Final Report

Thermal Movement Design Procedure for Steel and Concrete Bridges

A Report to the

National Cooperative Highway Research Program
NCHRP 20-07/106

by

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Chapter 1

Introduction

Introduction to Thermal Movements

Bridges expand and contract due to temperature change. This movement is accommodated by bearings and expansion joints or by deformation of the piers and abutments with integral construction. The AASHTO Specifications [1,2] provide simple guidelines, which divide the US into two climate (Cold and Mild) zones, but no guidance is provided as to what states or regions fall into the two zones. Metal bridges are designed for a minimum bridge temperature of 0^{O}F (-18^{O}C) and -30^{O}F (-34^{O}C) in the mild and cold climate zones, respectively, and AASHTO recommends a maximum bridge temperature of 120^{O}F (49^{O}C) in both zones. AASHTO recommends that concrete bridges use a maximum temperature which is $+30^{O}$F and $+35^{O}$F (+16.7 and $+19.4^{O}$C) larger than the installation temperature and a minimum bridge temperature which is -40^{O}F and -45^{O}F (-22.2 and -25^{O}C) smaller than the installation temperature in mild and cold climates, respectively. Movements are computed by the equation

$$\Delta = \alpha \, L \, \Delta T \,, \tag{1}$$

where α is the coefficient of thermal expansion, L is the expansion length, and ΔT is the difference between the extreme temperature and the installation temperature. No guidance is provided as to how the installation temperature should be defined for steel bridges, while concrete bridges effectively ignore the installation temperature issue. These AASHTO Specifications are essentially unchanged from those used in the first AASHTO Specification developed in the 1920's. There is no clear rationale or justification for these existing provisions

These thermal movements are used to design the expansion joints and bearings. It is important that the magnitude and direction of the design movements be predicted with reasonable accuracy, since extremely large forces occur if movement is restrained. These forces may lead to severe damage if movements are not properly accommodated through joints and bearings or some form of integral construction. Figure 1 is a photograph of typical damage that may occur if proper allowances for this movement are not made.

If the design movements are larger than truly needed, other problems may occur. Bridge engineers commonly use sliding pot bearings for movements larger than ±2.0 inches (±50 mm) whereas elastomeric bearings could be used if the movements were smaller. Bridge engineers commonly shift from strip seals or compression joints to modular expansion joints when the range of movement exceeds 4 or 5 inches (100 to 125 mm). If these changes to the joints and bearings are required by overly large design movements, significant increases in the initial bridge cost as well as increased maintenance costs must be expected. Integral construction often offers significant advantages in reduced maintenance and improved bridge performance, but the acceptability of this practice is often judged based on the expected bridge movements. Therefore, improperly estimated bridge movements may also lead to improper or reduced use of integral construction.

Figure 1. Photograph of Typical Damage Caused by Inadequate Allowances for Bridge Movement

Recent Research on Bridge Movements

Research [3] has addressed the design temperatures for bridge movements. This work was based on field measurements of bridges in England [4,5,6,7] and analytical studies [8,9,10,11] of heat flow and bridge movements under extreme temperature conditions in the US. The work [3] showed that bridge movements depend upon average bridge temperatures rather than air temperature. The actual calculation of the bridge temperature distribution is quite complex, but two simplified methods (the Emerson Method and Kuppa Method) for estimating the average bridge temperature were noted. The work [3] focused on steel bridges with concrete decks, and it showed that the Emerson Method consistently produced higher maximum average bridge temperatures and lower minimum average bridge temperatures than did the Kuppa Method. The work was compared to the limited field measurements. The comparison showed that the Emerson Method provided better estimates of the day to day temperature of steel bridges, but the Kuppa Method [3] provided much more accurate estimates of the extreme average bridge temperatures needed for determination of the bridge design movements. The Kuppa Method was then combined with historic weather data [12] to develop design maps for minimum and maximum average bridge temperatures for steel bridges with concrete decks. Figures 2 and 3 show the maps for the maximum and minimum average bridge temperature for the contiguous 48 states, respectively. These maps were based on statistical data from 1157 locations with a target minimum history of 60 years of continuous data. The average history was 70.7 years for the lower 48 states and 68.5 years for all sites and locations. These design maps show that steel bridges with concrete decks can often be designed for smaller movements than recommended by the present

AASHTO provisions [1,2]. Therefore, reduced initial bridge costs and lowered maintenance costs are sometimes possible through the selection of more economical and maintenance free joint and bearing systems. A few locations, however, will require larger movements than present AASHTO requirements, and thus fewer long term problems will be noted if these larger movements are used.

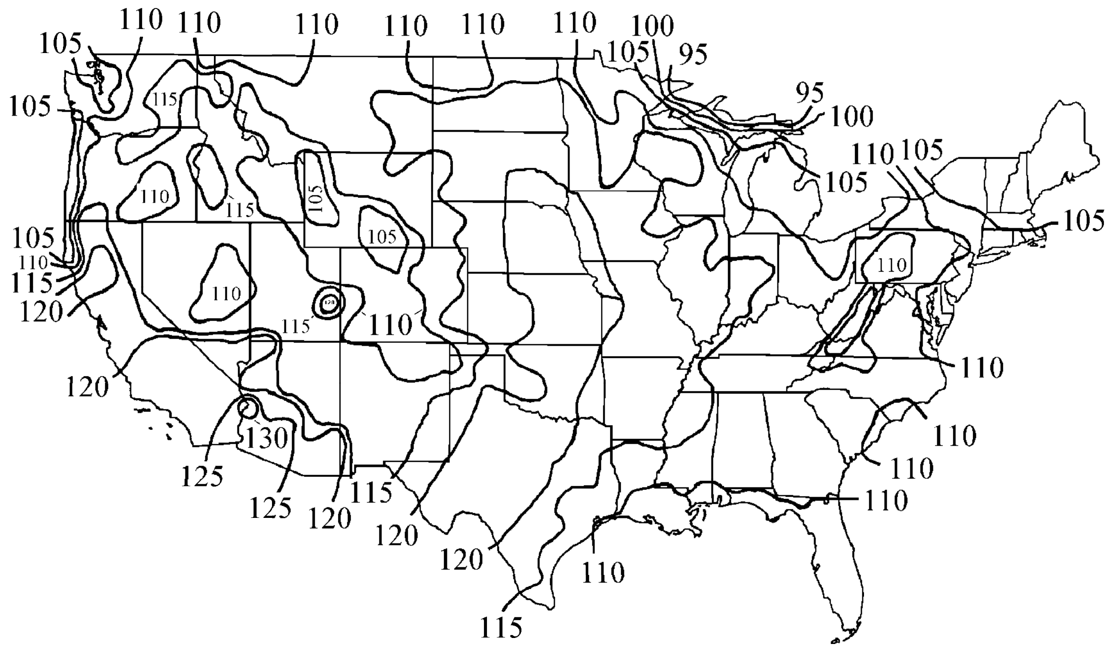

Figure 2. Proposed Design Map for Extreme Maximum Average Bridge Temperature for Steel Bridges with Concrete Decks for Lower 48 States

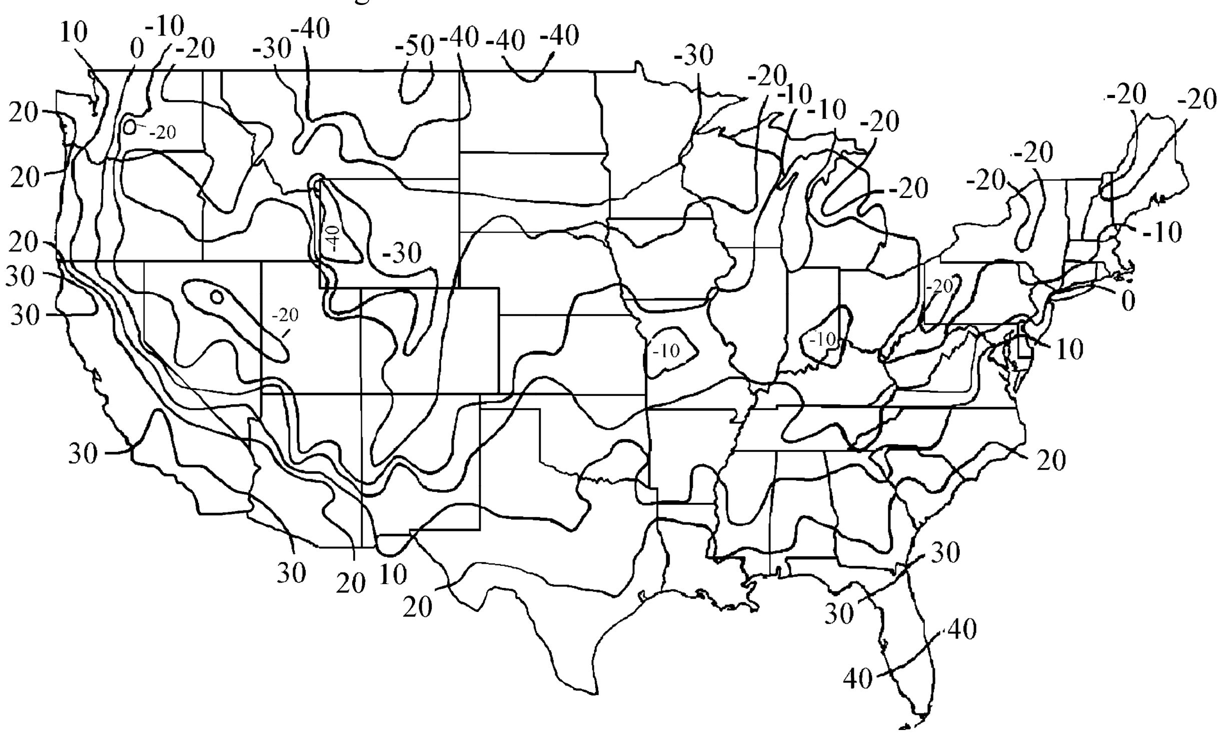

Figure 3. Proposed Design Map for Extreme Minimum Average Bridge Temperature for Steel Bridges with Concrete Decks for Lower 48 States

In addition to development of these design maps, the statistical variation of the day to day temperatures were evaluated and improved recommendations were developed for installation temperatures for joints and bearings on steel bridges. The results of this work were then compared to the limited available field measurement data, and the comparison suggests that the recommendations compare reasonably well to the measured data. These recommendations are generally believed to be conservative, because they neglect the beneficial effects of deformation of the structure due to temperature changes and the stiffness of joints and bearings.

Scope of this Research Study

These earlier recommendations were reviewed by the AASHTO Bridge Subcommittees on Steel Bridges (T14) and Joints and Bearings (T2). These Committees felt that the recommendations were an improvement to existing practice, but the Committee members wanted to extend these methods to concrete bridges so that consistent methods could be used for all bridges. This study was an funded under the NCHRP 20-7 program with the limited goal of extending this past work [3] to concrete bridges. Since this project extends the earlier work, the reader is referred to the earlier report [3] for many details of the methods used in the evaluation. This NCHRP 20-7 project was divided into five tasks. These include -

Task 1. Assemble the temperature and weather data needed to establish bridge design temperatures for concrete bridges throughout the US.

Task 2. Translate the weather data into estimated maximum and minimum bridge temperatures for concrete bridges.

Task 3. Perform an initial mapping of the high and low bridge temperatures, and develop contour maps for concrete bridges.

Task 4. Evaluate and correct the maps for local anomalies and verify the acceptability of the maps through comparison of map data to existing field measurements for concrete bridge temperatures.

Task 5. Correlate the concrete maps with the steel maps and develop consistent proposals for specification wording in both Standard Specification and LRFD format.

Tasks 1, 2, and 3 utilized basic methods developed and discussed in the earlier report [3], and as a result they are only briefly summarized in this report. Tasks 4 and 5 used methods developed in this earlier work, but the differences between steel and concrete bridges require a more in depth discussion of this work. As a result greater discussion of Tasks 4 and 5 is included in this report.

Scope of this Report

This report summarizes the results of this NCHRP 20-7 study. This first chapter introduces the research topic and provides an overview and summary of the past work and the issues of concern. Chapter 2 summarizes the theoretical considerations and simplified methods needed to understand and examine thermal movements of concrete bridges. Chapter 3 describes the application of these methods to bridges in all locations of the United States and the development of the design proposal for concrete bridges. Extreme bridge design temperatures are estimated for all parts of the United States with the data from sites selected from the earlier study but with data adapted to concrete bridges. Design strategies and installation temperature are also discussed in this chapter. Chapter 4 provides verification of these recommendations by comparison to field measurement data. Chapter 5 provides a brief summary and conclusion to this work. Appendices A and B summarize the locations and data used to establish the design recommendations presented in this report. Appendix C contains the proposed wording for changes to the AASHTO Standard Specification. Similar wording is proposed for the AASHTO LRFD Specifications in Appendix D. Appendix E describes some supplemental field measurement data obtained by the AASHTO T2 Joints and Bearings Committee to further evaluate the design recommendations.

Chapter 2

Prediction of Thermal Movements for Concrete Bridges

Prediction of Bridge Temperature

Bridge temperatures vary through the bridge cross section as a function of time. Temperature calculations are based on radiation, convection, and conduction heat flow, and these three mechanisms all contribute to the time dependent cross sectional variation. Radiation is the long distance transfer of heat from a hot body to a colder body. The sun heats the bridge during the day, and the bridge transmits radiant heat to the environment on cold nights. Convection is the transfer of heat from a solid (the bridge) to a moving air or fluid. Convection is influenced by the air temperature and is largely driven by the wind or by air currents caused by moving traffic. Convection lowers the extreme high temperatures of the bridge during the summer, and the extreme low temperatures during the winter. Radiation and convection contribute to the changes in the bridge surface temperature, and they drive the time dependent variations in the body. Conduction is the flow of heat within the body of the bridge. All solid bodies move toward a equilibrium temperature in the absence of outside influences, and conduction heat flow is the process by which this equilibrium condition is attained. Accurate determination of the bridge temperature requires consideration of all 3 components of heat flow in addition to other information including the cloud cover, air temperature, wind speed, angle of the sun, time of day, orientation of the structure with respect to the sun, and geometry and materials of the bridge.

Kuppa Method

Kuppa [13,14,15] used analytical methods developed and initially verified by others [11,12] to perform a heat flow analysis of both steel and concrete bridges in a wide range of climates. Her calculations focused on near extreme events, since these events control thermal movement design. She showed that temperature distribution within the bridge varies as a function of time and bridge type. The average bridge temperature, T_{Avg}, is based on equilibrium principles, and is integrated over the bridge cross section. Therefore,

$$T_{Avg} = \frac{\sum A_i E_i \alpha_i T_i}{\sum A_i E_i \alpha_i} \qquad (2)$$

where i represents the different temperature or material segments (or layers) of the bridge, A_i is the cross sectional area of the ith segment, E_i is the elastic modulus of the ith segment, α_i is the coefficient of thermal expansion of the ith segment, and T_i is the temperature of the ith segment. Kuppa's calculations included all bridge properties as well as conduction, convection, and radiation heat transfer. She considered actual air temperatures, cloud cover, precipitation, and wind velocity, since data was taken from US sites where complete climate data was available.

These calculations indicated that the extreme maximum and minimum average bridge temperatures depend upon the four day averages of the high and low air temperature, respectively. That is, the extreme maximum average bridge temperature, T_{AvgMax}, depends upon the average high air temperatures for 4 consecutive days in the hottest part of the summer, and the extreme minimum average bridge temperature, T_{AvgMin}, depends on the average of the low air temperature for 4 consecutive days in the coldest part of the winter. The 4 day averages were the same for both bridge types, but different relationships were noted for steel bridges with concrete decks, concrete girder bridges and concrete box girder bridges. The correlation was determined by a regression analysis of data obtained from detailed analysis for a number of locations in the US.

For concrete bridges, her method suggests that

$$T_{AvgMax} = \frac{T_{MaxAir1} + T_{MaxAir2} + T_{MaxAir3} + T_{MaxAir4}}{4} \quad 0.953 + 4.6 \qquad (^{o}F) \quad (3a)$$

$$T_{AvgMin} = \frac{T_{MinAir1} + T_{MinAir2} + T_{MinAir3} + T_{MinAir4}}{4} \quad 1.186 + 17.24 \qquad (^{o}F) \quad (3b)$$

Slight differences were noted for concrete box girder bridges and bridges with precast concrete girders, but these differences are not large enough to warrant separate design limits.

<u>Emerson Method</u>

Black [16,17] and Emerson [4,5,6,7] also developed empirical methods which permit estimation of bridge temperatures. These methods were based upon a correlation between the measured daily minimum average temperature, T_{AvgMin}, of the bridge and the mean of the measured night time low and previous day high shade temperatures, $T_{ShadePrevHigh}$ and $T_{NightLow}$, for a two day period. The data was recorded on five bridges in Great Britain, and daily temperatures were recorded over several years at each bridge site. Because of the location of the bridges and the short duration of the measurements, there was limited extreme temperature data. That is, extreme bridge temperatures are noted over a 50 or 100 year period, and this extreme data was not recorded during the short duration monitored at each bridge. T_{AvgMin} for a given day was then correlated to the 2 day average of the night time low and previous day high shade temperature through an empirical equation. This equation for concrete bridges can be approximately expressed -

$$T_{AvgMin} = \frac{T_{MaxAir1} + T_{MaxAir2} + T_{MinAir1} + T_{MinAir2}}{4} \quad 1.14 - 3.6 \qquad (^{o}C) \quad (4a)$$

or

$$T_{AvgMin} = \frac{T_{MaxAir1} + T_{MaxAir2} + T_{MinAir1} + T_{MinAir2}}{4} \quad 1.14 - 10.96 \qquad (^{o}F). \quad (4b)$$

The average minimum bridge temperature occurs early in the morning while the bridge is approaching a thermal equilibrium state. Emerson estimated the average maximum bridge temperature by adding a temperature range to the minimum value for that day. She observed that the maximum daily range of the average bridge temperature depended upon the type of bridge, season of the year, and the cloud cover. Table 1 illustrates these maximum temperature ranges for concrete bridges.

Table 1. Maximum Daily Temperature Ranges for Concrete Bridges

	Daily Temperature Range ^{o}F (^{o}C)		
	Clear and Sunny	Cloudy, but not overcast	Overcast/ rain, snow
Winter	5.4 (3)	1.8 (1.)	0 (0)
Spring/Autumn	10.8 (6)	5.4 (3)	1.8 (1)
Summer	10.8 (6)	7.2 (4)	3.6 (2)

The Emerson method is based upon air temperatures in the shade rather than normal weather station data or normal air temperatures. The shade air temperatures are measured under a bridge in a sheltered location, and as a result shade temperatures have less extreme variations than the normal air temperature. Therefore, the use of air temperature always overestimates the magnitude of bridge movements by the Emerson method.

The largest possible daily temperature range occurs for clear, summer days and the maximum average bridge temperature also occurs on similar days. Therefore the maximum average bridge temperature, T_{AvgMax}, for composite bridges can be conservatively estimated by Equation 5 where T_{AvgMin} is determined by Equation 4.

$$T_{AvgMax} = T_{AvgMin} + 10.8 \qquad (^{o}F) \qquad (5a)$$

$$T_{AvgMax} = T_{AvgMin} + 6 \qquad (^{o}C) \qquad (5b)$$

The estimate is conservative because it assumes that the largest range and highest temperature occur on the same day. The Emerson data and analysis focused on these average days rather than extreme temperature data, and the method is best suited for intermediate moderate temperatures rather than extreme bridge temperatures which control thermal movement design. As a result, the estimate is normally quite conservative in estimating extreme temperatures as will be noted in later discussion.

Selection of Locations for Evaluation of Bridge Temperatures

The Emerson and Kuppa models were used to estimate the extreme average bridge temperatures expected for concrete bridges throughout the United States. A computer based weather database, Climate Data [12], provided the input data for this analysis. Daily high and low temperatures, precipitation, and information for more than 10,000 weather stations throughout the US including Alaska and Hawaii are included in this computer database. Some locations have more than 100 years of continuous daily data, but other stations were moved to different sites in the same vicinity during the period of interest. Therefore the data from several files had to be combined for these later locations.

The goals of this research first required consideration of the largest number of locations possible, since many locations are needed to consider local variations in each state and geographical region. In addition, the data files must have a long period of continuous data to be statistically relevant compared to the life of the bridge. There is no uniform agreement as to the design bridge life in the US, but it is clear that 50 to 100 years contains the normal range of interest. Therefore, a minimum 60 year continuous temperature history was selected as the target minimum for this investigation, since this time period is a relevant length of time that was attainable with the existing data.

The tens of thousands of Climate Data [12] files were examined with the goal of selecting data files which satisfy the two different requirements in the previous study [3,18]. During this earlier work, the files were also examined for potential data errors, for gaps in the data, and other inconsistencies. A total of 1273 stations were selected to cover the US including Alaska and Hawaii. Most of these sites clearly satisfied the criteria of having 60 years of continuous data with acceptable accuracy, and these stations are listed in Appendix A. However, 116 of these locations were selected to provide additional information for several states with more limited data. These sites were chosen because of inadequate coverage of the maps in some sparsely populated areas, and a minimum continuous history of 40 years were required for these additional locations. The average length of the time considered in the temperature analysis was 70.7 years for the basic 1157 sites, and 68.5 years for all data including the supplemental data. This duration is significant compared to the expected life of bridges in the US and extreme design values obtained from this data are clearly relevant to bridges in this country.

Prediction of Bridge Temperatures for All Parts of US

The 1273 locations were then evaluated to determine T_{AvgMax} and T_{AvgMin} by both the Kuppa and Emerson methods. Each file was scanned to determine the historic high and low air temperatures. Upon locating the day, month and year of the historic high and low temperatures, the daily temperature record for a period from 5 days prior to the extreme and 5 days after the extreme were isolated as a window for further analysis. Within each window, the averages required by the Kuppa and Emerson methods were computed sequentially for each day until the extreme estimated bridge temperatures were obtained for concrete bridges.

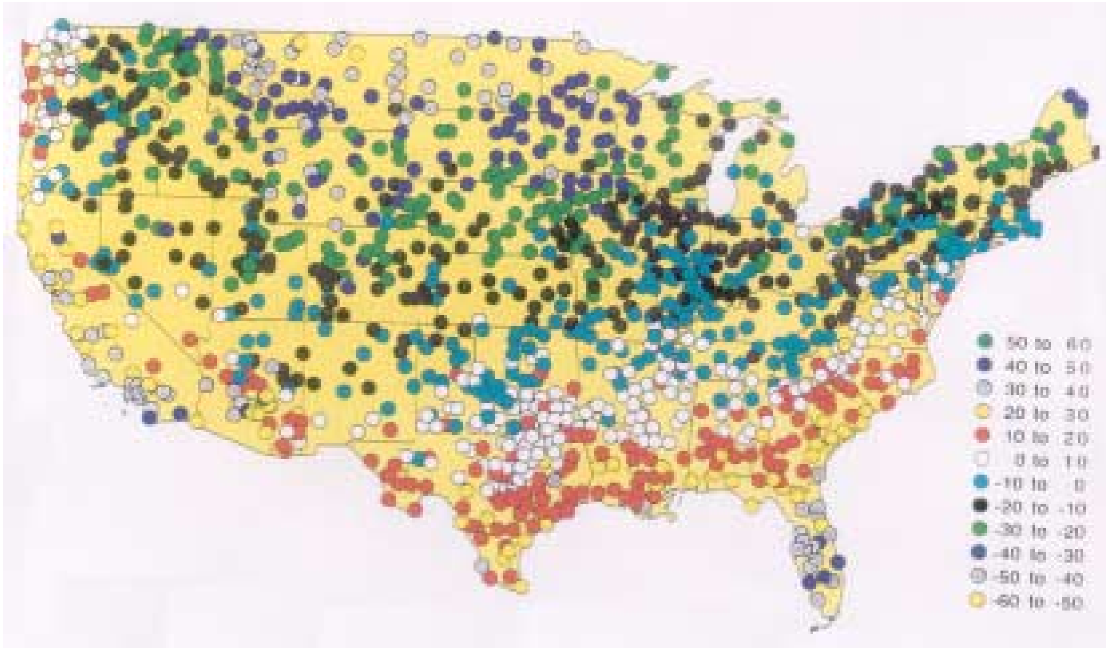

Figure 4. Extreme Minimum Average Bridge Temperature for Concrete Bridges by the Emerson Method

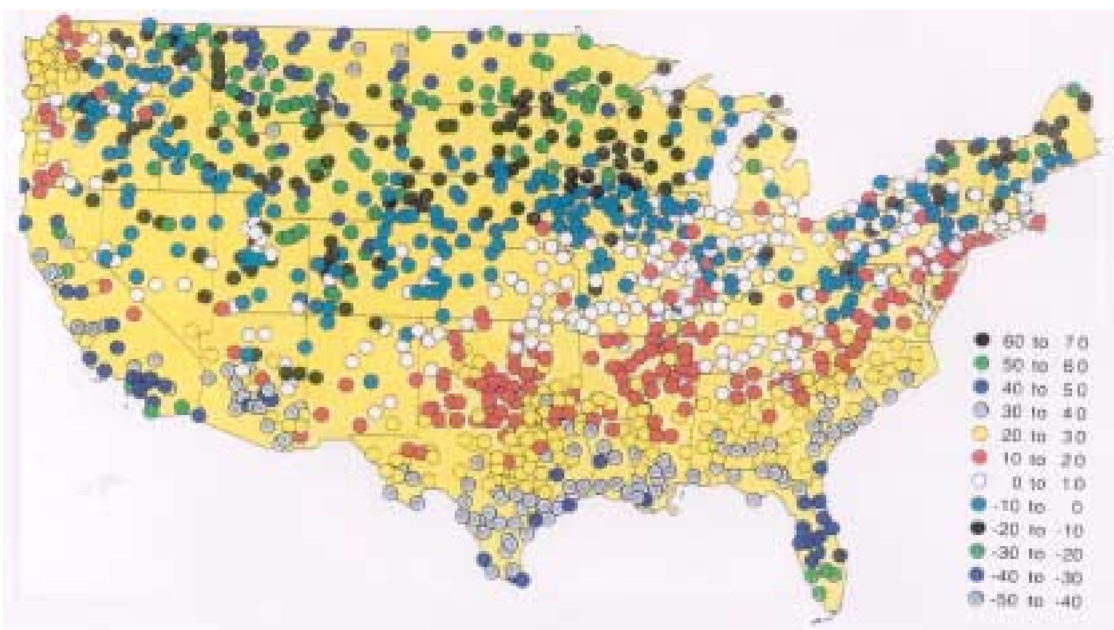

Figure 5. Extreme Minimum Average Bridge Temperature for Concrete Bridges by the Kuppa Method

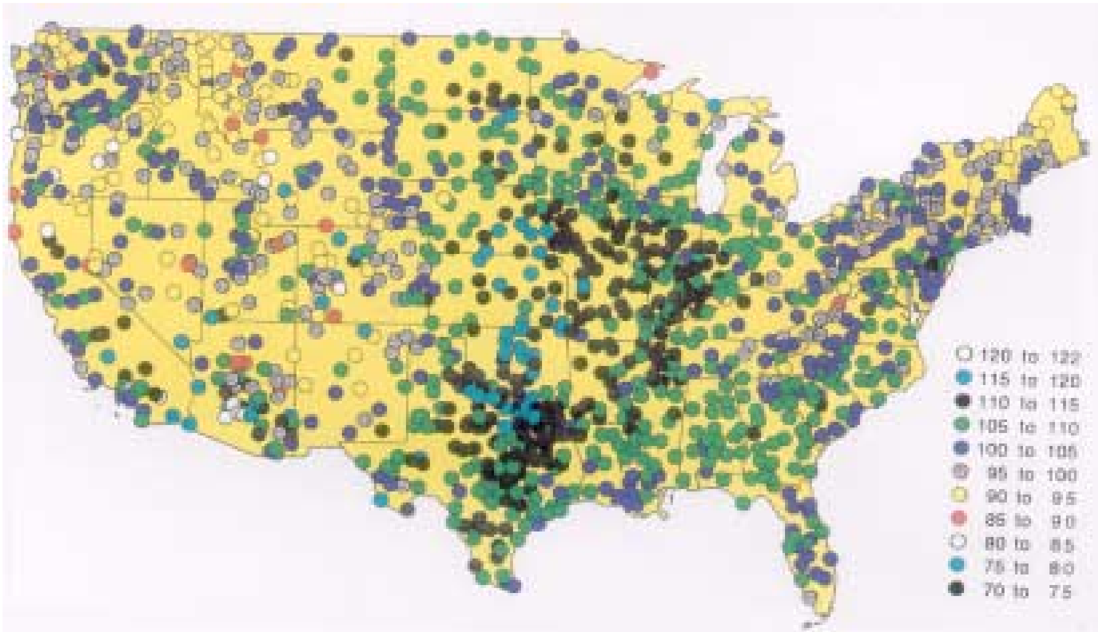

Figure 6. Extreme Maximum Average Bridge Temperature for Concrete Bridges by the Emerson Method

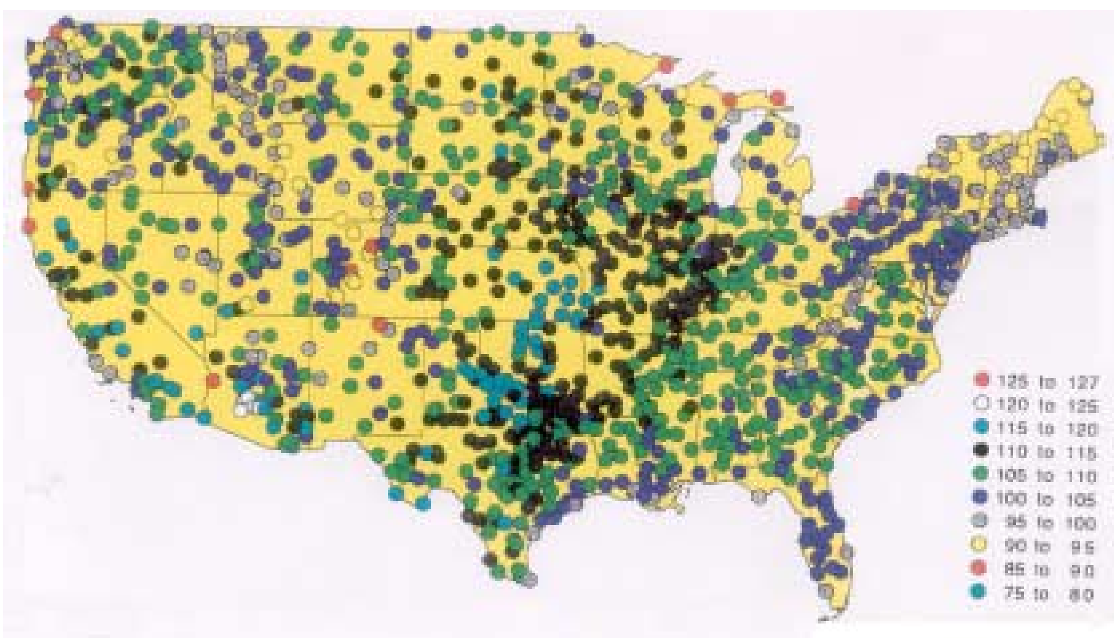

Figure 7. Extreme Maximum Average Bridge Temperature for Concrete Bridges by the Kuppa Method

The MapInfo computer program [19] then was used for mapping this data, since the program permitted rapid changes and adjustments to examine the effect of different parameters on bridge temperatures. Figures 4 and 5 show color coded maps for the extreme minimum average temperatures for concrete bridges in the continental 48 states by the Emerson and Kuppa Models, respectively. These maps are plotted in 10 degree increments, since there is very large variation in minimum average temperatures throughout the US. Figures 6 and 7 show the extreme maximum average bridge temperatures for concrete bridges in the 48 states by the Emerson and Kuppa methods, respectively. The maximum average temperature maps are plotted in 5 degree increments, since there is significantly less variation in maximum bridge temperature for different parts of the US than there is for minimum average temperatures. Comparison of Figures 4, 5, 6, and 7 show that there are consistent trends in average bridge temperatures in both models for both high and low temperatures. Locations which have extremely high or extremely low temperatures by the Emerson model also have the same extremes by the Kuppa Model. The two models trace each other well throughout the United States. However, the Emerson method is consistently conservative in predicting extreme low temperatures. That is, the Emerson model predicts lower minimum extreme temperatures than the Kuppa model. The difference is typically in the order of 8 to 10 degrees in the Fahrenheit scale. The Kuppa method commonly predicts maximum extreme temperatures which are higher than those predicted by the Emerson Method, but the difference is more commonly in the order of 3 to 5 degrees Fahrenheit. The compensating effects of the Kuppa and Emerson Models indicate that the range of temperature variation or the total design movement are similar by the two methods.

The bridge temperatures for Alaska and Hawaii were computed but are not displayed in maps in this report. Alaska has relatively wide variations of temperature, but the maps are sparse because large regions of the interior of Alaska have inadequate temperature data. Hawaii is dominated by coastal effects and elevation. Elevation is important only in that there are some relatively high mountains (in excess of 10,000 feet or 3,000 m) in the islands. This data shows that there is no need to provide maps for Hawaii. A conservative approach for Hawaii could utilize fixed values for the entire region, or as a more accurate and less conservative alternative an equation based upon elevation could be employed. The relative trends noted for the Kuppa and Emerson Methods for the contiguous 48 states are also noted for the Hawaii and Alaska data.

Earlier work[3] developed similar maps for steel bridges with concrete decks. Both methods show that steel bridges with concrete decks are expected to have larger movements than concrete bridges, but both methods show that the difference is much smaller than suggested by existing AASHTO Specifications [1,2]. Both methods suggest that concrete bridges in many locations in the US should have larger thermal movements than considered by the AASHTO Specifications. At the same time, there is little evidence of significant damage to concrete bridges with the present design practice. It should be noted however, the concrete bridges are also designed for movements due to creep and shrinkage, and the combination of the creep and shrinkage effects with the thermal movements will likely combine to reduce any damage potential. The combination of the maps of Figs. 4, 5, 6 and 7 with the maps developed in the previous study [3] are rational in that these bridge temperatures are based upon rational estimates including the actual weather data for regions throughout the US, the actual geometry of real bridges, and the best available data on field measurements of bridges. This rationality makes them attractive compared to the existing AASHTO limits which were developed on intuitive estimates more than 70 years ago. Both methods clearly show that bridges may be designed more economically for smaller movements in some parts of the country, but other parts of the US clearly will require consideration of somewhat larger design movements. The predictions provided by these calculations are still conservative, since they neglect the effect of restraint of movement due to resistance of bearings as well as normal deflections of piers and abutments.

