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| 16. Abstract <p>This report is the fifth out of six reports prepared for Research Project 0-1778 "TxDOT Rigid Pavement Database," conducted by the Center for Transportation Research (CTR) of the University of Texas at Austin and funded by the Texas Department of Transportation (TxDOT). Project 0-1778 documents the efforts conducted for the evaluation of hundreds of pavement test sections that rationally represent the concrete pavement network in Texas. Thousands of condition surveys have been conducted during the last thirty years, and all the information that has been collected has contributed to a better understanding of materials and their performance. In addition, the pavement database has technically contributed to the development of other research studies that have focused on evaluating different variables that affect pavement performance, such as aggregate type and placement season of concrete. This report presents examples of data analyses that can be conducted with the use of the information in the database. It describes various tasks that have been carried out to interpret the information contained in the database.</p> | | | | | |
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ANALYSES PERFORMED USING THE RIGID PAVEMENT DATABASE IN TEXAS

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1. Introduction

1.1 Background

Research Project 0-1778 “TxDOT Rigid Pavement Database” is distinguished from other research projects by its continuous effort to collect data on pavements in Texas. This endeavor has provided benefits and technical contributions to pavement research projects in Texas and other states. The continuity of this project guarantees periodic assessment of the performance of various pavements. Field collection tasks are regularly conducted to keep the data up to date. Likewise, and in conjunction with other research projects, additional tasks are conducted to improve the quality of the information stored in the database. An extension of the project granted for fiscal year (FY) 2004 allowed conducting a number of activities and analyses with the use of the data contained in the rigid pavement database (RPDB). The analyses presented throughout this report were prepared in agreement with the project director and the researchers in order to demonstrate the worth of this project and the importance of its continued funding.

1.2 Objective

The primary objective of this report was to present examples of data analysis that can be performed with the use of the information contained in the database. To accomplish this goal, special care has been taken to recognize the needs of other research studies, in Texas and throughout the United States, for data about pavements. The analyses presented in this report include a methodology for design purposes to predict the minimum concrete pavement temperature with the use of one or more climatic variables, a newly developed spalling criterion model for pavement overlay, and the significance of deflection for the selection of rehabilitation strategies. Additional objectives of this work include reporting the regular activities conducted for the project and recommending actions that will improve the overall quality of the RPDB.

1.3 Methodology

This report contains seven chapters that discuss different topics and tasks developed for the RPDB project. In order to present the information in an organized manner, the contents of the chapters in the report are summarized as follows.

Chapter 1 contains basic information about the project and states the objectives of the report according to the project objectives scheduled for the reporting period. Likewise, the methodology with which research tasks were conducted is provided, along with a brief description of the contents of all the chapters in the report.

Chapter 2 introduces the concepts of minimum concrete pavement temperature, distress prediction models and distress index, and pavement deflection. Section 2.1 describes how concrete temperature affects the performance of rigid pavements at the short and long terms, and an explanation of the initial and maximum temperature differential referenced to the setting concrete temperature is provided. Section 2.2 describes the evolution of the distress prediction model developed at the Center for Transportation Research (CTR) of the University of Texas at Austin. Previous and current considerations for the evaluation of the conditions of the pavements are discussed. In Section 2.3, pavement deflection is discussed with a focus on the relationship between pavement thickness and deflection at different locations along the pavement structure.

Chapter 3 presents the findings from pavement instrumentations performed during the summer of 2002. Research Report 0-1778-4 (Ref 1) documented the steps undertaken to install three sets of concrete temperature reading devices, or i-Buttons, in certain pavement sections in North Texas. In Chapter 3 of the present report, the activities performed to collect the temperature data are summarized, and the results obtained from the downloading process are shown. Likewise, two models based on available climatic data, which were derived to predict the minimum concrete temperature, are described. These two models were used to predict pavement temperature, and the results were compared with the actual temperature data recorded by i-Buttons in Amarillo. Finally, the possibility of using these models to predict the minimum concrete pavement temperature for various

regions in Texas is discussed. The ultimate goal of concrete temperature research would be to determine regional minimum concrete temperatures for use as input values in the design of new concrete pavements.

In an extension of the concept discussed in Chapter 2, in Chapter 4 an attempt is made to derive a spalling model for overlaying purposes. As has been described in Chapter 2, the distress prediction model in current use has limited application. The current distress model for nonoverlaid sections includes the effects of such distresses as punchouts and patches, but it does not consider the detriment caused by spalled cracks. With the use of the distress data, including spalling, stored in the database, a new and improved distress model is proposed to evaluate the condition of concrete pavements.

Chapter 5 focuses on the analysis of the current pavement deflection data stored in the database. An analysis of deflections for continuously reinforced concrete pavement (CRCP) sections, measured in 1988 in twelve districts, aids in the determination of whether the structural support for CRCP was good or poor for any given district. The analysis included 23,972 data points taken at 5,909 locations (at cracks and midspans between cracks) on overlaid and nonoverlaid sections. CRCPs were, for the most part, 8 in. thick, although some were 9 and 10 in. thick. Finally, a similar approach is recommended for application to more recent data, which will allow a further step in evaluating CRCP performance before and after overlay.

Because the analyses that can be conducted with the use of data from the RPDB are countless, Chapter 6 presents a few examples of the evaluations that can be conducted. Previous research projects have focused on particular evaluations, such as effect of aggregate type on crack spacing and distress occurrences or the effect of reinforcement steel content and coefficient of thermal expansion (CTE) of the aggregate on the performance of CRCPs. In this chapter, the analyses include the issues described in Chapter 3 about the prediction of concrete temperature, those in Chapter 4 about a distress model that incorporates spalling, and those in Chapter 5 about a deflection decision criterion for overlay.

Chapter 7 presents the conclusions of the work performed and the analyses conducted with the use of the data contained in the RPDB. Likewise, there are some recommendations for future tasks related to this project.

2. Design Models and Criteria

The structural design of concrete pavements depends on various factors such as the foundation soil and sub-base material qualities, traffic volume and loads, and concrete characteristics. This chapter focuses on the assessment of the effect of temperature on the latter variable—the concrete. The discussion emphasizes the significance of minimum concrete temperature variation at the short and long terms. Likewise, there is a discussion about the evolution of the distress index model developed at the Center for Transportation Research (CTR) of the University of Texas at Austin. Finally, an introduction to the relationship between pavement thickness and deflection provides the elements for further analysis in Chapter 5. Note that this chapter serves as an introductory tool for the topics presented in the following three chapters.

2.1 Minimum Concrete Pavement Temperature

Environmental conditions have a tremendous effect on the performance of concrete pavements and, therefore, have to be considered in the design. Among those climatic variables that affect pavements, probably air and concrete temperatures are the most critical. Air temperature exerts a direct effect on concrete temperature and, along with relative humidity and wind speed, affects the evaporation rate of fresh concrete. The cyclic variation of concrete temperature and its effects on the design and performance of pavements is still subject to investigation. Current design methods for concrete pavements usually consider the temperature drop effect in the concrete in the design of the longitudinal steel reinforcement. This temperature drop is calculated as the difference between the average concrete curing temperature and a minimum design temperature (Ref 2). The minimum design temperature is defined as the average daily low temperature for the coldest month of the year(s) and can be easily obtained from the U.S. National Climatic Data Center (NCDC), which maintains a weather database accessible through the Internet (Ref 3).

2.1.1 Background on Previous Research

Studies conducted during the late 1980s at the CTR provided useful guidelines for the development of the CRCP-4 program's analyses. At that time, Version 4 of the program required the input of certain environmental variables that affect CRCP. Those variables included curing temperature, minimum temperature expected after concrete gained full strength, the number of days after the concrete set before the minimum temperature occurred, and the minimum daily temperature. Research Project 249 (Ref 4) documented prevailing climatic conditions for Texas, and the results from this study were later used for the establishment of geographical regions in Texas that would allow the determination of the minimum temperature expected after concrete placement and setting. The establishment of these geographical regions also allowed the identification of a lower temperature bound that a particular region could experience one year after construction (Ref 5). Table 2.1 presents the results obtained from the research performed during the 1980s. The table lists the three zones into which the state was divided, the expected range of the minimum daily temperature, and the temperature of the coldest day of the year, which were used for design purposes.

Table 2.1 Minimum ambient temperatures expected in Texas

| Zone | Region | Range of Minimum Daily Temperature (°F) | Temperature of the Coldest Day of the Year (°F) |
|-------------|--------------------|--|--|
| I | Gulf Coast/Valley | 29-20 | 25 |
| II | East/South Central | 19-10 | 15 |
| III | North/West Texas | 9-0 | 5 |

Figure 2.1 displays a map with contours delimiting the three zones listed in Table 2.1. The climatic data obtained at Brownsville (Zone I), Port Arthur (Zone II), and Amarillo (Zone III) were used to represent the prevailing conditions in each zone in the map.

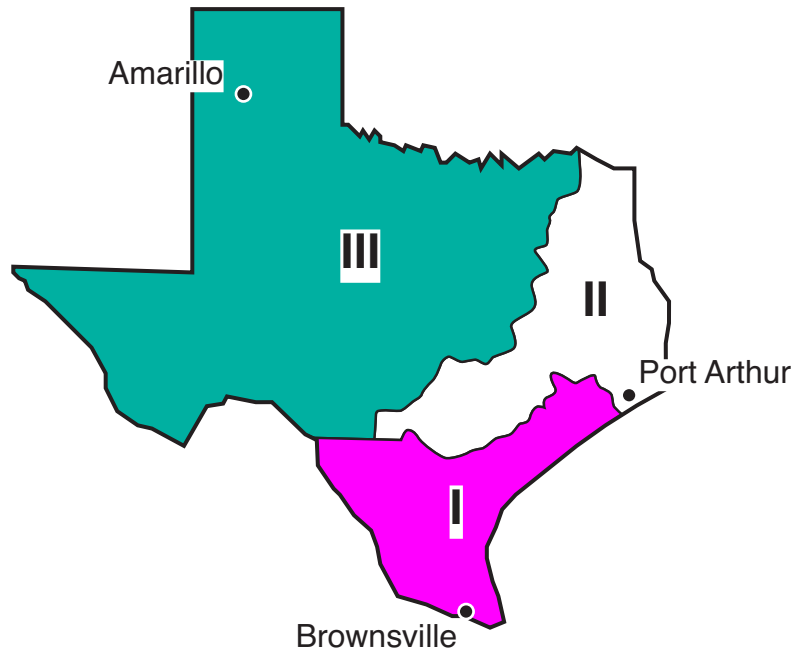


Figure 2.1 Defined climatic regions and representative cities

2.1.2 Temperature Drop

As ambient temperature drops, the temperature of fresh concrete also drops, causing the CRCP slab to move. This fluctuation in temperature has an effect on the strains and stresses of both concrete and reinforcement steel. The strains developed and their associated stresses are forces that contribute to the development of CRCP cracking. Mathematically, these strains are expressed by Equations 2.1 and 2.2, as follows:

$$\varepsilon_c = \alpha_c \Delta T \quad (2.1)$$

and

$$\varepsilon_s = \alpha_s \Delta T, \quad (2.2)$$

where

ϵ_c = concrete strain

ϵ_s = steel strain

α_c = concrete CTE

α_s = steel CTE

ΔT = temperature drop below placement temperature.

