and bottom slab are less than 5% of the allowable compressive stress for a 42MPa ultimate strength post-tensioned segmental concrete.

9. The maximum negative temperature gradient in the top slab, webs, and bottom slab were similar in magnitude to the negative design thermal gradient of the *AASHTO Segmental Guide Specification* [6] and NCHRP Report 276 [2].

10. The transverse analysis and recorded data from transverse concrete strain gauge devices indicate that the positive thermal gradient in the top slab produces large strains and or stresses in the top slab of a closed box structure. In this structure the analysis indicated the stresses are about 30% of the current allowable compressive stress for a 42MPa ultimate strength post-tensioned segmental concrete.

7.3 RECOMMENDATIONS

Several recommendations regarding thermal gradients can be made based on the instrumentation and analyses of the segmentally-constructed, post-tensioned, concrete, hollow box bridge span. Those recommendations are included in the following discussions.

7.3.1 Thermal Gradients Through The Depth of The Box

The measured positive thermal gradients occurring through the depth of the box in this structure were very close in magnitude and shape to those specified by the *AASHTO Segmental Guide Specification* [6] and NCHRP Report 276 [2]. Any deviations from the shape of the design gradient were small and the comparisons of the forces produced from restraint moments and restraint axial forces showed small deviations. This data suggests that there is no need to change the positive design thermal gradients specified by the *AASHTO Segmental Guide Specification* [6] and NCHRP Report 276 [2] for this type of concrete segmental box structure in Zone 2 with no topping. In addition there is nothing in this data which would support the proposed revisions to the positive thermal design gradients of the *AASHTO Segmental Guide Specification* [6].

The measured negative thermal gradients occurring through the depth of the box in this structure were close in magnitude and not quite as close in shape to those specified by the *AASHTO Segmental Guide Specification* [6] and NCHRP Report 276 [2]. The measured negative thermal gradients occurring through the depth of the box in this structure were even less close in magnitude and in shape to those specified by the proposed revisions to the *AASHTO Segmental Guide Specification* [6] and NCHRP Report 276 [2]. It appears from this data that the current negative design thermal gradients specified by the *AASHTO Segmental Guide Specification* [6] and NCHRP Report 276 [2] for this type of concrete segmental box structure in Zone 2 with no topping would be slightly unconserative for design of this structure if it were continuous. In addition this data would not support the proposed revisions to the negative thermal design gradients of the *AASHTO Segmental Guide Specification* [6]. There seems to be a need for additional research to adjust the shape of the negative design gradient from the *AASHTO Segmental Guide Specification* [6] and NCHRP Report 276 [2].

7.3.2 Positive And Negative Thermal Gradients Through The Top Slab For Transverse Design

There is currently no provision in the *AASHTO Segmental Guide Specification* [6] for a specific design thermal gradient to be applied to the top slab (for transverse design of the top slab) of a closed concrete box. From the transverse analysis shown in Chapter 6 and the thermally induced strains recorded from the transverse concrete strain gauge devices shown in

Chapter 6, it is apparent that the positive thermal gradient loading produced in the top slab is very significant to the transverse design. The data from thermocouples located in the webs and bottom slab show small positive thermal gradients occurring. However the transverse analysis shown in Chapter 6 indicates that their affect on the structural response in the transverse direction is small for this type of box (wings which almost fully shade the webs). In looking at the data from this structure is appears that the positive thermal gradient through the top slab is slightly larger in magnitude than that of the current design positive thermal gradient specified for the total depth of the structure in the *AASHTO Segmental Guide Specification* [6]. In addition the maximum positive gradient recorded in the top slab of this structure appears to be almost linear.

It is clear that there is a need for a design positive thermal gradient to be applied to the transverse design of closed segmental concrete box bridges. In the absence of other data, a non-linear gradient could be derived from the current positive thermal design gradient in the *AASHTO Segmental Guide Specification* [6] to be applied to the transverse design of closed segmental concrete box bridges. Figure 7.5 shows the proposed adaptation of the current positive thermal gradient recommended in the *AASHTO Segmental Guide Specification* [6] for the top slab.

Figure 7.5: Adaptation of current positive thermal design gradient of the AASHTO Segmental Design Guide[6]to the top slab thermal design gradient (Zone 2 - no topping).