Factors Affecting Bridge Design Temperatures

The Emerson and Kuppa data were analyzed [18] in detail to determine the factors which affect the bridge design temperatures for both steel and concrete bridges in addition to the local weather data. Bridge design temperatures are strongly affected by the elevation, the latitude, and the proximity of the location to the coast. Both the Emerson and Kuppa methods predict less extreme temperatures (lower maximum average bridge temperatures and higher minimum average bridge temperatures) for both steel and concrete bridges located near the coast. The opposite trend is noted for interior locations. Locations at more northerly latitudes and higher elevations tend to have lower maximum and minimum extreme average bridge temperatures for both models and both bridge types. These trends were examined [18,3] in great detail to determine if they could be employed in design equations. While the trends are clear, there are clearly no methods or equations for easily translating these trends into design. Under these circumstances it was determined that contour mapping was the most rational method of translating these trends into a design aid. All factors are consistently considered with a contour map, although the factors cause local anomalies in each map as can be seen in Figs. 4, 5, 6, and 7. The development of the contour maps and the evaluation of these local anomalies are addressed in the next chapter.

Selection of the Best Model for Bridge Design

The Emerson method is consistently conservative compared to the Kuppa method. That is, it predicts a somewhat larger movement range for both concrete bridges and steel bridges with composite decks. Both models produce similar temperature predictions, but it is important to have the best, most rational estimate for use in bridge design. It is important the design temperatures produce safe, serviceable bridges, but it is equally important that the design temperatures not be overly conservative. The two models were analyzed and evaluated in some detail, and ultimately it was determined that the Emerson model is overly conservative, and the Kuppa model is reasonable for extreme bridge temperatures. It should be recalled that the overall temperature range of the Emerson model is approximately 10 ^{o}F to 15 ^{o}F larger than the temperature range predicted by the Kuppa model for both bridge types. There are a number of reasons for determining that the Kuppa model is the proper model for use in bridge design.

1. The Emerson model is based upon the use of shade temperatures rather than normal air temperatures used in this study. Past work by Emerson shows that the air temperatures are more extreme than the shade temperatures, and as a result the use of air temperatures overestimates the extreme bridge temperature. This average overestimate is in the range of 1 ^{O}F or 2 ^{O}F for concrete bridges. Thus, it is a contributing factor, but it is clearly not the major factor in this determination.

2. The Emerson method utilized the approximation that the maximum bridge temperature occurred on the same day as the maximum bridge temperature range was noted. This invariably results in a conservative estimate of the design temperature range, but the degree of the conservative cannot be determined without more information regarding the weather conditions during that period.

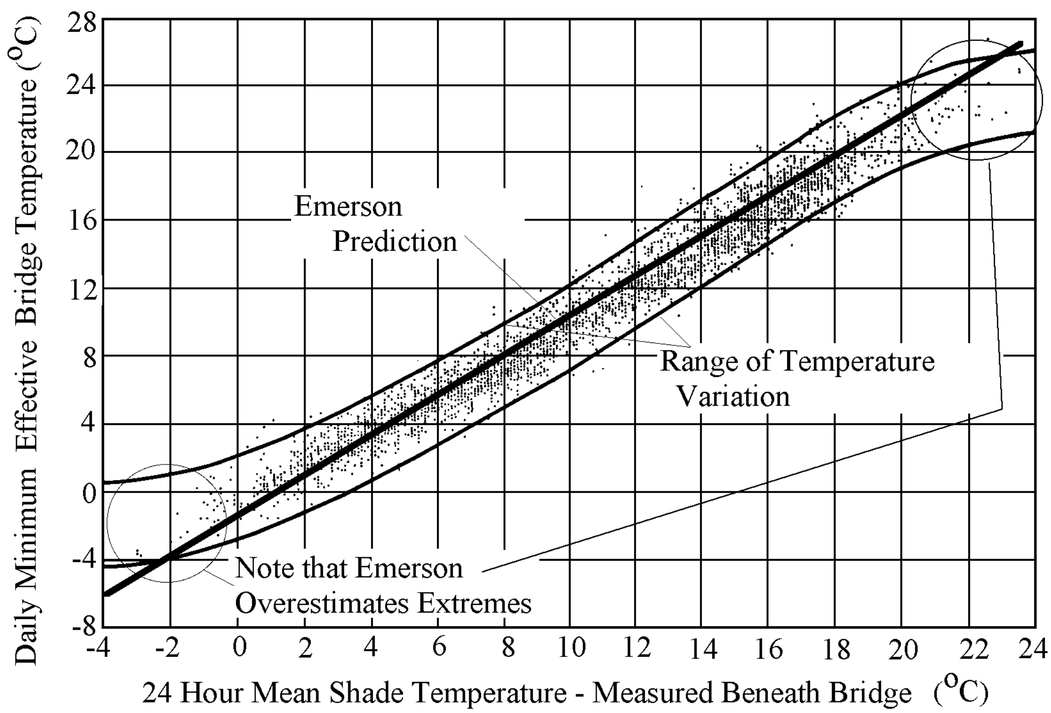

Figure 8. Comparison of Daily Measured Bridge Temperatures to Shade Temperatures Used to Establish Emerson Method

3. The Emerson method is focused on normal day to day temperatures rather than extreme design temperatures. In Fig. 8, the measured daily average minimum bridge temperature of a concrete bridge is plotted as a function of the average shade temperatures for the same bridge. A statistical evaluation of the data is performed and there is a strong correlation between the two measurements during normal conditions, and this correlation provides the basis of equations noted earlier. However, the actual correlation is an s-shaped curve as shown in the figure rather than a straight line as assumed in the predictive model.

The linear correlation is dominated by average conditions, since very little extreme data is available. Figure 8 shows that the few extreme measurements recorded by Emerson are less extreme that predicted by the linear regression model. Figure 8 shows that the maximum extreme bridge temperatures are always lower than the linear correlation line, and the minimum extreme bridge temperatures are always higher than that suggested by the correlation. This difference is quite large and accounts for the largest part of the conservatism noted in the Emerson method.

4. In a more positive light, the Kuppa model was developed for extreme temperatures rather than average conditions, and was developed based upon detailed analysis of bridges under US climates. The difference between the Kuppa and Emerson models are largely explained by the inherent conservatism of the Emerson formulation.

5. The Kuppa method is based primarily upon calculated results, but there is limited experimental data [13] which indicates good correlation between the calculations and field observations.

Based on this discussion, the Kuppa model was used to establish proposed design criteria for both concrete bridges and steel bridges with concrete decks. The design criteria for steel bridges with concrete decks are described in the earlier report [3], but the criteria for concrete bridges is developed in the next chapter.

Chapter 3

Proposed Design Criteria

Design Maps

Prior discussion has shown that contour maps are the most reasonable and rational technique for dealing with the parameters controlling bridge design temperatures. Both the Emerson and Kuppa methods can be used to predict temperatures and movements for concrete bridges, but the Emerson method focuses on typical day to day temperature conditions and is less reliable for the extreme temperature requirements. The Kuppa method is keyed to the extreme design temperatures, and results in a far better estimate of extreme average bridge temperatures. Therefore, design maps for concrete bridges were developed from the Kuppa extreme bridge temperatures.

Initially, contour lines were drawn precisely to the data provided in Figs. 5 and 7. Similar maps were also drawn for Alaska, although the original color coded map is not included in this report. The contours were drawn for 10^{o}F increments for minimum average bridge temperatures and for 5^{o}F increments for maximum average bridge temperatures. The maximum average bridge temperature used smaller increments because there is less variation in the maximum temperatures than in the minimum temperatures. The initial drafts of the contour maps were accurate to the data, but they were not directly useful for design. They were too complex and cluttered, because some contours were around a single point or a small group of points. As a result, these maps were reworked and simplified to provide more practical and useful design maps. This simplification procedure was the same general procedure as used for steel bridges in an earlier study [3] and used to develop Figs. 2 and 3. However, the maps for concrete bridges were drawn slightly less conservatively than were the maps for the steel bridges. There are two rational reasons for using reduced conservativism for concrete bridges as opposed to that used for steel bridges. First, the design maps generally showed that steel bridges could be designed for smaller movements than required by the present AASHTO [1.2] provisions, while the maps for concrete bridges more frequently require slightly larger movements than required by present AASHTO provisions. Further, there have been few problems reported with thermal movements with concrete bridges, and so it is difficult to justify large changes in their design. Second, concrete bridges are also designed for creep and shrinkage movements, and these added movements permit the bridge to deal with the greater uncertainty in the thermal movements.

As was previously done for steel bridges, each individual cluster or contour of the concrete bridge map was examined in detail. The maps are generally conservative estimates of the design temperatures, but excessive conservatism is not desirable. In some cases, individual contours or clusters were eliminated, because the data inside the contour was only slightly outside the norm for the region surrounding the contour. The restraint provided by even the most reliable joints and bearings will reduce thermal movements by a movement equivalent to several degrees of temperature change, and so this reserve readily permits these small changes.

In other cases the data inside the cluster was clearly justified, and small contours were conservatively joined to form a coherent region, since it is unrealistic to expect that pockets of more severe conditions would be isolated to the given region. Local geography and typography were considered in this evaluation, since locations with similar elevation and proximity to the coast have similar bridge temperatures. Small individual contours which shared similar geographic characteristics were more likely to be grouped that those that did not.

These corrections ultimately led to the proposed design maps. The proposed design map for extreme low average bridge temperature and extreme high average bridge temperatures for concrete bridges in the lower 48 states are provided in Figures 9 and 10. Despite the conservatism present in the design maps, comparison of Figs. 9 and 10 shows that only the north central regions of the US have a temperature range as large as the present AASHTO cold climate region for concrete bridges. However, concrete bridges located in the north central portion of the US sometimes have a significantly larger range of thermal movements than required in the present AASHTO provisions. Most of the southern tier of the US have a design temperature range which is smaller than the present AASHTO mild climate requirements. Many other locations have smaller movements than presently required by the present AASHTO Cold Climate condition for concrete bridges.

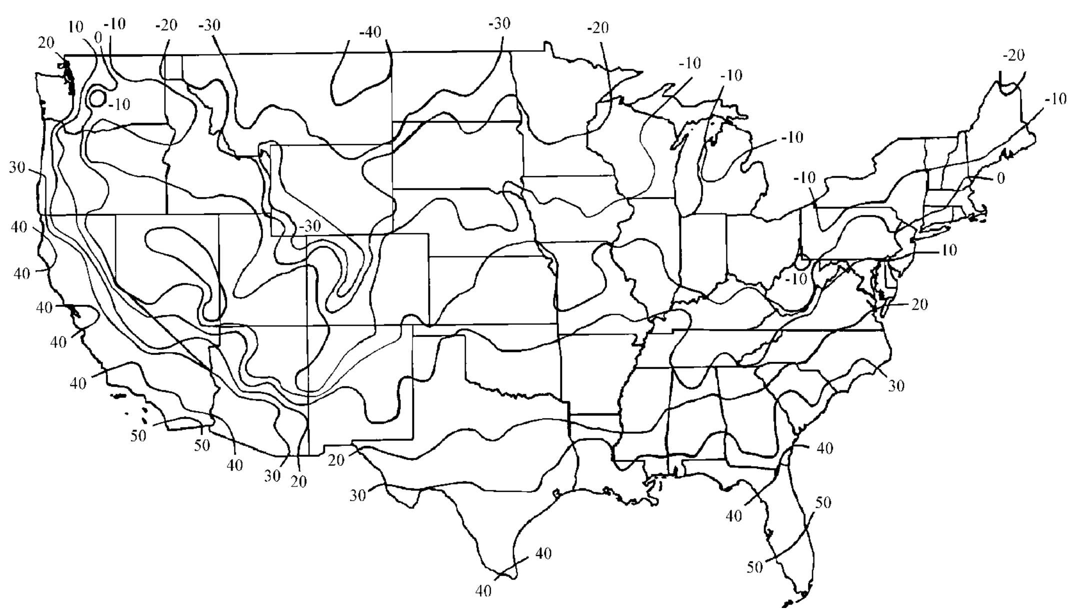

Figure 9. Proposed Design Map for Extreme Minimum Average Bridge Temperature for Concrete Bridges for Lower 48 States

The maximum and minimum average bridge temperatures directly translate into maximum and minimum design temperatures, $T_{MaxDesign}$ and $T_{MinDesign}$. To determine these design temperatures for a given location, the engineer should find the location of the bridge on these maps, and then select the maximum and minimum average bridge temperatures for that location. The selection can be conservatively made to the most conservative adjacent contour for that location or interpolation can be employed. For the most conservative selection, the temperature should be the higher adjacent temperature for the high temperature value and the lower adjacent temperature for the low temperature. The engineer should consider whether the contour represents a local high or low point in the mapping surface. For example, for a concrete bridge in southeast corner of Georgia by the most conservative method, $T_{MaxDesign}$ should be should be $+105^{O}F$ and $T_{MinDesign}$ should be $+30^{O}F$. A bridge in northwest corner of Minnesota should have a $T_{MaxDesign}$ of $+110^{O}F$ and a $T_{MinDesign}$ of $-30^{O}F$. A more accurate and less conservative method for determining the maximum and minimum design temperatures is to interpolate between adjacent contours. By the interpolation method, a concrete bridge in Harrisburg, PA, could rationally be designed for a $T_{MaxDesign}$ of approximately $103^{O}F$ and a $T_{MinDesign}$ of $4^{O}F$.

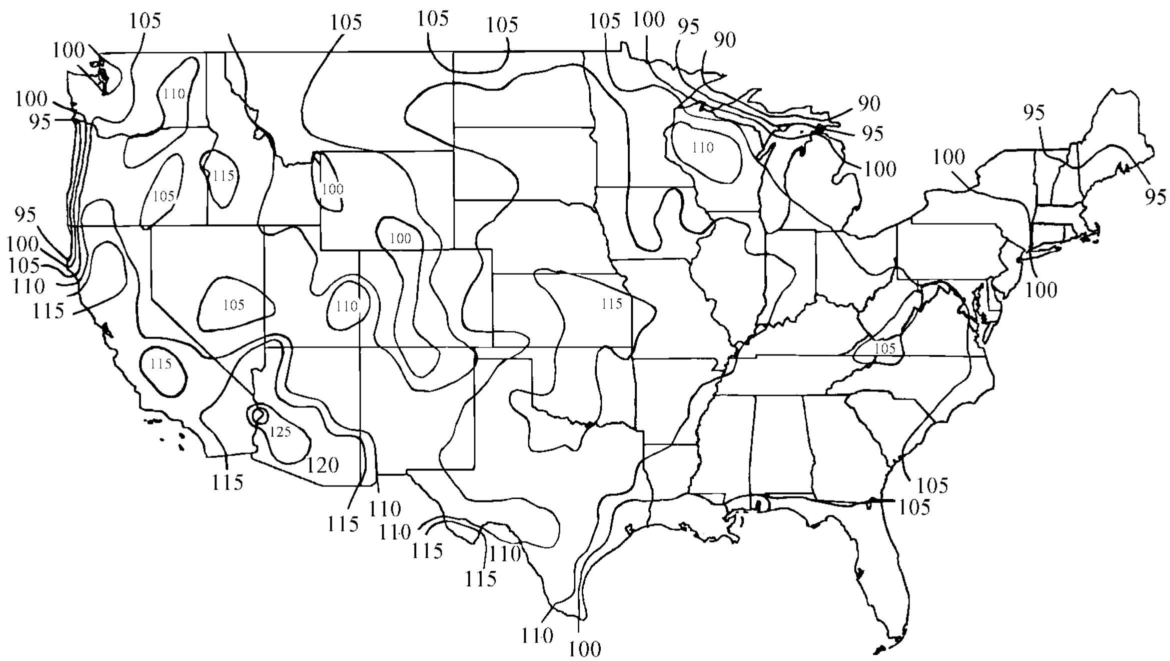

Figure 10. Proposed Design Map for Extreme Maximum Average Bridge Temperature for Concrete Bridges for Lower 48 States

Comparison of Figs 9 and 10 with Figs. 2 and 3 shows that the recommendations require significantly larger thermal movements for steel bridges with concrete decks than for concrete bridges, but the difference is significantly less than recommended in the present AASHTO provisions. On general, the maximum average bridge temperature is $5^{O}F$ to $7^{O}F$ higher for steel bridges than for concrete bridges. The minimum average bridge temperature for steel bridges with concrete decks is $8^{O}F$ to $12^{O}F$ lower than the minimum average temperature for concrete bridges.

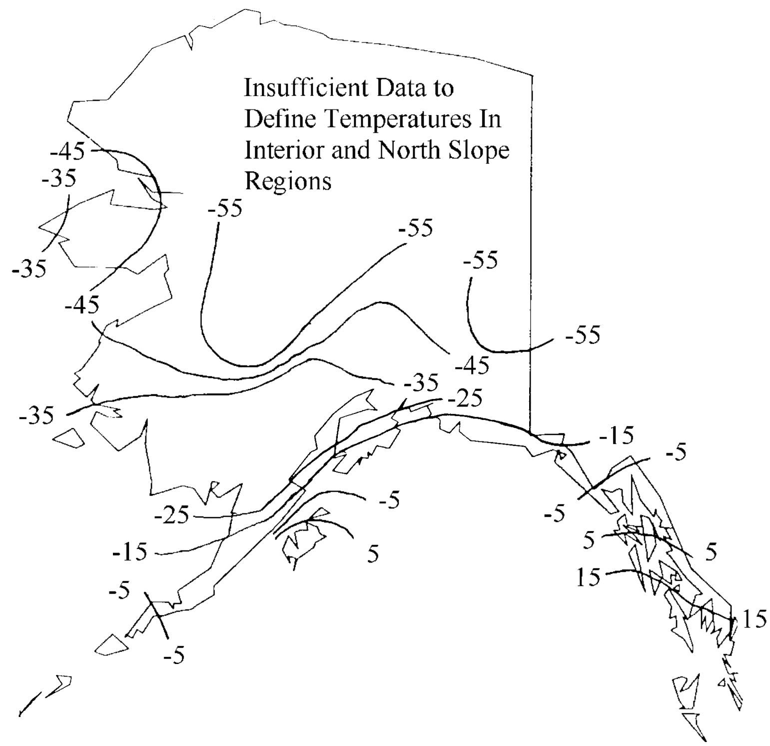

Figure 11. Proposed Design Map for Extreme Minimum Average Bridge Temperature for Concrete Bridges in Alaska

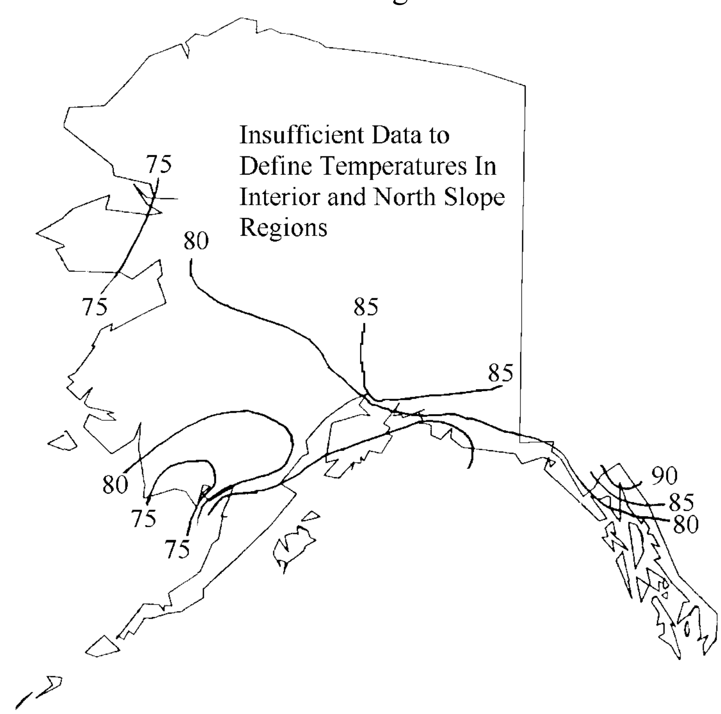

Figure 12. Proposed Design Map for Extreme Maximum Average Bridge Temperature for Concrete Bridges in Alaska

Figures 11 and 12 provide the proposed design maps for Alaska. In general, Alaska has larger design movement requirements than present AASHTO cold climate requirements for concrete bridges for most of the state. Further, the maps provide information only for the more populated regions of Alaska, since there is inadequate thermal data for the interior regions of Alaska to establish reliable design requirements.

Maps are not provided for Hawaii, because there is very little variation in extreme bridge temperature in that state. All variations in Hawaii can be attributed to changes in elevation. Quite high elevations are possible in Hawaii, but very little is lost by employing conservative estimates throughout the state since few bridges are located at the highest elevations. Therefore, a maximum design temperature of 100^{O}F (37.8^{O}C) and a minimum design temperature of 40^{O}F (4.4^{O}C) are recommended for Hawaii, since these are conservative for concrete bridges in all parts of that state.

Statistical Variation of Bridge Temperatures

The discussion to this point has focused on the extreme bridge temperatures for thermal movement design. However, it is logical to ask how frequently these extreme temperatures are achieved and how long they are maintained, since these factors also affect design. A more detailed analysis of the day to day variation of average bridge temperatures was performed for fifty of the 1273 locations considered in this study. The fifty sites were selected -

- to have at least one site in each state,
- to cover the full range of temperature variation throughout the US,
- and to include all types of urban and rural locations with a significant temperature history.

The daily minimum and maximum bridge temperatures were estimated by both the Kuppa and Emerson methods for each day of the full history for both steel and concrete bridges at each of these fifty locations. The daily high and low temperatures were then accumulated into a histogram for each site and the histograms were statistically analyzed to determine the general characteristics of the temperature variations. Figures 13, 14 and 15 are three histograms developed for steel bridges with concrete decks from this detailed analysis. Figure 13 is a histogram for Glasgow, Montana which is clearly one of the coldest locations in the continental lower 48 states. Figure 14 is a histogram for Boston, Massachusetts, which is an intermediate temperature region. Figure 15 is the histogram for Pasadena, California which is one of the mildest locations considered in this study.

Figure 16 shows the histogram for concrete bridges in Pasadena. Comparison of Figs. 15 and 16 shows that the characteristics of the daily temperature variation are similar in both steel and concrete bridges. Concrete bridges have a smaller range of variation of the bridge temperatures largely because of their larger thermal mass, but the statistical characteristics of the temperature variations are similar for the two bridge types. Comparison of the temperature characteristics for all climate conditions show a number of similar attributes. First, there is clearly a statistical variation in the day to day temperatures, but the results are skewed more toward the higher end of the temperature range. The extreme temperatures are marked in all three curves, and it can be seen that the extreme temperatures are only rarely achieved at all locations. However, the extreme low temperature is far more rare than the extreme high temperatures for all locations. The bottom 10% to 15% of the temperature range is typically penetrated only about one day out of every 10 years. This seldom penetrated low temperature range is larger in the cold regions than in the mild regions, because the overall temperature range is larger for these colder sites.

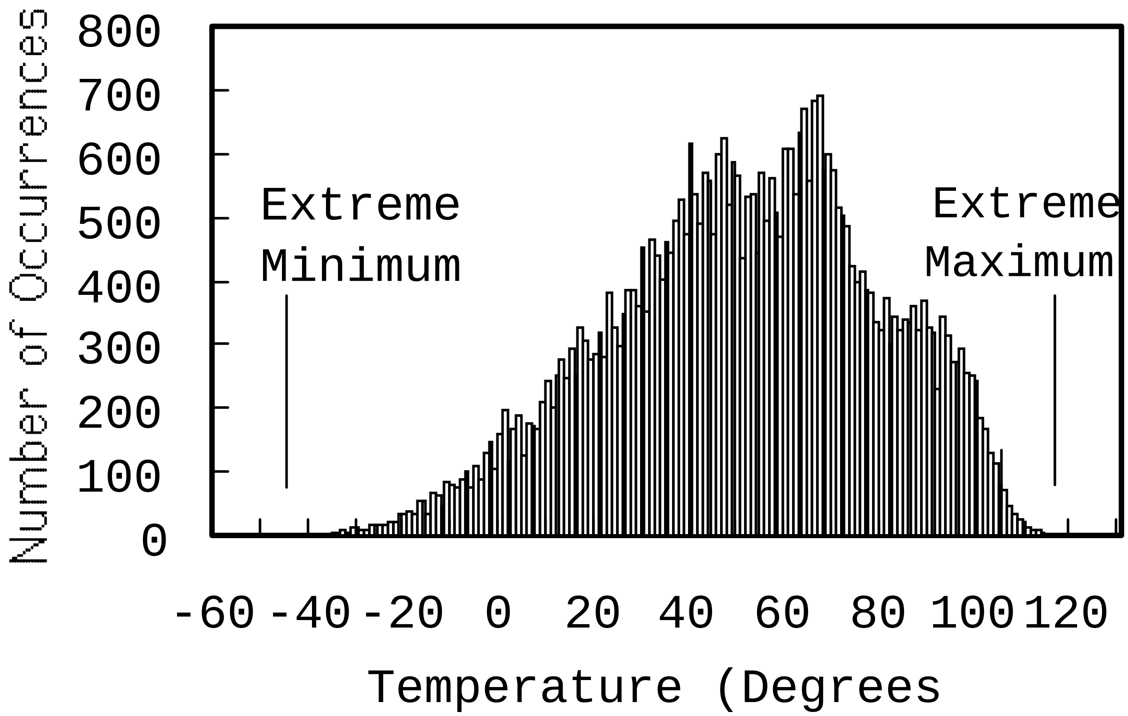

Figure 13. Histogram of the Daily Temperature for Steel Bridges with Concrete Decks in Glasgow, Montana

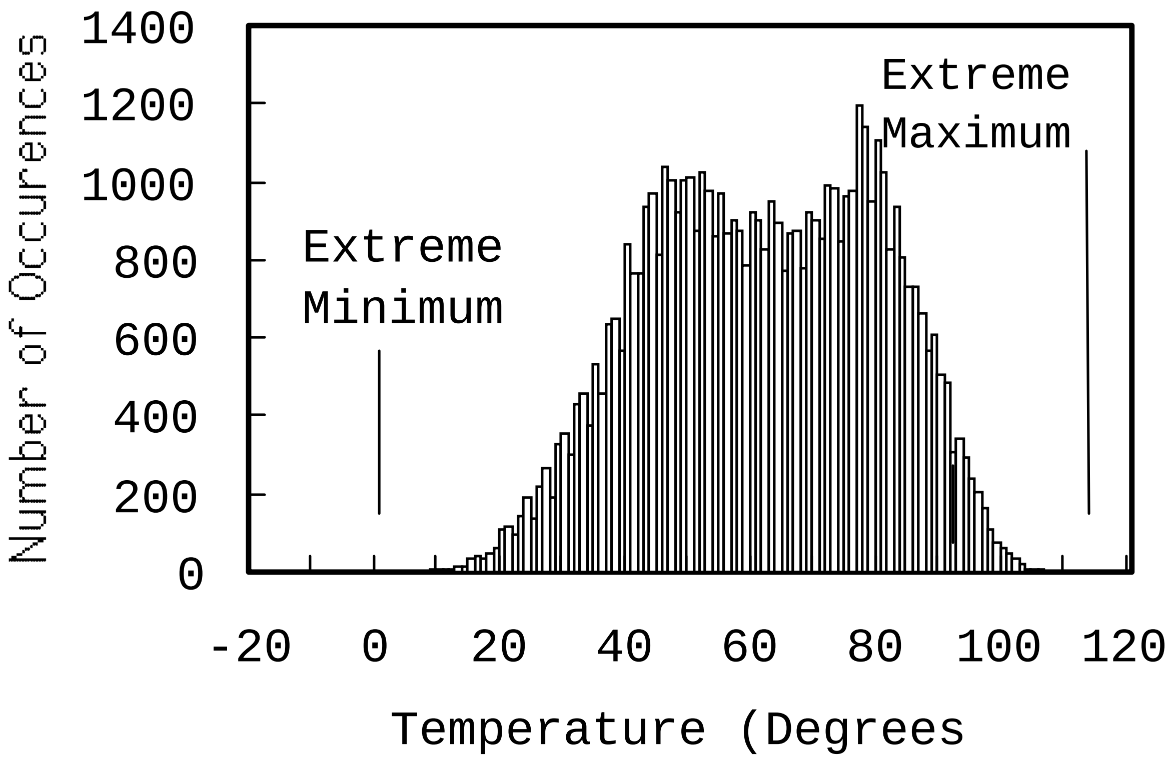

Figure 18. Histogram of the Daily Temperature for Steel Bridges with Concrete Decks in Boston, Massachusetts

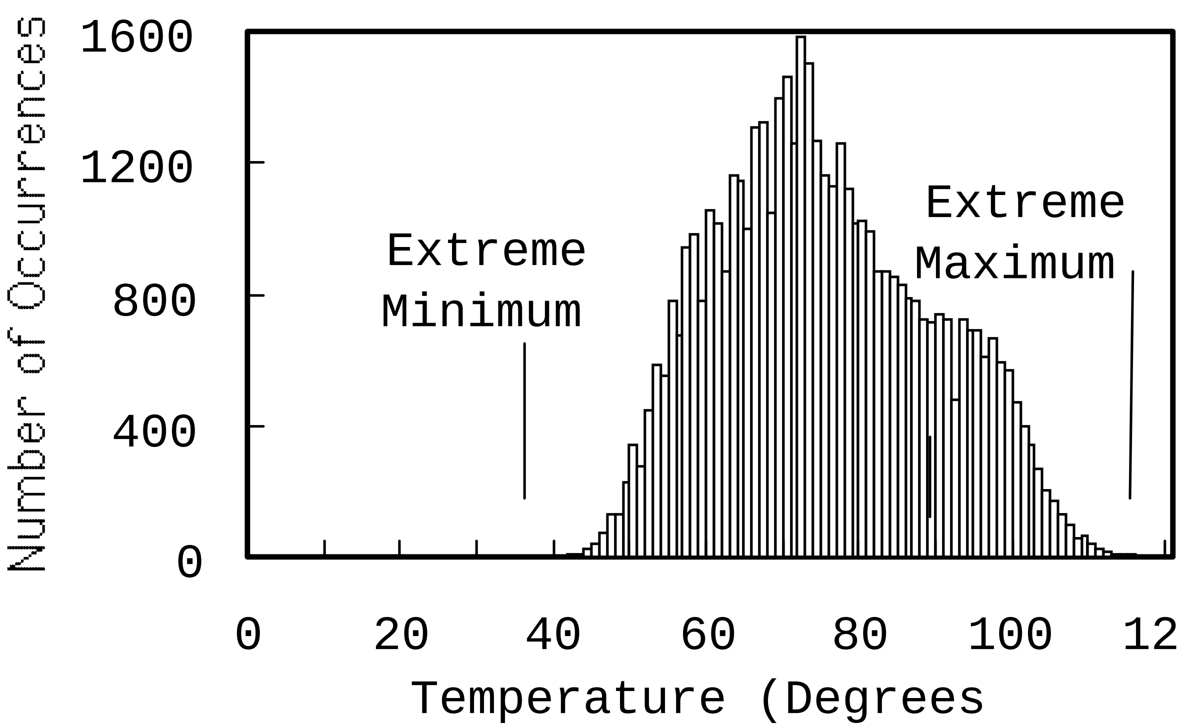

Figure 15. Histogram of the Daily Temperature Variation for Steel Bridges with Concrete Decks in Pasadena, California

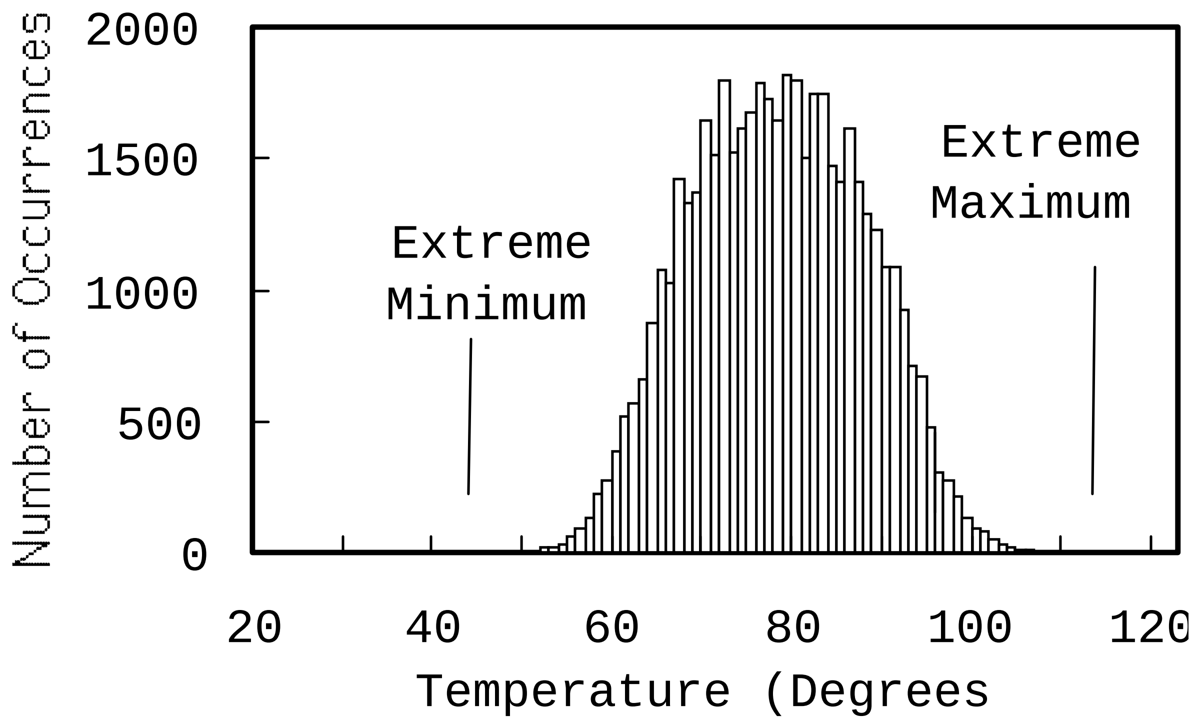

Figure 16. Histogram of Daily Temperature Variation for Concrete Bridges in Pasadena, California

While there is very large variations in the extreme low temperatures for these locations, the variations in the extreme high temperatures of the bridge are not that large. The mean temperature is also in the upper half of the temperature range. Examination of histograms from locations in all 50 states shows that the mean temperature typically is at the 60% to 65% point of the temperature range. The standard deviation of both the daily high and low bridge temperatures tend to be in the range of 13% to 16% of the total temperature range. This suggests that the bridge temperature will remain in a band that is approximately 26% to 32% of total temperature range for approximately 60% of the time. The bridge should remain within a temperature range of 52% to 64% of the total range approximately 90% of the life of the bridge. These factors have considerable impact upon the probable installation temperature, since the girder installation is likely to occur within this central temperature range.