From Equations 2.1 and 2.2 and from the direct relationship that exists between stress and strain, the stress might be expressed in terms of the CTE in a more general form, as is shown in Equation 2.3:

$$\sigma_s = f(\Delta T \dots). \quad (2.3)$$

Equation 2.3 is a simple representation of the relationship that exists between concrete temperature and steel stress in CRCs. In this equation, it is implied that anything that causes a differential of temperature in the concrete causes the increase or decrease of the stresses in the steel and, thus, the amount of reinforcement required. Furthermore, this equation is generally applied for the steel design in CRCP; however, its application might need to be revisited, as will be described in the following section.

2.1.3 Application and Implementation

The design of CRCP requires the input of climatic variables to determine the amount of longitudinal reinforcement steel required to keep concrete crack spacing within acceptable limits and crack width as narrow as possible to maintain sufficient crack interlock (acceptable load transfer). As was previously mentioned, the design of CRCP requires the input of the curing temperature, minimum temperature expected after concrete gains full strength, the number of days after the concrete is set before the minimum temperature occurs, and the minimum daily temperature. Figure 2.2 displays the cyclic behavior of concrete temperature for the short and long terms. The first sinusoidal curve shows that for

normal, not extreme, weather conditions, the concrete setting temperature occurs some time after placement and depends on various factors, including ambient temperature, mixing time, mixed concrete temperature, rate of cement hydration, and so forth. Therefore, the setting temperature reaches a higher value than the temperature of the concrete when it was placed. The differential of temperature between placement and setting times is defined by Equation 2.4, and its value has a tremendous effect on the early age of the pavement.

$$\Delta T_{initial} = T_{set} - T_{placement} \quad (2.4)$$

In general, a very high $\Delta T_{initial}$ value will cause the development of early cracks in the concrete, which in turn might develop spalling, delamination, or other distress. Current construction practices recommend limiting this temperature differential for the first seventy-two hours after placement, weather permitting.

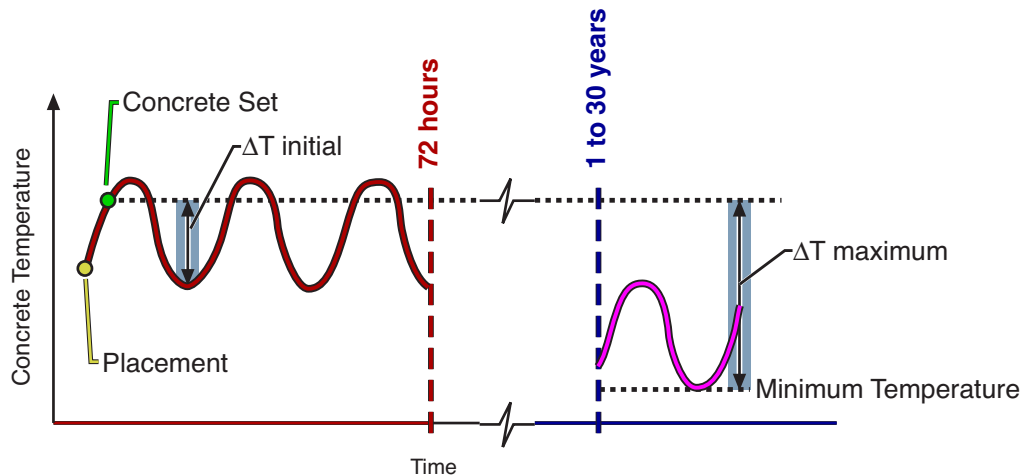


Figure 2.2 Variation of concrete temperature for short and long terms

With regard to the long term, Figure 2.2 shows that at some point in time, the pavement will experience a temperature drop that will be the maximum ever. What remains uncertain is when the maximum temperature drop after placement will happen; it might be one year or thirty years after construction. This event is defined by Equation 2.5 and is

graphically shown in Figure 2.2. The $\Delta T_{maximum}$ is estimated by subtracting the minimum concrete temperature experienced ever from the concrete setting temperature.

$$\Delta T_{maximum} = T_{set} - T_{minimum}. \quad (2.5)$$

The minimum concrete temperature shown in Equation 2.5 corresponds to one of the input variables for CRCP design, which determines the steel content required for the pavement. As was explained for Equation 2.3, the temperature differential is directly proportional to the steel stress. This means that, to withstand the stresses developed, more steel is required for higher values of $\Delta T_{maximum}$, making the pavement more costly.

Air Temperature Versus Concrete Temperature

When a CRCP is designed in the real world, the actual concrete temperature is unknown; thus, a surrogate input variable for the minimum concrete temperature is the minimum ambient temperature recorded for the coldest month of the year(s). Weather databases provide abundant information for ambient temperature and other variables; however, the real concrete temperature differs from the one used as surrogate. It is certain that if real, precise concrete temperatures were used in design, more realistic conditions would be simulated and more economic pavement designs would be achieved. Even though actual concrete temperature is not available for design, a temperature value might be obtained from experimentation that is at least closer to reality than the one used as surrogate. In general, a more realistic value of the minimum concrete temperature is provided by Equation 2.6. The minimum concrete temperature (*Min concrete temp*) is equal to the minimum ambient temperature (*Tminimum*) at the construction site, modified by a *K* factor:

$$\text{Min concrete temp} = K (T_{minimum}). \quad (2.6)$$

The simplicity of Equation 2.6 does not imply that *K* is a single factor; instead, it could be very complex, depending on the variables that it could represent. This factor is actually a function of different variables, including CRCP thickness (*D*), the CTE of the concrete (α), and so forth. In the course of Project 1778, fieldwork has been conducted that has resulted

in a good estimation of some of these variables and how much they contribute to the real concrete temperature. Chapters 3 and 6, respectively, detail the work performed for the development of this study and an application of the findings to data collected in Amarillo.

Chapter 3 focuses on the presentation of temperature data collected in three particular locations in North Texas. The data include concrete temperature at three depths plus ambient temperature data retrieved from the NCDC website. Figure 2.3 illustrates how these two temperature values relate to each other in the medium and long terms. In general, it is observed that actual concrete temperature is different from ambient temperature (design input), which has an effect on the design of CRCP.

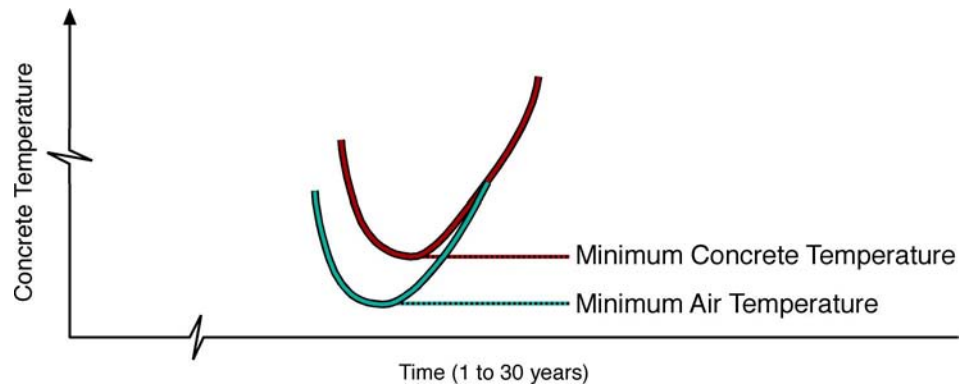


Figure 2.3 Minimum ambient and concrete temperatures for the medium and long terms

Finally, in Chapter 6 an algorithm is presented that employs some of the data collected in Amarillo, and a feasible approach is provided with which to estimate concrete temperature given ambient temperature and other parameters, so that a more rational minimum temperature value can be used for design.

2.2 Distress Index and Decision Criteria Index

The literature offers a variety of methods to evaluate the conditions of pavements by subjective and objective ways. Although subjective approaches help in the identification of problems and needs, objective approaches allow the evaluation of pavements in such a way that maintenance and rehabilitation tasks could be performed promptly, as they are

required. Among the objective evaluation methods, the most commonly used are regression analysis, factor analysis, and discriminant analysis. Revisions and applications of all the methods have shown that discriminant analysis is probably the best of the three because of the type of data that are analyzed and the obtained results (Ref 6). Before getting into the details and findings, a distinction among distress, distress index, and decision criteria index is required.

A *distress* is the visible deterioration of a pavement that is subject to traffic loads, environmental effects, and other external or internal factors that in the end cause physical damage to the structure. Depending on the type of pavement, different distresses have been identified. Common distresses in all type of pavements include patches, crack and joint spalls, punchouts, and so forth. In TxDOT's Rater's Manual (Ref 7), distresses commonly found in all types of pavements are described.

Distress index (DI) is the combination of various distress manifestations represented by a single number that rates the condition of the pavement. A simple mathematical form of the DI is represented in Equation 2.7. This is the basic equation used for the estimation of the DI, and it has been applied by various transportation agencies in the evaluation of pavements.

$$DI = A_0 + \sum_i^n \frac{m_i}{M_i} \quad (2.7)$$

where

A_0 = constant

n = number of distress types

m_i = amount of distress manifestation i

M_i = terminal condition of a pavement section if the distress type is an isolated occurrence.

Decision criteria index (DCI) is the combination of all the distresses found in a pavement section into a number that indicates its failure condition associated with age and traffic. DCI is differentiated from the DI because the first indicates whether a pavement has reached its terminal condition. Theoretically, the DCI should include riding quality, safety, and economics, but in general only distress effects are considered.

2.2.1 Previous Developments

Some of the models developed to date for the estimation of DI and DCI involve subjective preferences and decisions; other models involve some type of correlation procedure. All these models can be found in the literature (Ref 6). This section presents only the most relevant developments that have been applied in Texas. Those developments have evolved in such a way that different distresses have been evaluated at different times and were added to the models as required.

Among the objective evaluation methods, the ones that yielded the more encouraging results were the ones involving discriminant analysis. This is a statistical technique that classifies data into groups; its objective is to provide a discriminant equation so that the elements of each group can be separated. Once the discriminant equation is defined, a new element can be assigned to one of the defined groups. Research conducted at the CTR in the early 1980s applied this technique to develop an equation to discriminate CRCPs with an acceptable level of distress from CRCPs requiring overlay (Ref 6).