The proposed adaptation of the positive thermal design gradient from Figure 7.5 is compared to the data from the peak positive gradient derived from temperature differentials measured in the top slab of the structure on May 20 1995 from 8AM to 4PM (see Figure 7.6). This comparison shows that the proposed gradient is slightly smaller than the temperature differentials. However the restraint moment resulting from these two thermal gradients would be fairly close which, as previously discussed, is the largest component of thermally induced transverse strain in most structures. It is also felt that using a gradient derived from the current model would be more readily accepted than a completely new model. It is also clear that using any model would yield better results than using none. This gradient would also be applied through webs of deep structures if they received substantial solar radiation, otherwise it could be neglected for webs and bottom slabs (as demonstrated by analysis).

Figure 7.6: Comparison of the adaptation of the current positive thermal design gradient of the AASHTO Segmental Design Guide[6] (Zone 2 - no topping) to the derived maximum positive top slab thermal gradient.

The same adaptation can be applied to negative gradients as well. The largest negative thermal gradients measured through the members were found in the top slab. The maximum negative thermal gradients through the webs and bottom slabs were approximately 50 percent of the values found in the top slab. Since the negative thermal gradients recommended in the AASHTO Segmental Design Guide[6] are substantially smaller than those found in the recommended positive thermal gradient, and the webs and bottoms slab gradients are approximately 50 percent of the peak values found in the slabs, then the gradients in the webs in bottom slabs could be neglected in most cases. Figure 7.7 shows the proposed adaptation of the current negative thermal gradient recommended in the *AASHTO Segmental Guide Specification* [6] for the top slab.

Figure 7.7: Adaptation of current negative thermal design gradient of the AASHTO Segmental Design Guide[6] (Zone 2 - no topping) to the top slab negative thermal design gradient.

The proposed adaptation of the negative thermal design gradient from Figure 7.7 is compared to the data from the peak negative gradient derived from temperature differentials measured in the top slab of the structure on January 18 1996 from 9PM to 7AM (see Figure 7.8).

Figure 7.8: Comparison of the adaptation of the current negative thermal design gradient of the AASHTO Segmental Design Guide[6] (Zone 2 - no topping) to the derived maximum negative top slab thermal gradient

The proposed changes to thermal gradients used in the transverse design of closed concrete boxes for the *AASHTO Segmental Design Guide [6]* are as follows (changes in italics):

> **7.4.4 Differential Temperature**
>
> Positive and negative differential superstructure temperature gradients in Appendix A of the National Cooperative Highway Research Program Report 276 “Thermal Effects in Concrete Bridge Superstructures” [5]. *Positive and negative differential temperature gradients as shown below shall be applied to*

the transverse design of superstructure elements which receive substantial direct solar radiation:

APPENDIX - CALCULATIONS

Longitudinal Analysis Self-Equilibrating Stresses

$$\mathbf{P1} = \int_{1.993}^{2.134} \{ 8.534 \} \bullet [T_1 - (T_1 - T_2) \bullet \frac{2.134 - y}{.141}] \bullet E \bullet \alpha \bullet dy$$

$$\mathbf{P2} = \int_{1.880}^{1.933} \{ 8.534 \} \bullet [T_2 - (T_2 - T_3) \bullet \frac{1.993 - y}{.113}] \bullet E \bullet \alpha \bullet dy$$

$$\mathbf{P3} = \int^{1.880} \{ 0.742 + 15.93 \bullet (y - 1.524) \} \bullet [T_3 - (T_3 - T_4) \bullet \underline{1.880 - y}] \bullet E \bullet \alpha \bullet dy$$

1.524 .356

$$\mathbf{P4} = \int_{1.241}^{1.524} \{ 0.438 + 1.075 \bullet (y - 1.241) \} \bullet [T_4 - (T_4 - T_5) \bullet \frac{1.524 - y}{.283}] \bullet E \bullet \alpha \bullet dy$$

$$\mathbf{P5} = \int_{0.965}^{1.241} \{ 0.438 \} \bullet [T_5 - (T_5 - T_6) \bullet \frac{1.241 - y}{.276}] \bullet E \bullet \alpha \bullet dy$$

$$\mathbf{P6} = \int_{0.660}^{0.965} \{ 0.438 \} \bullet [T_6 - (T_6 - T_7) \bullet \frac{0.965 - y}{.305}] \bullet E \bullet \alpha \bullet dy$$