Installation Temperature and Design Movements

Installation temperature is presently not directly considered by the present AASHTO Standard Specifications [1] for concrete bridges, since an implied installation temperature is included in the recommended design temperature limits. There is no rational reason for ignoring the installation temperature for concrete bridges, nor is there any rational reason behind the present provision. The statistical variation in bridge temperatures provides a rational basis for estimating bridge installation temperatures. Further, installation temperatures are somewhat different for bearings and expansion joints. As a result, installation temperatures for bearings and expansion joints are discussed separately.

Design Movements of Bearings

Two strategies for installation temperature are proposed for bridge bearings. For mechanical bearings, bearings have been historically designed for an approximate mean intermediate temperature, and an offset chart is provided to adjust the bearing alignment to account for the temperature at installation. This procedure has worked reasonably well and there is little reason to change this practice. It must be noted however that the consequences of inadequate movement capacity for a mechanical bearing are quite severe, since serious damage may occur if a mechanical bearing tips over or rolls off its support. The offset table provides substantial protection against this catastrophic event, but there is inherent uncertainty in the construction and installation process. A 20^{o}F hedge is recommended to account for this uncertainty. Therefore, for these types of bearings it is recommended that the installation temperature be

$$T_{Install} = T_{MinDesign} + \frac{T_{MaxDesign} - T_{MinDesign}}{2} \tag{6a}$$

the design movements, Δ, should be

$$\Delta = \pm \alpha L \left(\frac{T_{MaxDesign} - T_{MinDesign}}{2} + 20^{o}F\right) \tag{6b}$$

where L is expansion length between the fixed support and movable bridge bearing, $T_{MaxDesign}$ is the maximum design temperature, and $T_{MinDesign}$ is the minimum design temperature. The bearing must then be offset by an amount

$$\text{Increment } \Delta = \alpha L \, 5^{o}F \tag{6c}$$

for each 5 degree increment that the air temperature deviates from $T_{Install}$. While this technique is primarily intended for mechanical bearings, it could also be used for PTFE and other low friction sliding surfaces or other bearings which can be easily offset during erection.

Offset charts are not effective for the installation of elastomeric bearings, since they can not be easily deformed to achieve the offset deformation. To address this issue, the temperature data for 50 locations in the US was analyzed in much greater detail as noted earlier. The daily high and low average bridge temperatures were estimated for the entire 60+ year history of each of the 50 locations and were statistically analyzed. Figures 13, 14, 15 and 16 are typical histograms of the data for these locations. All curves have some similar attributes as noted earlier. While there is clearly a statistical variation in bridge temperature, the mean temperature is in the upper half of the temperature range. This mean temperature is clearly the most probable installation temperature, since more days are closer to the mean than closer to the extremes. Further, construction is much more likely to proceed during hot summer days than during cold winter nights. Thus, the most probably installation temperature should be at approximately 60% to 70% of the temperature range above the minimum temperature. The bottom 10% to 15% of the temperature is typically penetrated only very rarely, and erection is unlikely during darkness during the winter months. As a result, the proposed installation temperature for all bridge types is

$$T_{Install} = T_{MinDesign} + 0.65^{*} (T_{MaxDesign} - T_{MinDesign}) \quad (7a)$$

and the design movement. Δ, is

$$\Delta = \pm \alpha L \, 0.65^{*} (T_{MaxDesign} - T_{MinDesign}) \, . \quad (7b)$$

There is still considerable uncertainty as to the true installation temperature, and it is necessary to consider how this uncertainty will impact bridge performance. The translational movement capacity of elastomeric bearings is controlled by the 50% strain limit (or rubber thickness must be two times to maximum movement). This clearly means that the greatest risk to the bearing is that the girders and bearings will be installed on the very hottest days of the historic record. The probability of this is small (in the order of one in 25,000). However, even if this occurs the maximum strain in the elastomer will be less than 77% on the coldest day in the life of the bridge. Further, it will be at this extreme strain only once in 60+ years. The decision to install the girder on the very hottest days and the very lowest historic bridge temperatures are independent events, and so the probability that this extreme strain will be achieved is clearly very low. When the very low probability of occurrence of these extreme strains is considered, the larger strain is quite tolerable. Present strain limits are based upon fatigue tests [20] performed at the University of Washington. In these past tests, elastomeric bearings were cycled to more than 25,000 cycles of shear strains in excess of 70%. This larger strain limit was rejected as a design limit, because the edge of the bearing tends to roll over at these large strains, and significantly greater cracking of the rubber was noted after 25,000 cycles. However, a few cycles of these larger strains are clearly tolerable with no damage. Further, much larger strains are invariably employed with seismic base isolation bearings. Therefore, girders must subsequently be lifted if the actual girder temperature at installation exceeds

$$\{T_{MinDesign} + 0.9 * (T_{MaxDesign} - T_{MinDesign})\}$$

or is less than

$$\{T_{MinDesign} + 0.25^{*} (T_{MaxDesign} - T_{MinDesign})\}.$$

The girder temperature of steel bridges at the time of installation may be taken as the air temperature at the time of setting. The girder temperature of concrete bridges at the time of installation may be taken as the average of the daytime high air temperature and the previous night low air temperature. The exceedance of the high temperature is possible for steel bridges, since construction is commonly completed in the summer, but the lower limit is an extremely unlikely event for either bridge type. This lifting operation will allow the elastomer to recover to an intermediate strain condition, and reduce the maximum strains to much more normal values.

This procedure permits the elastomeric bearing to be designed for an intermediate installation temperature and perform well without lifting the girders or adjusting the position. Elastomeric bearings are very forgiving as are elastomeric bearings. However, many bridge components are not so forgiving. Thus, while the proposed design movements are

$$-0.65*(T_{MaxDesign} - T_{MinDesign}) * \alpha * L$$

and

$$+0.35*(T_{MaxDesign} - T_{MinDesign}) * \alpha * L,$$

larger movements are required between structural elements with hard contact. Examples of this type of contact would be contact the girders with the abutment backwall, end contact of the length of slotted holes with anchor bolts, or other similar condition. It is proposed that the design movement for these “hard contact” elements be at least 150% of that noted above to account for the uncertainties in the installation temperature. No adjustments are required if these larger hard contact limits are maintained.

Design Movements of Expansion Joints

Expansion joints are normally installed after the girders are in place and after the concrete deck and abutments have been formed and placed. The positive and negative movements required of the expansion joint are determined by the formwork for concrete placement and the average bridge temperature when the formwork is completed. The average bridge temperature at the time the formwork is placed controls the gap for the expansion joint. The actual expansion joint system is compressed to fit the gap at the time and temperature at installation. It is recommended that the installation temperature of the joint, $T_{Install}$, be defined by

$$T_{Install} = \frac{T_{MaxAir} + T_{MinAir}}{2} \tag{8a}$$

where T_{MaxAir} is the maximum daily air temperature for the previous day, and T_{MinAir} is the minimum night time air temperature for the morning of the day that the joint gap is defined. There is still some uncertainty in the actual installation temperature, but this uncertainty is limited by the daily temperature range of the bridge. It is therefore recommended that the expansion joint be designed for the total movement, Δ, where

$$\text{Total Movement } \Delta = \pm \alpha L (T_{MaxDesign} - T_{MinDesign} + 30), \qquad (^{o}F) \tag{8b}$$

and where the expansion gap is

$$\text{Expansion Gap } \Delta = \pm \alpha L (T_{MaxDesign} - T_{Install} + 15). \qquad (^{o}F) \tag{8c}$$

Chapter 4

Verification of Design Proposals

Verification Requirements

Chapter 3 provided design recommendations for thermal movements. These recommendations have considerable impact on bridge design, but they are based upon simplified theoretical calculations. The underlying theories were checked in some past field measurements [6,11,13,15], and very good correlation between measured field behavior and computed behavior was noted. The simplifications to the theories have been calibrated to the more complex theoretical calculations. In this chapter, field data is also compared to the recommendations so that the recommendations can be fully accepted and adopted by bridge engineers.

For this comparison, measurements of temperatures and thermal movements are compared to the design recommendations and design models for bridges in different parts of the US for both winter and summer conditions. Chapter 2 has shown that temperature varies over the cross section of the bridge, and so temperatures must be measured at different locations on each bridge and averaged over the bridge cross section to accomplish this comparison. The measurements should ideally include approximately one week of winter and one week of summer data at each site. The bridges need not be continuously monitored, but measurements should be made at intervals throughout each day, since this is adequate to estimate the extreme daily temperatures. The field information must then be combined with the analytical research and the design maps and recommendations described earlier in this report.

Observations Based on Past Practice

One important observation in validating the design recommendations is the general observations as to how the recommendations fit with the experience of bridge engineers and the past performance noted in bridges throughout the US. Past research [3] has shown that the recommendations for steel bridges results in significant changes in thermal movement design for steel bridges. The recommendations for steel bridges will clarify the role of the installation temperature, and they will eliminate the present ambiguity of the cold and mild climate designations. Somewhat larger bridge design movements are required in a few north central parts of the US, but smaller thermal design movements are possible in most other parts of the country. The recommended changes to the thermal movement requirements for concrete bridges are relatively modest. Comparison of Figures 9 and 10 with existing AASHTO Provisions shows that the total temperature range for concrete bridges is very similar to the present AASHTO Mild and Cold Climate requirements for concrete bridges for most of the US. The interior northern part of the US including Illinois, Wisconsin, Minnesota, North Dakota, South Dakota, Montana, Wyoming, Nebraska and Iowa are the only regions where significant changes in the temperature range can be noted. Within, this region the recommendations result in somewhat larger ranges of thermal movement than the present AASHTO provisions. In many cases, the recommended temperature range is approximately $10^{O}F$ larger than the present AASHTO provisions, but in the extreme case the recommended temperature range for concrete bridges is nearly $70^{O}F$ larger than the AASHTO recommendations. There have not been a large number of problems reported with bridge thermal movements, and so it is logical to ask whether the recommendations are consistent with behavior observed by bridge engineers. As a result, a brief survey of bridge engineers in this critical region was completed. The survey showed that many of the bridge engineers within this region already recognized that the thermal movements for concrete bridges were larger than those predicted by AASHTO, and these engineers were already using much larger design movements for concrete bridges within their state.

Minnesota DOT will be most substantially affected by these revised recommendations, and an engineer from this organization [21] provided the strongest evidence for increasing design thermal movements for concrete bridges in that region. This engineer measured the relative distance between the bridge superstructure and the abutment for extreme winter and summer conditions for a concrete box girder bridge, a precast concrete girder bridge, and a four span continuous steel girder bridge. The concrete bridge measurements were taken on February 11, 1981, and July 20, 1982. The steel bridge measurements were taken for January 16, 1982, and July 20, 1982. Summertime measurements were taken when the air temperature was near $90^{O}F$ for all three bridge sites. The wintertime measurements were taken when the air temperature was approximately $-20^{O}F$. The days preceding the steel bridge measurements had $-60^{O}F$ wind chill factors, and the days preceding the winter measurements of the concrete bridges were cold but somewhat milder than that noted for the steel bridge. These observations show that the measurements were taken at near extreme temperatures for the climate, but certainly not at the extreme temperatures. The change in bridge expansion between the winter and summer data can be used to estimate a temperature range through application of Eq. 1 of this report. The application of Eq. 1 with this measured data shows that,

$$\Delta T = \frac{\Delta_{meas}}{\alpha L_{exp}} \tag{9}$$

where Δ_{meas} is the measured change in deflection of the bridge, α is the coefficient of thermal expansion of the bridge, and L_{exp} is the bridge length over which the expansion and contraction occurs. The change in temperature range, ΔT, was determined from these measured displacements. Design temperature ranges were determined from Figs 2 and 3 for the steel bridge and from Figs. 9 and 10 for the two concrete bridges and comparisons are tabulated in Table 2.

The tabulation shows that the measured temperature range compared well with the temperature range recommended by the design maps for both concrete bridges and steel bridges with concrete decks. The temperatures were not extreme at the time of the measurements, but were near extreme conditions. Table 2 shows that the measured temperature range is approaching the design temperature range, but is well within the design range. The measured ranges for the concrete bridges greatly exceed present AASHTO temperature recommendations. The relative proximity of the measured design movement to the extreme design limits is similar, since the weather was similar for all bridge types. This data provides substantial support for the recommended changes in the design procedure.

Table 2. Comparison of Design Recommendations to Minnesota Data

Bridge Type	Temperature Range, ΔT, Inferred from Displacement Measurements	Temperature Range Predicted by Design Map of this Report
Steel Girder Bridge with Concrete Deck	138^{o}F	147^{o}F
Precast Concrete Girder Bridge	110^{o}F	130^{o}F
Concrete Box Girder Bridge	107^{o}F	130^{o}F

Field Measurement Data

Field measurements are costly and time consuming, and there are virtually no existing field measurements specifically for thermal movements in bridges in the US. As a result, existing data obtained for other purposes from bridges at different locations in the US were examined to provide a correlation between the design recommendations and the field data. In general, the existing measurements were secondary measurements made during a study that was not focused on the thermal movement problem. However, this existing data can still be used as a check of the recommendations from this study.

University of Wyoming Study (Ref 22)

One very useful comparison between past field measurements and the design recommendations for steel bridges with concrete decks was provided from data reported by Croft, Puckett, and Dolan [22]. In this study, 7 steel girder bridges in Wyoming were measured for bridge temperatures and movements. The data from this study was relatively complete, because the study was started because of movement damage noted on sharply skew and curved bridges in Wyoming, but bridge geometry was the primary focus of the study. The temperatures and movements were measured at several locations of each bridge, and at several different times and dates at each location. The average bridge temperature was obtained by integration of the measured temperatures over the cross section of the bridge, since the average bridge temperature controls movement. Thus, detailed evaluation of the bridge geometry and design drawings was required for each bridge and temperature record to translate the measured data into the average value. Design drawings and other information about the geometry and location of the bridges were obtained from the Wyoming DOT. The integration is a slightly conservative evaluation, since it overestimates the extreme high temperature of the bridge and underestimates the extreme low temperatures of the bridge.

Wind River Bridge

The Wind River bridge is a sharply skew (approx. 60^{O}) bridge located in DuBois, Wyoming, with 3 continuous spans of approximately 437 feet total length. Temperatures were measured various times at 16 locations of the bridge from the evening of August 17, 1993 through early morning of August 20, 1993. Figure 17 shows a comparison of the measured average bridge temperatures as compared to the high temperature predicted by the simplified models used in this study with weather conditions noted on those dates and the extreme design limit is provided by the proposed design map in Fig. 2. The averages were computed by consideration of all measured data and the bridge geometry. Late August is normally a relatively hot summer month, but during this period weather was overcast and air temperatures and bridge temperatures were well below the extreme limits for this month in this region. The diamonds in this plot indicate the average bridge temperatures derived from the measured data. The solid squares with the connecting line define the maximum design temperature based on Fig. 2, and the solid triangles with connecting lines define the upper limit on the bridge temperature as defined by the Kuppa model for the weather conditions during the period of the field measurements. The figure clearly shows that the bridge temperature varies during the course of the day, but the design model accurately estimates the upper limits. The Kuppa model provides a reasonable estimate of the maximum bridge temperature for these specific weather conditions.

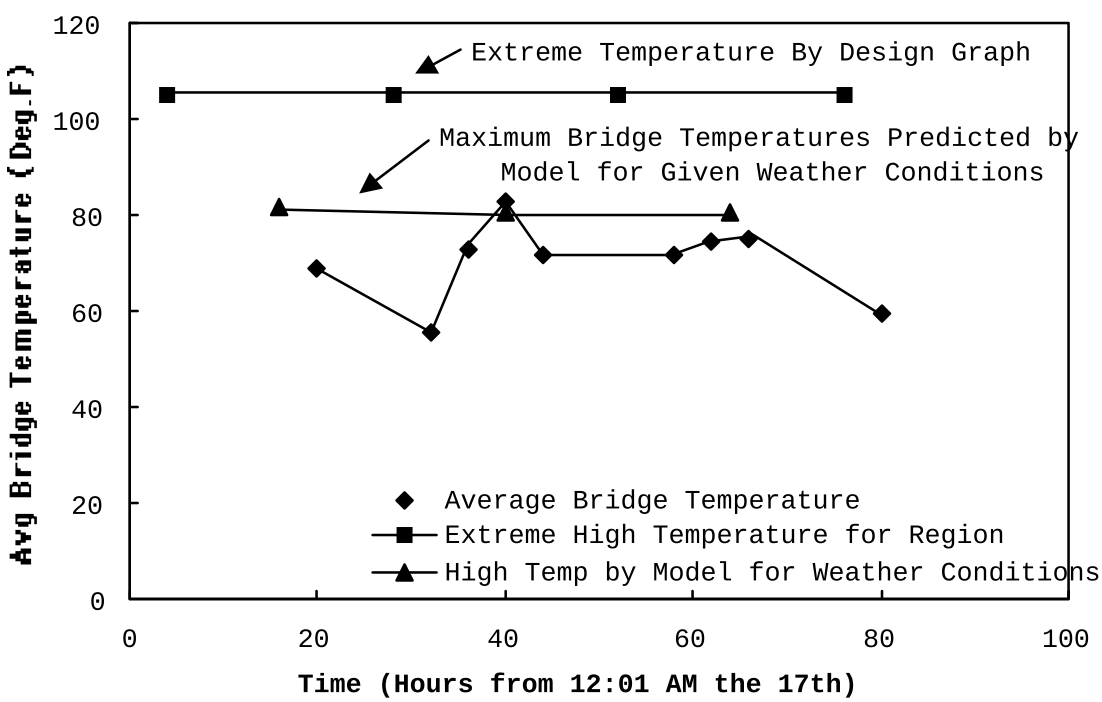

Figure 17. Measured Bridge Temperatures for Wind River Bridge and Comparison to Design Models

Casper Creek Bridge

The Casper Creek bridge carries Alternate US Routes 20 and 26 over Casper Creek in Casper, Wyoming. The bridge has 10 steel girders with a sharp skew (approximately 40^{o}), and the 3 continuous spans have a total length of approximately 148 feet. Measurements were taken from the afternoon of July 28, 1993 through morning of July 30, 1993. The weather was quite hot during these days and this data provides a comparison of hot summertime weather to the design maps. Temperatures were measured at various times at 4 locations of the bridge for whole period but at 16 locations for one day of the time period. Figure 18 shows a comparison of the measured average bridge temperatures as compared the high temperature predicted by the simplified models used in this study with weather conditions noted on those dates and the extreme design limit provided by the proposed design map in Fig. 2. The diamonds, squares and triangles again have the same meaning as illustrated in Fig. 17. Comparison of these data points again show that the Kuppa model again provides a good envelop for the maximum average bridge temperature, and the design limit is well above the daily maximum values as should be expected on a hot day which is below record temperatures. The bridge temperature again varies during the day, because of variation of the position of the sun and air temperature.

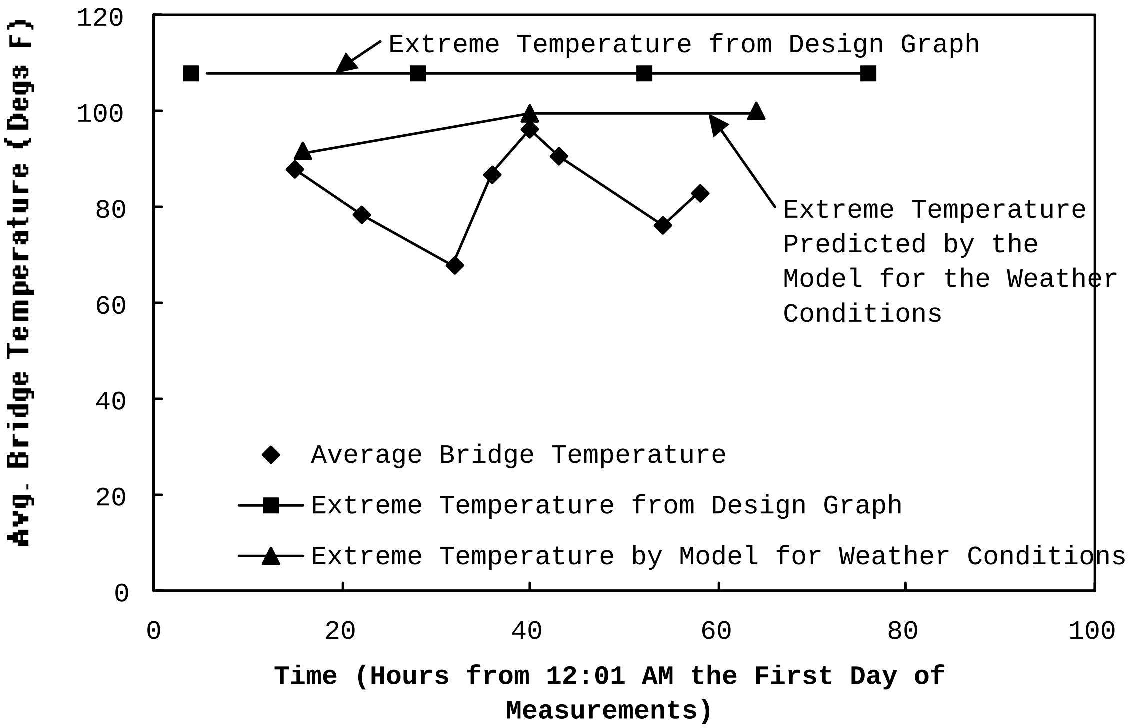

Figure 18. Measured Bridge Temperatures for Casper Creek Bridge and Comparison to Design Models

Other Wyoming Bridges

In addition,

- the Burlington Northern Railroad Overpass, in Gillette, at the Wyoming-Montana State Line,
- the Laramie Union Pacific Railroad Bridge, in Laramie, Wyoming,
- Herrick lane separation over Interstate 80 in Laramie, Wyoming,
- the Casper Street, Mills Spur east overpass over the C&NR railroad in Casper, Wyoming,
- and the Curtis Street Bridge over the Laramie River in Laramie, Wyoming

were included in this study. Analyses were completed for all the data for all of these bridges. In general, the correlation obtained for these other bridges were similar to that noted for Figs. 17 and 18 and as a result they are not repeated here. The Laramie Union Pacific Railroad Bridge provided some of the potentially most useful data, because it included wintertime and summertime measurements. There is a clear shortage of winter data, but the winter data for this bridge was not useful in evaluating the design recommendations, because there were clear and obvious discrepancies in the data. Among the more obvious discrepancies, it was noted that the air temperature reported for the bridge site varied approximately 20^{o}F from the temperatures reported by the US Weather Service for that location. The design model is based upon US Weather data and as a consequence this discrepancy would not provide a rational check of the procedures used in this report.

New Jersey DOT Study (Refs 23, 24, 25, and 26)

During the early 1970's, the New Jersey DOT performed automatic monitoring of two simple span steel girder bridges with composite decks. Data was recorded at two-hour intervals from December 28, 1972, through December 28, 1973. Temperatures and displacements were measured at numerous locations on each bridge. This data was recorded 25 years ago and was documented differently than the University of Wyoming data. Therefore, comparisons must be made differently for this data than for that included in Figs 17 and 18. The comparison again used a weighted average of the monthly extreme thermocouple measurements to estimate extreme average bridge temperatures for each month and these are compared to the design limits for the two bridges in Figs. 19 and 20. The weighting procedure used for these two bridges produces a more conservative temperature estimate than for the bridges in the Wyoming study, because it is also assumed that extreme temperatures reported in the study all occurred at the same instant at all locations. This assumption is incorrect, and it adds to the conservatism in the measured estimates. That is, the maximum average bridge temperature estimated from this data always is larger than the true average bridge temperature, and the minimum average bridge temperature always is smaller than the true value. The estimated temperatures are well within the band defined by the design maps for the entire year, but they are clearly approaching the design limits during summer and winter extremes. This further verifies the reliability of the design model.

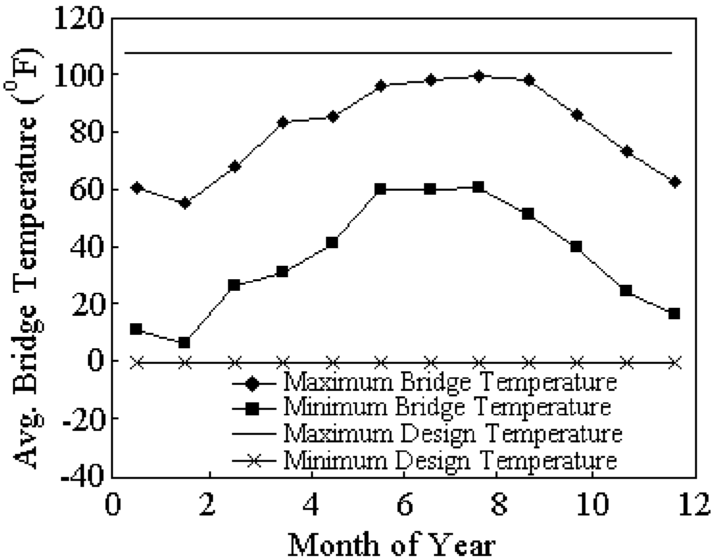

Figure 19. Comparison of Measured Bridge Temperatures to Design Limits for Bridge 1

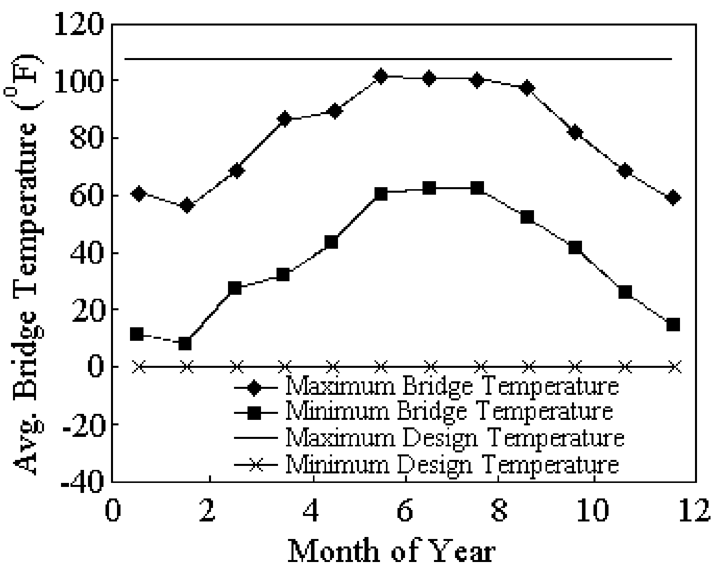

Figure 20. Comparison of Measured Bridge Temperatures to Design Limits for Bridge 5

New York DOT (Ref 30)

New York DOT performed a limited series of temperature measurements on a curved steel girder bridge during the period August 31, 1994, through September 2, 1994. Temperatures were measured at a very limited number of locations at the top of the concrete deck and in the steel girders. Average maximum bridge temperatures were estimated for the bridge as 86^{O}F from the measured data for this period, and the Kuppa model predicted a maximum average bridge temperature of 86.2^{O}F from the weather data for that location for this period. The design map indicates a maximum design temperature of 109^{O}F for this region. This comparison suggests very good correlation between the design recommendations, the analytical methods used to establish these recommendations, and the measured behavior.

Louisiana State University Study (Ref 33)

Pentas, Avent, Gopu, and Rebello [23] measured temperatures and movements at expansion joints on several segments of the US 190 crossing of the Atchafalaya River at Krotz Springs, Louisiana. This is a multi-span bridge with a number of expansion segments. One single span segment is a steel girder with a concrete slab. The other 3 segments are multiple span prestressed concrete girders. The goal of this research appears to be directed toward a better understanding of the design and behavior of expansion joints. Thus, the thermal movements and temperatures were important but of secondary interest to the global project. The temperature data was not adequately documented to fully evaluate the proposed design recommendations. Measurements were taken intermittently for approximately one day per month through late 1987, 1988, and part of 1989. The elongation and contraction could then be used to infer a design temperature range by methods similar to those noted in the Minnesota DOT work. However, the researchers did not separate the effects of the steel span from the concrete span in the temperature evaluation. Nevertheless, the maximum temperature range noted for any part of the bridge was 70^{O}F and the maximum range when it is applied to entire bridge length was approximately 55^{O}F. These again compare very well with the Kuppa model and the design recommendations, since the maximum range for steel bridges with concrete decks is 100^{O}F and a range of approximately 80^{O}F would be obtained if the Kuppa method were used to develop design maps for the concrete alternative. The proposed methods appear to be both conservative and accurate based on this data.

Measurements Provided by AASHTO T2 Committee

A final comparison between design models and recommendations and field measurements was made with the aid of data provided by members of the AASHTO T2 Technical Committee for Bearings and Expansion Devices. State bridge engineers from this committee monitored the movements and temperatures of 41 bridges in Colorado, Illinois, Iowa, Kentucky, Maine, Missouri, Montana, New Jersey, North Carolina, and Pennsylvania during the hottest and coldest periods of 1999 through 2001. More detailed information on the types of bridge, location of bridges, and the measured data is provided in Appendix E. The appendix summarizes the data and provides identifiers for each bridge used in this chapter.

Evaluation of Design Recommendations

Temperatures were measured at the bridge girders and at the top and bottom of the bridge deck on a hot summer afternoon and a cold winter morning. In addition, the gap at movable joints was measured to estimate total range of bridge movement between winter and summer for this time period. The measurements included steel girders with concrete decks, steel and concrete box girders, prestressed concrete girders and several other bridge types. The high and low average bridge temperature for each bridge was estimated from this measured data through application of Eq. 2. The state bridge engineers provided drawings of each bridge to aid in application of this equation. Table 3 summarizes data from 35 of these bridges. Six bridges were excluded from the table. One of the excluded bridges was from Kentucky and 5 were from Pennsylvania. Some of the excluded bridges were truss bridges and these bridges are not covered by the proposed design recommendations. However, others were excluded, because of incomplete information on the bridge or the measurement data. In particular, the drawings of several of the excluded bridges were more than 60 years old. The drawings had been photographed and copied several times, and it was not possible to read some dimensions that are required for the averaging process from those older drawings. As a result, these 6 bridges were excluded from all comparisons that follow. A number of the bridges in Table 3 have wintertime data only. The wintertime only data does not provide the full desired information, but this data is still useful, because of the general shortage of winter data.

The extreme temperatures were always well within the design temperature range. In no case, did any of bridge measured temperatures come closer than 11^{o}F (6.1^{o}C) of the design limit, and this proximity was very rare. This measured temperature range was always smaller than the existing AASHTO recommendations for a mild climate for all steel bridges in all locations, but the measurements for concrete bridges were commonly larger that the existing AASHTO temperature ranges. The measured temperature range was always between 40% and 60% of the proposed design temperature range for all bridge types. At first glance, this may suggest that the recommended design limits are overly conservative, but Fig. 21 shows that that clearly is not the case. The design temperature limits are based upon the extreme bridge temperatures that will occur only once in a 60-year period. The measurements were typically completed over a 6-month period, and a normal statistical distribution as illustrated in Fig. 21 indicates that the expected maximum variation for 6 months of measurement data is 69% of the variation expected over a 60-year period. In addition, the notes provided in Appendix E show that many of the measurements were not accumulated on the very hottest or coldest days expected for that region during the 6 month period. Therefore, the 40% and 60% variation is quite appropriate as compared to the 69% maximum expected range. It does not appear that the recommendations are too conservative when these field measurements are considered. At the same time, the measured temperature range exceeded the present AASHTO recommended temperature range for several concrete bridges during this six-month period. This provides further definitive evidence that the present AASHTO design limits underestimate the thermal design movements for this bridge type.