Early Findings—Gutierrez de Velasco (1982)

Distress data were collected for a number of overlaid and nonoverlaid sections. With the use of discriminant analysis, the reasons leading to overlay were determined. On the basis of data from the two groups—overlaid and nonoverlaid CRCPs—an equation was developed to differentiate between the groups. The outcome of this analysis was a DCI, and its magnitude was used as a DI. The data in the analysis included distresses collected during the 1974 and 1978 condition surveys. On those occasions, several distress manifestations were recorded, including punchouts, patches, minor and severe spalling, and

pumping. Some pavements surveyed in 1974 had been overlaid when revisited in 1978, and this study led to an idea of the reasons leading to overlay.

With the use of discriminant analysis, equations were obtained for continuous and jointed pavements, and for both cases the scores can be interpreted as follows. A positive score indicates that the section is in good condition, not requiring overlay; if the score is zero or negative, the section is considered to be failed. Likewise, the larger the magnitude of the score, the better the condition of the pavement. Equation 2.8 presents the discriminant equation for CRCPs, and Equation 2.9 shows the results obtained for jointed concrete pavements (JCPs).

$$Z_c = 1.0 - 0.065FF - 0.015MS - 0.009SS \quad (2.8)$$

where

- Z_c = discriminant score for CRCP
- FF = failures per mile
- MS = percent minor spalling
- SS = percent severe spalling.

$$Z_j = 1.0 - 0.028C - 0.004S - 0.007P - 0.019F \quad (2.9)$$

where

- Z_j = discriminant score for JCP
- C = cracking, ft. per 1,000 ft.²
- S = spalling, ft. per 1,000 ft.²
- P = patching, ft. per 1,000 ft.²
- F = faulting in wheel path, in. per 1,000 ft.

In Equation 2.8, the failures per mile are normalized. The most important variable is failures per mile, followed by minor spalling and severe spalling. In Equation 2.9, a greater weight is assigned to cracking, and lower weights are assigned to spalling and patching. This means that cracking (cracked slabs) influences the decision of overlaying a pavement section more than any other distress.

Subsequent Findings—Chia-Pei Chou (1988)

As was previously mentioned, Gutierrez de Velasco's research derived DI and DCI indexes on the basis of pavement data collected during the 1974 and 1978 condition surveys. Later, in 1988, Chia-Pei Chou continued that work and modified the equation used for the estimation of Z_c for CRCPs. Chou's work updated the existing model and suggested that in addition to pavements with positive (good condition pavement) and zero or negative (failed pavement) zeta scores, there are also pavements that fall in a *zone of conflict*, which are pavements whose condition classification is uncertain within the reliability of the analysis.

In order to update the previous models, Chou performed a more comprehensive study that analyzed data collected over a time frame of ten years. Condition survey data were collected, stored, and reduced for the analysis. Again, input variables for the analysis included historical distress data, and the outcome included the discriminant function and its mathematical expression that can be used as a DI. The type of distresses considered in the original analysis included minor spalling, severe spalling, minor punchouts, severe punchouts, and patches; however, it was found that including the coefficients for both types of spalling led to misleading results. As a result, another equation was developed that did not include both types of spalling. After a series of mathematical transformations, a final model was obtained for the estimation of the DI, as is shown in Equation 2.10 (Ref 8).

$$Z'' = 1.0 - 0.0071MPUNT - 0.3978SPUNT - 0.4165PATCH \quad (2.10)$$

where

Z'' = DI or zeta score

$MPUNT$ = \ln (minor punchouts per mile +1)

$SPUNT$ = \ln (severe punchouts per mile +1)

$PATCH$ = \ln (total patches per mile +1)

The criterion for major rehabilitation of a pavement is a DI less than or equal to zero ($Z'' \leq 0$). The natural logarithm model in Equation 2.10 is an improvement over the linear model in Equation 2.8. This type of logarithmic transformation is commonly used for data that grow with age, and it results in the best fit for the data analyzed here.

2.2.2 Problems and Needs

The present serviceability index (PSI) concept, developed by Carey and Irick in 1960 and used at the AASHO road test, is the basis of all the DIs used nowadays. In Texas, data regarding pavement distresses have been collected for several years by the CTR, and a rigid pavement database is maintained with current distress information for hundreds of pavement sections. Among research studies performed with the data contained in the database, several have used the DI concept as a tool to identify pavements with good or poor performance. Initially, in 1982 Gutierrez de Velasco developed a DI model obtained from discriminant analysis. At that time, discriminant analysis yielded a model that had two main limitations: (1) It was a linear model based on the assumption that data were normally distributed for both overlaid and nonoverlaid sections; and (2) the most important variable was *failures per mile* in general. This meant that all distresses, except for spalling, had the same weight. However, for the pavement conditions at the time, the DI equation was functional.

Later, in 1988 Chia-Pei Chou improved the previous model for CRCP sections by making the data more comprehensive, which yielded better results. In fact, the development of the DI is thought to be a progressive task. According to the information contained in the database, pavements in Texas showed no major spalling through 1990, and thus it was not until the late 1980s that Chou tried to reevaluate the coefficients for minor and severe spalling. Contrary to what was expected, and as it has been previously explained, the final model for DI dropped the terms for spalling.

Finally, because spalling has occurred with more frequency in the last eight to ten years and has led to pavement failure, the DI model should be reevaluated to include condition survey data from the latest collection efforts. By analyzing more recent data, the concept of spalling could be added to Chou's equation, and a more reliable prediction model for overlay could be obtained. In Chapter 4, this work will be described and the findings will be presented.

2.3 Deflection

The deflection of a pavement is probably the best indicator of its ability to withstand loads, although conducting deflection testing is still on the expensive side of the activities performed to monitor pavements. This explains why deflection testing is commonly conducted at a network level for pavement management purposes. As for the measurement of deflections, there are various criteria. The data contained in the RPDB dates from 1988 and includes measurements at two main locations: at cracks and at midspans. Measuring deflections at these locations has helped in the assessment of load transfer capability across cracks and joints.

For CRCPs, load transfer at cracks is measured as the ability of the concrete slab to transfer traffic loads from one side of the crack to the other. When load transfer is reduced to a certain limit, punchouts start to appear. Load transfer efficiency (LTE) can be measured with the use of one of the approaches proposed in the literature, such as Teller's procedure (Ref 9). Equation 2.11 shows how LTE can be estimated across a crack or joint. In this example, if LTE equals zero, then no load is being transferred. In contrast, if LTE equals

100, then the load transfer is perfect and there is no differential displacement across the crack or joint.

$$LTE = \frac{2W_u}{W_u + W_l} \quad (2.11)$$

where

LTE = LTE, percentage

W_u = deflection on an unloaded slab

W_l = deflection on an adjacent loaded slab.

An analysis performed with the use of deflection data from the RPDB (detailed in Chapter 5) demonstrates the relevance of comparing deflection values before and after overlay conditions, indicating that deflection could be used as a criterion for overlay. Likewise, analysis of the data reveals that overlay protection might prevent the pavement from deflecting excessively and developing punchouts if the pavement is overlaid on time. Once the LTE is almost null, the overlay does not work in the same way and structural improvement might be slight. Figure 2.4 shows how two pavements with two different thicknesses (D_1 and D_2 , and $D_2 > D_1$) and subject to the same traffic load conditions (W_{18}) experience different deflections. The thicker pavement is expected to show lower deflection values than the thinner pavement. Although this is acceptable from common sense, the key issue is to define how much thicker a pavement should be to withstand the projected loads for the design period.

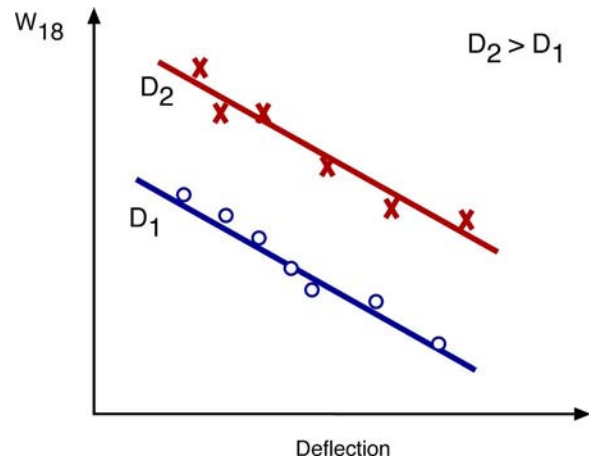


Figure 2.4 Comparison of pavement thicknesses, traffic, and deflection

3. Prediction of Minimum Concrete Temperature

3.1 Pavement Temperature Data Collection

As part of the tasks reported for the RPDB project, Report 1778-4 presented the initial developments in the installation of i-Buttons in three locations in North Texas: Amarillo, Childress, and Wichita Falls. Likewise, to supplement research tasks performed in other TxDOT projects, more of these temperature-recording devices have been installed in other locations in Texas. These efforts have been conducted in a joint effort between CTR's researchers and TxDOT personnel. To date, more than 150 i-Buttons are installed in concrete pavements in additional locations including El Paso, Van Horn, Austin, and Houston. As can be seen, these devices are installed in such a way that allows coverage of almost every climatic condition in Texas, which was the original intention.

I-Buttons are able to store temperature data for almost six months in a row; therefore, downloading recorded data should be done at least twice each year. In all locations, data has been collected for winter 2002 and summer 2003. The winter 2003 data are still being collected, and the downloading process will be done in late April 2004. At that time, i-Buttons will be reset to continue data collection. Of all the devices installed, only two have failed.

Although data are available for all the locations mentioned above, this section of the report presents only the results obtained from the North Texas locations—that is, for Amarillo, Childress, and Wichita Falls—and includes a brief description of the field activities conducted to retrieve the data recorded between October 2002 and February 2003. The information is summarized in charts, and the contents of three text files are contained in Appendix A. This appendix includes the data for one i-Button for each location (Amarillo, Wichita Falls, and Childress). The files containing all the raw data are available in Microsoft Excel format.

3.1.1 Wichita Falls Data

The i-Buttons in Wichita Falls were installed in a concrete pavement section located on US 281, southbound outside lane. The section is a 12-in. thick bonded concrete overlay (BCO). The precise location of the devices can be easily found on the road, because they were placed 1.3 miles south of Milepost 194. The global positioning system (GPS) coordinates of their location are N 33° 51' 3.36" and longitude W 98° 29' 15.12".

With regard to the data collection tasks, only a few inconveniences delayed the job. First, the software used to download the data experienced some problems, and a series of pop-up windows made the downloading task tedious. The pop-up windows were caused by an i-Button that was not providing any data because its battery ran out of life. Second, because the cable that extended from the computer to the i-Button in the pavement was very short, the computer had to be laid directly on the ground to retrieve the data. This fact, apparently simple, delayed collection tasks for more than two hours. Another factor that complicated the tasks was reflection caused by sunlight, which did not allow a clear image of the computer screen. Figure 3.1 displays how the computer had to be placed on the pavement to retrieve the data.