$$\mathbf{P7} = \int_{0.507}^{0.660} \{ 0.438 \} \bullet [T_7 - (T_7 - T_8) \bullet \frac{0.660 - y}{.153}] \bullet E \bullet \alpha \bullet dy$$

$$\mathbf{P8} = \int_{0.304}^{0.507} \{ 0.681 - 1.197 \bullet (y - 0.304) \} \bullet [T_8 - (T_8 - T_9) \bullet \frac{0.507 - y}{.203}] \bullet E \bullet \alpha \bullet dy$$

$$\mathbf{P9} = \int_{0.279}^{0.304} \{ 0.681 \} \bullet [T_9 - (T_9 - T_{10}) \bullet \frac{0.304 - y}{.025}] \bullet E \bullet \alpha \bullet dy$$

$$\mathbf{P10} = \int_{0.000}^{0.279} \{ 1.981 + 1.197 \bullet (y - 1.241) \} \bullet [T_{10} - (T_{10} - T_{11}) \bullet \frac{0.279 - y}{.279}] \bullet E \bullet \alpha \bullet dy$$

$$\mathbf{P} = \sum_{i=1}^{10} Pi$$

Longitudinal Analysis Self-Equilibrating Stresses

$$\mathbf{M1} = \int_{1.993}^{2.134} \{ 8.534 \} \bullet [T_1 - (T_1 - T_2) \bullet \frac{2.134 - y}{.141}] \bullet y \bullet E \bullet \alpha \bullet dy$$

$$\mathbf{M2} = \int_{1.880}^{1.933} \{ 8.534 \} \bullet [T_2 - (T_2 - T_3) \bullet \frac{1.993 - y}{.113}] \bullet y \bullet E \bullet \alpha \bullet dy$$

$$\mathbf{M3} = \int^{1.880} \{ 0.742 + 15.93 \bullet (y - 1.524) \} \bullet [T_3 - (T_3 - T_4) \bullet \underline{1.880 - y}] \bullet y \bullet E \bullet \alpha \bullet dy$$

1.524 .356

$$\mathbf{M4} = \int_{1.241}^{1.524} \{ 0.438 + 1.075 \cdot (y - 1.241) \} \cdot \left[T_4 - (T_4 - T_5) \cdot \frac{1.524 - y}{.283} \right] \cdot y \cdot E \cdot \alpha \cdot dy$$

$$\mathbf{M5} = \int_{0.965}^{1.241} \{ 0.438 \} \cdot \left[T_5 - (T_5 - T_6) \cdot \frac{1.241 - y}{.276} \right] \cdot y \cdot E \cdot \alpha \cdot dy$$

$$\mathbf{M6} = \int_{0.660}^{0.965} \{ 0.438 \} \cdot \left[T_6 - (T_6 - T_7) \cdot \frac{0.965 - y}{.305} \right] \cdot y \cdot E \cdot \alpha \cdot dy$$

$$\mathbf{M7} = \int_{0.507}^{0.660} \{ 0.438 \} \cdot \left[T_7 - (T_7 - T_8) \cdot \frac{0.660 - y}{.153} \right] \cdot y \cdot E \cdot \alpha \cdot dy$$

$$\mathbf{M8} = \int_{0.304}^{0.507} \{ 0.681 - 1.197 \cdot (y - 0.304) \} \cdot \left[T_8 - (T_8 - T_9) \cdot \frac{0.507 - y}{.203} \right] \cdot y \cdot E \cdot \alpha \cdot dy$$

$$\mathbf{M9} = \int_{0.279}^{0.304} \{ 0.681 \} \cdot \left[T_9 - (T_9 - T_{10}) \cdot \frac{0.304 - y}{.025} \right] \cdot y \cdot E \cdot \alpha \cdot dy$$

$$\mathbf{M10} = \int_{0.000}^{0.279} \{ 1.981 + 1.197 \cdot (y - 1.241) \} \cdot \left[T_{10} - (T_{10} - T_{11}) \cdot \frac{0.279 - y}{.279} \right] \cdot y \cdot E \cdot \alpha \cdot dy$$

$$\mathbf{M} = \sum_{i=1}^{10} Mi$$

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