Table 3. Summary of Field Measurements

Bridge Identification see Appendix E	Measured High Temp. ^oF (^oC)	Measured Low Temp. ^oF (^oC)	Recommended High Temp ^oF (^oC)	Recommended Low Temp. ^oF (^oC)	Notes
CO-1	88.4 ^oF (31.3^oC)	39 ^oF (3.9 ^oC)	118 ^oF (47.8 ^oC)	-20 ^oF (-28.9 ^oC)	
CO-2	90.4 ^oF (32.4 ^oC)	42.4 ^oF (5.8 ^oC)	110 ^oF (43.3 ^oC)	-20 ^oF (-28.9 ^oC)	
CO-3	94.8 ^oF (34.9 ^oC)	40.1 ^oF (4.5 ^oC)	118 ^oF (47.8 ^oC)	-20 ^oF (-28.9 ^oC)	
CO-4	90.4 ^oF (32.5 ^oC)	40 ^oF (4.5 ^oC)	110 ^oF (43.3 ^oC)	-20 ^oF (-28.9 ^oC)	
IL-1	90 ^oF (32.2 ^oC)	41.8 ^oF (5.5 ^oC)	110 ^oF (43.3 ^oC)	0 ^oF (-17.8 ^oC)	
IL-2	85.4 ^oF (29.7 ^oC)	11 ^oF (-11.7 ^oC)	115 ^oF (46.1 ^oC)	-5 ^oF (-20.6 ^oC)	
IL-3	86.3 ^oF (30.2 ^oC)	16 ^oF (-8.9 ^oC)	115 ^oF (46.1 ^oC)	-5 ^oF (-20.6 ^oC)	
IL-4	83.1 (28.4 ^oC)	19 ^oF (-7.2 ^oC)	110 ^oF (43.3 ^oC)	0 ^oF (-17.8 ^oC)	
IL-5	88.6 ^oF (31.5 ^oC)	33 ^oF (0.6 ^oC)	115 ^oF (46.1 ^oC)	-5 ^oF (-20.6 ^oC)	
IL-6	85.7 ^oF (29.8 ^oC)	22 ^oF (-5.6 ^oC)	110 ^oF (43.3 ^oC)	0 ^oF (-17.8 ^oC)	
IA-1	----	9.8 ^oF (-12.3 ^oC)	120 ^oF (48.9 ^oC)	-20 ^oF (-28.9 ^oC)	Winter only
IA-2	----	6 ^oF (-14.4 ^oC)	110 ^oF (43.3 ^oC)	-10 ^oF (-23.3 ^oC)	Winter only
KY-1	95.7 ^oF (35.4 ^oC)	37.6 ^oF (3.1 ^oC)	112 ^oF (44.4 ^oC)	-10 ^oF (-23.3 ^oC)	
KY-2	93.8 ^oF (34.4 ^oC)	37.8 ^oF (3.2 ^oC)	105 ^oF (40.6 ^oC)	-28 ^oF (-33.3 ^oC)	
ME-1	----	18 ^oF	105 ^oF	-23 ^oF	Winter only

		(-7.8 ^oC)	(40.6 ^oC)	(-30.6 ^oC)	
ME-2	----	16.3 ^oF (-8.8 ^oC)	105 ^oF (40.6 ^oC)	-23 ^oF (-30.6 ^oC)	Winter only
ME-3	----	13.1 ^oF (-10.5 ^oC)	105 ^oF (40.6 ^oC)	-23 ^oF (-30.6 ^oC)	Winter only
ME-4	----	34.3 ^oF (1.3 ^oC)	105 ^oF (40.6 ^oC)	-23 ^oF (-30.6 ^oC)	Winter only
ME-5	----	16.5 ^oF (-8.6 ^oC)	105 ^oF (40.6 ^oC)	-23 ^oF (-30.6 ^oC)	Winter only
MO-1	----	41.8 ^oF (5.4 ^oC)	119 ^oF (48.3 ^oC)	-10 ^oF (-23.3 ^oC)	Winter only
MO-2	----	31.4 ^oF (-0.3 ^oC)	119 ^oF (48.3 ^oC)	-10 ^oF (-23.3 ^oC)	Winter only
MO-3	----	37.7 ^oF (3.2 ^oC)	119 ^oF (48.3 ^oC)	-10 ^oF (-23.3 ^oC)	Winter only
MO-4	----	36.4 ^oF (2.4 ^oC)	113 ^oF (45 ^oC)	0 ^oF (-17.8 ^oC)	Winter only
MO-5	----	44 ^oF (6.7 ^oC)	113 ^oF (45 ^oC)	0 ^oF (-17.8 ^oC)	Winter only
MT-1	65.7 ^oF (18.7 ^oC)	17.6 ^oF (-8 ^oC)	110 ^oF (43.3 ^oC)	-40 ^oF (-40 ^oC)	
MT-2	65.9 ^oF (18.8 ^oC)	16.3 ^oF (-8.7 ^oC)	105 ^oF (40.6 ^oC)	-40 ^oF (-40 ^oC)	
NJ-1	----	34.7 ^oF (1.5 ^oC)	100 ^oF (37.8 ^oC)	8 ^oF (-13.3 ^oC)	Winter only
NJ-2	----	35 ^oF (1.6 ^oC)	108 ^oF (42.2 ^oC)	0 ^oF (-17.8 ^oC)	Winter only
NC-1	95 ^oF (35 ^oC)	40.5 ^oF (4.7 ^oC)	110 ^oF (43.3 ^oC)	25 ^oF (-3.9 ^oC)	
NC-2	98.6 ^oF (37 ^oC)	33.9 ^oF (1 ^oC)	110 ^oF (43.3 ^oC)	15 ^oF (-9.4 ^oC)	
PA-1	83.5 ^oF (28.6 ^oC)	16.7 ^oF (-8.5 ^oC)	105 ^oF (40.6 ^oC)	-15 ^oF (-26.1 ^oC)	
PA-2	84.4 ^oF	24.6 ^oF	105 ^oF	-15 ^oF	

	(29.1 °C)	(-4.1 °C)	(40.6 °C)	(-26.1 °C)	
PA-3	78.3 °F (25.7 °C)	15.5 °F (-9.2 °C)	100 °F (37.8 °C)	-10 °F (-23.3 °C)	
PA-4	76.2 °F (24.6 °C)	43 °F (6.1 °C)	105 °F (40.6 °C)	0 °F (-17.8 °C)	
PA-5	76.2 °F (24.5 °C)	45 °F (6.7 °C)	105 °F (40.6 °C)	0 °F (-17.8 °C)	

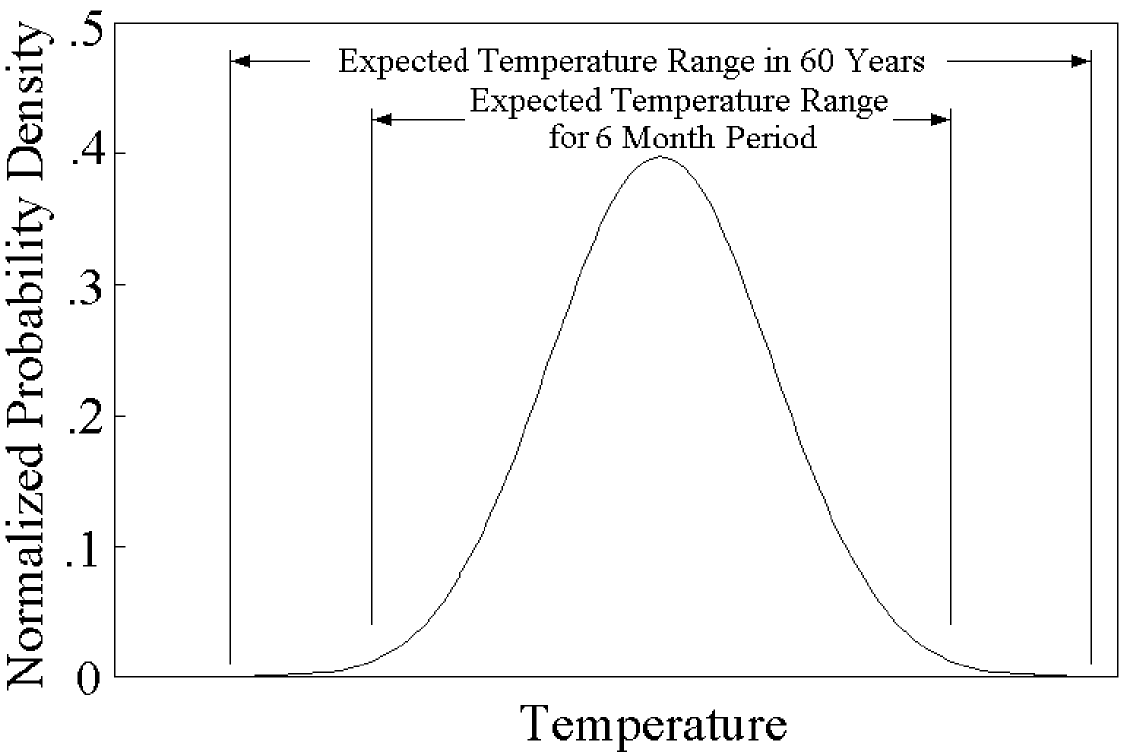

Figure 21. Statistical Variation in Bridge Temperatures

Validity of the Kuppa Model

The Kuppa model was used for establishing the design recommendations included in this report. This model was developed for predicting the extreme maximum and minimum average bridge temperatures (rather than the day to day bridge temperatures) based upon the long term weather data. The T2 committee data is not sufficient to provide an accurate evaluation of the Kuppa model, because -

- long term temperature data is not available at the bridge site, and
- the measured conditions are not quite extreme conditions as shown in Fig. 21 and prior discussion.

However, longer term data was often available for locations within 20 miles of the bridge site, and this can be used to provide approximate verification of the model as shown in Figs. 22 and 23.

Figure 22 shows the temperature predicted by the Kuppa model divided by the measured average bridge temperature for summertime measurements. The bridges are identified by state. The variation is generally less than approximately 5%, and the variation is slightly greater for concrete bridges than for steel bridges. Figure 23 shows the difference between the high and low temperature for the Kuppa model and the measured average bridge temperatures. The hollow squares are steel bridges and the solid squares are concrete bridges. The bridges are again identified by state. The scatter is larger in the low temperature estimates than for the high temperature estimates shown in Fig. 22. This should be expected because there is much wider variation in low temperatures than in high temperatures as noted in the discussion of Chapters 2 and 3. It should be recalled that the temperature range for the Kuppa model is based on temperature data recorded at a site near but not at the bridge site. The variation is smaller for the steel bridges than for the concrete bridges. Some winter time data is not included in this table, because the bridge measurements were made during 2001, and the required wintertime weather data for this period is not yet available. These comparisons show that the Kuppa model is a good indicator of the extreme average bridge temperatures for both winter and summer conditions.

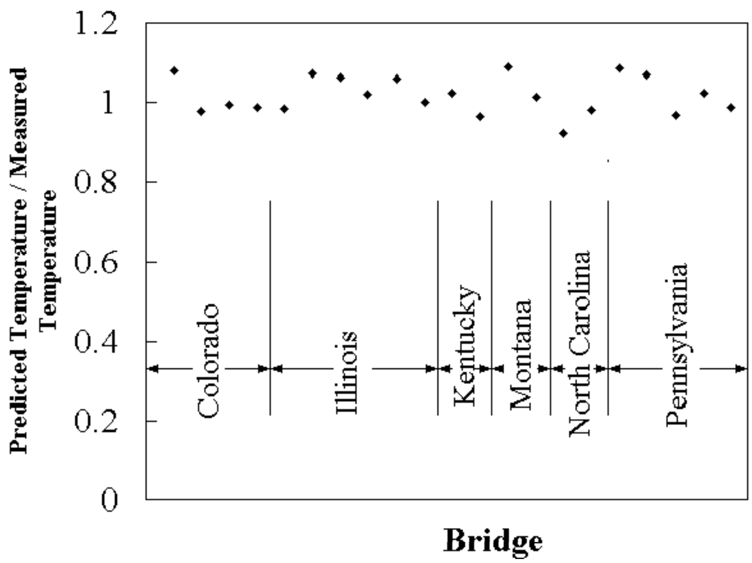

Figure 22. Approximate Comparison of the Measured Maximum Average Bridge Temperature with the Predicted Maximum Average Bridge Temperature from Kuppa Model and Data from a Nearby Site

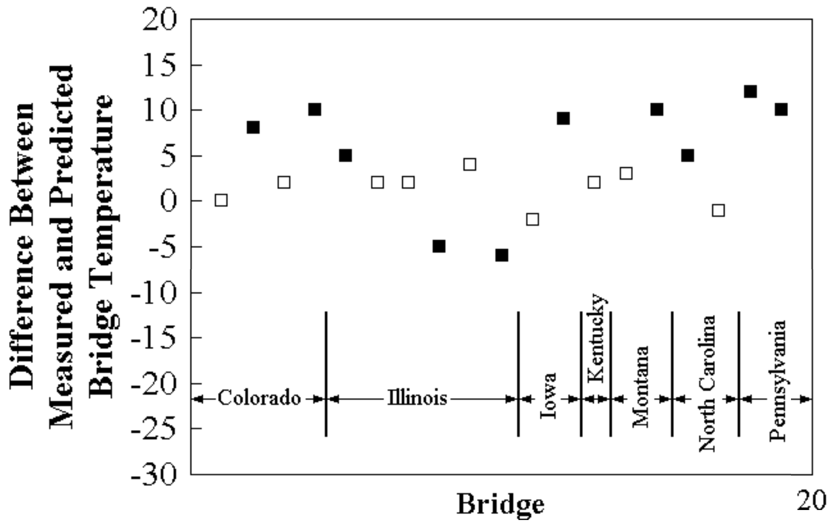

Figure 23. Approximate Comparison of the Measured Minimum Average Bridge Temperature with the Predicted Minimum Average Bridge Temperature from Kuppa Model and Data from a Nearby Site

<u>Evaluation of Bridge Temperatures Based Upon Measured Movements</u>

Displacements were measured for the 35 bridges listed in Table 2. Joint displacements were measured for each of those bridges, and for the 21 bridges with both winter and summer displacement data, the change in bridge expansion between the winter and summer data can be used to estimate a temperature range through application of Eq. 9 of this report. These 21 bridges are bridges from Colorado, Illinois, Kentucky, Montana, North Carolina, and Pennsylvania. Table 4 summarizes the results of the application of Equation 9 and compares them to the measured bridge temperature from Table 3. It can be seen that the comparison is very good. While the application of Eq. 9 is intuitively easy it is practically impossible for a number of the 21 bridges. Most of these difficulties center around determination of L_{exp}. For a straight, right bridge, L_{exp} is normally the distance between the fixed point of the bridge and the expansion joint where deflections were measured on the bridge. However, many of the bridges included in this survey had multiple fixed points, because the bridge superstructure was integral or partially integral with the bridge piers. Under these conditions, the thermal expansion is partially taken by the deflection and deformation of the bridge piers. The length, L_{exp}, is unclear under these conditions, and Eq. 9 can not be applied. In other cases, cases the fixity of the multiple span bridge was unclear from the limited drawings provided with the bridge data. Only one of the 4 Colorado bridges are included in this table, because CO-3 and CO-4 where fixed at interior piers and L_{exp} could not be determined. Only two of 6 Illinois bridges (IL-3 and IL-5) are included in Table 3, because these bridges also were all long multiple span bridges that were nominally "fixed" at more than one interior pier. In addition, several of the measured movements for the Illinois bridges contained thermal movement from two different superstructure systems. KY-2 was excluded from Table 2 because this bridge had two internal fixed piers, and the expansion length was unclear without information regarding the pier stiffness. PA-5 and NC-2 were excluded for similar reasons. As would be expected, the measured movement was generally much smaller for these excluded bridges than predicted if L_{exp} were set as the length between the centroid of the fixed points and the measured joint. This occurs, because the forces and moments needed to deform and deflect the internal piers always reduce the magnitude of the bridge movement.

Table 4. Inferred Temperature Range from Measured Bridge Movements

Bridge Ident.	Measured Movement Range (inches)	Bridge ΔT_{meas} ^oF from Table 2	Expansion Length (inches)	ΔT_{disp} ^oF as Determined from Eq. 9	Ratio of ΔT_{meas} to ΔT_{disp}
CO-2	1.06"	48 ^oF	4393"	44 ^oF	0.92
IL-3	2.50"	70.3 ^oF	5579"	69 ^oF	0.98
IL-4	0.44"	55.6 ^oF	1176"	58 ^oF	1.04
KY-1	0.50"	58.1 ^oF	1835"	42 ^oF	0.73
MT-1	0.55"	48.1 ^oF	1558"	54 ^oF	1.12
MT-2	0.95"	49.6 ^oF	3540"	49 ^oF	0.99
NC-1	0.35"	54.5 ^oF	1034"	62 ^oF	1.14
PA-1	1.88"	66.3 ^oF	4284"	68 ^oF	1.03
PA-2	1.06"	59.8 ^oF	3636"	53 ^oF	0.89
PA-3	0.50"	62.8 ^oF	1020"	75 ^oF	1.19
PA-4	0.31"	33.2 ^oF	2760"	20 ^oF	0.60

The ratios provided in Table 4 are all very close to 1.0, and this shows that the techniques used to determine average bridge temperature lead to accurate estimates of bridge movement as described in Chapter 2. It should be noted that the bridge movements were typically measured to the nearest $^1/_{16}$th or $^1/_8$th of an inch, and this limiting accuracy becomes quite important since the measured movements were relatively small. A $^1/_{16}$th inch error in the measured movement leads to a 13% error in the ratio provided in Table 4, if the total movement is approximately 0.5 inches. Table 4 shows that the average ratio is approximately 0.98 for all specimens, and the bridges with variations greater than approximately 5% all had small total bridge movement, and so the differences are readily explained by the accuracy of the measurements. This table shows that there is good agreement between the two movement predictions.

Final Observations

This comparison with field measurements has provided good support for the proposed design methods and the simplified analytical procedures used to develop these methods. The comparisons with field measurements indicate that the design recommendations are conservative, in that no single observed temperature was close to the upper or lower design temperatures. This is appropriate, since Chapter 3 has shown that these extreme limits are likely to be achieved no more than about once every ten years. At the same time, they show that the recommendations are not overly conservative in that a number of concrete bridges provided measured temperatures and thermal movements that exceed the existing AASHTO design recommendations. Further, the measured movements and temperatures appear to be appropriate with the 60+ history used for the extreme design temperatures included in the design recommendations. In view of the good correlation between the field data and the observed behavior, proposed wording for specification provisions for the adoption of the recommendations in Chapter 4 are included in the appendices. Appendix C contains recommendations that can be used for the AASHTO Standard Specifications, and Appendix D contains recommendations for the AASHTO LRFD Specifications.

Chapter 5

Summary and Conclusions

Results of the Research Study

This report has summarized a study into thermal movements of concrete bridges. The work followed the rational developed in an earlier study related to thermal movements for steel bridges with concrete decks. The work was entirely an analytical study, which included no experiments or field measurements, but verification of the analytical methods was made against existing field measurement data and temperature and movement measurements provided by various state bridge engineerings. Analytical methods were examined and evaluated, and two simplified procedures were investigated in some detail. Rational design recommendations were then developed. A number of points are worth emphasizing -

1. Proposed design maps were developed for concrete bridges, and they are provided in Chapter 3. Comparison of these maps to the maps developed for steel bridges with concrete decks in the earlier study [3] showed that concrete bridges will generally sustain smaller thermal movements than steel bridges, but the difference is not as large as the present AASHTO provision would suggest.

2. Comparison of the proprosed design maps for concrete bridges with the present AASHTO provisions, shows that the proposed maps will result in similar thermal design movements for concrete bridges for most of the US, but somewhat larger movements will be required in the north central regions of the country. Checks with bridge engineers in this region indicate that this larger movement potential is already recognized in some states. Measured average bridge temperatures and bridge movements for these concrete bridges exceeded the present AASHTO design limits for several bridges.

3. The installation temperature for the bearing and the expansion joint presently require consideration in bridge design. The variation in bridge temperature was analyzed, and rational and economical procedures for dealing with bridge temperatures are proposed. Two methods are proposed for bridge bearings and a separate method is proposed for expansion joints.

   For mechanical bearings and other bearings that can be installed with an offset, the standard offset chart procedures is retained, and a proposal for establishing the movement limits and offset chart is discussed and included in the recommended provisions.

   For elastomeric bearings and other bearings that cannot be installed with an offset, an alternate method is proposed. This alternate method utilizes the inherent flexibility and deformability of elastomeric bearings. However, lifting the girder to release the strain is recommended if the bearings are installed at an extreme temperature. However, this will be required only for a very small number of bridges. This requirement can be avoided entirely if limited care is employed in the girder installation procedure.

For expansion joints, a method analogous to the offset chart procedure is proposed. This method recognizes the movement of an expansion joint is largely determined by the temperatures when formwork for the concrete deck and abutment are placed, since the gap at that time should be appropriate to the temperatures at the same time.

4. The design recommendations were verified by comparison to field measurements on bridges throughout the United States. The measured bridge data always indicated average bridge temperatures that were well within the design limits for that region, but at the same time extreme summertime high temperatures and extreme wintertime low tempberatures approached the design limits in all cases.

5. In view of the good correlation with observed field behavior. proposed wording for modifying the AASHTO Standard Specifications are included in Appendix C, and proposals for the AASHTO LRFD Specifications are included in Appendix D.

Recommendations

It is believed that the design recommendations are valid and appropriate for use in bridges in the US. Field data has been used to check and verify the design recommendations and approximate analytical methods used to derive these recommendations. However, the cost of obtaining field data, limits the available supply of this data. As a result, it is recommended that additional data be accumulated to better understand the bridge movement issue.

Further, movements in skew and curved bridges are quite complex. Further analytical work and field observations are needed for the movements in bridges with these more complex geometries.

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Appendix A

Basic 60 Year Data Locations

STATION	ST	LAT	LONG	ELEV (ft)	BEG YR	END YR	NUM REC
ANNISTON FAA AP	al	33.58	85.85	610	1948	1994	47
AUBURN 3 SW	al	32.57	85.52	730	1928	1970	43
BIRMINGHAM FAA AP	al	33.57	86.75	630	1930	1994	65
BREWTON 3 SSE	al	31.07	87.05	90	1928	1994	67
CAMDEN 3 NW	al	32.03	87.32	240	1961	1994	34
CLANTON	al	32.85	86.63	580	1920	1994	65
EUFAULA	al	31.87	85.15	200	1930	1967	38
EVERGREEN	al	31.45	86.93	290	1933	1994	38
FAYETTE	al	33.68	87.82	370	1930	1994	53
GADSDEN	al	34.02	86.00	570	1930	1958	25
GREENSBORO	al	32.70	87.58	220	1890	1994	68
GREENVILLE	al	31.85	86.65	470	1920	1994	68
MOBILE WSO AP	al	30.68	88.25	210	1900	1994	48
MONTGOMERY WSO AP	al	32.30	86.40	220	1948	1994	47
SAINT BERNARD	al	34.17	86.82	800	1930	1994	65
SCOTTSBORO	al	34.68	86.05	620	1927	1994	68
SELMA	al	32.42	87.00	150	1930	1994	65
THOMASVILLE	al	31.92	87.73	410	1930	1994	65
TROY	al	31.78	85.95	500	1930	1994	65
BLYTHEVILLE	ar	35.92	89.90	250	1930	1994	65
CALICO ROCK 2 WSW	ar	36.12	92.17	350	1930	1994	39
CAMDEN 1	ar	33.60	92.82	120	1930	1994	65
CONWAY	ar	35.10	92.45	330	1897	1994	97
CORNING	ar	36.40	90.58	290	1930	1994	65
DUMAS	ar	33.88	91.48	160	1930	1994	65
EL DORADO FAA AP	ar	33.22	92.80	250	1930	1994	65
HARRISON	ar	36.23	93.12	1170	1930	1975	46
HOPE 3 NE	ar	33.72	93.55	380	1892	1994	84
JONESBORO 4 N	ar	35.88	90.70	390	1890	1994	102
LITTLE ROCK FAA AP	ar	34.73	92.23	260	1897	1994	98
MARIANNA 2 S	ar	34.73	90.77	230	1899	1994	84
MORRILTON	ar	35.13	92.73	280	1919	1994	74
MOUNTAIN HOME 1 NNW	ar	36.33	92.38	800	1902	1994	82
NEWPORT	ar	35.60	91.27	230	1930	1994	65
OZARK	ar	35.50	93.85	490	1930	1994	65
PINE BLUFF	ar	34.22	92.02	220	1887	1994	107
PORTLAND	ar	33.23	91.50	120	1909	1994	83
PRESCOTT	ar	33.80	93.38	310	1930	1994	65
SEARCY	ar	35.25	91.75	250	1930	1994	65
STUTTGART 9 ESE	ar	34.47	91.42	200	1890	1994	80
TEXARKANA FAA AP	ar	33.45	94.00	360	1930	1991	62
BAGDAD	az	34.57	113.17	3710	1929	1994	58
BISBEE 2 WNW	az	31.47	109.93	5600	1985	1992	8
BOWIE	az	32.33	109.48	3770	1902	1994	80
BUCKEYE	az	33.37	112.58	870	1893	1994	102
CHILDS	az	34.35	111.70	2650	1915	1994	80
CLIFTON	az	33.05	109.28	3460	1908	1994	87
CROWN KING	az	34.20	112.33	5920	1914	1994	26
DUNCAN	az	32.75	109.12	3660	1901	1994	55
FLORENCE	az	33.03	111.38	1510	1892	1994	87
FORT VALLEY	az	35.27	111.73	7350	1909	1994	86
GILA BEND	az	32.95	112.72	740	1892	1994	96
GLOBE	az	33.38	110.78	3550	1894	1975	76
GRAND CANYON HDQS	az	36.05	112.13	6890	1903	1957	55
HILLSIDE 4 NNE	az	34.48	112.88	3320	1899	1994	41
HOLBROOK	az	34.90	110.17	5080	1893	1994	100
JEROME	az	34.75	112.10	4950	1897	1994	98
KINGMAN	az	35.18	114.05	3360	1901	1967	67
LITCHFIELD PARK	az	33.50	112.37	1030	1917	1994	78
MARICOPA 4 N	az	33.12	112.03	1160	1960	1994	35
MC NARY	az	34.07	109.85	7320	1933	1994	62
MIAMI	az	33.40	110.88	3560	1914	1994	81
ORACLE	az	32.60	110.78	4600	1893	1948	53
PARKER 6 NE	az	34.18	114.22	410	1893	1994	102
PAYSON R S	az	34.23	111.33	4850	1909	1974	66
PORTAL 4 SW	az	31.88	109.20	5390	1965	1994	30
PRESCOTT	az	34.57	112.43	5210	1898	1994	97
ROOSEVELT 1 WNW	az	33.67	111.15	2210	1905	1994	90
SACATON	az	33.07	111.75	1290	1908	1994	87
SAFFORD	az	32.83	109.72	2900	1898	1973	50

STATION	ST	LAT	LONG	ELEV (ft)	BEG YR	END YR	NUM REC
SAINT JOHNS	az	34.52	109.38	5790	1901	1994	90
SAN SIMON	az	32.27	109.23	3610	1903	1994	42
SELIGMAN	az	35.32	112.88	5250	1904	1994	90
SIERRA ANCHA	az	33.80	110.97	5100	1914	1979	49
SIERRA VISTA	az	31.55	110.30	4680	1982	1994	13
SNOWFLAKE	az	34.50	110.08	5640	1897	1994	95
TEMPE 3 S	az	33.38	111.93	1180	1905	1952	48
TOMBSTONE	az	31.70	110.05	4610	1893	1994	99
TUBA CITY	az	36.13	111.23	4980	1900	1994	84
TUCSON U OF A	az	32.25	110.95	2440	1894	1994	101
WHITERIVER 1 SW	az	33.83	109.97	5120	1900	1994	90
WICKENBURG	az	33.98	112.73	2050	1908	1994	86
WILLCOX	az	32.30	109.85	4180	1903	1994	91
WILLIAMS	az	35.25	112.18	6750	1897	1994	95
WINDOW ROCK 4 SW	az	35.62	109.12	6900	1937	1994	57
WINSLOW WSO AP	az	35.02	110.73	4890	1898	1994	90
YUMA CITRUS STN	az	32.62	114.65	190	1920	1994	75
ALTURAS R S	ca	41.50	120.55	4400	1931	1994	64
BAKERSFIELD	ca	35.38	119.02	400	1927	1937	11
BARSTOW	ca	34.90	117.03	2160	1913	1980	51
BRAWLEY 2 SW	ca	32.95	115.55	0	1927	1994	68
CHICO UNIV FARM	ca	39.70	121.82	190	1906	1994	89
COALINGA 1 SE	ca	36.13	120.35	660	1911	1941	31
DAVIS 2 WSW EXP FRM	ca	38.53	121.77	60	1917	1994	78
ESCONDIDO	ca	33.12	117.08	660	1931	1979	49
HANFORD 1 S	ca	36.30	119.65	250	1927	1994	68
HAPPY CAMP R S	ca	41.80	123.37	1120	1931	1994	64
HETCH HETCHY	ca	37.95	119.78	3870	1931	1994	64
INDEPENDENCE	ca	36.80	118.20	3950	1927	1994	67
INDIO FIRE STATION	ca	33.73	116.27	-210	1927	1994	66
KING CITY	ca	36.20	121.13	320	1927	1994	61
LAGUNA BEACH	ca	33.55	117.78	40	1928	1994	67
LIVERMORE	ca	37.67	121.77	480	1930	1994	65
LONG BEACH	ca	33.77	118.20	30	1927	1969	43
MODESTO	ca	37.65	121.00	90	1931	1994	64
NAPA STATE HOSPITAL	ca	38.28	122.27	60	1917	1994	78
NEVADA CITY	ca	39.25	121.03	2780	1931	1994	64
PALM SPRINGS	ca	33.83	116.50	430	1927	1994	68
PALMDALE	ca	34.58	118.10	2600	1931	1994	64
PASADENA	ca	34.15	118.15	860	1927	1994	68
PASO ROBLES	ca	35.63	120.68	700	1901	1994	66
POMONA CAL POLY	ca	34.07	117.82	740	1927	1994	68
RED BLUFF WSO AP	ca	40.15	122.25	340	1933	1994	62
REDDING FIRE STN 2	ca	40.58	122.40	580	1931	1979	49
REDLANDS	ca	34.05	117.18	1320	1927	1994	68
RIVERSIDE FIRE STN 3	ca	33.95	117.38	840	1927	1994	68
SAINT HELENA	ca	38.50	122.47	230	1931	1994	64
SAN BERNARDINO CO HOSP	ca	34.13	117.27	1130	1927	1994	68
SAN DIEGO WSO AP	ca	32.73	117.17	10	1927	1994	68
SANTA BARBARA	ca	34.42	119.68	10	1927	1994	68
SANTA MARIA WSO AP	ca	34.90	120.45	250	1948	1994	47
SANTA ROSA	ca	38.45	122.70	170	1931	1994	64
SCOTIA	ca	40.48	124.10	140	1931	1994	64
SONORA R S	ca	37.83	120.38	1750	1931	1994	64
TAHOE CITY	ca	39.17	120.13	6230	1931	1994	64
TULELAKE	ca	41.97	121.47	4040	1932	1994	62
TUSTIN IRVINE RANCH	ca	33.73	117.78	120	1927	1994	68
UKIAH	ca	39.15	123.20	630	1906	1994	89
VISALIA	ca	36.33	119.30	330	1927	1994	68
AKRON 4 E	co	40.15	103.15	4540	1918	1994	41
BURLINGTON	co	39.32	102.27	4170	1918	1994	77
CHEYENNE WELLS	co	38.82	102.35	4250	1918	1994	77
COLLBRAN	co	39.25	107.97	5980	1900	1994	92
CORTEZ	co	37.37	108.55	6210	1929	1994	66
CRESTED BUTTE	co	38.87	106.97	8860	1910	1994	85
DELTA	co	38.75	108.07	4930	1900	1994	94
DILLON 1 E	co	39.63	106.03	9070	1910	1994	85
DURANGO	co	37.28	107.88	6600	1900	1991	92
EADS	co	38.48	102.78	4210	1918	1994	77
FORT COLLINS	co	40.58	105.08	5000	1900	1994	95
FRASER	co	39.95	105.83	8560	1909	1974	66

STATION	ST	LAT	LONG	ELEV (ft)	BEG YR	END YR	NUM REC
GLENWOOD SPRINGS 1 N	co	39.57	107.33	5820	1902	1994	93
GRAND JUNCTION WSO AP	co	39.10	108.55	4850	1900	1994	95
GUNNISON	co	38.53	106.93	7660	1900	1994	95
HOLLY	co	38.05	102.12	3390	1918	1994	75
JULESBURG	co	41.00	102.25	3470	1918	1994	76
LAMAR	co	38.08	102.62	3620	1918	1994	77
LAS ANIMAS	co	38.07	103.22	3890	1930	1994	65
LIMON 10 SSW	co	39.15	103.77	5560	1918	1971	53
MEEKER	co	40.03	107.90	6240	1900	1994	60
MONTROSE 2	co	38.48	107.88	5830	1903	1994	92
PAGOSA SPRINGS	co	37.27	107.02	7110	1906	1993	67
PAONIA 1 SW	co	38.87	107.60	5580	1905	1994	64
RIFLE	co	39.53	107.80	5320	1910	1994	83
ROCKY FORD 2 SE	co	38.03	103.70	4170	1918	1994	77
SILVERTON	co	37.82	107.67	9270	1906	1994	89
STEAMBOAT SPRINGS	co	40.48	106.83	6760	1908	1994	87
TELLURIDE	co	37.93	107.82	8800	1900	1994	89
WRAY 1 E	co	40.08	102.18	3520	1918	1994	76
HARTFORD BRAINARD FLD	ct	41.73	72.65	20	1920	1994	75
MOUNT CARMEL	ct	41.40	72.90	180	1936	1994	59
STORRS	ct	41.80	72.25	650	1888	1994	106
DOVER	de	39.15	75.52	30	1948	1994	47
GEORGETOWN 5 SW	de	38.63	75.45	50	1948	1994	47
LEWES	de	38.77	75.13	20	1948	1994	47
NEWARK UNIV FARM	de	39.67	75.73	90	1948	1994	47
WILMINGTON WSO AP	de	39.67	75.60	80	1948	1994	47
APALACHICOLA WSO AP	fl	29.73	85.03	20	1931	1994	64
ARCADIA	fl	27.23	81.85	60	1931	1994	64
AVON PARK 2 W	fl	27.60	81.53	150	1931	1994	64
BARTOW	fl	27.90	81.85	120	1931	1994	64
BELLE GLADE EXP STN	fl	26.65	80.63	20	1924	1994	71
BROOKSVILLE CHIN HILL	fl	28.62	82.37	240	1931	1994	64
CRESCENT CITY	fl	29.43	81.52	60	1931	1994	39
DE FUNIAK SPRINGS	fl	30.73	86.12	230	1931	1994	64
DELAND 1 SSE	fl	29.02	81.30	30	1931	1994	64
EVERGLADES	fl	25.85	81.38	10	1931	1994	64
FEDERAL POINT	fl	29.75	81.53	10	1931	1994	64
FORT MYERS FAA AP	fl	26.58	81.87	20	1931	1994	63
FORT PIERCE	fl	27.47	80.35	30	1931	1994	64
GAINESVILLE UNI OF FLA	fl	29.65	82.35	170	1903	1963	61
KISSIMMEE 2	fl	28.28	81.42	60	1959	1994	36
LAKE ALFRED EXP STN	fl	28.10	81.72	140	1905	1994	72
LAKE CITY 2 E	fl	30.18	82.60	200	1931	1994	64
MADISON 4 N	fl	30.53	83.43	180	1931	1994	64
MOORE HAVEN LOCK 1	fl	26.83	81.08	40	1930	1994	65
PLANT CITY	fl	28.02	82.13	120	1931	1994	64
SAINT LEO	fl	28.33	82.27	190	1931	1994	64
TAMPA WSCMO AP	fl	27.97	82.53	20	1900	1994	63
TITUSVILLE	fl	28.62	80.82	50	1931	1994	64
ALBANY 3 SE	ga	31.53	84.13	180	1892	1994	98
AMERICUS 3 SW	ga	32.05	84.25	490	1930	1994	65
ATHENS WSO AP	ga	33.95	83.32	800	1948	1994	47
ATLANTA WSO AP	ga	33.65	84.43	1010	1930	1994	65
BAINBRIDGE	ga	30.92	84.58	120	1892	1977	79
BLAIRSVILLE EXP STN	ga	34.85	83.93	1920	1931	1994	64
BROOKLET 1 W	ga	32.38	81.68	190	1930	1994	65
BRUNSWICK	ga	31.17	81.50	10	1930	1994	65
CLAYTON 1 SSW	ga	34.87	83.40	1880	1927	1994	68
DAHLONEGA	ga	34.53	83.98	1430	1930	1994	65
DUBLIN 3 S	ga	32.50	82.90	220	1930	1994	65
EXPERIMENT	ga	33.27	84.28	930	1926	1994	69
GAINESVILLE	ga	34.30	83.85	1170	1930	1994	65
GLENNVILLE	ga	31.93	81.92	170	1930	1994	65
HARTWELL	ga	34.35	82.92	690	1930	1994	65
HAWKINSVILLE	ga	32.28	83.47	270	1930	1994	65
MILLEN 4 N	ga	32.87	81.97	200	1930	1994	65
MOULTRIE 2 ESE	ga	31.17	83.75	340	1926	1994	69
QUITMAN 2 NW	ga	30.80	83.58	190	1896	1994	99
ROME	ga	34.25	85.15	620	1930	1994	65
TALBOTTON 1 NE	ga	32.70	84.53	710	1930	1994	65
THOMASVILLE 3 NE	ga	30.88	83.93	260	1892	1994	102
TIFTON EXP STATION	ga	31.48	83.53	370	1922	1994	73