Figure 3.1 Tools required to retrieve data from i-Buttons

The i-Button that ran out of power was the one located at the top of the pavement, just 1 in. under the pavement surface. The remaining two i-Buttons from the set provided the data without a problem, and both were reset to collect new temperature data starting on July 27, 2003, and ending on January 27, 2004. The devices were reprogrammed in such a way that temperature data will be collected in 120-minute intervals between the two dates. Figure 3.2 shows all the raw data collected for the winter 2002 season. As can be seen, only two sets of data are presented, which correspond to the two buttons that survived in the pavement—the one at mid-depth and the one at the bottom.

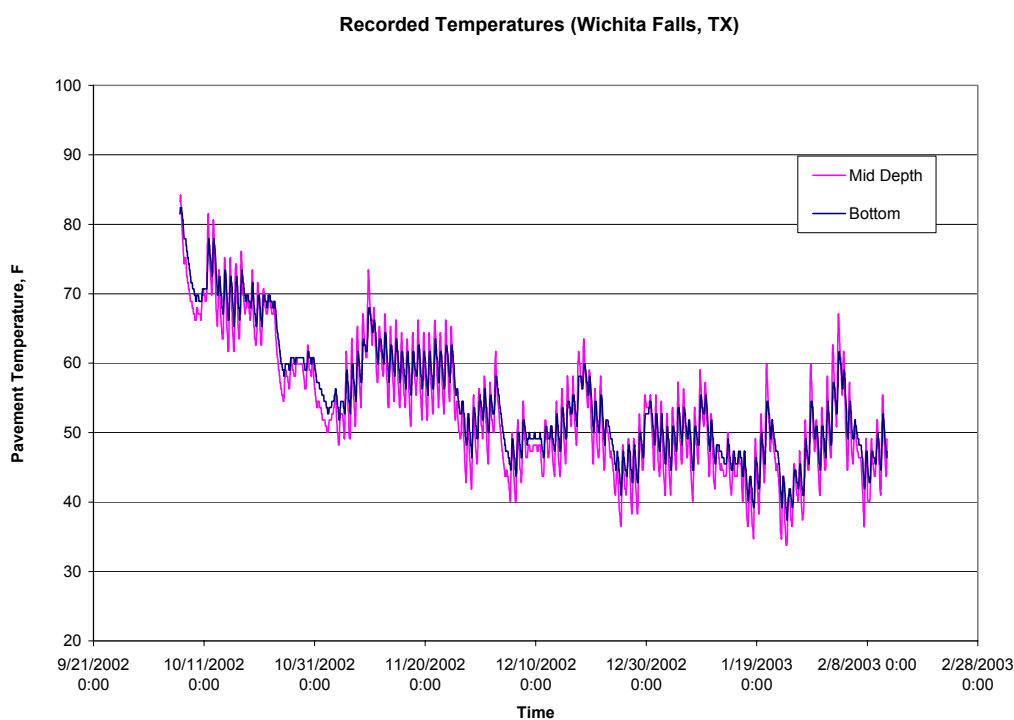


Figure 3.2 Raw concrete temperature data collected in Wichita Falls

As can be seen in Figure 3.2, the pavement temperatures were recorded from October 6, 2002, until February 11, 2003. As was expected, the maximum temperature values were recorded in October, and the temperatures descended throughout the winter, reaching the minimum values at the end of January. Temperatures increased in early February but then dropped again. The maximum temperature registered at mid-depth was 84.2 °F, and the

minimum temperature at this same point was 33.8 °F. For the bottom of the slab, the maximum temperature recorded was 82.4 °F and the minimum was 37.4 °F. Given these values, the temperature differentials experienced from October to February were 50.4 °F for the mid-depth i-Button and 45.0 °F for the bottom i-Button.

3.1.2 Childress Data

The i-Buttons in Childress were located on US 287, northbound outside lane. The pavement is a 12-in. thick CRCP. The precise location of the devices was 1.6 miles north of Milepost 232. The location is in Childress County, and GPS coordinates are latitude N 34° 26' 15.89" and longitude W 100° 14' 5.08". The TxDOT district office is located between 3 and 4 miles north of the i-Buttons location.

Data collection tasks at this location went well, and no inconveniences delayed the job. The temperature data were downloaded without any trouble. At this location, the three i-Buttons provided the temperatures as expected, and to make the collection tasks easier, an extension cable was acquired at a hardware store so that the downloading process could be done from inside the car. The use of the extension cable facilitated the tasks in several ways. First, it was much safer to work inside the vehicle, because this highway carries high traffic volumes and lots of heavy trucks. In addition, the sunlight reflection on the computer screen was eliminated, and therefore, the downloading of all the data and the reprogramming of the i-Buttons was done in approximately 15 minutes. Figure 3.3 displays how the extension cable went from the i-Buttons in the pavement to the inside of the vehicle.



Figure 3.3 Downloading temperature data from inside the vehicle

The i-Buttons provided information about the concrete pavement temperatures at the top, mid-depth, and bottom. The data obtained in Childress were for the same period of time as those in Wichita Falls, starting on October 6, 2002, and ending on February 11, 2003. Temperature measurements were taken at 90-minute intervals. Again, as in Wichita Falls, the i-Buttons were reset and reprogrammed to collect new data from July 27, 2003, until January 27, 2004. This time, the devices were scheduled to read temperatures every 120 minutes (or 2 hours) so that more time could be covered between downloading tasks. Figure 3.4 displays the raw data collected for the winter 2002 season. As can be seen, the temperatures were recorded by the three i-Buttons at top, mid-depth, and bottom. As was predicted, the maximum temperature values were recorded in October; later, the temperatures decreased, reaching the minimum values at the end of January. An upward shift of the values was observed for the first 4 days in February, and then the temperature dropped again. The maximum recorded temperature at the top of the pavement was 93.2 °F, the minimum temperature at this location was 23.9 °F, and therefore, the maximum temperature differential experienced during the 4 months (October–February) was 69.3 °F. For the mid-depth i-Button, the maximum temperature was 87.8 °F, the minimum temperature was 32.9 °F, and the maximum temperature differential was 54.9 °F. Finally, for the bottom fiber of the slab, the maximum temperature was 84.2 °F, the minimum temperature was 35.6 °F, and the maximum temperature differential was 48.6 °F. All these values are reasonable and in agreement with common sense, because it should be expected that the top fiber of the slab, which is exposed to the air, will experience more extreme conditions. The opposite is observed for the “heart” of the pavement, where temperature differentials are not as critical. Note that during very cold days, the top of the slab reached temperature values below the freezing point, 32 °F. This is in fact an issue that should be regarded from different perspectives in pavement research.

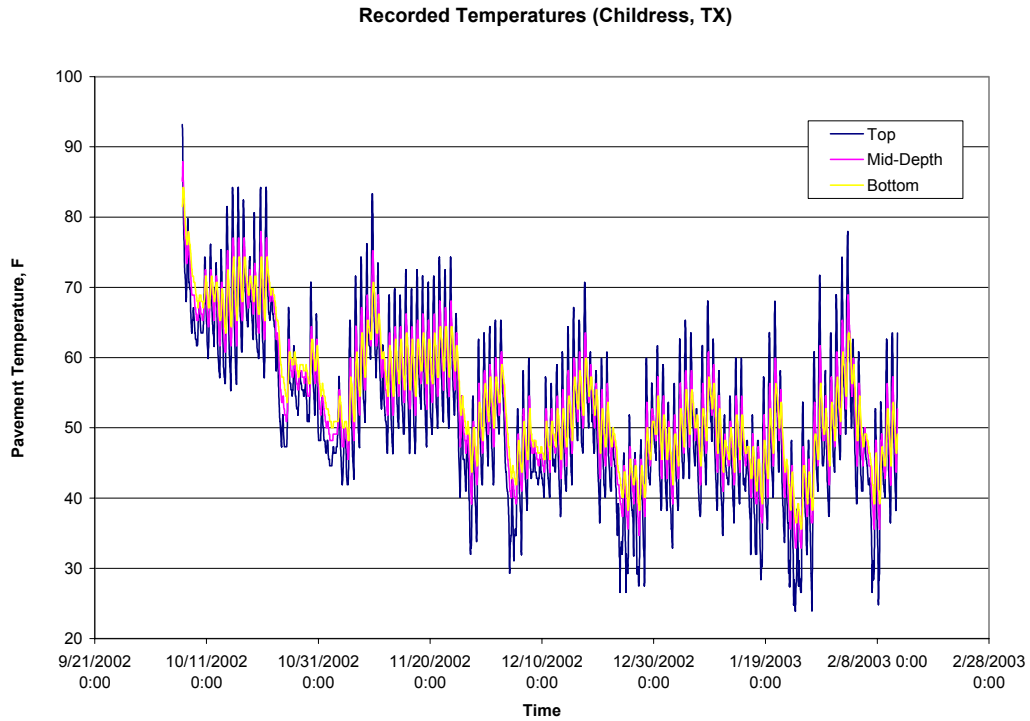


Figure 3.4 Raw concrete temperature data collected in Childress

3.1.3 Amarillo Data

The i-Buttons in Amarillo were installed on a concrete pavement section located near IH 40. It was not possible to install the devices on the interstate highway owing to safety issues. The set of i-Buttons was installed outside the TA Travel Center, which is a large-truck maintenance facility located on the southbound frontage road of IH 40. The area is in Amarillo County, and GPS coordinates of the i-Buttons are latitude N 35° 11' 27.81" and longitude W 101° 45' 36.03". This pavement is 8 in. thick and was constructed in 2001.

In Amarillo, as in Childress, the data collection effort was very successful, and no delays were experienced. The three i-Buttons were able to record and keep the temperature data until they were retrieved. The downloading process was performed, and the devices were reset and reprogrammed to continue collecting data for the same period of time as that for

the other two locations. Figure 3.5 shows the moment when the i-Buttons were plugged to the computer to download the data.



Figure 3.5 Downloading temperature data in Amarillo

For illustrative purposes, Figure 3.6 displays a member of the field crew downloading the data from the inside of the vehicle, and Figure 3.7 shows the alligator, or jumper connectors, that connect to the cable tips protruding from the uncovered hole drilled in the pavement.



Figure 3.6 Data downloading from inside the vehicle



Figure 3.7 I-Button cable tips plugged to computer

The i-Buttons were placed at the top, mid-depth, and bottom of the slab. Temperature data were obtained from October 6, 2002, until February 11, 2003, and records are available for 90-minute intervals. The i-Buttons were reprogrammed to collect new data in the same way that the devices at the other locations were. Figure 3.8 presents the raw temperature data collected for the winter 2002 season for the top, mid-depth, and bottom locations.