STATION	ST	LAT	LONG	ELEV (ft)	BEG YR	END YR	NUM REC
TOCCOA	ga	34.58	83.32	1020	1930	1994	65
WARRENTON	ga	33.42	82.65	510	1930	1994	65
WAYCROSS 4 NE	ga	31.25	82.32	150	1930	1994	64
WEST POINT	ga	32.87	85.18	580	1930	1994	65
ALGONA 3 W	ia	43.07	94.30	1230	1893	1994	102
AMES 3 SW	ia	42.00	93.65	1000	1893	1964	72
ATLANTIC 1 NE	ia	41.42	95.00	1200	1893	1994	102
CEDAR RAPIDS 1	ia	42.03	91.58	820	1895	1994	100
CLARINDA	ia	40.73	95.03	1030	1893	1994	102
CORNING	ia	41.00	94.75	1220	1896	1994	99
DELAWARE 3 WSW	ia	42.47	91.42	980	1899	1975	69
DENISON	ia	42.03	95.33	1400	1900	1994	95
FAIRFIELD	ia	41.03	91.95	740	1896	1994	94
FAYETTE	ia	42.83	91.80	1010	1900	1994	95
GLENWOOD 3 SW	ia	41.00	95.77	990	1893	1994	92
GRINNELL 3 SW	ia	41.72	92.73	910	1893	1994	87
GUTHRIE CENTER	ia	41.68	94.52	1180	1895	1994	100
HAMPTON 2 NW	ia	42.75	93.20	1220	1893	1994	93
INDIANOLA	ia	41.37	92.55	940	1893	1994	102
KEOSAUQUA	ia	40.73	91.97	630	1893	1994	102
LE MARS	ia	42.80	96.17	1200	1896	1994	99
LOGAN	ia	41.63	95.80	1050	1893	1994	102
MAQUOKETA 2 W	ia	42.07	90.70	680	1897	1994	87
MARSHALLTOWN	ia	42.07	92.93	870	1893	1994	102
MASON CITY	ia	43.15	93.20	1130	1896	1994	97
MOUNT AYR 4 SW	ia	40.68	94.30	1240	1893	1994	101
NEW HAMPTON	ia	43.05	92.32	1160	1897	1994	98
NEWTON	ia	41.70	93.05	490	1899	1994	48
ONAWA	ia	42.02	96.10	1060	1899	1994	96
OSKALOOSA	ia	41.32	92.65	830	1893	1994	102
ROCK RAPIDS	ia	43.43	96.17	1350	1893	1994	99
ROCKWELL CITY	ia	42.40	94.62	1210	1894	1994	100
SPENCER 1 N	ia	43.17	95.15	1330	1895	1994	88
STORM LAKE 2 E	ia	42.63	95.18	1430	1893	1994	99
TIPTON	ia	41.78	91.12	770	1902	1994	93
WASHINGTON	ia	41.28	91.68	760	1893	1994	102
WATERLOO	ia	42.52	92.33	840	1895	1950	56
WEBSTER CITY	ia	42.47	93.80	1170	1893	1994	97
ABERDEEN EXP STN	id	42.95	112.83	4410	1914	1994	81
ARROWROCK DAM	id	43.60	115.92	3280	1916	1994	79
AVERY R S	id	47.25	115.80	2490	1913	1968	56
BONNERS FERRY 1 SW	id	48.68	116.32	1860	1907	1994	69
CALDWELL	id	43.67	116.68	2370	1904	1994	91
CAMBRIDGE	id	44.57	116.68	2650	1931	1994	64
CHALLIS	id	44.50	114.23	5180	1931	1994	64
DEER FLAT DAM	id	43.58	116.75	2510	1916	1994	57
DRIGGS	id	43.73	111.12	6120	1930	1994	64
DUBOIS EXP STN	id	44.25	112.20	5450	1925	1994	70
GRACE	id	42.58	111.73	5550	1931	1994	64
GRAND VIEW 2 W	id	43.00	116.13	2400	1933	1994	62
HILL CITY 1 W	id	43.30	115.05	5000	1931	1994	64
IDAHO CITY	id	43.83	115.83	3970	1931	1994	64
JEROME	id	42.73	114.52	3740	1919	1994	56
KELLOGG	id	47.55	116.17	2320	1905	1994	89
MACKAY R S	id	43.92	113.62	5900	1931	1994	64
MC CALL	id	44.90	116.12	5030	1930	1994	65
MONTPELIER R S	id	42.32	111.30	5960	1931	1991	61
MOSCOW U OF IDAHO	id	46.73	116.97	2660	1893	1994	102
NEZPERCE	id	46.25	116.25	3150	1901	1994	45
OAKLEY	id	42.23	113.88	4600	1931	1994	64
PARMA EXP STN	id	43.80	116.95	2220	1922	1994	73
PIERCE R S	id	46.50	115.80	3170	1922	1962	41
POCATELLO 2	id	42.87	112.47	4440	1899	1994	13
POTLATCH 3 NNE	id	46.97	116.88	2600	1915	1994	77
PRIEST RIVER EXP STN	id	48.35	116.83	2380	1911	1994	84
RUPERT 1 E	id	42.62	113.67	4200	1931	1994	49
SALMON AIRPORT	id	45.12	113.88	3950	1930	1967	38
SANDPOINT EXP STATION	id	48.28	116.57	2100	1910	1994	85
SHOSHONE 1 WNW	id	42.97	114.43	3950	1931	1994	64
TWIN FALLS 2 NNE	id	42.58	114.47	3690	1905	1974	70
ALEDO	il	41.23	90.73	720	1901	1994	94
ANNA 1 E	il	37.47	89.23	650	1901	1994	94

STATION	ST	LAT	LONG	ELEV (ft)	BEG YR	END YR	NUM REC
AURORA	il	41.75	88.35	640	1901	1994	94
BLOOMINGTON NORMAL	il	40.52	89.00	790	1901	1977	77
CARBONDALE SEWAGE PLANT	il	37.73	89.17	390	1910	1994	85
CARLINVILLE	il	39.28	89.87	630	1901	1994	94
CHARLESTON	il	39.48	88.17	680	1901	1994	94
CHICAGO MIDWAY AP 3 SW	il	41.73	87.77	620	1928	1994	67
DANVILLE	il	40.13	87.65	560	1901	1994	88
DECATUR	il	39.83	89.02	620	1901	1994	94
DIXON 1 NW	il	41.85	89.48	700	1901	1994	94
DU QUOIN 4 SE	il	38.00	89.25	420	1901	1994	94
EFFINGHAM	il	39.13	88.53	600	1901	1994	87
FAIRFIELD RADIO WFIW	il	38.38	88.32	430	1901	1994	94
FLORA 5 NW	il	38.68	88.57	500	1901	1994	94
GALVA	il	41.17	90.05	860	1901	1994	94
GRIGGSVILLE	il	39.72	90.73	700	1901	1989	89
HAVANA	il	40.30	90.05	440	1901	1966	66
HILLSBORO	il	39.15	89.48	630	1901	1994	94
HOOPESTON 1 NE	il	40.47	87.67	710	1902	1994	93
LA HARPE	il	40.58	90.97	700	1901	1994	94
LINCOLN	il	40.17	89.37	580	1906	1994	89
MARENGO	il	42.25	88.60	820	1901	1994	94
MC LEANSBORO 2 ENE	il	38.10	88.50	480	1901	1994	94
MOUNT CARROLL	il	42.08	89.98	700	1901	1994	94
MOUNT VERNON 3 NE	il	38.35	88.87	490	1901	1994	94
MT CARMEL 4 NW	il	38.45	87.78	470	1902	1977	73
NEW BURNSIDE	il	37.58	88.77	560	1901	1964	64
OLNEY 2 S	il	38.70	88.07	0	1901	1994	94
OTTAWA 4 SW	il	41.33	88.92	530	1901	1994	94
PALESTINE	il	39.00	87.62	520	1901	1994	94
PANA	il	39.38	89.08	700	1901	1994	94
MORRISONVILLE 4 SE	il	39.38	89.40	640	1901	1971	71
MORRISON	il	41.82	89.97	600	1901	1994	94
MINONK	il	40.90	89.05	750	1901	1994	94
WINDSOR	il	39.43	88.60	690	1904	1994	91
WHITE HALL 1 E	il	39.43	90.38	580	1902	1994	93
WALNUT	il	41.55	89.60	690	1901	1994	94
URBANA	il	40.10	88.23	740	1903	1994	92
SYCAMORE	il	41.98	88.68	840	1901	1965	65
SPARTA	il	38.13	89.72	520	1901	1994	94
RUSHVILLE	il	40.12	90.55	660	1901	1994	94
PONTIAC	il	40.88	88.63	650	1903	1994	92
PARIS WATERWORKS	il	39.63	87.73	680	1901	1994	94
ANGOLA	in	41.63	84.98	1010	1901	1994	80
BERNE	in	40.67	84.95	860	1910	1994	85
BLOOMINGTON INDIANA U	in	39.17	86.52	830	1901	1994	93
COLUMBUS	in	39.20	85.92	620	1901	1994	94
FARMLAND 5 NNW	in	40.25	85.15	970	1901	1994	75
KOKOMO 7 SE	in	40.42	86.05	860	1901	1994	84
LA PORTE	in	41.60	86.72	810	1901	1994	48
MARION 2 N	in	40.57	85.67	790	1901	1994	94
OOLITIC PURDUE EXP FARM	in	38.88	86.55	650	1902	1994	81
PAOLI	in	38.55	86.48	560	1901	1994	94
PLYMOUTH POWER SUBSTN	in	41.33	86.32	790	1905	1989	85
PRINCETON 1 W	in	38.35	87.58	480	1901	1994	94
RICHMOND WATERWORKS	in	39.85	84.85	970	1901	1968	68
RUSHVILLE SEWAGE PLANT	in	39.60	85.45	960	1901	1994	94
SCOTTSBURG	in	38.70	85.77	550	1901	1994	94
VALPARAISO WATERWORKS	in	41.52	87.03	800	1901	1994	86
WASHINGTON	in	38.65	87.17	490	1901	1994	94
WEST LAFAYETTE 6 NW	in	40.47	87.00	710	1901	1994	90
WHITESTOWN	in	40.00	86.35	940	1901	1994	94
ROCKVILLE	in	39.77	87.23	690	1901	1994	94
ASHLAND	ks	37.20	99.77	1970	1900	1994	95
COLUMBUS 1 SW	ks	37.17	94.85	900	1900	1994	95
EL DORADO	ks	37.82	96.83	1340	1904	1994	91
ELKHART 6 NNE	ks	37.08	101.85	3600	1900	1994	90
ELLSWORTH	ks	38.72	98.23	1530	1904	1994	91

STATION	ST	LAT	LONG	ELEV (ft)	BEG YR	END YR	NUM REC
EMPORIA 1 S	ks	38.38	96.18	1080	1900	1954	55
HAYS 1 S	ks	38.87	99.33	2010	1900	1994	95
HEALY	ks	38.60	100.62	2850	1901	1994	94
HORTON	ks	39.67	95.52	1030	1900	1994	95
INDEPENDENCE	ks	37.25	95.70	780	1900	1994	95
IOLA	ks	37.92	95.40	960	1905	1958	54
LARNED	ks	38.18	99.08	2000	1904	1994	91
MANHATTAN	ks	39.20	96.58	1070	1900	1994	95
MC PHERSON	ks	38.38	97.67	1500	1900	1994	95
OTTAWA	ks	38.62	95.28	900	1900	1994	95
PHILLIPSBURG 1 SSE	ks	39.73	99.32	1910	1900	1994	94
SAINT FRANCIS	ks	39.77	101.80	3360	1908	1994	87
TRIBUNE 1 W	ks	38.47	101.77	3640	1900	1994	85
WINFIELD NO 1	ks	37.23	96.98	1140	1900	1994	95
ASHLAND	ky	38.45	82.62	560	1932	1994	63
BOWLING GREEN FAA AP	ky	36.97	86.42	550	1932	1994	63
FARMERS 2 S	ky	38.12	83.55	680	1932	1994	63
GREENSBURG	ky	37.25	85.50	590	1932	1994	63
HENDERSON 7 SSW	ky	37.75	87.63	430	1932	1994	63
HOPKINSVILLE	ky	36.83	87.50	590	1932	1994	63
MAYFIELD RADIO WNGO	ky	36.78	88.63	380	1967	1994	28
MIDDLESBORO	ky	36.60	83.73	1180	1928	1994	65
OWENSBORO 3 W	ky	37.77	87.15	410	1932	1994	63
SHELBYVILLE 1 E	ky	38.20	85.20	730	1932	1994	63
ALEXANDRIA	la	31.32	92.47	90	1930	1994	65
BATON ROUGE WSO AP	la	30.53	91.15	60	1930	1994	65
CLINTON	la	30.87	91.02	180	1930	1975	40
COVINGTON 4 NNW	la	30.53	90.12	40	1930	1994	65
CROWLEY 2 NE	la	30.25	92.37	30	1930	1994	52
DONALDSONVILLE 4 SW	la	30.07	91.03	30	1930	1994	65
FRANKLINTON 3 SW	la	30.82	90.18	150	1956	1994	39
HOUMA	la	29.58	90.73	20	1930	1994	65
JEANERETTE 5 NW	la	29.95	91.72	20	1930	1994	59
LAKE CHARLES WSO AP	la	30.12	93.22	10	1962	1994	33
LAKE PROVIDENCE	la	32.80	91.17	100	1930	1994	64
LEESVILLE	la	31.15	93.27	240	1930	1994	63
MONROE FAA AP	la	32.52	92.05	80	1930	1994	57
MORGAN CITY	la	29.68	91.18	10	1930	1994	65
NATCHITOCHES	la	31.77	93.08	130	1930	1994	65
NEW ORLEANS WSCMO AP	la	29.98	90.25	0	1954	1994	41
PLAIN DEALING	la	32.90	93.68	290	1930	1994	65
RUSTON-LA TECH UNIV	la	32.52	92.65	280	1930	1994	65
SAINT JOSEPH 3 N	la	31.95	91.23	80	1930	1994	65
SHREVEPORT WSO AP	la	32.47	93.82	250	1930	1994	65
WINNSBORO 5 SSE	la	32.10	91.72	80	1930	1994	65
AMHERST	ma	42.38	72.53	150	1926	1994	69
BLUE HILL WSO	ma	42.22	71.12	630	1926	1994	69
BOSTON WSO AP	ma	42.37	71.03	20	1920	1994	75
EAST WAREHAM	ma	41.77	70.67	20	1926	1994	69
HYANNIS	ma	41.67	70.30	50	1930	1994	64
LAWRENCE	ma	42.70	71.17	60	1926	1994	69
BALTIMORE WSO CI	md	39.28	76.62	90	1893	1994	102
EASTPORT	me	44.92	67.00	90	1926	1994	69
FARMINGTON	me	44.68	70.15	420	1926	1994	69
LEWISTON	me	44.10	70.22	180	1926	1994	69
PORTLAND WSMO AP	me	43.65	70.32	60	1920	1994	75
PRESQUE ISLE	me	46.65	68.00	600	1926	1994	69
ANN ARBOR U OF MICH	mi	42.30	83.72	900	1897	1994	98
BERGLAND DAM	mi	46.58	89.55	1300	1888	1994	47
BIG RAPIDS WATERWORKS	mi	43.70	85.48	930	1899	1994	96
CHATHAM EXP FARM	mi	46.35	86.93	880	1901	1988	78
COLDWATER STATE SCHOOL	mi	41.95	85.00	980	1898	1994	97
EAST LANSING EXP FARM	mi	42.70	84.47	890	1910	1960	51
FAYETTE 4 SW	mi	45.67	86.72	750	1931	1994	64
HART	mi	43.70	86.37	680	1921	1994	48
IRON MOUNTAIN WATERWKS	mi	45.78	88.08	1060	1931	1994	64
IRONWOOD	mi	46.47	90.18	1430	1901	1994	94
MONROE	mi	41.92	83.40	590	1931	1994	64
SAULT STE MARIE WSO	mi	46.47	84.37	720	1931	1994	63
TRAVERSE CITY FAA AP	mi	44.73	85.58	620	1896	1994	99

STATION	ST	LAT	LONG	ELEV (ft)	BEG YR	END YR	NUM REC
BAUDETTE	mn	48.72	94.62	1080	1932	1994	63
BRAINERD	mn	46.37	94.20	1180	1908	1994	43
CANBY	mn	44.72	96.28	1240	1932	1994	62
CASS LAKE	mn	47.38	94.62	1300	1911	1994	47
CHASKA	mn	44.80	93.58	720	1911	1994	47
CLOQUET	mn	46.70	92.52	1270	1900	1994	85
CROOKSTON NW EXP STN	mn	47.80	96.62	880	1890	1994	105
DETROIT LAKES 1 NNE	mn	46.83	95.85	1380	1932	1994	62
GRAND MARAIS	mn	47.73	90.35	610	1932	1994	63
GRAND MEADOW	mn	43.70	92.57	1350	1932	1994	63
GRAND RAPIDS FOREST LAB	mn	47.23	93.50	1310	1915	1994	80
GULL LAKE DAM	mn	46.42	94.35	1220	1919	1994	48
HALLOCK	mn	48.77	96.95	820	1932	1994	63
ITASCA UNIV OF MINN	mn	47.22	95.20	1490	1912	1994	83
LITTLE FALLS 1 N	mn	45.98	94.35	1120	1932	1994	63
MINN-ST PAUL WSO AP	mn	44.88	93.22	830	1891	1994	104
MORA	mn	45.88	93.30	990	1932	1994	63
MORRIS WC EXP STN	mn	45.58	95.88	1140	1886	1994	109
PINE RIVER DAM	mn	46.67	94.12	1250	1901	1994	94
REDWOOD FALLS FAA AP	mn	44.55	95.08	1030	1932	1994	62
WADENA 3 S	mn	46.40	95.15	1350	1932	1994	63
WASECA EXPERIMENT STN	mn	44.07	93.52	1150	1915	1994	80
WHEATON	mn	45.80	96.48	1020	1948	1994	48
WILLMAR STATE HOSPITAL	mn	45.13	95.02	1130	1932	1994	63
WINNEBAGO	mn	43.77	94.17	1110	1932	1994	63
WINONA	mn	44.05	91.63	650	1932	1994	63
ZUMBROTA	mn	44.30	92.67	990	1932	1994	63
ARCADIA	mo	37.58	90.62	930	1918	1994	76
BETHANY	mo	40.25	94.05	950	1918	1994	77
BOLIVAR 1 NE	mo	37.60	93.42	1080	1918	1994	69
BRUNSWICK	mo	39.42	93.12	650	1918	1994	77
CARUTHERSVILLE	mo	36.20	89.67	280	1918	1994	77
CASSVILLE RANGER STN	mo	36.68	93.87	1340	1918	1994	36
CHILLICOTHE RADIO KCHI	mo	39.80	93.55	790	1918	1980	63
CLINTON	mo	38.40	93.77	770	1918	1994	77
ELSBERRY 1 S	mo	39.15	90.78	450	1931	1994	64
FARMINGTON	mo	37.70	90.38	940	1918	1994	77
FULTON	mo	38.85	91.95	870	1918	1994	77
GREENVILLE 6 N	mo	37.20	90.45	490	1922	1994	73
JACKSON	mo	37.37	89.67	440	1930	1994	64
JEFFERSON CITY WTR PLT	mo	38.58	92.15	670	1918	1994	77
KIRKSVILLE RADIO KIRX	mo	40.22	92.58	970	1918	1994	77
LEBANON 2 W	mo	37.67	92.65	1280	1918	1994	77
LEXINGTON 3 NE	mo	39.20	93.87	830	1918	1994	77
LOCKWOOD	mo	37.38	93.95	1080	1918	1994	77
MARYVILLE 2 E	mo	40.35	94.83	990	1918	1994	77
MOUNTAIN GROVE 2 N	mo	37.15	92.27	1450	1918	1994	77
NEOSHO	mo	36.87	94.37	1010	1918	1994	77
NEVADA SEWAGE PLANT	mo	37.85	94.40	740	1918	1994	77
POPLAR BLUFF R S	mo	36.77	90.40	370	1918	1994	77
ROLLA UNIV OF MO	mo	37.95	91.77	1180	1918	1994	76
SAINT CHARLES	mo	38.78	90.50	470	1918	1994	77
SALEM	mo	37.63	91.53	1200	1918	1994	77
SIKESTON	mo	36.87	89.60	300	1926	1959	34
STEFFENVILLE	mo	39.97	91.88	690	1918	1994	77
TARKIO	mo	40.45	95.38	1040	1918	1993	75
UNIONVILLE	mo	40.48	93.00	1060	1918	1993	74
WARRENSBURG	mo	38.75	93.73	870	1918	1994	75
WARRENTON 1 N	mo	38.82	91.13	850	1918	1994	76
BROOKHAVEN CITY	ms	31.55	90.45	430	1930	1994	65
CLARKSDALE	ms	34.20	90.57	170	1930	1994	65
COLUMBIA	ms	31.25	89.83	160	1930	1994	65
CORINTH CITY	ms	34.92	88.52	390	1930	1994	65
FOREST 3 S	ms	32.32	89.48	480	1930	1994	65
GREENVILLE	ms	33.38	91.02	130	1920	1994	75
HERNANDO	ms	34.83	90.00	360	1930	1994	64
HOLLY SPRINGS 2 N	ms	34.80	89.43	500	1930	1962	33
JACKSON 4 NW	ms	32.33	90.23	320	1930	1971	42
LOUISVILLE	ms	33.13	89.07	580	1930	1994	65
NATCHEZ	ms	31.55	91.38	200	1930	1994	65
POPLARVILLE EXP STN	ms	30.85	89.55	310	1930	1994	65
PORT GIBSON 1 NW	ms	31.97	91.00	120	1930	1994	65
STATE UNIVERSITY	ms	33.47	88.78	190	1930	1994	65
STONEVILLE EXP STN	ms	33.43	90.92	130	1930	1994	65
TUPELO	ms	34.25	88.72	290	1930	1969	40
UNIVERSITY	ms	34.38	89.53	380	1930	1994	65
VICKSBURG MILITARY PK	ms	32.35	90.85	260	1967	1994	28
WAYNESBORO 3 WNW	ms	31.68	88.68	200	1930	1953	24
YAZOO CITY 5 NNE	ms	32.90	90.38	110	1960	1994	35
AUGUSTA	mt	47.48	112.38	4070	1896	1994	98
BALLANTINE	mt	45.95	108.13	3000	1919	1990	72
BIG SANDY	mt	48.17	110.12	2700	1921	1994	73
BIG TIMBER	mt	45.83	109.95	4100	1894	1994	95
BILLINGS WATER PLANT	mt	45.77	108.48	3100	1894	1994	101
BRIDGER	mt	45.30	108.92	3680	1900	1994	88
BROWNING	mt	48.57	113.02	4360	1894	1980	82
BUTTE FAA AP	mt	45.95	112.50	5540	1880	1994	102
CASCADE 5 S	mt	47.22	111.72	3390	1904	1994	91
CHOTEAU AIRPORT	mt	47.82	112.17	3950	1893	1994	90
CONRAD AIRPORT	mt	48.17	111.97	3540	1911	1994	81
CROW AGENCY	mt	45.60	107.45	3030	1898	1991	92
CULBERTSON	mt	48.15	104.50	1920	1900	1994	87
CUT BANK FAA AP	mt	48.60	112.37	3840	1903	1994	89
DILLON W M C E	mt	45.20	112.63	5230	1895	1994	97
EAST ANACONDA	mt	46.10	112.92	5510	1905	1980	76
EKALAKA	mt	45.88	104.53	3430	1896	1994	99
FLATWILLOW 4 ENE	mt	46.85	108.32	3140	1913	1994	82
FORKS 4 NNE	mt	48.78	107.47	2600	1915	1994	80
FORT ASSINNIBOINE	mt	48.50	109.80	2610	1917	1994	78
FORTINE 1 N	mt	48.78	114.90	3000	1906	1994	89
GLASGOW	mt	48.18	106.63	2090	1893	1994	65
GLENDIVE	mt	47.10	104.72	2080	1893	1994	102
GREAT FALLS	mt	47.52	111.30	3350	1893	1956	59
HAUGAN 3 E (DEBORGIA)	mt	47.38	115.35	3120	1912	1989	78
HAVRE WB CITY	mt	48.57	109.67	2490	1879	1961	70
HELENA WSO AP	mt	46.60	112.00	3890	1893	1994	102
HERON 2 NW	mt	48.08	116.00	2240	1912	1994	83
HUNTLEY EXP STN	mt	45.92	108.25	2990	1911	1994	84
JORDAN	mt	47.32	106.90	2590	1905	1994	80
KALISPELL WSO AP	mt	48.30	114.27	2970	1899	1994	96
LEWISTOWN FAA AP	mt	47.07	109.45	4150	1896	1994	91
LIBBY 1 NE R S	mt	48.40	115.53	2140	1895	1994	87
LIMA	mt	44.65	112.58	6270	1898	1994	80
LIVINGSTON	mt	45.67	110.57	4490	1895	1981	83
MALTA	mt	48.35	107.87	2260	1906	1972	67
MILES CITY	mt	46.40	105.82	2360	1893	1982	82
MISSOULA 2 WNW	mt	46.88	114.03	3170	1893	1966	72
MOCCASIN EXP STN	mt	47.05	109.95	4300	1909	1994	85
NORRIS MADISON P H	mt	45.48	111.63	4750	1907	1994	88
OVANDO	mt	47.02	113.13	4110	1899	1976	78
PHILIPSBURG R S	mt	46.32	113.30	5270	1955	1994	40
PLEVNA	mt	46.42	104.50	2770	1910	1994	84
POLSON	mt	47.68	114.17	2990	1906	1994	81
RAPELJE 4 S	mt	45.92	109.25	4130	1908	1994	87
RED LODGE	mt	45.18	109.25	5580	1894	1994	99
ROUNDUP	mt	46.45	108.53	3230	1914	1994	75
SAINT IGNATIUS	mt	47.32	114.10	2900	1896	1994	91
SAVAGE	mt	47.45	104.35	1990	1905	1994	90
STANFORD	mt	47.15	110.22	4280	1927	1964	38
STEVENSVILLE	mt	46.52	114.10	3380	1911	1994	84
THOMPSON FALLS R S	mt	47.60	115.35	2440	1911	1956	46
TRIDENT	mt	45.95	111.48	4040	1922	1994	73
TROUT CREEK R S	mt	47.87	115.62	2360	1960	1994	35
VALIER	mt	48.32	112.25	3810	1911	1994	84
WEST GLACIER	mt	48.50	113.98	3150	1949	1994	46
WEST YELLOWSTONE	mt	44.65	111.10	6660	1924	1994	71
WHITE SULPHUR SPRINGS	mt	46.53	110.92	5160	1894	1978	68
ALBEMARLE	nc	35.37	80.18	610	1933	1994	62
ASHEBORO 2 W	nc	35.70	79.83	870	1933	1994	62
BELHAVEN	nc	35.55	76.63	10	1933	1994	40
CHARLOTTE WSO AP	nc	35.22	80.93	700	1948	1994	47
CLINTON 2 NE	nc	35.02	78.28	160	1971	1994	24

STATION	ST	LAT	LONG	ELEV (ft)	BEG YR	END YR	NUM REC
EDENTON	nc	36.05	76.62	20	1933	1994	62
FAYETTEVILLE	nc	35.07	78.87	100	1933	1994	62
GASTONIA	nc	35.28	81.18	760	1890	1994	66
GOLDSBORO 4 SE	nc	35.33	77.97	110	1900	1994	94
GREENSBORO WSO AP	nc	36.08	79.95	890	1933	1994	62
GREENVILLE	nc	35.62	77.38	30	1933	1994	48
MONROE 4 SE	nc	34.97	80.50	580	1933	1994	62
MORGANTON	nc	35.75	81.68	1160	1933	1994	62
MOUNT AIRY	nc	36.52	80.62	1030	1893	1994	102
NEW BERN FAA AP	nc	35.07	77.05	20	1948	1994	47
RALEIGH STATE UNIV	nc	35.78	78.70	4080	1921	1994	71
REIDSVILLE	nc	36.35	79.63	830	1901	1963	51
SALISBURY	nc	35.68	80.48	700	1893	1994	102
SHELBY 2 NNE	nc	35.32	81.53	920	1893	1994	63
STATESVILLE 2 NNE	nc	35.82	80.88	950	1901	1994	91
WAYNESVILLE 1 E	nc	35.48	82.97	2660	1894	1994	96
WELDON	nc	36.43	77.60	80	1903	1971	69
WILMINGTON WSO AP	nc	34.27	77.90	70	1933	1994	62
BOTTINEAU	nd	48.83	100.45	1640	1898	1994	92
BOWMAN COURT HOUSE	nd	46.18	103.38	2980	1915	1994	77
CROSBY	nd	48.90	103.30	1950	1909	1994	86
DICKINSON EXP STATION	nd	46.88	102.80	2460	1903	1994	92
EDGELEY 3 WNW	nd	46.37	98.77	1560	1901	1994	84
FESSENDEN	nd	47.65	99.62	1620	1932	1994	61
GRAND FORKS UNIVERSITY	nd	47.93	97.08	830	1932	1994	63
HANSBORO 4 NNE	nd	49.00	99.35	1540	1932	1994	63
HETTINGER	nd	45.98	102.65	2680	1916	1994	79
JAMESTOWN STATE HOSP	nd	46.88	98.68	1470	1881	1994	96
LANGDON EXP FARM	nd	48.75	98.33	1620	1907	1994	88
LISBON	nd	46.43	97.67	1090	1932	1994	63
MANDAN EXP STATION	nd	46.80	100.90	1750	1913	1994	82
MINOT EXP STATION	nd	48.18	101.30	1770	1905	1994	90
NAPOLEON	nd	46.50	99.77	1980	1901	1994	94
OAKES 2 S	nd	46.13	98.08	1310	1922	1994	47
WAHPETON 3 N	nd	46.32	96.60	960	1897	1994	94
ALBION 1 S	ne	41.67	97.98	1750	1893	1994	100
ALLIANCE 1 WNW	ne	42.10	102.90	3990	1896	1994	95
ATKINSON	ne	42.55	98.97	2130	1906	1994	89
BRIDGEPORT	ne	41.67	103.10	3660	1897	1994	98
BROKEN BOW 2 W	ne	41.42	99.68	2500	1894	1994	101
CULBERTSON	ne	40.22	100.83	2610	1889	1994	96
DAVID CITY	ne	41.25	97.13	1620	1897	1994	98
FAIRBURY 2 SSE	ne	40.12	97.17	1340	1895	1994	100
FRANKLIN	ne	40.10	98.97	1860	1888	1990	103
GORDON 3 W	ne	42.80	102.25	3640	1909	1994	83
GOTHENBURG	ne	40.93	100.17	2590	1894	1994	101
GRAND ISLAND WSO AP	ne	40.97	98.32	1840	1900	1994	95
HARRISON	ne	42.68	103.88	4850	1914	1994	81
HARTINGTON	ne	42.60	97.27	1370	1893	1994	102
KEARNEY	ne	40.70	99.10	2170	1931	1994	64
KIMBALL	ne	41.23	103.67	4710	1893	1994	97
LINCOLN AGRONOMY FARM	ne	40.85	96.62	1200	1921	1968	48
MADRID	ne	40.85	101.55	3200	1900	1994	93
MITCHELL 5 E	ne	41.95	103.68	4080	1909	1994	80
OSHKOSH	ne	41.40	102.35	3380	1913	1994	82
PAWNEE CITY	ne	40.10	96.15	1190	1903	1994	92
WEST POINT	ne	41.83	96.72	1260	1892	1994	102
BERLIN	nh	44.45	71.18	930	1926	1994	68
CONCORD WSO AP	nh	43.20	71.50	350	1921	1994	74
DURHAM	nh	43.15	70.95	70	1926	1994	69
HANOVER	nh	43.70	72.28	600	1926	1994	69
KEENE	nh	42.92	72.27	480	1926	1994	69
BELLEPLAIN STA FOREST	nj	39.25	74.87	30	1926	1994	69
BELVIDERE	nj	40.83	75.08	280	1926	1981	56
CANOE BROOK	nj	40.75	74.35	180	1931	1994	64
CHARLOTTEBURG RESERVOIR	nj	41.03	74.43	760	1926	1994	69
FLEMINGTON	nj	40.50	74.87	180	1926	1994	69
HIGHTSTOWN 2 W	nj	40.27	74.57	100	1931	1994	64
INDIAN MILLS 2 W	nj	39.80	74.78	100	1926	1994	69

STATION	ST	LAT	LONG	ELEV (ft)	BEG YR	END YR	NUM REC
LAMBERTVILLE	nj	40.37	74.95	60	1931	1994	64
LONG VALLEY	nj	40.78	74.78	550	1931	1994	64
MOORESTOWN	nj	39.97	74.97	50	1926	1994	69
NEW BRUNSWICK EXP STN	nj	40.47	74.43	90	1912	1968	57
PLAINFIELD	nj	40.60	74.40	90	1931	1994	64
SOMERVILLE 3 NW	nj	40.60	74.63	160	1931	1994	64
ALBUQUERQUE WSFO AP	nm	35.05	106.62	5310	1931	1994	64
BLOOMFIELD 3 SE	nm	36.67	107.97	5810	1925	1994	67
BRANTLEY DAM	nm	32.52	104.38	3210	1987	1994	8
CIMARRON 4 SW	nm	36.47	104.95	6540	1904	1994	91
CLAYTON WSO AP	nm	36.45	103.15	4970	1896	1992	87
CLOVIS 3 SSW	nm	34.37	103.20	4280	1910	1994	85
CORONA	nm	34.25	105.58	6650	1931	1977	47
ELK 2 E	nm	32.95	105.30	5710	1895	1994	54
FORT BAYARD	nm	32.80	108.15	6140	1897	1994	98
MOSQUERO 1 NE	nm	35.80	103.93	5470	1926	1994	69
ROSWELL WSO AP	nm	33.40	104.53	3640	1893	1972	80
SANTA FE	nm	35.68	105.90	7200	1874	1972	99
SOCORRO	nm	34.08	106.88	4590	1931	1994	64
TUCUMCARI 4 NE	nm	35.20	103.68	4090	1904	1994	91
BATTLE MOUNTAIN	nv	40.65	116.93	4510	1928	1945	18
CALIENTE	nv	37.62	114.52	4400	1931	1994	64
FALLON EXP STN	nv	39.45	118.78	3970	1928	1994	67
LAMOILLE P H	nv	40.68	115.47	6290	1928	1972	45
LAS VEGAS	nv	36.17	115.13	2010	1928	1956	29
LOVELOCK	nv	40.18	118.47	3980	1928	1994	67
MC GILL	nv	39.40	114.77	6300	1928	1994	67
MINA	nv	38.38	118.10	4550	1928	1994	67
MINDEN AIRPORT	nv	39.00	119.75	4710	1928	1994	67
RENO WSFO AP	nv	39.50	119.78	4400	1937	1994	58
TONOPAH AIRPORT	nv	38.07	117.08	5430	1954	1994	41
WINNEMUCCA WSO AP	nv	40.90	117.80	4300	1928	1994	67
YERINGTON	nv	39.00	119.28	4380	1928	1994	67
ALBANY WSO AP	ny	42.75	73.80	280	1938	1994	57
ALFRED	ny	42.25	77.80	1740	1926	1994	69
ANGELICA	ny	42.30	78.03	1420	1926	1994	69
AUBURN 2 NE	ny	42.93	76.53	770	1926	1994	62
BINGHAMTON WB CITY	ny	42.10	75.92	860	1926	1968	43
BUFFALO WSFO AP	ny	42.93	78.73	710	1922	1994	73
CANTON	ny	44.58	75.17	410	1922	1994	73
CHAZY	ny	44.88	73.43	170	1926	1994	69
COOPERSTOWN	ny	42.70	74.92	1240	1926	1994	69
DANNEMORA	ny	44.72	73.72	1340	1926	1994	69
DELHI 2 SW	ny	42.25	74.93	1350	1926	1994	61
ELMIRA	ny	42.08	76.82	860	1926	1994	69
FREDONIA	ny	42.45	79.30	760	1926	1994	69
GENEVA SCS	ny	42.88	77.02	620	1921	1968	44
HEMLOCK	ny	42.78	77.62	900	1926	1994	69
ITHACA CORNELL UNIV	ny	42.45	76.45	960	1926	1994	69
JAMESTOWN	ny	42.10	79.25	1390	1926	1960	35
LITTLE FALLS CITY RES	ny	43.07	74.87	900	1926	1994	69
LOCKPORT 2 NE	ny	43.18	78.65	520	1926	1994	69
LOWVILLE	ny	43.80	75.48	860	1926	1994	68
MORRISVILLE	ny	42.90	75.65	1300	1926	1993	63
NEW YORK CENTRAL PARK	ny	40.78	73.97	130	1876	1994	119
NORWICH 1 NE	ny	42.53	75.50	1120	1926	1994	69
OGDENSBURG HOSP 3 NE	ny	44.73	75.45	280	1926	1994	69
OSWEGO EAST	ny	43.47	76.50	350	1926	1994	69
PORT JERVIS	ny	41.38	74.68	470	1926	1994	69
POUGHKEEPSIE CAA AP	ny	41.63	73.88	150	1948	1994	46
ROCHESTER WB AP	ny	43.12	77.67	550	1926	1994	69
SETAUKET	ny	40.95	73.10	40	1926	1994	69
TROY LOCK AND DAM	ny	42.75	73.68	20	1932	1994	40
WALDEN 1 ESE	ny	41.55	74.17	380	1973	1994	22
WANAKENA RANGER SCHOOL	ny	44.15	74.90	1510	1926	1994	69
WATERTOWN	ny	43.97	75.87	500	1926	1994	69
CADIZ	oh	40.27	81.00	1260	1903	1994	92
CANFIELD 1 S	oh	41.02	80.77	1140	1917	1994	78
CINCINNATI ABBE WSMO	oh	39.15	84.52	760	1916	1982	67
COLUMBUS OHIO STN	oh	40.00	83.02	760	1900	1957	58

STATION	ST	LAT	LONG	ELEV (ft)	BEG YR	END YR	NUM REC
UNIV							
GREENVILLE WATER PL	oh	40.10	84.65	1020	1900	1994	95
HILLSBORO	oh	39.20	83.62	1100	1900	1994	95
HIRAM	oh	41.30	81.15	1230	1900	1994	95
KENTON	oh	40.65	83.60	1000	1900	1994	95
LIMA SEWAGE PLANT	oh	40.72	84.13	850	1929	1994	66
NAPOLEON	oh	41.38	84.12	680	1900	1962	63
NORWALK WST WTR TRT PL	oh	41.27	82.62	670	1900	1994	95
WOOSTER EXP STATION	oh	40.78	81.92	1020	1900	1994	95
ADA	ok	34.78	96.68	1020	1907	1994	88
ARDMORE	ok	34.20	97.15	860	1901	1994	94
BRISTOW	ok	35.83	96.38	820	1915	1994	80
CHANDLER 1	ok	35.70	96.88	950	1901	1994	94
CHICKASHA	ok	35.03	97.95	1090	1901	1966	65
DURANT-USDA	ok	34.02	96.38	660	1901	1994	94
EL RENO 1 N	ok	35.55	97.97	1320	1893	1994	69
ENID	ok	36.42	97.87	1250	1894	1994	98
FREDERICK	ok	34.40	99.02	1300	1905	1994	90
HOLDENVILLE	ok	35.08	96.40	860	1901	1994	94
HOLLIS	ok	34.70	99.92	1630	1923	1994	71
JEFFERSON	ok	36.72	97.80	1050	1897	1994	97
KINGFISHER 2 SE	ok	35.85	97.90	1100	1897	1994	97
NEWKIRK	ok	36.88	97.05	1150	1898	1994	97
OKEMAH	ok	35.43	96.30	940	1912	1994	83
PAULS VALLEY 4 WSW	ok	34.73	97.28	940	1900	1994	95
PAWHUSKA	ok	36.67	96.35	840	1898	1994	97
SMITHVILLE 1 W	ok	34.47	94.67	840	1888	1994	48
STILLWATER 2 W	ok	36.12	97.10	900	1893	1994	101
WAURIKA	ok	34.17	98.00	880	1910	1994	85
ANTELOPE 1 NW	or	44.92	120.73	2840	1931	1994	60
BAKER FAA AP	or	44.83	117.82	3370	1948	1994	47
BEND	or	44.07	121.28	3660	1928	1994	67
BROOKINGS	or	42.05	124.28	70	1931	1994	64
CASCADIA	or	44.40	122.48	860	1931	1994	64
CONDON	or	45.23	120.18	2860	1928	1994	67
CRATER LAKE NPS HQ	or	42.90	122.13	6480	1931	1994	63
DALLAS 2 NE	or	44.95	123.28	290	1928	1994	61
DANNER	or	42.93	117.33	4230	1931	1994	64
DUFUR	or	45.45	121.13	1330	1904	1994	68
ENTERPRISE	or	45.43	117.27	3790	1931	1981	51
EUGENE WSO AP	or	44.12	123.22	360	1939	1994	56
FOREST GROVE	or	45.53	123.10	180	1928	1994	67
GRANTS PASS	or	42.42	123.33	960	1928	1994	67
HEPPNER	or	45.37	119.55	1890	1928	1994	67
HERMISTON 2 S	or	45.82	119.28	620	1928	1994	67
HOOD RIVER EXP STN	or	45.68	121.52	500	1928	1994	67
KLAMATH FALLS 2 SSW	or	42.20	121.78	4100	1928	1994	67
LAKEVIEW 2 NNW	or	42.22	120.37	4780	1928	1994	67
MC MINNVILLE	or	45.23	123.18	150	1928	1994	64
MEDFORD WSO AP	or	42.38	122.88	1300	1928	1994	67
MILTON FREEWATER	or	45.95	118.42	970	1928	1994	67
MORO	or	45.48	120.72	1870	1928	1994	67
NEWPORT	or	44.58	124.05	140	1931	1994	64
PARKDALE	or	45.52	121.58	1710	1928	1969	42
PENDLETON WSO AP	or	45.68	118.85	1490	1928	1994	67
PORTLAND RFC CITY	or	45.53	122.67	30	1928	1973	46
PRINEVILLE 4 NW	or	44.35	120.90	2840	1928	1994	67
PROSPECT 2 SW	or	42.73	122.52	2480	1931	1994	64
REDMOND FAA AP	or	44.27	121.15	3060	1949	1994	46
ROSEBURG AP	or	43.23	123.37	510	1931	1965	35
SALEM WSO AP	or	44.92	123.02	200	1928	1994	67
SEASIDE	or	45.98	123.92	10	1930	1994	65
SQUAW BUTTE EXP STN	or	43.48	119.72	4660	1937	1994	58
THREE LYNX	or	45.12	122.07	1120	1931	1994	64
UKIAH	or	45.13	118.93	3360	1931	1994	63
UNION EXP STN	or	45.22	117.88	2770	1928	1994	67
VALE	or	43.98	117.25	2240	1928	1994	66
MADRAS	or	44.63	121.13	2230	1928	1994	67
DRAIN	or	43.67	123.32	290	1948	1994	48
ALTOONA HORSESHOE CURVE	pa	40.50	78.48	1500	1926	1967	42

STATION	ST	LAT	LONG	ELEV (ft)	BEG YR	END YR	NUM REC
BROOKVILLE FAA AIRPORT	pa	41.15	79.10	1420	1926	1962	37
CORRY	pa	41.92	79.63	1440	1926	1994	69
COUDERSPORT 1 NNE	pa	41.78	78.02	1690	1929	1954	24
DERRY 4 SW	pa	40.30	79.33	1060	1926	1994	65
EBENSBURG SEWAGE PLANT	pa	40.47	78.73	1940	1964	1994	31
EMPORIUM	pa	41.50	78.23	1040	1969	1994	26
ERIE WSO AP	pa	42.08	80.18	730	1926	1994	69
FRANKLIN	pa	41.38	79.82	990	1926	1994	69
GREENVILLE 2 NE	pa	41.42	80.37	1130	1926	1994	69
HARRISBURG FAA AP	pa	40.22	76.85	340	1926	1991	66
HAWLEY 1 E	pa	41.48	75.17	890	1926	1994	63
JOHNSTOWN	pa	40.33	78.92	1210	1926	1993	68
LANCASTER 2NE PUMP STN	pa	40.05	76.28	260	1926	1974	49
LOCK HAVEN	pa	41.13	77.42	550	1926	1977	52
MONTROSE	pa	41.83	75.87	1560	1926	1994	68
PALMERTON	pa	40.80	75.62	410	1926	1994	68
PHILADELPHIA SHAWMONT	pa	40.03	75.25	70	1926	1957	32
PITTSBURGH WSO CI	pa	40.45	80.00	750	1926	1979	54
READING WB CITY	pa	40.33	75.97	270	1926	1973	48
RIDGWAY	pa	41.42	78.75	1360	1926	1994	69
STATE COLLEGE	pa	40.80	77.87	1170	1926	1994	69
STROUDSBURG	pa	41.00	75.18	480	1926	1994	69
TOWANDA 1 ESE	pa	41.75	76.42	750	1926	1994	69
UNIONTOWN 1 NE	pa	39.92	79.72	960	1926	1994	69
WARREN	pa	41.85	79.15	1210	1926	1994	69
WELLSBORO 3 S	pa	41.70	77.27	1860	1926	1994	68
WILLIAMSPORT WSO AP	pa	41.25	76.92	520	1948	1994	47
YORK 3 SSW PUMP STN	pa	39.92	76.75	390	1926	1994	69
KINGSTON	ri	41.48	71.53	100	1926	1994	69
BLACKVILLE 3 W	sc	33.37	81.32	320	1930	1994	65
CALHOUN FALLS	sc	34.08	82.58	530	1930	1994	65
CHARLESTON WSO AP	sc	32.90	80.03	40	1930	1994	65
CHERAW	sc	34.70	79.88	140	1930	1994	65
CLEMSON UNIVERSITY	sc	34.68	82.82	820	1930	1994	65
COLUMBIA UNIV OF SC	sc	33.98	81.02	240	1930	1994	65
CONWAY	sc	33.83	79.05	20	1930	1994	65
GEORGETOWN 2 E	sc	33.35	79.25	10	1930	1994	61
KINGSTREE 1 SE	sc	33.65	79.82	60	1930	1994	65
LAURENS	sc	34.50	82.03	590	1930	1994	65
LITTLE MOUNTAIN	sc	34.20	81.42	710	1930	1994	65
SANTUCK	sc	34.63	81.52	520	1930	1994	65
SUMMERVILLE	sc	32.98	80.18	40	1930	1994	65
WINNSBORO	sc	34.37	81.08	560	1930	1994	65
YEMASSEE	sc	32.68	80.85	30	1930	1994	65
GREENVILLE WB AIRPORT	sc	34.85	82.35	1020	1930	1962	33
ABERDEEN WSO AP	sd	45.45	98.43	1300	1932	1994	63
ACADEMY 2 NE	sd	43.50	99.07	1680	1898	1994	97
ALEXANDRIA	sd	43.65	97.78	1350	1932	1994	63
ARMOUR	sd	43.32	98.35	1510	1896	1994	99
BRITTON	sd	45.78	97.75	1340	1913	1994	82
BROOKINGS 2 NE	sd	44.32	96.77	1640	1893	1994	102
CAMP CROOK	sd	45.55	103.98	3120	1896	1994	98
CENTERVILLE 6 SE	sd	43.05	96.90	1260	1905	1994	90
CLARK	sd	44.88	97.73	1780	1896	1994	97
COTTONWOOD 2 E	sd	43.97	101.87	2410	1909	1994	86
DUPREE	sd	45.05	101.60	2370	1922	1994	73
EUREKA	sd	45.78	99.63	1870	1877	1994	88
FAIRFAX	sd	43.03	98.88	1930	1902	1956	55
FAITH 2 W	sd	45.03	102.08	2550	1926	1994	69
FAULKTON 1 NW	sd	45.03	99.13	1570	1896	1994	98
FORESTBURG 3 NE	sd	44.03	98.07	1230	1896	1994	99
GANN VALLEY 2 NW	sd	44.05	99.03	1740	1920	1994	75
GREGORY	sd	43.23	99.43	2160	1930	1994	65
HOT SPRINGS	sd	43.43	103.47	3540	1908	1994	87
MC LAUGHLIN	sd	45.82	100.82	2000	1919	1994	48
MELLETTE	sd	45.15	98.50	1290	1896	1994	99
ALLARDT	tn	36.38	84.87	1680	1928	1994	67
BOLIVAR WTR WKS	tn	35.27	88.98	460	1930	1994	65
BROWNSVILLE	tn	35.58	89.25	330	1930	1994	65

STATION	ST	LAT	LONG	ELEV (ft)	BEG YR	END YR	NUM REC
CHATTANOOGA WSO AP	tn	35.03	85.20	680	1928	1994	67
COVINGTON 1 W	tn	35.57	89.67	310	1928	1994	67
CROSSVILLE EXP STN	tn	36.02	85.13	1810	1912	1994	83
DOVER 1 W	tn	36.48	87.85	480	1928	1994	50
FRANKLIN SEWAGE PLANT	tn	35.93	86.87	660	1928	1994	67
JACKSON EXP STN	tn	35.62	88.83	400	1900	1994	95
KINGSPORT	tn	36.52	82.53	1280	1931	1994	57
KNOXVILLE WSO AP	tn	35.80	84.00	950	1910	1994	85
LEWISBURG EXP STN	tn	35.45	86.80	790	1928	1994	67
MC MINNVILLE	tn	35.68	85.80	940	1927	1994	68
MEMPHIS FAA-AP	tn	35.05	90.00	270	1940	1994	55
MILAN	tn	35.98	88.83	470	1930	1992	63
NEWPORT 1 NW	tn	35.98	83.20	1040	1927	1994	68
ROGERSVILLE 1 NE	tn	36.42	82.98	1360	1927	1994	68
SAVANNAH 6 SW	tn	35.15	88.32	420	1927	1994	68
TULLAHOMA	tn	35.35	86.20	1050	1928	1994	67
WAYNESBORO	tn	35.30	87.77	750	1927	1994	68
ALBANY	tx	32.73	99.28	1420	1901	1994	94
ALICE	tx	27.73	98.07	200	1911	1994	77
ALPINE	tx	30.37	103.67	4480	1900	1994	66
ALVIN (HOU AREA WSO)	tx	29.42	95.22	40	1903	1993	36
ANAHUAC	tx	29.78	94.67	20	1931	1994	64
ANGLETON 2 W	tx	29.15	95.45	30	1913	1994	82
ANSON	tx	32.77	99.90	1700	1898	1994	38
ATHENS 3 SSE	tx	32.17	95.83	460	1903	1994	46
AUSTIN WSO AP	tx	30.30	97.70	600	1930	1994	65
BALLINGER 5 WSW	tx	31.73	100.05	1640	1897	1994	96
BALMORHEA	tx	30.98	103.75	3220	1923	1994	72
BAY CITY WATERWORKS	tx	28.98	95.98	50	1912	1994	59
BEAUMONT CITY	tx	30.10	94.10	20	1901	1981	75
BEEVILLE 5 NE	tx	28.45	97.70	260	1901	1994	93
BLANCO	tx	30.10	98.42	1370	1897	1994	97
BOERNE	tx	29.80	98.72	1420	1904	1994	91
BONHAM	tx	33.60	96.18	570	1903	1994	90
BOQUILLAS RANGER STN	tx	29.18	102.97	1880	1910	1994	32
BOWIE	tx	33.57	97.85	1120	1897	1994	70
BRACKETTVILLE	tx	29.32	100.42	1120	1897	1994	71
BRADY	tx	31.12	99.33	1720	1897	1994	62
BRAZORIA	tx	30.55	95.57	30	1897	1929	32
BRECKENRIDGE	tx	32.75	98.93	1180	1898	1994	71
BRENHAM	tx	30.15	96.40	350	1902	1994	93
BROWNFIELD 2	tx	33.18	102.27	3300	1953	1994	42
BURNET	tx	30.73	98.23	1280	1896	1994	96
CAMERON	tx	30.85	96.98	390	1908	1994	87
CARRIZO SPRINGS	tx	28.53	99.88	640	1912	1994	71
CHILDRESS 3 W	tx	34.43	100.25	1970	1897	1946	29
CLARENDON	tx	34.93	100.88	2700	1904	1994	89
CLARKSVILLE 2 NE	tx	33.63	95.03	440	1903	1994	90
CLEBURNE	tx	32.33	97.40	780	1907	1994	86
COLEMAN	tx	31.83	99.43	1730	1897	1994	97
COLLEGE STATION FAA AP	tx	30.58	96.35	310	1951	1994	44
CORSICANA	tx	32.08	96.47	430	1897	1994	95
CRANE	tx	31.38	102.33	2630	1929	1994	36
CROCKETT	tx	31.30	95.45	350	1904	1994	61
CROSBYTON	tx	33.50	101.25	3010	1897	1994	97
CUERO	tx	29.08	97.32	180	1901	1994	87
DALHART EXP STN	tx	36.02	102.58	4000	1906	1953	48
DALLAS FAA AP	tx	32.85	96.85	440	1897	1994	65
DANEVANG 1 W	tx	29.07	96.22	70	1897	1994	97
DENTON 2 SE	tx	33.20	97.10	630	1913	1994	82
DILLEY	tx	28.67	99.17	580	1916	1994	76
DUBLIN	tx	32.10	98.33	1500	1897	1994	96
EAGLE PASS	tx	28.70	100.48	810	1897	1994	95
EMORY	tx	32.87	95.73	460	1897	1994	36
ENCINAL	tx	28.03	99.37	560	1908	1994	87
FALFURRIAS	tx	27.23	98.13	120	1907	1994	88
FLATONIA	tx	29.67	97.12	520	1908	1994	87
FORT DAVIS	tx	30.60	103.88	4890	1902	1994	21
FOWLERTON 2 NW	tx	28.48	98.87	340	1913	1994	43
FREDERICKSBURG	tx	30.27	98.87	1750	1896	1994	75
FORT WORTH INTL WSO AP	tx	32.83	97.05	540	1897	1973	27

STATION	ST	LAT	LONG	ELEV (ft)	BEG YR	END YR	NUM REC
GAIL	tx	32.77	101.45	2530	1912	1994	37
GAINESVILLE	tx	33.63	97.13	760	1897	1987	89
GALVESTON WB AP	tx	29.27	94.85	10	1948	1963	16
GATESVILLE	tx	31.43	97.77	850	1900	1994	83
GEORGETOWN	tx	30.63	97.68	750	1896	1983	54
GILMER 2 W	tx	32.73	94.98	390	1929	1994	66
GRAHAM	tx	33.10	98.58	1050	1897	1994	85
GRANDFALLS 3 SSE	tx	31.30	102.83	2440	1909	1994	47
GRAPEVINE DAM	tx	32.97	97.05	590	1897	1994	61
GREENVILLE 7 NW	tx	33.20	96.22	610	1900	1994	94
GROVETON	tx	31.07	95.13	350	1930	1994	31
HALLETTSVILLE 2 N	tx	29.47	96.95	280	1897	1994	97
HARLINGEN	tx	26.20	97.67	30	1912	1994	81
HENRIETTA	tx	33.82	98.20	900	1897	1994	94
HEREFORD	tx	34.82	102.40	3820	1905	1994	67
HICO	tx	31.98	98.03	1030	1916	1994	79
HILLSBORO	tx	32.02	97.12	550	1903	1994	92
JEFFERSON 4 NE	tx	32.83	94.32	240	1903	1994	17
JEWETT	tx	31.35	96.15	510	1904	1991	31
KENT 8 SE	tx	31.00	104.10	4860	1988	1994	7
KERRVILLE	tx	30.05	99.15	1640	1897	1974	75
KILLEEN 3 S	tx	31.07	97.73	910	1978	1994	17
KINGSVILLE	tx	27.55	97.88	70	1902	1993	55
LAMESA 1 SSE	tx	32.70	101.93	2970	1927	1994	68
LAMPASAS	tx	31.05	98.18	1020	1897	1994	97
LIBERTY	tx	30.05	94.80	40	1904	1994	91
LLANO	tx	30.75	98.68	1040	1903	1994	92
LONGVIEW	tx	32.47	94.73	330	1902	1994	81
LUFKIN FAA AP	tx	31.23	94.75	280	1906	1994	88
LULING	tx	29.67	97.65	400	1897	1994	95
MARATHON	tx	30.22	103.23	4090	1897	1994	61
MARFA 2	tx	30.30	104.02	4710	1958	1994	37
MARLIN 3 NE	tx	31.33	96.85	370	1902	1994	56
MARSHALL	tx	32.53	94.35	350	1908	1994	87
MATAGORDA 2	tx	28.70	95.97	10	1927	1994	68
MC CAMEY	tx	31.13	102.20	2450	1932	1994	63
MEMPHIS	tx	34.73	100.53	2090	1905	1994	87
MENARD	tx	30.92	99.78	1950	1897	1994	61
MEXIA	tx	31.68	96.48	540	1904	1994	90
MIAMI	tx	35.70	100.63	2760	1905	1994	90
MOUNT PLEASANT	tx	33.17	95.00	430	1905	1994	79
NACOGDOCHES	tx	31.60	94.65	310	1900	1973	74
NEW BRAUNFELS	tx	29.73	98.12	710	1897	1994	97
PALESTINE 2 NE	tx	31.78	95.60	470	1930	1994	63
PAMPA 2	tx	35.57	100.97	3150	1964	1994	31
PARIS	tx	33.67	95.57	540	1896	1994	94
PEARSALL	tx	28.88	99.08	640	1902	1994	46
PECOS	tx	31.42	103.50	2610	1904	1994	66
PIERCE 1 E	tx	29.23	96.18	110	1904	1994	91
PLAINVIEW	tx	34.18	101.70	3370	1908	1994	87
PORT ISABEL	tx	26.07	97.22	20	1928	1994	62
PORT LAVACA 2	tx	28.62	96.63	20	1901	1988	66
PRESIDIO	tx	29.57	104.38	2550	1927	1994	68
QUANAH 5 SE	tx	34.25	99.68	1500	1904	1994	91
RAYMONDVILLE	tx	26.48	97.80	30	1913	1994	82
RIO GRANDE CITY 3 W	tx	26.38	98.87	180	1897	1994	75
ROBERT LEE (LCRA 55)	tx	31.90	100.48	1780	1908	1994	37
ROCKPORT	tx	28.02	97.05	10	1959	1994	36
ROCKSPRINGS	tx	30.02	100.22	2400	1932	1994	56
SAN ANTONIO WSFO	tx	29.53	98.47	790	1946	1994	49
SAN MARCOS	tx	29.85	97.95	610	1897	1994	95
SAN SABA	tx	31.18	98.72	1200	1901	1994	49
SANDERSON	tx	30.15	102.40	2820	1897	1994	48
SEALY	tx	29.78	96.13	190	1919	1994	71
SEMINOLE	tx	32.72	102.67	3340	1922	1994	73
SEYMOUR	tx	33.60	99.25	1290	1905	1994	81
SHERMAN	tx	33.63	96.62	720	1897	1994	95
SIERRA BLANCA	tx	31.18	105.35	4590	1897	1994	34
SMITHVILLE	tx	30.02	97.15	320	1921	1994	73
SNYDER	tx	32.72	100.92	2340	1911	1994	84
SONORA	tx	30.57	100.65	2140	1902	1994	60
SPEARMAN	tx	36.18	101.18	3100	1927	1994	68

STATION	ST	LAT	LONG	ELEV (ft)	BEG YR	END YR	NUM REC
SPUR	tx	33.48	100.85	2310	1911	1994	62
STAMFORD 1	tx	32.93	99.78	1640	1911	1994	20
STEPHENVILLE WSMO	tx	32.22	98.18	1310	1921	1994	57
STRATFORD	tx	36.35	102.08	3690	1911	1994	66
SULPHUR SPRINGS	tx	33.15	95.63	500	1897	1994	63
TAHOKA	tx	33.17	102.48	3120	1913	1994	68
TAYLOR	tx	30.57	97.42	570	1929	1994	66
TEMPLE	tx	31.08	97.37	700	1897	1994	97
THROCKMORTON 2 W	tx	33.18	99.20	1400	1924	1994	71
TILDEN	tx	28.42	98.53	350	1903	1994	46
TULIA 6 NE	tx	34.60	101.70	3500	1897	1919	20
TYLER CAA AP	tx	32.35	95.40	530	1898	1954	19
VALENTINE 10 WSW	tx	30.50	104.63	4420	1897	1976	7
VERNON 4 S	tx	34.08	99.30	1200	1904	1994	65
WACO WSO AP	tx	31.62	97.22	500	1930	1994	63
WAXAHACHIE	tx	32.42	96.85	630	1897	1994	97
WEATHERFORD	tx	32.77	97.82	1070	1902	1994	93
WICHITA FALLS WSO AP	tx	33.97	98.48	990	1907	1994	73
WILLS POINT	tx	32.70	96.02	520	1905	1994	75
ZAPATA	tx	26.88	99.30	320	1909	1994	36
ALTON	ut	37.43	112.48	7040	1928	1994	67
BLANDING	ut	37.62	109.47	6130	1904	1994	90
BLUFF	ut	37.28	109.55	4320	1928	1994	67
CASTLE DALE	ut	39.20	111.02	5620	1928	1994	57
CORINNE	ut	41.55	112.12	4230	1928	1994	67
DESERET	ut	39.28	112.65	4590	1928	1994	67
DUCHESNE	ut	40.17	110.40	5510	1906	1994	83
ELBERTA	ut	39.95	111.95	4680	1928	1992	65
ESCALANTE	ut	37.77	111.60	5810	1901	1994	92
FILLMORE	ut	38.95	112.32	5120	1928	1994	67
FORT DUCHESNE	ut	40.28	109.87	5050	1928	1994	67
HEBER	ut	40.50	111.42	5630	1928	1994	67
HIAWATHA	ut	39.48	111.02	7280	1921	1992	72
LEVAN	ut	39.57	111.87	5300	1928	1994	67
LOGAN UTAH ST U	ut	41.75	111.80	4790	1928	1994	67
MANTI	ut	39.25	111.63	5740	1928	1994	67
MILFORD WSMO	ut	38.43	113.02	5030	1928	1993	66
MOAB 4 NW	ut	38.60	109.60	3970	1928	1994	67
MYTON	ut	40.20	110.07	5080	1928	1994	66
OAK CITY	ut	39.38	112.33	5070	1928	1994	67
OGDEN SUGAR FACTORY	ut	41.23	112.03	4280	1928	1994	67
RICHFIELD RADIO KSVC	ut	38.77	112.08	5270	1928	1994	67
SALT LAKE CITY NWSFO AP	ut	40.78	111.95	4220	1948	1994	47
SCIPIO	ut	39.25	112.10	5310	1928	1994	67
SNAKE CREEK P H	ut	40.55	111.50	6000	1928	1994	67
SPANISH FORK P H	ut	40.08	111.60	4720	1928	1994	67
ST GEORGE	ut	37.12	113.57	2760	1928	1994	67
TOOELE	ut	40.53	112.30	5070	1919	1994	48
UTAH LAKE LEHI	ut	40.37	111.90	4500	1928	1994	67
VERNAL AP	ut	40.45	109.52	5260	1928	1994	67
WENDOVER AUTOB	ut	40.73	114.03	4240	1924	1994	63
ZION NATL PARK	ut	37.22	112.98	4050	1928	1994	67
BUCHANAN	va	37.53	79.68	880	1930	1994	62
CHARLOTTESVILLE 2 W	va	38.03	78.52	870	1948	1994	47
CHATHAM	va	36.82	79.40	640	1930	1994	65
CULPEPER	va	38.47	78.00	420	1930	1990	58
FARMVILLE 2 N	va	37.33	78.38	450	1914	1994	66
FREDERICKSBURG NAT PK	va	38.32	77.45	90	1930	1994	64
HOPEWELL	va	37.30	77.30	40	1930	1994	65
LANGLEY AIR FORCE BASE	va	37.08	76.35	10	1930	1994	65
LINCOLN	va	39.12	77.72	500	1930	1994	65
LYNCHBURG WSO AP	va	37.33	79.20	920	1930	1994	65
PENNINGTON GAP	va	36.75	83.05	1510	1931	1994	64
SALTVILLE	va	36.88	81.77	1720	1930	1948	19
STUART	va	36.63	80.27	1350	1930	1960	30
WOODSTOCK 2 NE	va	38.90	78.47	660	1930	1994	65
WYTHEVILLE 1 S	va	36.93	81.08	2450	1930	1994	65
CORNWALL	vt	43.95	73.22	490	1926	1994	69
NEWPORT	vt	44.93	72.20	770	1930	1994	65
NORTHFIELD	vt	44.17	72.65	670	1926	1994	49
SAINT JOHNSBURY	vt	44.42	72.02	700	1926	1994	69
ABERDEEN	wa	46.97	123.82	10	1931	1994	64
ANACORTES	wa	48.52	122.62	30	1931	1994	64
BICKLETON	wa	46.00	120.30	3000	1931	1994	64
BUCKLEY 1 NE	wa	47.17	122.00	690	1931	1994	64
CEDAR LAKE	wa	47.42	121.73	1560	1931	1994	64
CENTRALIA	wa	46.72	122.95	190	1931	1994	64
CHELAN	wa	47.83	120.03	1120	1890	1994	38
CLE ELUM	wa	47.18	120.95	1930	1931	1994	63
CLEARBROOK	wa	48.97	122.33	60	1931	1994	64
COLFAX 1 NW	wa	46.88	117.38	1960	1893	1994	48
COLVILLE AP	wa	48.55	117.88	1890	1948	1987	40
CONCRETE PPL FISH STN	wa	48.55	121.77	200	1931	1994	64
CUSHMAN DAM	wa	47.42	123.22	760	1931	1973	43
DARRINGTON R S	wa	48.25	121.60	550	1931	1994	64
DAYTON 1 WSW	wa	46.32	118.00	1560	1931	1994	64
DIABLO DAM	wa	48.72	121.15	890	1931	1994	63
ELLENSBURG	wa	46.97	120.55	1480	1901	1994	83
EPHRATA FAA AP	wa	47.32	119.52	1260	1949	1994	46
FORKS 1 E	wa	47.95	124.37	350	1931	1994	64
GOLDENDALE	wa	45.75	120.83	1650	1931	1972	42
LA CROSSE	wa	46.82	117.88	1480	1931	1994	64
LANDSBURG	wa	47.38	121.97	540	1931	1994	64
LIND 3 NE	wa	47.00	118.58	1630	1931	1994	63
LONGVIEW	wa	46.15	122.92	10	1931	1994	64
NESPELEM 2 S	wa	48.13	118.98	1890	1931	1991	61
NEWPORT	wa	48.18	117.05	2140	1927	1994	68
NORTHPORT	wa	48.92	117.78	1320	1920	1994	70
OLGA 2 SE	wa	48.62	122.80	80	1891	1994	104
OMAK 2 NW	wa	48.43	119.53	1230	1931	1994	54
PALMER 3 ESE	wa	47.30	121.85	920	1931	1994	64
PROSSER 4 NE	wa	46.25	119.75	900	1931	1994	64
PUYALLUP 2 W EXP STN	wa	47.20	122.33	50	1931	1994	64
RAINIER PARADISE R S	wa	46.78	121.73	5430	1948	1994	47
RAINIER LONGMIRE	wa	46.75	121.82	2760	1931	1978	48
SEATTLE TAC WSCMO AP	wa	47.45	122.30	450	1931	1994	48
SEDRO WOOLLEY	wa	48.50	122.23	60	1931	1994	64
SEQUIM	wa	48.08	123.10	180	1931	1980	50
SNOQUALMIE FALLS	wa	47.55	121.85	440	1931	1994	64
SPOKANE WSO AP	wa	47.63	117.53	2360	1889	1994	106
STEHEKIN 4 NW	wa	48.35	120.72	1270	1931	1994	64
VANCOUVER 4 NNE	wa	45.68	122.65	210	1898	1994	97
WALLA WALLA 3 W	wa	46.05	118.40	800	1931	1962	32
WATERVILLE	wa	47.65	120.07	2620	1931	1994	64
WENATCHEE	wa	47.42	120.32	640	1877	1994	65
WILBUR	wa	47.75	118.67	2230	1900	1994	95
DARLINGTON	wi	42.68	90.12	930	1901	1994	90
EAU CLAIRE	wi	44.82	91.50	770	1901	1960	60
HANCOCK EXP FARM	wi	44.12	89.53	1080	1903	1994	92
MADISON WB CITY	wi	43.08	89.40	940	1905	1963	59
MARSHFIELD EXP FARM	wi	44.65	90.13	1250	1913	1994	82
MERRILL	wi	45.18	89.68	1250	1905	1994	90
MINOCQUA DAM	wi	45.88	89.73	1590	1905	1994	90
PLYMOUTH	wi	43.75	87.98	870	1910	1994	85
SOLON SPRINGS	wi	46.35	91.82	1080	1920	1994	48
SPOONER EXP FARM	wi	45.82	91.88	1100	1911	1994	84
STURGEON BAY	wi	44.87	87.33	660	1905	1994	90
VIROQUA 2 NW	wi	43.57	90.90	1200	1901	1994	92
WATERTOWN	wi	43.18	88.73	820	1924	1994	71
WEST BEND	wi	43.40	88.18	940	1924	1994	71
BAYARD	wv	39.27	79.37	2380	1926	1994	69
CHARLESTON 1	wv	38.35	81.65	600	1926	1974	49
CLARKSBURG 1	wv	39.27	80.35	950	1926	1994	69
ELKINS WSO AP	wv	38.88	79.85	1990	1926	1994	69
FAIRMONT	wv	39.47	80.13	1300	1926	1994	69
FLAT TOP	wv	37.58	81.10	3340	1931	1994	64
GLENVILLE 1 ENE	wv	38.93	80.82	720	1926	1994	68
HUNTINGTON 1	wv	38.42	82.37	680	1926	1957	32
KEARNEYVILLE WSO	wv	39.38	77.88	550	1930	1994	65
MANNINGTON 1 N	wv	39.55	80.35	980	1926	1979	54
MARLINTON	wv	38.22	80.08	2150	1926	1994	25
MARTINSBURG FAA AP	wv	39.40	77.98	530	1926	1994	69
PARKERSBURG 1 E	wv	39.27	81.53	640	1926	1994	69