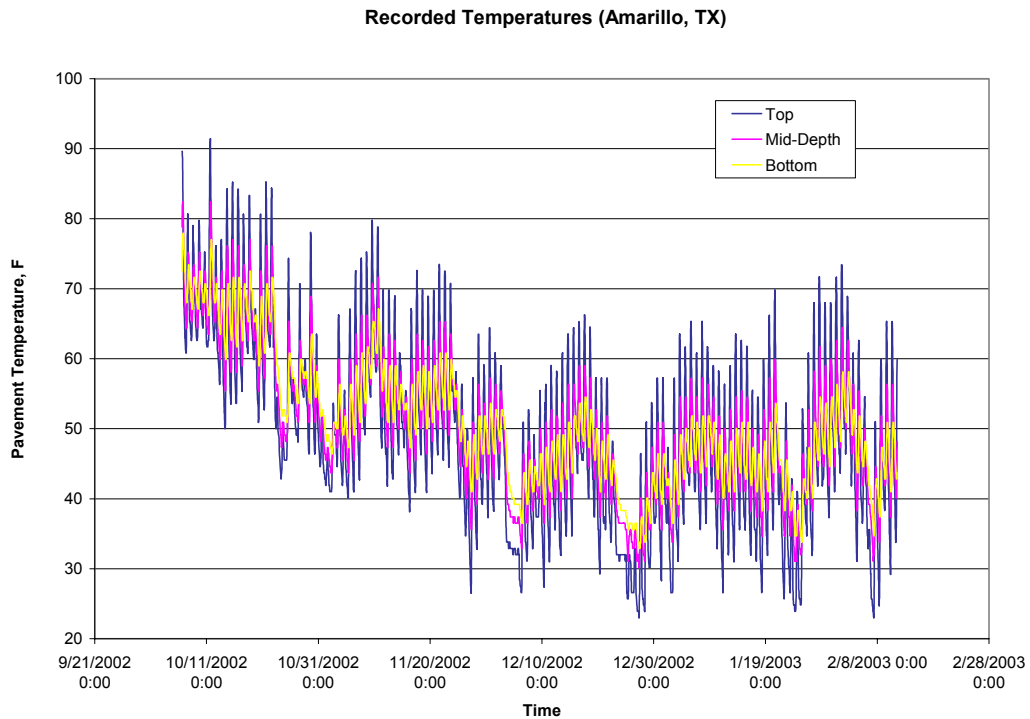


Figure 3.8 Raw concrete temperature data collected in Amarillo

In Amarillo, very low temperatures were reached at various times, starting in late November and continuing through February. Freezing temperatures were reached at various times. Maximum temperatures were recorded in October and gradually decreased with time. The maximum recorded temperature at the top of the pavement was 91.4°F, and the minimum temperature was 23.0°F; therefore, the maximum temperature differential during the monitoring period was 68.4°F. For the i-Button located at mid-depth, the maximum temperature was 82.4°F, the minimum temperature was 30.2°F, and the maximum temperature differential was 52.2°F. The bottom of the slab experienced a

maximum temperature of 77.9°F, a minimum temperature of 32.9°F, and a maximum temperature differential of 45.0°F.

3.1.4 Summary of Data Collection

The developments of the RPDB project have gone beyond expectations. In addition to managing the existing pavement database, which contains information about hundreds of pavement sections, a great effort has been made to perform additional research tasks. Installation of i-Buttons in various locations in Texas has helped to monitor temperature history for both new and old concrete pavement. For the three North Texas locations previously mentioned, pavement instrumentation and data downloading processes have been conducted with great success. It is expected that future tasks related to these projects will be developed with no delay and will provide valuable information that will enrich the RPDB's contents. In the end, all the information that is being compiled as part of the tasks in this project will serve pavement design engineers and state agencies in the identification of key factors that affect the design and performance of rigid concrete pavements. For instance, the temperature-monitoring tasks just described will aid in a more realistic assessment of the $\Delta T_{maximum}$ value that was described in Chapter 2. As a result, current CRCP design theory and computer-aided programs could be improved by using a mechanistic–empirical approach for temperature-related matters.

3.2 Temperature Prediction Models

As was previously mentioned, there are several approaches from which to predict concrete pavement temperature. In Chapter 2, Equation 2.6 showed the basic form for temperature prediction, which is the minimum concrete temperature equal to the minimum air temperature at the specific location, modified by one or more factors; in other words, it is a function of a set of variables. The more variables included in a model, the more complex it becomes; however, it gives more realistic outcomes. This section serves as an introduction to Chapter 6, which presents applications of the models described in the following paragraphs.

3.2.1 Ambient Temperature Based Model

This is probably the simplest model that might be used for the prediction of minimum concrete temperature. The algorithm uses solely the ambient temperature at a given location to provide the predicted concrete temperature during the cold season. Because this model is fairly simplistic, it provides reasonable results. This model can be used everywhere; the only input required is the ambient temperature, which can be obtained with a calibrated thermometer. The NCDC database provides abundant information about this parameter with a time frame of at least 30 years for almost every location in Texas; therefore, with such readily available data, this model can easily provide the concrete pavement temperature just by inputting the ambient temperature.

3.2.2 Multivariate Model

This is a tailored model that uses as many climatic variables as are available. The more variables included, the more accurate the model is; however, as a result it is more complex and difficult to adapt to specific locations where no data are available. The proposed model described in Chapter 6 uses three variables—ambient temperature, wind speed, and solar radiation—that have the most significant impact on concrete pavement temperature. Because this model involves more variables, it gives better results than the model that uses only ambient temperature. Unfortunately, solar radiation values are not available for many locations in Texas, which limits the applicability of this model. As technology advances and sites are instrumented, this model will become applicable on a broad basis.

3.3 Feasibility for Statewide Application

Because availability of solar radiation data in Texas is limited, only the temperature data collected in Amarillo were used in an example of the application of the two models previously described. Solar radiation information was not available for Childress or Wichita Falls. It is hoped that in the near future, as these data are collected and made available to the public, the multivariate model will be implemented for pavement design purposes. In fact, the CRCP-11 program might include a predicted minimum concrete pavement temperature module, which will divide Texas into various regions according to

their specific climatic conditions and will perform CRCP designs tailored for those conditions. In the end, a sophisticated tool for pavement design that considers predicted minimum concrete temperatures for the medium and long terms will be available, allowing more accurate results, more economical designs, and, more important, better-performing and less costly pavements.

4. Spalling Model for Overlay

4.1 Development of Spalling

Transverse cracking is an undesirable, inevitable distress associated with concrete pavements, especially CRCP. In some cases, these cracks are aggravated by the development of spalls in the concrete. Spalls initiate at one or both edges of a crack and then continue developing unpredictably. It is well understood that the severity and extent of spalls depend on many factors, such as concrete components, construction process, curing technique, climatic parameters, and lastly traffic loads. The interaction of all these variables either triggers or halts the development of spalls. Severe spalls result in a more distressed pavement, which consequently becomes weak because it is structurally opened to environmental factors. When a pavement section is severely spalled, its riding quality immediately drops and major maintenance tasks might be needed.

4.1.1 General Concepts

Spalling is defined as “the cracking, breaking, chipping, or fraying of the concrete edges within a few feet of a joint or crack” (Ref 10). Crack and joint spalling are common distresses found in JCPs and CRCPs. Concrete spalling gives an indication of crack movement and deterioration. Some factors contributing to this type of distress include the following.

- Presence of incompressible material in the joints or cracks that restrain slab expansion
- Slab curling and warping caused by temperature and moisture gradients
- Presence of alkali–aggregate reactivity (AAR) or D-cracking
- Insufficient air voids in the concrete mixture
- Poor freeze–thaw durability
- Localized weak areas in the concrete caused by poor consolidation
- Improper placement
- Corrosion of embedded steel
- High CTE of the aggregate

Among the concrete properties that contribute to spalling are workability, durability-related characteristics, coarse aggregate–mortar bond, and strength. These properties are related to aggregate gradation, size, mineralogy, texture, strength, and elastic modulus. In the early 1980s, concrete pavement research indicated that crack and joint spalling was a distress that required special observation and consideration for the estimation of pavement distress indexes (PDIs). However, because the occurrence of spalling was infrequent, its inclusion in those indexes produced biased results. Gutierrez de Velasco (Ref 6) proposed a couple of models for the prediction of minor and severe spalling. These models were based on distress data collected during condition surveys performed in 1974.

4.1.2 Previous Developments

Gutierrez de Velasco’s model for prediction of minor spalling was exponential, based on the fact that the maximum amount of spalling in a section was 100 percent. Thus, the asymptotic model was of the form shown in Equation 4.1.

$$MS = A_0 + A_1 \cdot \exp(B \cdot X_1) \quad (4.1)$$

where

MS = percent of minor spalling

X_1 = age at time i

A_0 , A_1 , and B = constants

The value of B was estimated by using condition survey data from 1974, as is shown in Equation 4.2.

$$B = \ln \frac{(1.0 - MS_1 / 100)}{X_1} \quad (4.2)$$

By combining Equations 4.1 and 4.2, the prediction of minor spalling can be estimated as shown in Equation 4.3.

$$MS_2 = A_0 + A_1 \cdot \exp(B \cdot X_2) \quad (4.3)$$

where

MS_2 = predicted percentage of minor spalling at future age

MS_1 = percentage of minor spalling at condition survey time

X_2 = pavement age at time of prediction, years

X_1 = pavement age at condition survey time, years

$A_0 = 92.357$

$A_1 = -87.764$

A_0 and A_1 coefficients were obtained from regression analysis. The study included 139 points, and the estimated correlation coefficient for the model was $R^2 = .846$.

For severe spalling, Gutierrez de Velasco used the same reasoning as that applied for minor spalling. He derived the model shown in Equation 4.4.

$$SS_2 = A_0 + A_1 \cdot \exp(B \cdot X_2) \quad (4.4)$$

where

SS_2 = predicted percentage of severe spalling at future age

SS_1 = percentage of severe spalling at condition survey time

X_2 = pavement age at time of prediction, years

X_1 = pavement age at condition survey time, years

$A_0 = 93.804$

$A_1 = -92.857$

Analogous to that in Equation 4.2, the constant B was in his model was estimated as shown in Equation 4.5.

$$B = \ln \frac{(1.0 - SS_1 / 100)}{X_1} \quad (4.5)$$

Again, A_0 and A_1 coefficients were obtained from regression analysis in a study including 139 point values. The correlation coefficient for the model was $R^2 = .860$.

One of the major pitfalls of the equations proposed by Gutierrez de Velasco lies in the fact that both consider the percentage of spalling rather than the actual number of spalls per mile, which would be more appropriate for the estimation of a DI. The reason he used percentage of spalling was because it was the parameter used during condition surveys in 1974.

Later, in 1988 Chia-Pei Chou (Ref 8) modified those models previously presented, and it was observed that the inclusion of the minor and severe spalling terms in the computation of the DI was misleading for two reasons: (1) because of its negative signs and (2) because the resulting coefficients for these terms were small as compared with the coefficients of other distresses, such as punchouts and patches. Therefore, Chou developed another model that did not include the effect of spalling in concrete. This model was presented in Chapter 2, Equation 2.10.