STATION	ST	LAT	LONG	ELEV (ft)	BEG YR	END YR	NUM REC
SPENCER 1 SE	wv	38.80	81.35	740	1926	1994	69
WARDENSVILLE R M FARM	wv	39.10	78.58	960	1926	1994	69
WHEELING WARWOOD DAM 12	wv	40.10	80.70	660	1926	1976	51
WHITE SULPHUR SPRINGS	wv	37.80	80.30	1920	1926	1994	69
WILLIAMSON	wv	37.67	82.28	670	1926	1993	68
BEDFORD 2 SE	wy	42.87	110.90	6330	1899	1967	69
BORDER 3 N	wy	42.25	111.03	6110	1902	1993	92
CHEYENNE WSFO AP	wy	41.15	104.82	6120	1915	1994	80
CHUGWATER	wy	41.75	104.82	5280	1915	1994	80
CLEARMONT 5 SW	wy	44.58	106.45	4060	1881	1994	39
CODY	wy	44.55	109.07	4990	1915	1994	79
COLONY	wy	44.93	104.20	3570	1915	1994	80
DOUGLAS	wy	42.77	105.38	4800	1915	1962	43
EVANSTON 1 E	wy	41.27	110.95	6810	1890	1994	97
FARSON	wy	42.12	109.43	6590	1915	1994	75
GILLETTE 9 ESE	wy	44.27	105.32	4640	1925	1994	70
GREEN RIVER	wy	41.53	109.47	6090	1915	1994	80
LARAMIE	wy	41.32	105.58	7200	1915	1961	47
LUSK	wy	42.77	104.47	5020	1915	1994	78
MEDICINE BOW	wy	41.90	106.20	6570	1948	1994	45
MORAN 5 WNW	wy	43.85	110.58	6790	1915	1994	80
NEWCASTLE	wy	43.85	104.22	4410	1918	1994	77
PINE BLUFFS	wy	41.18	104.07	5080	1919	1988	65
POWELL	wy	44.75	108.77	4380	1915	1981	67
RIVERTON	wy	43.02	108.38	4950	1918	1994	77
SHERIDAN FIELD STN	wy	44.83	106.83	3750	1920	1994	75
SOUTH PASS CITY	wy	42.47	109.80	7840	1915	1994	60
SUNDANCE	wy	44.40	104.35	4750	1915	1994	80
TORRINGTON EXP FARM	wy	42.08	104.22	4100	1922	1994	73
WHEATLAND 4 N	wy	42.12	104.95	4640	1915	1994	80

Appendix B

Supplemental Data Locations

STATION	ST	LAT	LONG	ELEV (ft)	BEG YR	END YR	NUM REC
ANCHORAGE PARK STRIP	ak	61.22	149.87	90	1964	1973	10
ANGOON	ak	57.50	134.58	10	1949	1989	40
ANNETTE WSO AP	ak	55.03	131.57	110	1949	1994	46
ANNEX CREEK	ak	58.32	134.10	20	1949	1989	41
BARROW WSO AP	ak	71.30	156.78	30	1949	1994	46
BARTER ISLAND WSO AP	ak	70.13	143.63	40	1949	1988	40
BETHEL WSO AP	ak	60.78	161.80	130	1949	1994	46
BIG DELTA FAA/AMOS AP	ak	64.00	145.73	1270	1937	1994	56
COLD BAY WSO AP	ak	55.20	162.72	100	1950	1994	45
CORDOVA FAA FSS AP	ak	60.50	145.50	40	1949	1994	46
DILLINGHAM FAA AP	ak	59.05	158.52	90	1951	1994	39
EIELSON FIELD	ak	64.67	147.10	550	1949	1994	46
EKLUTNA PROJECT	ak	61.47	149.17	40	1952	1994	43
ELMENDORF AF BASE	ak	61.25	149.80	190	1951	1994	44
GULKANA FSS/AMOS	ak	62.15	145.45	1570	1949	1994	46
HAINES TERMINAL	ak	59.27	135.45	180	1957	1988	32
HOMER WSO AP	ak	59.63	151.50	90	1932	1994	63
ILIAMNA FAA AP	ak	59.75	154.92	190	1939	1994	54
JUNEAU AP	ak	58.37	134.58	10	1949	1994	46
KASILOF 3 NW	ak	60.37	151.38	70	1931	1994	60
KETCHIKAN	ak	55.35	131.65	80	1949	1994	46
KING SALMON WSO AP	ak	58.68	156.65	50	1955	1994	40
KITOI BAY	ak	58.18	152.35	20	1954	1994	41
KODIAK NAS	ak	57.75	152.52	20	1931	1972	42
KOTZEBUE WSO AP	ak	66.87	162.63	10	1949	1994	46
LITTLE PORT WALTER	ak	56.38	134.65	10	1949	1994	46
MATANUSKA AGRI EXP STN	ak	61.57	149.27	150	1917	1994	78
MC GRATH WSO AP	ak	62.97	155.62	340	1941	1994	54
MC KINLEY PARK	ak	63.72	148.97	2070	1949	1994	46
NOME WSO AP	ak	64.50	165.43	10	1949	1994	46
PALMER IAS	ak	61.60	149.10	230	1949	1994	46
PETERSBURG	ak	56.82	132.97	0	1949	1983	34
PUNTILLA	ak	62.10	152.75	1830	1949	1994	46
SHEMYA USAF BASE	ak	52.72	174.10	120	1949	1994	43
SITKA MAGNETIC OBSY	ak	57.05	135.33	70	1899	1989	87
TALKEETNA WSCMO AP	ak	62.30	150.10	350	1949	1994	46
TANANA FAA AP	ak	65.17	152.10	230	1949	1994	46
TOK	ak	63.35	143.03	1620	1954	1994	41
UNALAKLEET WSO AP	ak	63.88	160.80	20	1949	1994	46
UNIVERSITY EXP STN	ak	64.85	147.87	480	1931	1994	64
WALES	ak	65.62	168.05	10	1949	1994	45
WRANGELL	ak	56.48	132.37	40	1949	1994	46
YAKUTAT WSO AP	ak	59.52	139.67	30	1949	1994	46

STATION	ST	LAT	LONG	ELEV (ft)	BEG YR	END YR	NUM REC
HALEAKALA R S 338	hi	20.77	156.25	7030	1949	1994	46
HANA AP 355	hi	20.80	156.02	60	1950	1994	45
HAWAII VOLONS NP HQ 54	hi	19.43	155.27	3870	1949	1994	46
HILO WSO AP 87	hi	19.72	155.07	30	1949	1994	46
HONOLULU WSFO 703 AP	hi	21.33	157.92	10	1949	1994	46
KAHULUI WSO AP 398	hi	20.90	156.43	50	1954	1994	41
KAILUA 446	hi	20.90	156.22	700	1949	1994	46
KAINALIU 73.2	hi	19.53	155.93	1500	1949	1994	46
KANEOHE MAUKA 781	hi	21.42	157.82	190	1949	1994	46
LAHAINA 361	hi	20.88	156.68	50	1949	1994	46
LIHUE 1020	hi	21.98	159.37	210	1949	1963	15
MAUNA LOA SLOPE OBS 39	hi	19.53	155.58	11150	1955	1994	40
NAALEHU 14	hi	19.07	155.58	800	1954	1994	41
OOKALA 223	hi	20.02	155.28	430	1949	1993	45
OPAEULA 870	hi	21.57	158.03	1060	1949	1994	46
WAIALUA 847	hi	21.58	158.12	30	1949	1994	46

STATION	ST	LAT	LONG	ELEV (ft)	BEG YR	END YR	NUM REC
BOULDER	co	40.03	105.28	5420	1948	1994	47
BYERS 5 ENE	co	39.75	104.13	5100	1948	1994	47
CANON CITY	co	38.42	105.22	5360	1948	1994	47
CHEESMAN	co	39.22	105.28	6880	1948	1994	47
CHERRY CREEK DAM	co	39.65	104.85	5650	1951	1994	44
COCHETOPA CREEK	co	38.43	106.77	8000	1948	1994	47
COLORADO SPRGS WSO AP	co	38.82	104.72	6090	1948	1994	47
BELFAST	me	44.40	69.00	20	1948	1994	47
CARIBOU WSO AP	me	46.87	68.02	620	1939	1994	56
CORINNA	me	44.92	69.27	220	1948	1994	47
FORT KENT	me	47.25	68.58	520	1945	1994	50
GARDINER	me	44.22	69.78	140	1948	1994	47
GREENVILLE	me	45.47	69.60	1030	1920	1975	56
JACKMAN	me	45.63	70.27	1180	1951	1994	44
RIPOGENUS DAM	me	45.88	69.18	970	1948	1994	47
ALMA	mi	43.38	84.67	760	1948	1994	47
ALPENA WSO AP	mi	45.07	83.57	690	1948	1994	47
BOYNE FALLS	mi	45.17	84.92	740	1961	1994	34
MULLEN	ne	42.05	101.05	3250	1948	1994	47
MULLEN 21 NW	ne	42.27	101.33	3450	1952	1994	42
VALENTINE WSO AP	ne	42.87	100.55	2590	1948	1994	47
OROGRANDE	nm	32.38	106.10	4180	1948	1994	47
PASAMONTE	nm	36.30	103.73	5650	1948	1994	47
QUEMADO RANGER STN	nm	34.35	108.50	6880	1948	1992	45
RED RIVER	nm	36.70	105.40	8680	1948	1994	47
ADAVEN	nv	38.12	115.58	6250	1928	1982	55
BEOWAWE	nv	40.60	116.48	4700	1949	1994	46
CARSON CITY	nv	39.15	119.77	4650	1948	1994	47
DENIO	nv	41.97	118.63	4190	1951	1994	43
DYER 4 SE	nv	37.62	118.03	4980	1950	1994	45
ELY WSO AP	nv	39.28	114.85	6260	1948	1994	47
EUREKA	nv	39.52	115.97	6540	1952	1994	40
LEHMAN CAVES N M	nv	39.00	114.22	6830	1948	1987	40
LEONARD CREEK RANCH	nv	41.52	118.72	4220	1954	1994	41
ALTUS DAM	ok	34.88	99.30	1530	1948	1994	47
ANTLERS	ok	34.25	95.63	520	1948	1994	47
BEAVER	ok	36.82	100.53	2470	1948	1994	47
BUFFALO	ok	36.83	99.62	1800	1948	1994	47
CHATTANOOGA 3 NE	ok	34.45	98.62	1150	1948	1994	47
ERICK 4 E	ok	35.20	99.80	1990	1948	1994	47
IDABEL	ok	33.88	94.82	460	1948	1994	47
MIAMI	ok	36.88	94.88	810	1948	1994	47
AUSTIN 3 S	or	44.58	118.50	4210	1948	1994	47
BURNS WSO CI	or	43.58	119.05	4140	1939	1980	42
HART MOUNTAIN REFUGE	or	42.55	119.65	5620	1939	1994	56
P-RANCH REFUGE	or	42.82	118.88	4200	1955	1994	40
SENECA	or	44.13	118.97	4660	1949	1994	46
LONG VALLEY	sd	43.47	101.50	2470	1948	1994	47
MURDO	sd	43.88	100.72	2300	1948	1994	47
BOYSEN DAM	wy	43.42	108.18	4640	1948	1994	47
DIVERSION DAM	wy	43.23	108.93	5580	1948	1994	47
DIXON	wy	41.03	107.53	6360	1922	1978	57
KAYCEE	wy	43.72	106.63	4660	1948	1994	47
LANDER WSO AP	wy	42.82	108.73	5370	1948	1994	47
MIDWEST	wy	43.40	106.28	4820	1948	1994	47
MUDDY GAP	wy	42.37	107.47	6250	1950	1994	44
PAVILLION	wy	43.25	108.68	5440	1948	1994	47

Appendix C

Proposed Revisions to the AASHTO Standard Specifications

General Comments

This report contains recommendations which may ultimately lead to revisions of the AASHTO Specifications. This appendix contains a first draft of possible changes to the AASHTO Standard Specifications, which incorporate recommendations of this report. Several parts of the both the design and construction specification may require minor revision. The best locations for inserting all of these requirements are not clear to the author. Existing provisions and commentary to the new provision proposals are italicized.

Existing AASHTO Standard Specification Provisions

3.16 THERMAL FORCES

Provision shall be made for stresses or movements resulting from variations in temperature. The rise and fall in temperature shall be fixed for the locality in which the structure is to be constructed and shall be computed from an assumed temperature at the time of erection. Due consideration shall be given to the lag between air temperature and the interior temperature of massive concrete members or structures.

The range of temperature shall generally be as follows:

Metal Structures

Moderate climate, from 0 to 120^{o}F

Cold climate, from -30 to 120^{o}F

Concrete Structures	*Temperature Rise*	*Temperature Fall*
Moderate Climate...	*30^{o}F*	*40^{o}F*
Cold Climate	*35^{o}F*	*45^{o}F*

Proposed Revisions to AASHTO Standard Specification Provisions

3.16 THERMAL FORCES AND MOVEMENTS

Provision shall be made for stresses and movements resulting from variations in temperature. The provisions shall be based upon a maximum design temperature, $T_{MaxDesign}$, a minimum design temperature, $T_{MinDesign}$, and a design installation temperature, $T_{Install}$. The design installation temperature shall depend upon the component under consideration and the time of erection.

Temperature Rise = $T_{MaxDesign}$ - $T_{Install}$

Temperature Fall = $T_{Install}$ - $T_{MinDesign}$

Commentary: Note that the notation has changed to make the various material fit together, but this is no change from existing AASHTO.

3.16.1. Concrete Bridges

For all concrete bridges, $T_{MaxDesign}$ shall be determined from the contours of Fig. 3.16.1.1 and $T_{MinDesign}$ shall be determined from the contours of Fig. 3.16.1.2. The design values for locations between contours shall be determined by linear interpolation. As a alternative method, the largest adjacent contour may be used to define $T_{MaxDesign}$, and the smallest adjacent contour may be used to define $T_{MinDesign}$. Both the minimum and maximum design temperatures shall be noted on the drawings for installation of the girders, expansion joints and bearings.

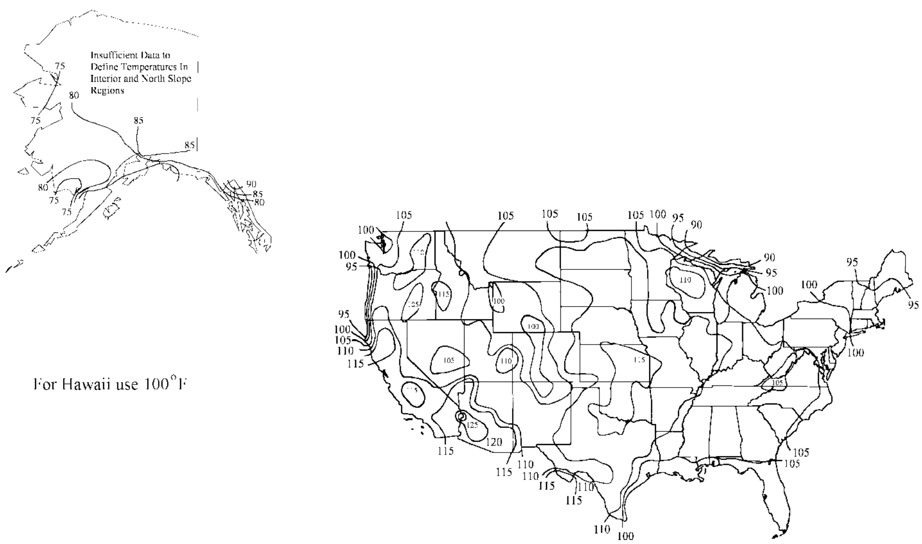


Figure 3.16.1.1. Contour Maps for Establishing $T_{MaxDesign}$ for Concrete Bridges

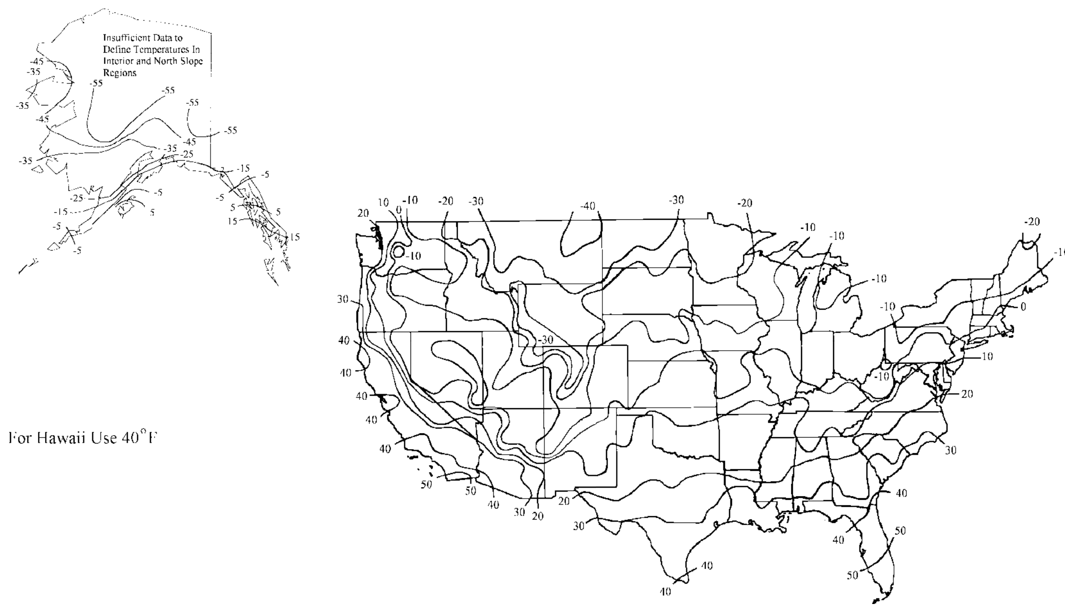
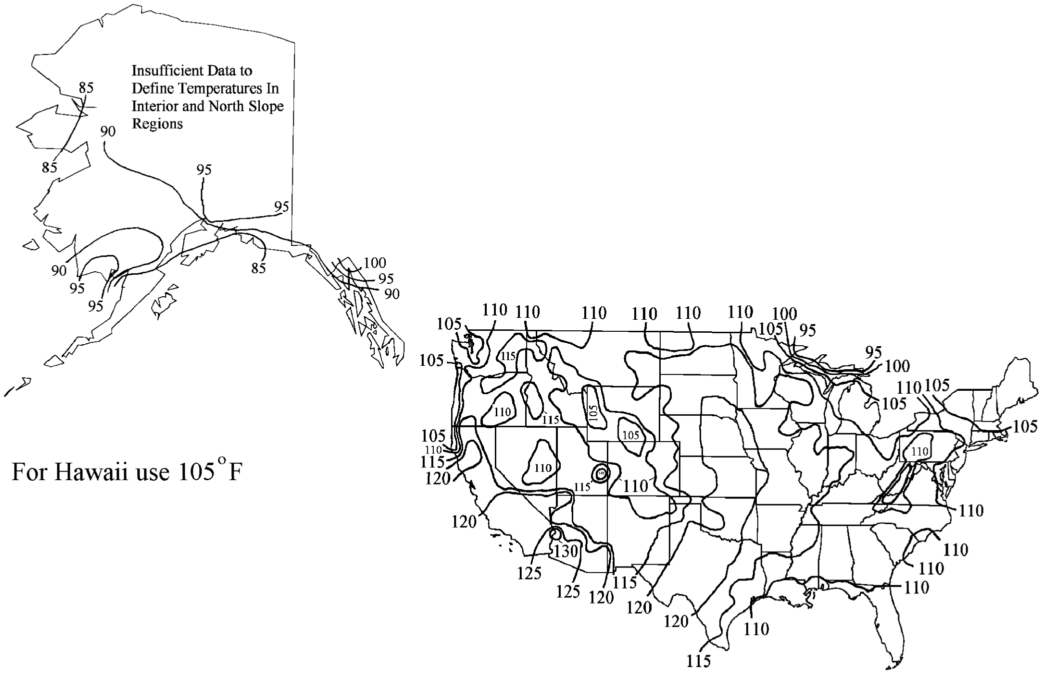


Figure 3.16.1.2. Contour Maps for Establishing $T_{MinDesign}$ for Concrete Bridges

Commentary: The temperatures provided in the above maps are extreme average bridge temperatures. The above maps are developed upon an analysis of past weather data for locations throughout the US and consideration of the heat flow through a concrete bridge. A minimum 60 year history was used for this analysis, but the average history for the locations used to determine the extreme average bridge temperatures is 70.7 years.

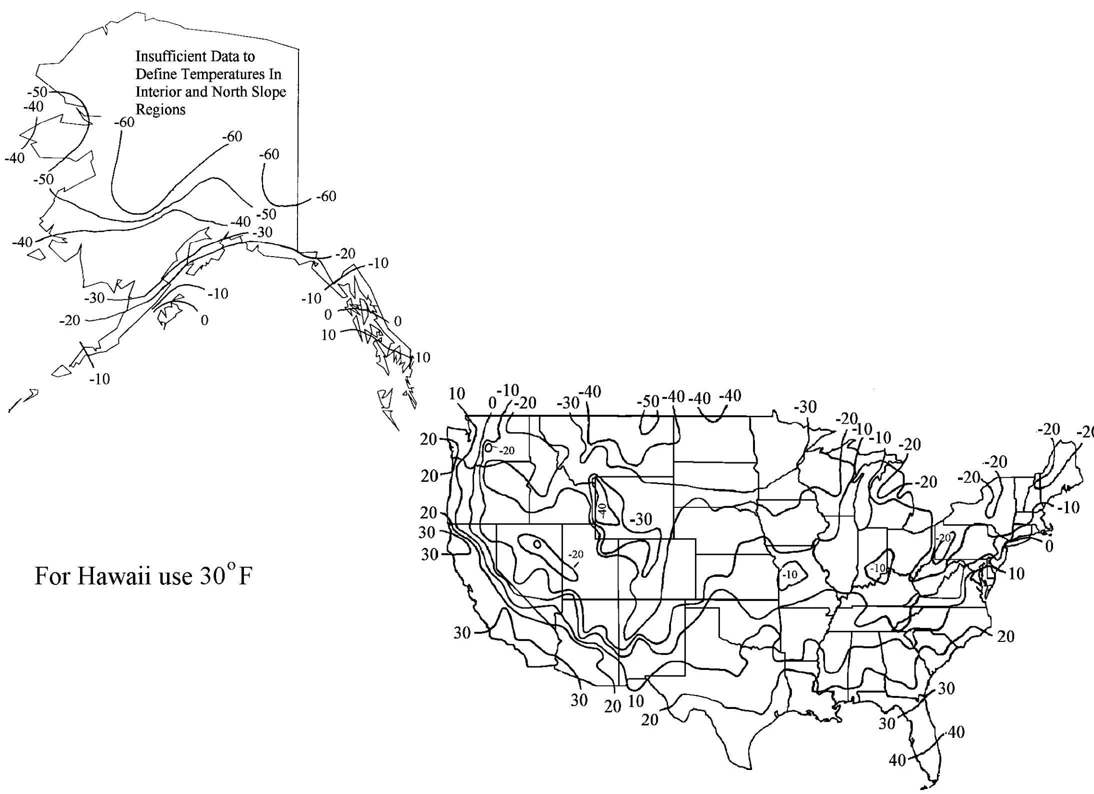
3.16.2 Steel Girder Bridges with Concrete Decks

For all steel girder bridges including box girders, plate girders and rolled shapes with a concrete deck, $T_{MaxDesign}$ shall be determined from the contours of Fig. 3.16.2.1 and $T_{MinDesign}$ shall be determined from the contours of Fig. 3.16.2.2. The design values for locations between contours shall be determined by linear interpolation. As a alternative method, the largest adjacent contour may be used to define $T_{MaxDesign}$, and the smallest adjacent contour may be used to define $T_{MinDesign}$. Both the minimum and maximum design temperatures shall be noted on the drawings for installation of the girders, expansion joints and bearings.



For Hawaii use 105°F

Figure 3.16.2.1. Contour Maps for Establishing $T_{MaxDesign}$ for Steel Bridges with Concrete Decks



For Hawaii use 30°F

Figure 3.16.2.2. Contour Maps for Establishing $T_{MinDesign}$ for Steel Bridges with Concrete Decks

Commentary: The temperatures provided in the above maps are extreme average bridge temperatures. The above maps are developed from an analysis of past weather data for locations throughout the US and consideration of the heat flow and average temperature as a function of time on steel bridges.. A minimum 60 year history was used for this analysis, but the average history for the locations used to determine the extreme average bridge temperatures is 70.7 years.

3.16.3 All Other Metal Bridges

For all other metal bridges, $T_{MaxDesign}$ shall be the larger of $120^{o}F$ or the value determined from the contours of Fig. 3.16.2.1 and $T_{MinDesign}$ shall be the lower of $0^{o}F$ or the value determined from the contours of Fig. 3.16.2.2. The design values for locations between contours shall be determined by linear interpolation. Both the minimum and maximum design temperatures shall be noted on the drawings for installation of the girders, expansion joints and bearings.

3.16.2.1 Installation Temperature and Design Movements for Elastomeric Bearings

The design installation temperature for bridges with elastomeric pads and bearings shall be defined by

$$T_{Install} = T_{MinDesign} + 0.65 * (T_{MaxDesign} - T_{MinDesign}) \qquad \text{(Eq. 3.16.2.1a)}$$

and the design movement for the elastomeric bearing, Δ, shall be

$$\Delta = \pm\, \alpha\, L\, 0.65 * (T_{MaxDesign} - T_{MinDesign})\,. \qquad \text{(Eq. 3.16.2.1b)}$$

where L is the expansion length and α is the coefficient of thermal expansion. The minimum gap, Δ_g, to avoid hard contact between the steel girders and other girders, piers, abutments or major structural members shall be

$$\Delta_g = \alpha\, L\, 0.9 * (T_{MaxDesign} - T_{MinDesign})\,. \qquad \text{(Eq 3.16.2.18b)}$$

Commentary: Elastomeric bearings cannot be set in an offset position as can most other bearing systems. On the other hand, elastomeric bearings are very forgiving of infrequent deformations which exceed their nominal deformation capacity. Other bearing systems do not possess this forgiving nature or the reserve deformational capacity. This provision is based upon statistical evaluation of the time and temperature dependent movements of bridge girders, and it is designed to permit direct installation of the bridge girders without repositioning the bearings or girders after the initial installation and without resorting to an offset chart.

3.16.2.2 Installation Temperature and Design Movements for Other Bearings

Mechanical bearings, PTFE sliding surfaces, and bearings which may be offset during erection shall have a design installation temperature of

$$T_{Install} = T_{MinDesign} + 0.5 * (T_{MaxDesign} - T_{MinDesign}) \qquad \text{(Eq. 3.16.2.2a)}$$

and the design movement for the bearing, Δ, and the minimum gap, Δ_g, to avoid hard contact between the steel girders and other girders, piers, abutments or major structural members shall be

$$\Delta = \pm\,\alpha\, L\,(T_{MaxDesign} - T_{Install} + 20^{o}F)\ . \qquad \text{(Eq. 3.16.2.2b)}$$

where L is the expansion length and α is the coefficient of thermal expansion.

Commentary: An offset chart for erection of these girders and alignment of bearings is required with this design method.

3.16.2.3 Installation Temperature and Design Movements for Expansion Joints

Expansion joint shall have a design installation temperature of

$$T_{Install} = \frac{T_{MaxAir} + T_{MinAir}}{2} \qquad \text{(Eq. 3.16.2.3a)}$$

where T_{MaxAir} is the maximum daily air temperature for the previous day, and T_{MinAir} is the minimum night time air temperature for the morning of the day that the formwork for the joint gap is installed. The total movement,□, shall be

$$\text{Total Movement } \Delta = \alpha\, L\,(T_{MaxDesign} - T_{MinDesign} + 30^{o}F) \qquad \text{(Eq. 3.16.2.3a)}$$

where L is the expansion length and α is the coefficient of thermal expansion. The placement of the joint shall be then accommodate a temperature rise and fall as defined by the maximum and minimum design temperatures and the installation temperature.

Commentary: The installation temperature and the total movement gap for expansion joints depend upon the formwork used for placement of the bridge deck and the approach slab. Therefore, the installation temperature is based upon the average of the high and low temperature on the day preceeding installation of the formwork for the joint. This still leaves a slight uncertainty in the actual joint placement. However, the uncertainty is limited by the daily temperature variation, and so the 30^{o}F margin is used to overcome that uncertainty.

3.16.3 All Other Metallic Bridges

For all other metallic bridges which do not qualify for section 3.16.2, $T_{MaxDesign}$ shall be 120^{o}F, and $T_{MinDesign}$ shall be 0^{o}F for mild climates and -30^{o}F for cold climates. The rise and fall in temperature shall be fixed for the locality in which the structure is to be constructed and shall be computed from an assumed temperature at the time of erection. Due consideration shall be given to the lag between air temperature and the interior temperature of massive concrete members or structures.

Commentary: Note that the provisions for steel truss bridges and other metal structures not included in 3.16.2 are not changed from the existing AASHTO.

DIVISION II
CONSTRUCTION

X.XX INSTALLATION OF BRIDGE GIRDERS AND BEARINGS

Bridge girders that are designed by the provisions of 3.16.2 shall be installed to satisfy the installation temperature criteria outlined here. The air temperature at the time of placement of the steel girders on the elastomeric bearings shall be recorded and noted as the true installation temperature, $T_{TrueInstall}$. $T_{TrueInstall}$ for concrete bridges may be taken as the average of the daytime high air temperature and the previous night low air temperature for the day of installation.

X.XX.1 Bridges with Elastomeric Bearings

For bridges with elastomeric bearings and movements designed by 3.16.2.2, if

$$T_{TrueInstall} > T_{MinDesign} + 0.9 * (T_{MaxDesign} - T_{MinDesign})$$

or if

$$T_{TrueInstall} < T_{MinDesign} + 0.2 * (T_{MaxDesign} - T_{MinDesign}),$$

the girders shall be relifted and the elastomeric bearings allowed to relax to there neutral position at a later date. At the time of the relifting the air temperature, T_{Air}, shall satisfy the requirement that

$$0.4 * (T_{MaxDesign} - T_{MinDesign}) < \{T_{Air} - T_{MinDesign}\} < 0.7 * (T_{MaxDesign} - T_{MinDesign}) .$$

Commentary: The provision looks somewhat difficult but it is quite easily met. For girders installed on very hot summer days, relifting of the girders will be avoided if the girders are placed only in the early morning. The low temperature requirement is more easily met, because construction is unlikely to proceed during the coldest days at most locations, and the very low temperatures will occur only during the early morning during hours of darkness. Concrete bridges are unlikely to be affected by this provision, because $T_{TrueInstall}$ is dependent upon the average daily air temperature.

X.XX.2 Bridges with Other Bearing Systems

For bridges with other bearings and movements designed by 3.16.2.3, the bearing shall be offset by an amount

$$\text{Increment } \Delta = \alpha L \, 5^{o}F$$

for each $5^{o}F$ variation of the true installation temperature from the design installation temperature.

Commentary: This is attempting to do what has always been done with mechanical bearings, PTFE sliding surfaces, and other bearing types. An offset chart is provided by the bridge engineer, and the erector offsets the bearing to account for variation between the true installation temperature and the design installation temperature.

Appendix D

Proposed Revisions to the AASHTO LRFD Specifications

General Comments

This appendix contains a first draft of possible changes that may be made to the AASHTO LRFD Specification. These changes would incorporate recommendations of this report. Several parts of the specification may require minor revision, since some provisions address design issues while others address construction issues. The best locations for inserting these construction requirements is not clear, and the AASHTO Committee may have better suggestions. Existing provisions and commentary to the new provision proposals are italicized.

Existing AASHTO LRFD Specification Provisions

3.12.2 Uniform Temperature

3.12.2.1 TEMPERATURE RANGES

In the absence of more precise information, the ranges of temperature shall be as specified in Table 1. The difference between the extended lower or upper boundary and the base construction temperature assumed in the design shall be used to calculate thermal deformation effects.

Table 3.12.2.1-1 – Temperature Ranges

CLIMATE	*STEEL OR ALUMINUM*	*CONCRETE*	*WOOD*
Moderate	*0 ^{o} to 120 ^{o} F*	*10 ^{o} to 80 ^{o} F*	*10 ^{o} to 75 ^{o} F*
Cold	*-30 ^{o} to 120 ^{o} F*	*0 ^{o} to 80 ^{o} F*	*0 ^{o} to 75 ^{o} F*

3.12.2.2 SETTING TEMPERATURE

The setting temperature if the bridge, or any component thereof shall be taken as the actual air temperature averaged over the 24 hour period immediately preceding the setting event.

Proposed Revisions to AASHTO LRFD Specification Provisions

3.12.2 Uniform Temperature

3.12.2.1 TEMPERATURE RANGES

The temperature range shall be defined as the difference between the maximum design temperature, $T_{MaxDesign}$, and the minimum design temperature,

$T_{MinDesign}$. The design movements shall be obtained from based upon the maximum temperature rise or temperature fall from the design installation temperature, $T_{Install}$. The design installation temperature shall depend upon the component under consideration and the time of erection.

Temperature Rise = $T_{MaxDesign}$ - $T_{Install}$
Temperature Fall = $T_{Install}$ - $T_{MinDesign}$

Commentary: Note that the notation has changed to make the various material fit together, but this is no change from existing AASHTO.

3.12.2.1.1. Concrete Bridges

For all concrete bridges, $T_{MaxDesign}$ shall be determined from the contours of Fig. 3.12.2.1.1.1 and $T_{MinDesign}$ shall be determined from the contours of Fig. 3. 12.2.1.1.2. The design values for locations between contours shall be determined by linear interpolation. As a alternative method, the largest adjacent contour may be used to define $T_{MaxDesign}$, and the smallest adjacent contour may be used to define $T_{MinDesign}$. Both the minimum and maximum design temperatures shall be noted on the drawings for installation of the girders, expansion joints and bearings.

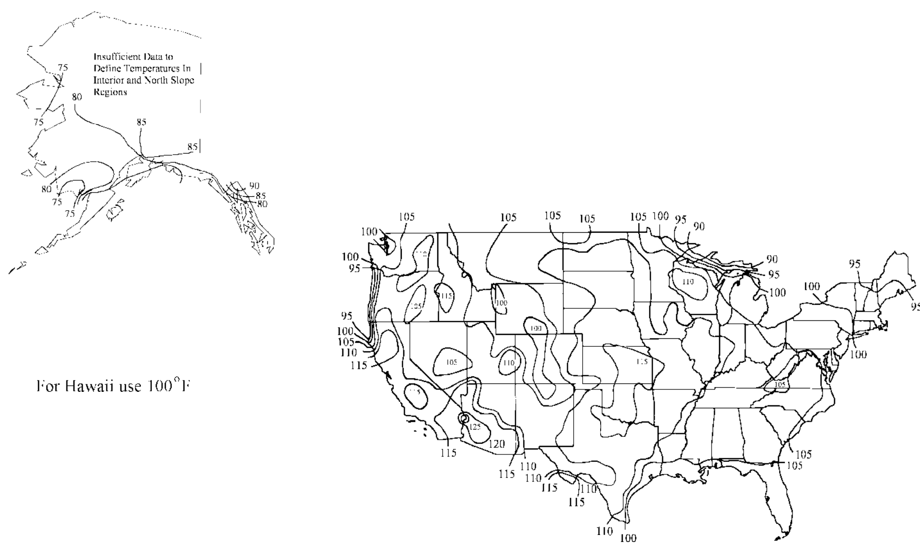


Figure 3.12.2.1.1.1. Contour Maps for Establishing $T_{MaxDesign}$ for Concrete Bridges

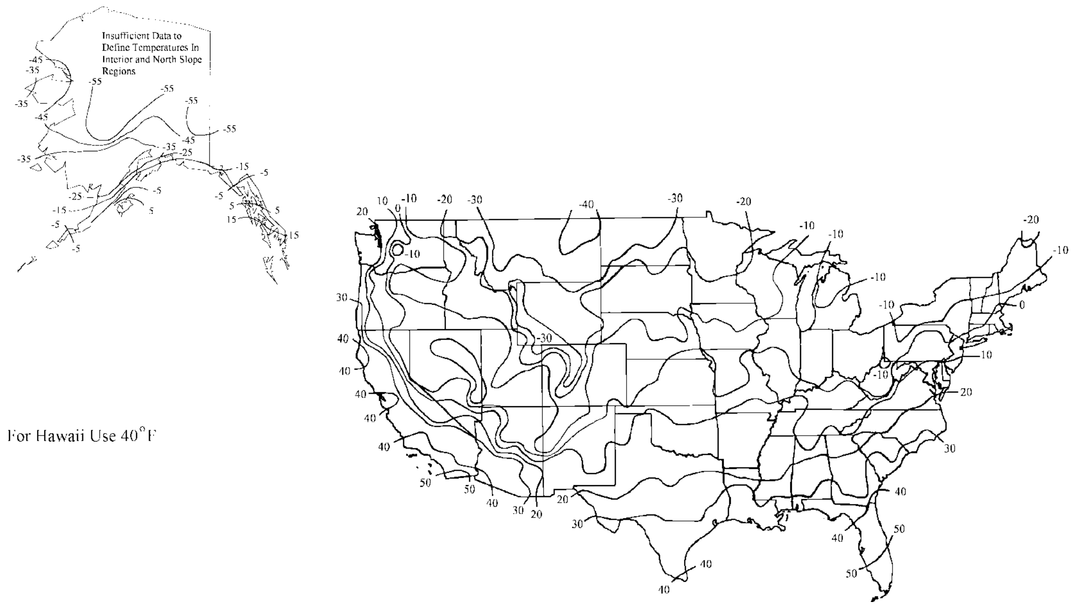


Figure 3.12.2.1.1.2. Contour Maps for Establishing $T_{MinDesign}$ for Concrete Bridges

Commentary: The temperatures provided in the above maps are extreme average bridge temperatures. The above maps are based upon an analysis of past weather data for locations throughout the US. A minimum 60 year history was used for this analysis, but the average history for the locations used to determine the extreme average bridge temperatures is 70.7 years.

3.12.2.1.2 Steel Girder Bridges with Concrete Decks

For all steel girder bridges including box girders, plate girders and rolled shapes with a concrete deck, $T_{MaxDesign}$ shall be determined from the contours of Fig. 3.12.2.1.2.1 and $T_{MinDesign}$ shall be determined from the contours of Fig. 3.12.2.1.2.2. The design values for locations between contours shall be determined by linear interpolation. As a alternative method, the largest adjacent contour may be used to define $T_{MaxDesign}$, and the smallest adjacent contour may be used to define $T_{MinDesign}$. Both the minimum and maximum design temperatures shall be noted on the drawings for installation of the girders, expansion joints and bearings.

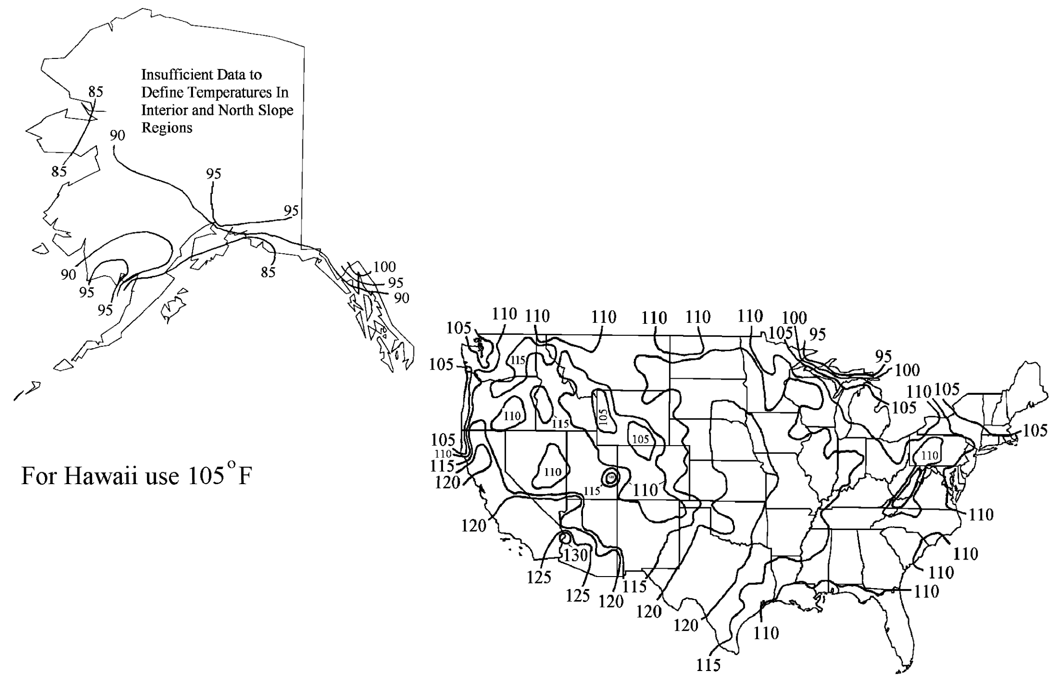


Figure 3.12.2.1.2.1. Contour Maps for Establishing $T_{MaxDesign}$ for Steel Bridges with Concrete Decks

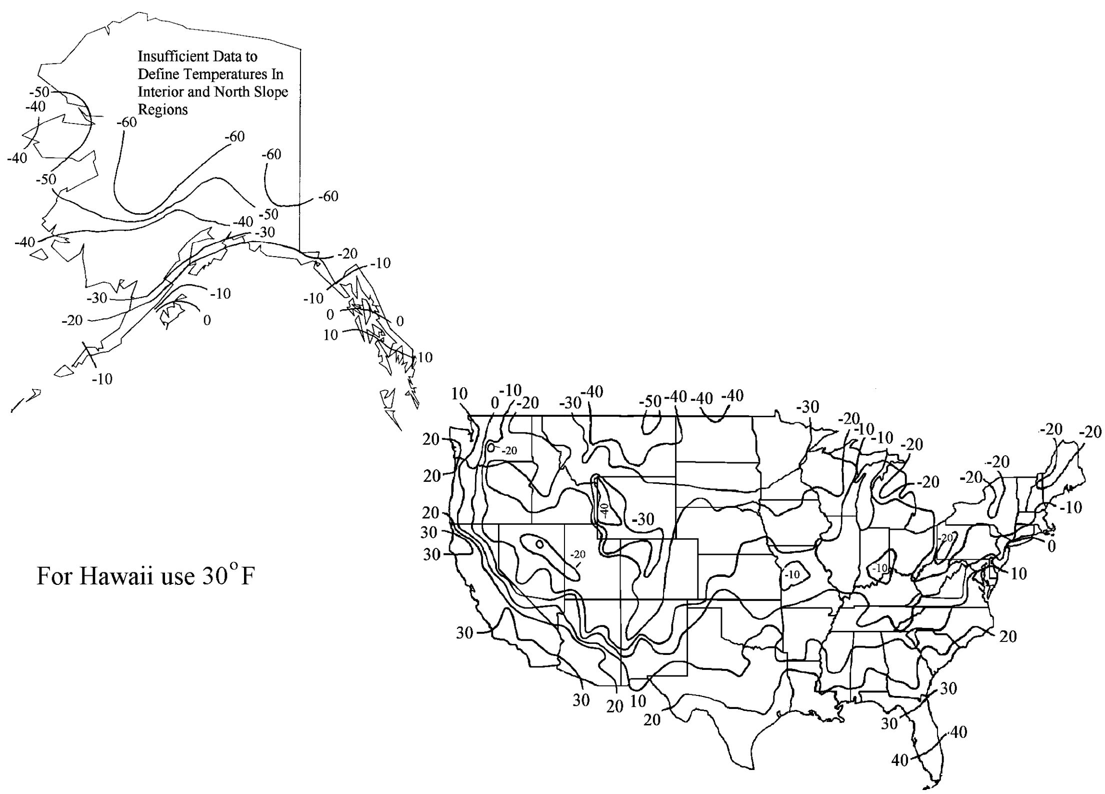


Figure 3.12.2.1.2.2. Contour Maps for Establishing $T_{MinDesign}$ for Steel Bridges with Concrete Decks

Commentary: The temperatures provided in the above maps are extreme average bridge temperatures. The above maps are developed from an analysis of past weather data for locations throughout the US. A minimum 60 year history was used for this analysis, but the average history for the locations used to determine the extreme average bridge temperatures is 70.7 years.

3.12.2.1.3 All Other Metal Bridges

For all other metal bridges, $T_{MaxDesign}$ shall be the larger of $120^{o}F$ or the value determined from the contours of Fig. 3.12.2.1.2.1 and $T_{MinDesign}$ shall be the lower of $0^{o}F$ or the value determined from the contours of Fig. 3.12.2.1.2.2. The design values for locations between contours shall be determined by linear interpolation. Both the minimum and maximum design temperatures shall be noted on the drawings for installation of the girders, expansion joints and bearings.

3.12.2.1.4 Timber Bridges

For timber bridges, $T_{MaxDesign}$ shall be $75^{o}F$ and $T_{MinDesign}$ shall be $0^{o}F$ if the bridge location has more than 14 days per year with average temperature below 32 ^{o}F and 10 ^{o}F for other locations.

- **Installation Temperature and Design Movements**

3.12.2.2.1 Elastomeric Bearings

The design installation temperature for bridges with elastomeric pads and bearings shall be defined by

$$T_{Install} = T_{MinDesign} + 0.65 * (T_{MaxDesign} - T_{MinDesign}) \quad \text{(Eq. 3.12.2.2.1.1)}$$

and the design movement for the elastomeric bearing, Δ, shall be

$$\Delta = \pm \alpha L\ 0.65 * (T_{MaxDesign} - T_{MinDesign})\ . \quad \text{(Eq. 3. 12.2.2.1.2)}$$

where L is the expansion length and α is the coefficient of thermal expansion. The minimum gap, Δ_g, to avoid hard contact between the steel girders and other girders, piers, abutments or major structural members shall be

$$\Delta_g = \alpha L\ 0.9 * (T_{MaxDesign} - T_{MinDesign})\ . \quad \text{(Eq 3. 12.2.2.1.3)}$$

Commentary: Elastomeric bearings cannot be set in an offset position as can most other bearing systems. On the other hand, elastomeric bearings are very forgiving of infrequent deformations *which exceed their nominal deformation capacity. Other bearing systems do not possess this forgiving nature or the reserve deformational capacity. This provision is based upon statistical evaluation of the time and temperature dependent movements of bridge girders, and it is designed to permit direct installation of the bridge girders without repositioning the bearings or girders after the initial installation and without resorting to an offset chart.*

3.12.2.2.2 Other Bearings

Mechanical bearings, PTFE sliding surfaces, and bearings which may be offset during erection shall have a design installation temperature of

$$T_{Install} = T_{MinDesign} + 0.5 * (T_{MaxDesign} - T_{MinDesign}) \quad \text{(Eq. 3.12.2.2.2.1)}$$

and the design movement for the bearing, Δ, and the minimum gap, Δ_g, to avoid hard contact between the steel girders and other girders, piers, abutments or major structural members shall be

$$\Delta = \pm \alpha L (T_{MaxDesign} - T_{Install} + 20^{o}F) . \quad \text{(Eq. 3. 12.2.2.2.2)}$$

where L is the expansion length and α is the coefficient of thermal expansion.

Commentary: An offset chart for erection of these girders and alignment of the bearings is required with this design method.

3.12.2.2.3 Expansion Joints

Expansion joint shall have a design installation temperature of

$$T_{Install} = \frac{T_{MaxAir} + T_{MinAir}}{2} \quad \text{(Eq. 3.12.2.2.3.1)}$$

where T_{MaxAir} is the maximum daily air temperature for the previous day, and T_{MinAir} is the minimum night time air temperature for the morning of the day that the formwork for the joint gap is installed. The total movement, Δ, shall be

$$\text{Total Movement } \Delta = \alpha L (T_{MaxDesign} - T_{MinDesign} + 30^{o}F) \quad \text{(Eq. 3. .12.2.2.3.2)}$$

where L is the expansion length and α is the coefficient of thermal expansion. The placement of the joint shall be then accommodate a temperature rise and fall as defined by the maximum and minimum design temperatures and the installation temperature.

Commentary: The installation temperature and the total movement gap for expansion joints depend upon the formwork used for placement of the bridge deck and the approach slab. Therefore, the installation temperature is based upon the average of the high and low temperature on the day preceeding installation of the formwork for the joint. This still leaves a *slight uncertainty in the actual joint placement. However, the uncertainty is limited by the daily temperature variation, and so the $30^{o}F$ margin is used to overcome that uncertainty.*

DIVISION II
CONSTRUCTION

X.XX INSTALLATION OF BRIDGE GIRDERS AND BEARINGS

Bridge girders that are designed by the provisions of 3.12.2.2.1 shall be installed to satisfy the installation temperature criteria outlined here. The air temperature at the time of placement of the girders on the elastomeric bearings shall be recorded and noted as the true installation temperature, $T_{TrueInstall}$. $T_{TrueInstall}$ for concrete bridges may be taken as the average of the daytime high air temperature and the previous night low air temperature for the day of installation.

X.XX.1 Bridges with Elastomeric Bearings

For bridges with elastomeric bearings and movements designed by 3.12.2.2.1, if

$$T_{TrueInstall} > T_{MinDesign} + 0.9 * (T_{MaxDesign} - T_{MinDesign})$$

or if

$$T_{TrueInstall} < T_{MinDesign} + 0.2 * (T_{MaxDesign} - T_{MinDesign}),$$

the girders shall be relifted and the elastomeric bearings allowed to relax to there neutral position at a later date. At the time of the relifting the air temperature, T_{Air}, shall satisfy the requirement that

$$0.4 * (T_{MaxDesign} - T_{MinDesign}) < \{T_{Air} - T_{MinDesign}\} < 0.7 * (T_{MaxDesign} - T_{MinDesign}) .$$

Commentary: The provision looks somewhat difficult but it is quite easily met. For girders installed on very hot summer days, relifting of the girders will be avoided if the girders are placed only in the early morning. The low temperature requirement is more easily met, because construction is unlikely to proceed during the coldest days at most locations, and the very low temperatures will occur only during the early morning during hours of darkness. Concrete bridges are unlikely to be affected by this provision, because $T_{TrueInstall}$ is dependent upon the average daily air temperature.

X.XX.2 Bridges with Other Bearing Systems

For bridges with other bearings and movements designed by 3.16.2.3, the bearing shall be offset by an amount

$$\text{Increment } \Delta = \alpha L \, 5^{o}F$$

for each $5^{o}F$ variation of the true installation temperature from the design installation temperature.

Commentary: This is attempting to do what has always been done with mechanical bearings, PTFE sliding surfaces, and other bearing types. An offset chart is provided by the bridge engineer, and the erector offsets the bearing to account for variation between the true installation temperature and the design installation temperature.

Appendix E

Summary Information on Bridges Included in the AASHTO T2 Committee Bridge Temperature Data

Figure E-1. General Information from the Test Data

Identifier	State	Bridge Location	Bridge Type	Information
CO-1	Colorado	Bear Creek	Steel girder with concrete deck and asphalt overlay	Hot and cold temperatures are not extreme
CO-2	Colorado	Eastbound Walnut Viaduct Ramp D	Prestressed concrete girder	Hot and cold temperatures are not extreme
CO -3	Colorado	SH 470 Ramp C	Steel box girder w/asphalt overlay	Hot and cold temperatures are not extreme
CO-4	Colorado	I 25 Ramp E	Post-tensioned concrete box girder w/ asphalt overlay	Hot and cold temperatures are not extreme
IL-1	Illinois	FAI 408 Illinois River	Segmental concrete box w/ asphalt overlay	Not too cold or hot days.
IL-2	Illinois	Il 54 Sangamon River	Steel girder with concrete deck	Pretty cold winter day but only warm summer day
IL-3	Illinois	Il 29 Sangamon River	Steel girder with concrete deck	Pretty cold winter day but only warm summer day
IL-4	Illinois	Il 48 S. Fork Sangamon River	Prestressed concrete girder	Cold day but getting warmer at the time of winter measurements and warm spring day.

IL-5	Illinois	TR 94 FAI 72	Steel box girder w/concrete girder and asphalt overlay	Fairly cold winter day but not too hot summer day
IL-6	Illinois	FAI 155 N.B Sugar Creek	Prestressed concrete girder	Pretty cold winter day but not too hot summer day
IA-1	Iowa	SR 175 over DesMoines River	Steel girder with concrete deck	Winter Data Only
IA-2	Iowa	SR 141 over Little Beaver Creek	Prestressed concrete girder	Winter Data Only
KY1	Kentucky	Highland Ave over I 471	Steel girders over arch	Winter data about Freezing and summer day not too hot
KY-2	Kentucky	Railway Crossing Ky 14 Boone County	Prestressed concrete girder	Winter data about Freezing and summer day not too hot
Not Used	Maine	Fish River Bridge in Eagle Lake	Steel girder with concrete deck and asphalt overlay	Winter Data Only
ME-1	Maine	Presque Isle Stream Bridge	Steel girder with concrete deck	Winter Data Only
ME-2	Maine	Second St Bridge in Millinocket	Steel girder with concrete deck	Winter Data Only
ME-3	Maine	Bulls Eye Bridge in Penobscot	Steel girder with concrete deck and asphalt overlay	Winter Data Only
ME-4	Maine	Route 6 and 15 over Kennbec River	Steel girder with concrete deck and asphalt overlay	Winter Data Only

ME-5	Maine	Saco River Bridge	Steel girder with concrete deck and asphalt overlay	Winter Data Only
MO-1	Missouri	Truman Blvd over US 50	Steel girder with concrete deck	Winter Data Only
MO-2	Missouri	SR 94 over Auxvasse Creek	Steel girder with concrete deck and asphalt overlay	Winter Data Only
MO-3	Missouri	US 54 over Richland Creek	Steel girder with concrete deck and asphalt overlay	Winter Data Only
MO-4	Missouri	US 54 over Hillers Creek	Prestressed concrete girder	Winter Data Only
MO-5	Missouri	Roue D over Route 50	Prestressed concrete girder	Winter Data Only
MT-1	Montana	SR 69 over I15	Steel girder with concrete deck	
MT-2	Montana	SR 518 over I15	Prestressed concrete girder	
NJ-1	New Jersey	I 195 ov er Watsons Creek	Prestressed Concrete Girder	Winter Data Only
NJ-2	New Jersey	I 195 under access road Hamilton and Bordenton	Steel girder with concrete deck	Winter Data Only
NC-1	North Carolina	I 440 over Raleigh Blvd	Prestressed concrete girder	
NC-2	North Carolina	Poole Road over I440	Steel girder with concrete deck and asphalt overlay	
PA-1	Pennsylvania	SR 62 over Allegheny River	Steel girder with concrete deck	
PA-2	Pennsylvania	13th Street over French Creek	Steel girder with concrete deck	

PA-3	Pennsylvania	US 6 over Allegheny River	Prestressed concrete box girder	
PA-4	Pennsylvania	SR 1009 over I99	Concrete box beam	
PA-5	Pennsylvania	SR 1013 over Little Juanita	Presstressed concrete girder	
Not Used	Pennsylvania	PA 59 over Kinzua Creek	Steel Truss Bridge	Drawings are quite old and some critical data was not legible. Bridge was not used.
Not Used	Pennsylvania	PA 453 over Railroad and Juanita River	Continuous riveted steel plate girder bridge	Drawings are quite old and there was uncertainty as to some aspects of thedesign. Not included in the data.
Not Used	Pennsylvania	SR 3009 over Blair Run Gap	Two Span concrete box girder bridge	This bridge was not used because of ambiguity in data and support conditions
Not Used	Pennsylvania	SR 2011 over Clover Creek	Standard Steel Rolled Girder Bridge	Older bridge and design drawings were not clear. Not Used.
Not Used	Pennsylvania	SR 4004 over Burgoon Run	Standard Steel Rolled Girder Bridge	Older bridge and design drawings were not clear. Not Used.

Figure E-2. Summertime Data

Bridge Identifier	Summertime Data												
	Date	Morning Data						Afternoon Data					
		Shade Temp °F	Temp Top of Deck °F	Temp Bottom of Deck °F	Beam Temp °F	Weather	Joint Opening	Shade Temp °F	Temp Top of Deck °F	Temp Bottom of Deck °F	Beam Temp °F	Weather	Joint Opening
CO-1	8/2/00	82.5	86.3	80.1	81.4	Hazy	1.62	87.7	92.8	85.1	87	Cloudy	1.56
CO-2	8/2/00	83.5	99.8	82.3	85.4	Hazy	2.06	88	101.6	85.4	85.8	Cloudy	1.81
CO -3	8/2/00	79.3	90.5	78.1	79.9	Hazy	25.9	86.5	110.8	84.3	89.1	Cloudy Hazy	24.88
CO-4	8/2/00	84.5	97.9	81.5	84.7	Hazy	5.06	88.9	98.8	85	89.4	Cloudy	5
IL-1	8/29/00	74	75	81	79	Foggy Overcast	34.5	88	103	86	86	Hot Sunny	34.62
IL-2	5/31/00	73	72	71	70	Cloudy	3.31	84	99	82	79	Sunny	2.88
IL-3	5/31/00	75	74	71	70	Partly Sunny	2.25	84	99	86	82	Sunny	1.75
IL-4	5/31/00	70	72	70	72	Cloudy	3.0	84	97	82	76	Partly Sunny	2.88
IL-5	8/29/00	75	77	77	76	Foggy Overcast	2.44	86	102	88	88	Hot Sunny	2.31

IL-6	8/30/00	62	61	64	67	Clear Sunny	3.56	85	103	84	77	Clear Sunny	3.25
IA-1	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data
IA-2	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data
KY-1	8/15/00	70	76	76	76.5	Overcast	2.5	90	109	85	90	Partly Cloudy	2.5
KY-2	8/15/00	73	78	80	79	Partly Cloudy	4.5	90	105	94	86	--	4.75
Not Used	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data
ME-1	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data
ME-2	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data
ME-3	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data
ME-4	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data
ME-5	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data
MO-1	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data

MO-2	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data
MO-3	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data
MO-4	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data
MO-5	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data	No summer data
MT-1	5/2/00	52	55.3	53.8	51.6	Cloudy	3.66	64	74.5	62.3	61.5	Partly Cloudy	3.54
MT-2	5/2/00	58.1	58.9	57.1	57.6	Cloudy	5.73	65.3	75.1	65.5	61.4	Partly Cloudy	5.65
NJ-1	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data
NJ-2	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data	2nd set of winter data
NC-1	9/12/00	No data	No data	No data	No data	No data	No data	87	108	93	85	Clear	2.38
NC-2	9/12/00	No data	No data	No data	No data	No data	No data	89	120	91	87	Clear	1.62
PA-1	6/16/00	72	73	76	74	Overcast	4.06	83	92	81	82	Cloudy	3.88
PA-2	8/9/00	75.8	77.1	76.1	75.5	Overcast	2	82.4	88.3	83.8	82.7	Sunny	2.19
PA-3	8/25/00	58.7	63.3	63.1	61.6	Fog	1.12	74.6	91.9	76.5	71.8	Sunny	1.0

PA-4	9/1/00	68	71	NA	70	Overcast	0.88	77	87	NA	73	Cloudy	0.81
PA-5	9/1/00	70	72	69	69	Overcast	1.44	74	87	73	70	Cloudy	1.38
Not Used	8/25/00	60.4	68.5	61.2	60.1	Fog	3.12	68.3	84.7	71.1	67.6	Sunny	3.06
Not Used	9/1/00	71	73	NA	69	Overcast	4.25	76	86	NA	73	Cloudy	4.12
Not Used	9/1/00	69	67	67	66	Overcast	0.38	70	71	69	68	Cloudy	0.38
Not Used	9/1/00	70	71	68	68	Overcast	0.62	74	81	72	70	Cloudy	0.62
Not Used	9/1/00	69	70	69	68	Overcast	0.62	75	84	72	71	Cloudy	0.62

Figure E-3. Wintertime Data

Bridge Identifier	Wintertime Data												
	Date	Morning Data						Afternoon Data					
		Shade Temp °F	Temp Top of Deck °F	Temp Bottom of Deck °F	Beam Temp °F	Weather	Joint Opening	Shade Temp °F	Temp Top of Deck °F	Temp Bottom of Deck °F	Beam Temp °F	Weather	Joint Opening
CO-1	3/9/00	38.5	38.7	39.8	38.4	Partly Cloudy	2	40.9	44.5	42.8	42.5	Cloudy Light Snow	2
CO-2	3/9/00	40.5	45.8	41.1	40.8	Partly Cloudy	2.88	43.1	42.1	41.8	41.5	Cloudy Light Snow	2.94
CO -3	3/9/00	45.8	45.8	37.9	36.5	Partly Cloudy	26.44	47.6	46.5	38.1	36.3	Cloudy Light Snow	26.31
CO-4	3/9/00	41.5	44.5	39.1	38.8	Partly Cloudy	6.5	41.6	44.8	40.5	40.1	Cloudy Light Snow	6.38
IL-1	3/17/00	31	39	42	43	Clear Sunny	36.88	48	64	47	47	Clear Sunny	36.47
IL-2	1/27/00	11	10	12	11	Clear Sunny	5.25	27	38	26	29	Clear Sunny	4.62
IL-3	1/27/00	16	15	17	16	Clear Sunny	4.25	32	45	35	33	Clear Sunny	3.62

IL-4	1/27/00	21	21	17	19	Clear Sunny	3.69	27	38	33	26	Clear Sunny	3.44
IL-5	3/17/00	30	33	34	33	Clear Sunny	2.81	47	60	49	47	Clear Sunny	2.75
IL-6	3/17/00	18	22	22	22	Cloudy	4.31	19	29	22	22	Cloudy	4.25
IA-1	12/13/00	7.3	6.6	12.3	12.3	Calm Light Snow	4.42	15.3	14.3	12.3	15.7	Calm Snow	4.25
IA-2	12/13/00	4.1	3.3	8.4	6.2	Cloudy Windy	2.12	12.2	13.1	10.9	9.4	Cloudy Light Snow	2.21
KY-1	3/17/00 2/23/00	31.2	37.2	39.4	34.6	Cloudy	3.0	65.6	74.4	62	65.1	Partly Cloudy	2.75
KY-2	3/17/00	30.8	35.5	37.5	39.6	Partly Cloudy	5.19	39.4	59.8	48.3	47.4	--	5.06
Not Used	1/28/01	22.8	20.5	--	--	Snowing	2.11	24.5	26.2	--	--	Sunny	2.05
ME-1	1/28/01	20.3	16.9	17.7	19.4	Overcast	2.38	27.5	30.4	24.5	27.5	Sunny	2.32
ME-2	1/27 & 1/29/01	21.5	14.7	20.3	14.3	Sunny	1.15	33.1	25.5	27	20.5	Overcast	1.11
ME-3	2/4/01	1.4	15.5	18.2	7.9	Sunny	2.76	30.3	27	24.2	32.4	Partly Cloudy	2.64
ME-4	2/20 & 2/21/01	27	37.8	31.4	33.6	Windy	1.46	53	40.1	38.3	47.9	Sunny	1.42

ME-5	1/5/01	14.3	18.1	17.1	14.4	Partly Cloudy	3.04	32.1	30.4	22.5	28.9	Partly Cloudy	2.95
MO-1	2/6/01	41	40	42	43	Sunny	1.5	50	66	52	52	Partly Sunny	1.44
MO-2	2/6/01	31	31	33	30	Mostly Sunny	2.62	51	46	45	47	Sunny	2.44
MO-3	2/6/01	36	34	40	40	Sunny	0.88	44	48	42	44	Sunny	0.88
MO-4	2/6/01	37	34	37	38	Sunny	1.75	46	58	43	44	Sunny	1.56
MO-5	2/6/01	41	40	46	45	Sunny	2.31	49	58	45	46	Partly Sunny	2.25
MT-1	2/11/00	17.5	17.3	17.4	18.2	Cloudy	4.09	33.3	38.8	31.5	32.2	Partly Cloudy	3.94
MT-2	2/11/00	15	14.5	16.1	17.3	Cloudy	6.60	28.5	30	28.4	24.4	Partly Cloudy	6.43
NJ-1	1/31/01	35.6	34.2	34.4	35.1	Partly Cloudy	2.12	47	53.3	44.2	43.8	Partly Cloudy	2.62
NJ-2	1/31/01	33.4	34.9	35.4	34.7	Partly Cloudy	3.0	49.3	49.2	48.2	49.8	Partly Cloudy	2.88
NC-1	2/21/00	31	31	44	46	Clear	2.75	No data	No data	No data	No data	No data	No data
NC-2	2/21/00	31	30	40	32	Clear	2.25	No data	No data	No data	No data	No data	No data
PA-1	1/29/01	16.5	17	18.9	16	Overcast	5.94	34	34.9	33.2	34.7	Overcast	5.56

PA-2	1/29/01	15.9	22.3	28.4	21.6	Overcast	3.06	34.1	34.4	34.9	35.9	Drizzle	2.88
PA-3	2/22/01	12	16	14	16	Overcast	1.50	18	21.6	19.9	19.2	Flurries	1.50
PA-4	3/29/00	43	43	NA	43	Cloudy Rain	1.12	49	49	NA	44	Cloudy	1.12
PA-5	3/29/00	45	43	43	45	Cloudy Rain	3.38	48	50	45	45	Cloudy	3.38
Not Used	2/22/01	11	14	13.6	12.4	Overcast	4.12	18	26	21	19	Flurries	3.88
Not Used	3/29/00	45	44	43	42	Cloudy	3.75	49	50	45	46	Cloudy	3.69
Not Used	3/29/00	42	38	41	39	Cloudy Rain	0.38	49	47	43	43	Mostly Cloudy	0.38
Not Used	3/29/00	44	39	40	41	Cloudy Rain	0.62	46	45	44	44	Mostly Cloudy	0.62
Not Used	3/29/00	43	39	41	39	Cloudy Rain	0.62	45	42	43	42	Mostly Cloudy	0.62