As concrete pavements aged and reached lower riding qualities, it was noticed that the appearance of spalling became more frequent as well. It was not until the late 1980s or early 1990s that spalling in concrete pavements manifested broadly, especially in CRCPs. Spalling started to occur in pavement sections across the state on a random basis, but some regions, especially the Houston district, started having more serious problems. Indeed, TxDOT Project 0-4398, in an effort conducted jointly with Project 0-1778, focused on the study of a particular group of sections located in Houston on SH 6. These sections, which were clear examples of the spalling problem in concrete pavements, led to the development

of a new DI model (Ref 11). This model might be applied to rate the conditions of a pavement section that shows spalling; however, it does not differentiate between minor and severe distresses.

4.1.3 Spalling Classification

Although spalling is treated as a common pavement distress these days, there is no clear definition in the literature for the classification of the problem. The Pavement Management Information System (PMIS) Rater's Manual edited by TxDOT (Ref 7) describes a spalled crack as "a crack that shows signs of chipping on either side, along some or all of its width." It states that the spalled crack must display edge chipping or secondary cracking at least 3 in. long on either side of the crack. However, the width and depth of the crack is not considered, which yields ambiguous pavement ratings.

Nevertheless, although a clear differentiation between minor and severe spalling is not available, the RPDB project has made progress on this subject. When condition surveys are conducted, both minor and severe crack spalling are noted in the survey form. Photographs are taken to document exemplary distresses, and those images are stored in the RPDB photo database. Thus, the severity of a spall is well defined within the context of the RPDB project. Figure 4.1 displays an isolated spall typically rated as minor. This pavement section is located on SH 6 in the district of Houston.



Figure 4.1 Isolated minor crack spall found in a CRCP

Figure 4.2 displays a CRCP section with typical minor spalling. This particular section is located on IH 10 in the district of Houston. The spalls were found during a condition survey performed in the summer of 2002. According to information obtained from the surveying crew, as of press time of this report the spalls have not deteriorated considerably.



Figure 4.2 Series of minor spalls found in a CRCP on IH 10 in Houston

Figure 4.3 displays a zoomed image of a severe spall found in a CRCP on IH 10 in Houston. According to the records, this spall was initially spotted during the condition survey conducted in 1996; however, at that time it was a minor spall with chipping of the concrete present only at one edge of the crack. It can be seen that in over seven years its condition has aggravated, and it is now a severe spall requiring patching. Notice how this particular spall presents problems in length (transverse direction) and width (longitudinal direction), which have progressed over time. The depth of the distress did not deteriorate at such a rapid pace, which is the main reason why corrective maintenance was not performed right away. This in fact helped in tracking the evolution of the problem.



Figure 4.3 Severe spall found in a CRCP on IH 10 in Houston

Figure 4.4 shows the appearance of a severely spalled CRCP section located on SH 6 in Houston. The section was constructed during the winter of 1990 as part of TxDOT Project 1244. The main objective of that project was to investigate the effects and interactions of construction season, concrete coarse aggregate, and the content of reinforcement steel on the performance of a series of test sections. Details about the performance of the sections can be found elsewhere (Ref 12). Figure 4.4 shows that many of the transverse cracks that

appeared in the CRCP were repaired with the use of a special mortar to deter the progress of the cracks. Unfortunately, it was observed that not long after the cracks were repaired, the mortar detached from the surface of the concrete owing to traffic loads and environmental effects. Because of the recurring spalling problems in this section, TxDOT decided to overlay the CRCP with a thin asphalt layer. This section is part of the special studies (satellite factorials) of the RPDB project, and it will continue to be monitored to track the performance of the AC overlay.



Figure 4.4 Severely spalled section located on SH 6 in Houston

4.1.4 Rating and Significance

As has been previously stated and done with all other distresses observed in pavements, two main factors are considered when conducting condition surveys. The severity of a spall basically indicates whether the distress is minor or severe. Although the photos previously shown provide a general idea of how to discriminate one from the other, it is important to have a more defined criterion. For instance, a spall in general is defined as when the chipped concrete in the crack (at one or both edges of the crack) adds up to 12 in.

in length. This means that the spall can be continuous or discontinuous for a given crack. As long as the cumulative length of the chipped concrete adds up to 12 in. in a single crack, a spall is counted.

Thus, the first aspect to be identified is the severity of the spall. For the purposes of this project, we have classified spalls as minor or severe. A minor spall is one that is more than 12 in. long, equal to or less than 1/2 in. wide, and between 1/8 and 1/4 in. deep. Likewise, a severe spall is identified when it is more than 12 in. long, over than 1/2 in. wide, and more than 1/4 in. deep. Figure 4.5 summarizes the dimensional characteristics for (1) minor spalls and (2) severe spalls.

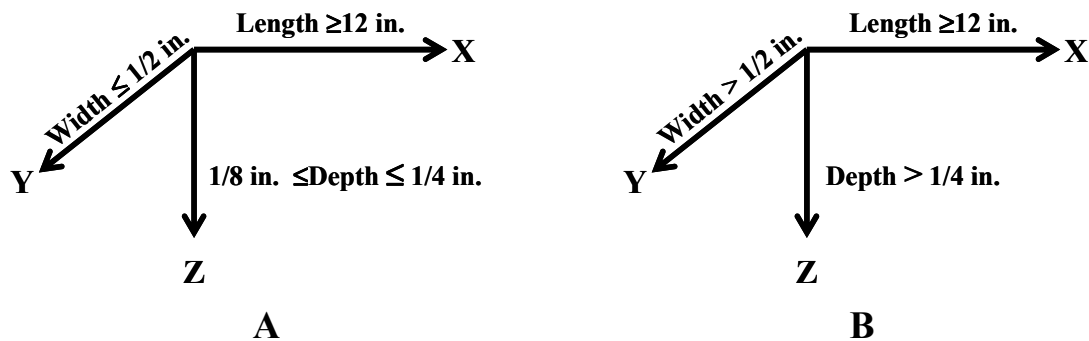


Figure 4.5 Dimensions used to identify minor and severe spalls

The next characteristic to be documented in a spalled pavement is the extent of the spalls. During surveying practices, the number of minor and severe spalls is noted for the entire length of the section, which is written in the survey form. Next, distress data are typed into Microsoft Excel, and the number of spalls (and other distresses) is computed per mile of length of the pavement. The number of spalls per mile measure provides a better understanding of the general condition of the pavement than percentage of spalling, as has been used in the past in the spalling prediction models developed by Gutierrez de Velasco.

This number of spalls per mile is the value input into the DI model that was developed with the use of the data from TxDOT Project 0-4398. Nevertheless, the overall number of spalls is used regardless of whether they are minor or severe. Therefore, in the RPDB project the model was refined, and a more reliable model was obtained that uses different coefficients for minor and severe spalling. Those coefficients used for the calculation of the DI for any pavement were estimated with the use of comprehensive data contained in the RPDB and not from only a few sections from a single pavement project.

4.2 Requirements for Overlay on the Basis of Spalling

This section presents the model that was obtained for the estimation of the DI on the basis of previous models developed at CTR. The model incorporates the spalling data collected during the latest collection efforts for the RPDB project. As has been previously mentioned, in the past spalling data were not considered significant and, therefore, were left out of the DI models. It is important to note that crack spalling is now considered in more recent models. In addition, crack spalling catalyzes the development of other pavement distresses that together deteriorate the structural integrity of the pavement at a very rapid pace. The results of the model presented in this section can be used to rationally recommend the construction of a pavement overlay that will improve the structural adequacy of the pavement or, at least, to schedule maintenance tasks to delay the quick drop in pavement serviceability caused by the roughness of the pavement surface.

Equation 4.6 presents the model developed for the estimation of the DI. This model was developed with the use of data collected for CRCPs, and it should be applied using good engineering judgment. As can be observed from the data stored in the RPDB, a practical indicator that a pavement needs at least an AC overlay is when the number of failures per mile per year for the given section is more than two over its life. Those failures include punchouts, patches, and spalls, as is indicated by the model. When the rate of failures per mile per year exceeds this criterion, the rehabilitation procedure might indicate a thick AC overlay, a BCO, or an unbonded concrete overlay.

$$Z'' = 1.0 - 0.0071MPUNT - 0.3978SPUNT - 0.4165PATCH - \\ -0.1315MSPALL - 0.2228SSPALL \quad (4.6)$$

where

Z'' = DI or zeta score

$MPUNT$ = ln (minor punchouts per mile +1)

$SPUNT$ = ln (severe punchouts per mile +1)

$PATCH$ = ln (total patches per mile +1)

$MSPALL$ = ln (minor spalls per mile +1)

$SSPALL$ = ln (severe spalls per mile +1).

In all cases, the rehabilitation criterion governs whether the value of the DI or Z'' is zero or negative.

5. Performance of CRCP: Deflection Data

5.1 Background

Pavement deflection is a parameter that provides a good indication about the structural condition of the structure. For current pavement evaluations, deflections obtained from the falling weight deflectometer (FWD) represent a valuable tool in determining the elastic properties of the layers in the pavement. TxDOT collects FWD information at the network level, and that information is available through the PMIS database. Although the deflection data that can be obtained from the PMIS are not comprehensive, at least they provide a general idea of how pavements deflect in certain areas of the state. Those deflection values are a function of the properties of the pavement structure and the load applied for the test.

TxDOT usually conducts deflection testing by using at least two testing devices, the rolling dynamic deflectometer (RDD) and the FWD. The RDD provides a continuous load deflection profile of the pavement and is an invaluable tool for detecting weak spots along the structure that the FWD might not notice, basically because the FWD is used for discrete testing. Therefore, the two equipments are normally used and complement each other. Usually, further testing is conducted with the FWD where the RDD has identified problematic, highly deflecting areas.

5.2 Interpretation of Deflection Data

The information obtained from the FWD can be used for various purposes, but in general, it helps in assessing the structural capacity of the pavement structure and in designing pavement overlays. This section presents a methodology that might be used for the evaluation of the feasibility of constructing an AC overlay on CRCP on the basis of deflection measurements. This methodology is called the *deflection criterion*, and it is based on the calculation of two components, the deflection ratio and the stress ratio. The following paragraphs discuss the details, estimation, interpretation, and application of these components, which are used to decide whether a CRCP might or might not be rehabilitated.

The deflection criterion is based on two components: stress calculations and deflection measurements taken at the cracks and the midspan of pavement slabs. These components are expressed as ratios: the deflection ratio and the stress ratio. The next sections will explain these ratios and how they are put together to become a decision element for AC overlays on CRCP.

The deflection criterion concept was developed with the use of the data contained in the RPDB. This information was obtained from twelve districts; it was collected in 1988; and it includes a total of 23,972 data points taken at 5,909 different locations within the districts. Tested sections were mostly 8-in. thick CRCPs (overlaid and nonoverlaid) as well as some 9- and 10-in. thick sections. The data for all the sections were carefully checked with the use of a computer program as well as human judgment (Ref 16).

5.2.1 Deflection Ratio

This deflection ratio is computed with the use of the two types of deflection values at the crack and at slab midspan. To obtain these data, measurements are taken first across transverse cracks and then on continuous slabs of pavement with no cracks between deflection sensors, which are the readings considered to be taken at the midspan. The arrangement of the FWD sensors is different for these two types of measurements. To measure deflections at the midspan, the sensors are usually arranged in normal order, starting with Sensor 1 at the loading point and continuing with the other six sensors spaced every 12 in. center to center. To measure deflections at the cracks, the sensors are arranged differently, as is illustrated in Figure 5.1. Sensor 1 is positioned at one side of the crack, Sensor 2 is placed at the opposite side of the crack, and Sensors 3 and 4 are positioned next, using the same 12 in. distance criterion from sensor to sensor. Finally, Sensor 5 is positioned 24 in. from Sensor 4, and Sensors 6 and 7 are placed 12 in. apart from each other. This arrangement provides load deflection basins that are easily interpreted and provide good load transfer capability data at cracks.

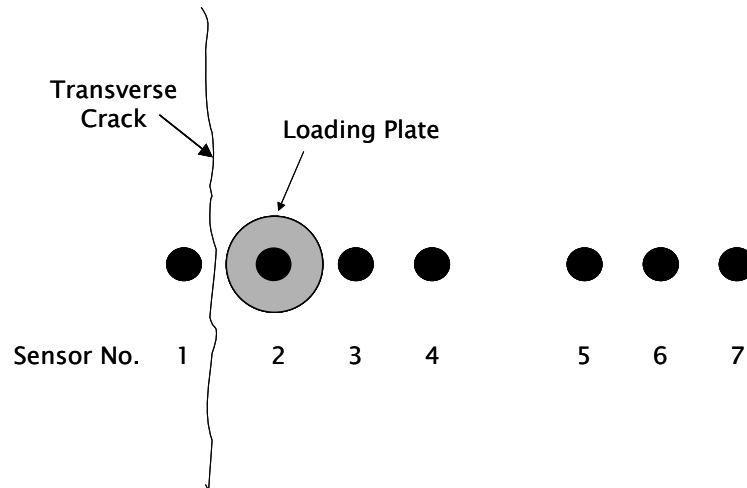


Figure 5.1 FWD sensor arrangement for deflection measurement at cracks

After measuring deflections at midspan and at cracks, the deflection ratio can be estimated with Equation 5.1.

$$\text{Deflection Ratio} = \frac{\text{Deflection at crack}}{\text{Deflection at midspan}} \quad (5.1)$$

The deflection ratio is in fact a measurement of load transfer, and it is inversely proportional to the structural capacity of the pavement. If deflection measurements at crack and at midspan are equal for a given location, the load transfer at that location is 100 percent, which indicates good structural capacity. In reality, for a conventional CRCP pavement, the ratio most likely will be greater than 1.0, meaning that at that location, there was a loss of LTE owing to the appearance of the crack. Likewise, it is unlikely that the deflection at a crack will be smaller than that at the midspan, in which case the deflection ratio will be less than 1.0.

If the ratio is equal to 1.0, the pavement's structural capacity is sufficient so that it does not require rehabilitation. For deflection ratios greater than 1.0, some type of rehabilitation may be required. The more the ratio deviates from a value of 1.0, the more expensive the

rehabilitation strategy will be, because this ratio equates to the structural condition. The decision criteria for rehabilitation can be found elsewhere (Ref 11). The recommended solutions, from least expensive to most expensive, are the thin AC overlay (minimal structural benefit), the BCO (considerable structural improvement for an existing pavement that is not too deteriorated), and finally the unbonded concrete overlay (for cases that need considerable structural improvement).

5.2.2 Stress Ratio

Similar to the deflection ratio, a stress ratio can be calculated to determine the structural contribution of an overlay by comparing the stresses calculated before and after the overlay. The stress ratio is defined in Equation 5.2.

$$\text{Stress Ratio} = \frac{\text{Stress without overlay}}{\text{Stress with overlay}} \quad (5.2)$$

This stress ratio concept is based on the principle that a thin AC overlay placed on top of a CRCP will provide a minimal structural contribution, and thus, the ratio is expected to be very close to 1.0 for this type of rehabilitation. Higher values that depart more from a ratio of 1.0 will indicate that a different kind of rehabilitation (e.g., a thicker overlay) is more appropriate. The stronger overlay will have a more significant structural contribution. In practice, a thickness for an AC overlay is assumed, and the stresses with and without this layer are estimated. It is expected that a more structural type of rehabilitation will be necessary as the ratio departs further from 1.0.

5.3 Overlay Selection on the Basis of Deflection

The concepts of deflection ratio and stress ratio and their interaction serve as a tool that aids in selecting the overlay type to be constructed in a rehabilitation project. A conceptual illustration of the deflection criterion was developed by plotting the deflection ratio versus the stress ratio and delimiting boundary areas of adequacy for three rehabilitation

strategies: AC overlay, BCO, and unbonded concrete overlay. These plots are represented in Figure 5.2. Cases in which the ratios have values less than 1.0 do not require an overlay, at least from the structural standpoint. The definition of the threshold values for this concept can be found in the literature (Ref 11). This section only summarizes those results obtained from the research conducted in that project.

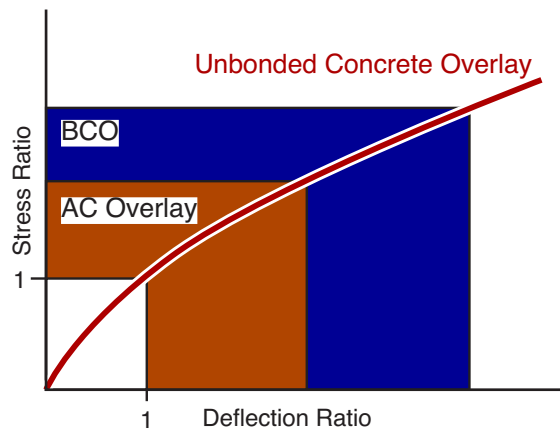


Figure 5.2 Applicability of different types of overlay according to a deflection criterion

The deflection criterion threshold values adopted for an AC overlay correspond to a stress ratio between 1.0 and 1.3 and a deflection ratio between 1.0 and 1.2. Chapter 6 presents a case study in which this deflection criterion is applied and construction of an AC overlay is recommended. Similarly, for a low-modulus concrete, a BCO is advisable when the deflection ratio is less than 1.7 for 8- and 10-in. thick pavements and less than 1.85 for 12-in. thick pavements. For a high-modulus concrete, a BCO is advisable if the deflection ratio is less than 1.25 for 8- and 10-in. thick pavements and less than 1.40 for 12-in. thick pavements. Cases rendering values beyond these limits are better suited for unbonded concrete overlays.

6. Application of Models

The information contained in the RPDB is almost boundless, and therefore, the analyses that can be conducted with this information are quite extensive. In fact, other research studies have used the data contained in the RPDB to perform further research; for instance, Project 1244 concentrated on analyzing a number of pavement sections built in the Houston district with different concrete aggregates and in different seasons. The main objective of these types of studies is to evaluate the performance of pavements given different design and construction procedures and materials.

Because it is unfeasible to try to cover every type of analysis that can be performed with the information from the RPDB in this report, only examples are provided. Analyses conducted in the past and documented in previous reports include the effect of aggregate type on the performance of the pavement including crack spacing, occurrence of distresses per mile, and so forth. This time, other types of analyses are presented and include the presentation of a minimum concrete temperature prediction model that uses already available climatic data as input, as was described in Chapter 3. Likewise, the improved spalling model described in Chapter 4 is applied to a series of pavement sections, comparing the results obtained from the previously developed model with the ones obtained from the improved model. Finally, an example of the application of pavement deflection as a criterion for overlay is also presented.

6.1 Prediction of Minimum Concrete Temperature

This section complements the concepts defined in Chapters 2 and 3 regarding the prediction of the minimum concrete temperature. As has been previously mentioned, the goal is to estimate the minimum concrete temperature that will be expected after construction. In the past, this design input value was assumed to be equal to the minimum ambient temperature observed; however, this assumption has somehow led pavement design engineers to obtain thicker or overdesigned pavements, which in turn are more expensive.

This analysis is based on the assumption that the minimum concrete temperature during any cold season (winter) will be always higher than the minimum ambient temperature observed at the same location, as shown in Figure 6.1. On the basis of this assumption, the value of the $\Delta T_{maximum}$ can be calculated for any pavement design with a greater level of accuracy than has been done previously. In general, the better this parameter is estimated, the less expensive the pavement will be; in other words, the structure will not be oversized.

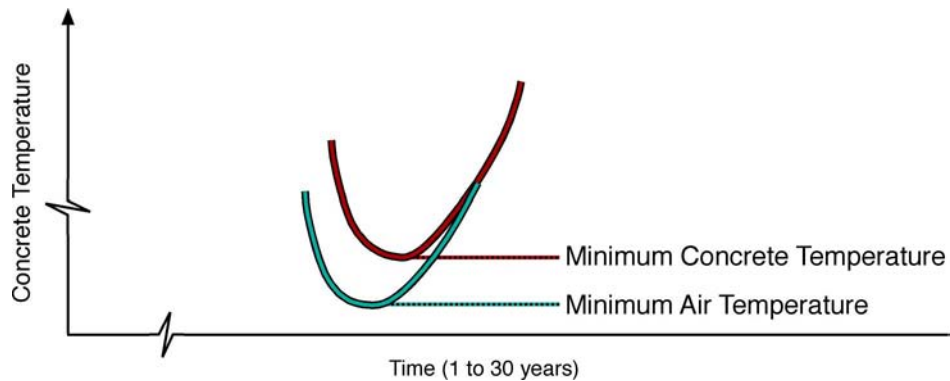


Figure 6.1 Relationship between minimum concrete and air temperatures

To demonstrate the applicability of the RPDB in the estimation of the minimum concrete temperature, the data that were retrieved with the i-Buttons were used. This section presents the analysis performed with the data recorded in Amarillo.

6.1.1 Ambient Temperature Based Model

This is the simplest model that might be used for prediction of the minimum concrete temperature. The algorithm uses solely the ambient temperature at a given location to provide the predicted concrete temperature during the cold season. Because this model is fairly simplistic, it provides reasonable results. This model can be used everywhere; the only input required is the ambient temperature. The NCDC database provides abundant information about this parameter with a time frame of at least 30 years for almost every

location in Texas; therefore, with such readily available data, this model can easily provide the concrete pavement temperature just by inputting the ambient temperature.

The first step in the process was to download the temperature data from the buried i-Buttons. These data were plotted as shown in Figure 6.2. A total of 2,048 points were plotted for each i-Button location (top, mid-depth, and bottom) and correspond to all the recorded data (raw data). As can be seen in the figure, the curves clearly show maximum and minimum temperature points and the ranges of those temperatures for each location. The next step was to compare those concrete temperature values with ambient temperature values obtained from the NCDC database (Ref 3). Once values were compared, linear correlation equations were established for the three i-Button locations. The correlations obtained for those locations are shown in Figures 6.3–6.5.

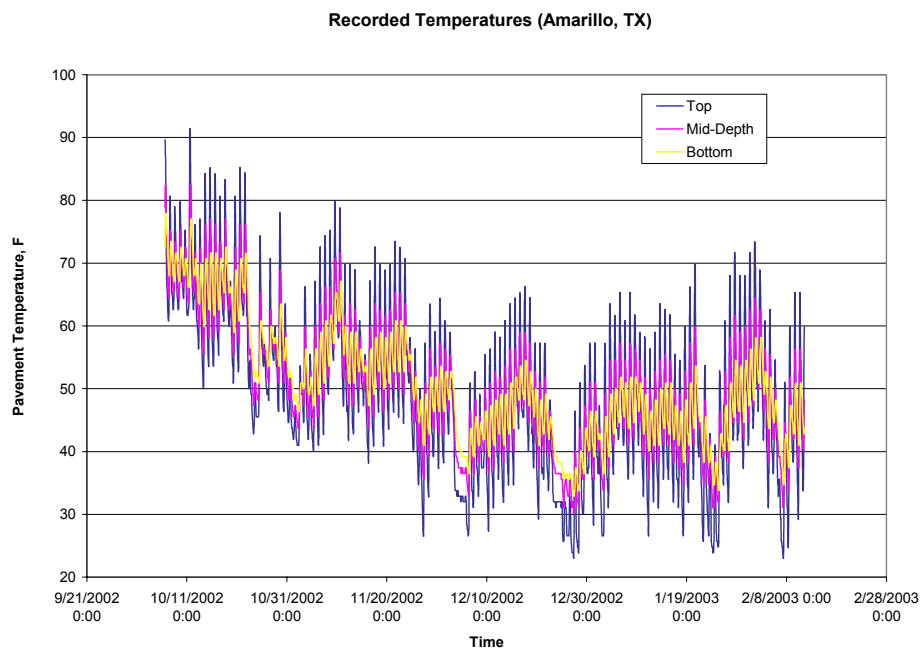


Figure 6.2 Concrete temperatures recorded in Amarillo, TX

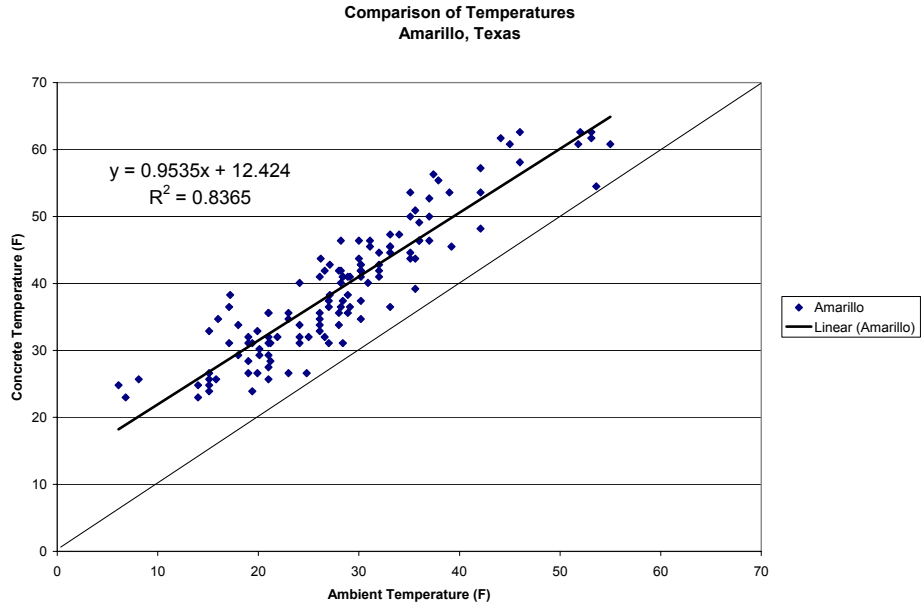


Figure 6.3 Correlation between concrete and ambient temperatures (top)

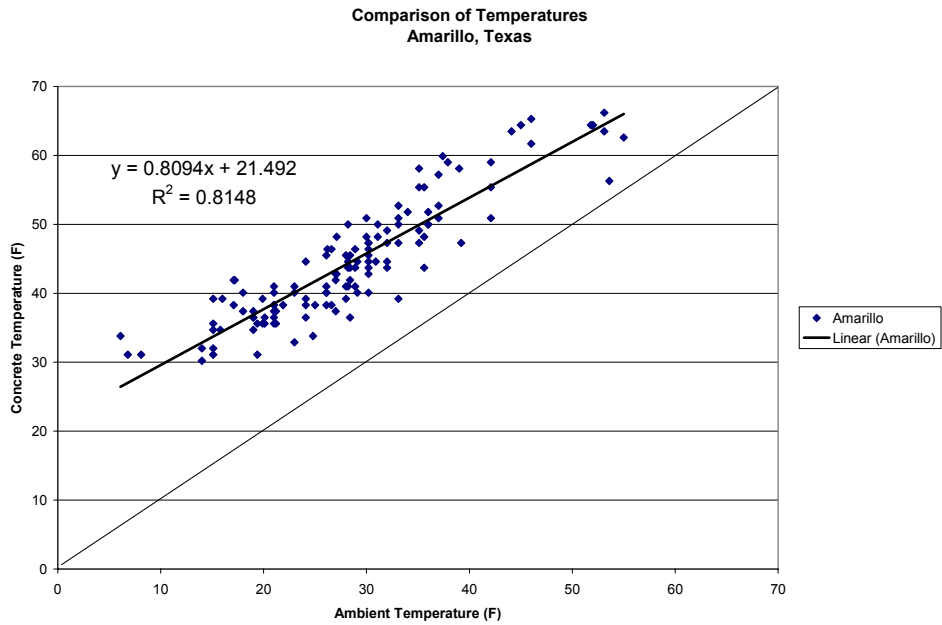


Figure 6.4 Correlation between concrete and ambient temperatures (mid-depth)

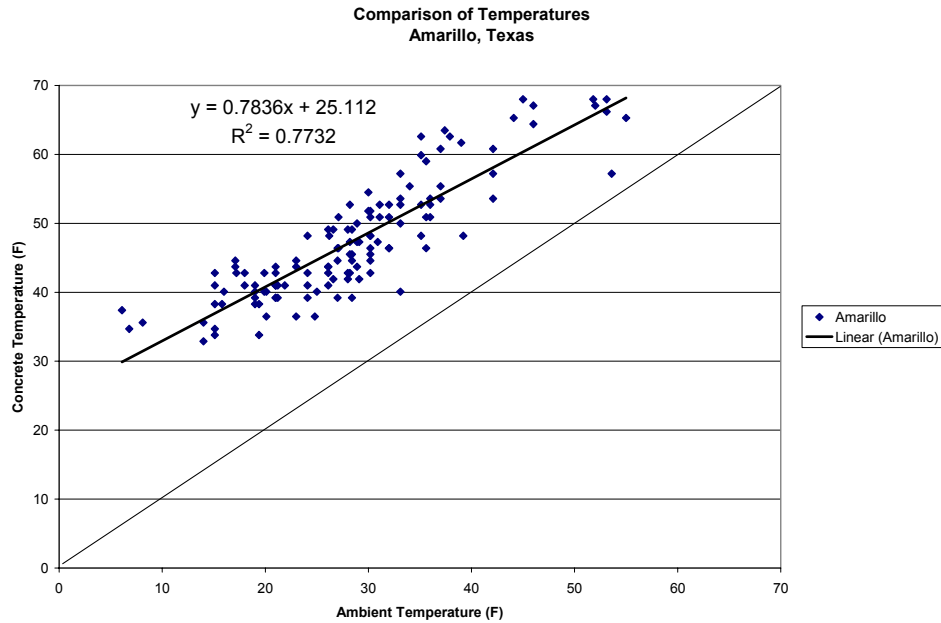


Figure 6.5 Correlation between concrete and ambient temperatures (bottom)

As can be seen from Figures 6.3–6.5, there is a good correlation between the ambient temperatures and the concrete temperatures. Likewise, it can be seen that the best correlation coefficient (0.83) was obtained for the i-Button placed at the top of the concrete slab and the lowest correlation coefficient value (0.77) was obtained for the i-Button placed at the bottom of the slab. This is reasonable because the deeper the concrete temperature is measured, the less probable the ambient temperature by itself will be able to predict it.

Once the correlations between those temperatures shown in Figures 6.3–6.5 were obtained, the next step was to use the estimated linear regression equations to calculate the concrete temperature given the ambient temperature and then plot those curves along with the corresponding measured i-Button temperatures. Figures 6.6–6.8 display the obtained curves for each location for top, mid-depth, and bottom. In all cases, the blue line represents the ambient temperature obtained from the NCDC website, the pink line corresponds to the measured i-Button temperature, and the red line is the predicted temperature.

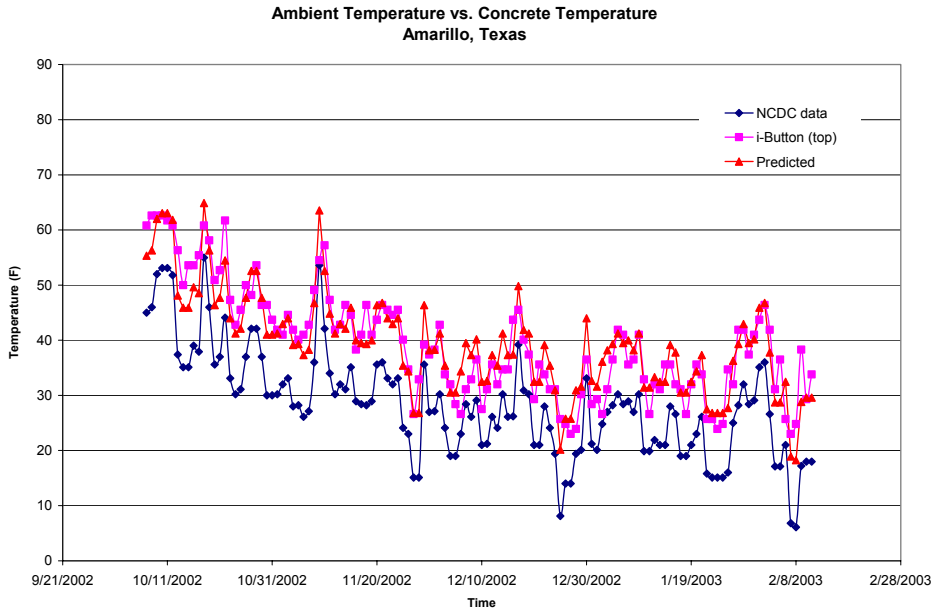


Figure 6.6 Ambient, i-Button, and predicted temperatures (top)

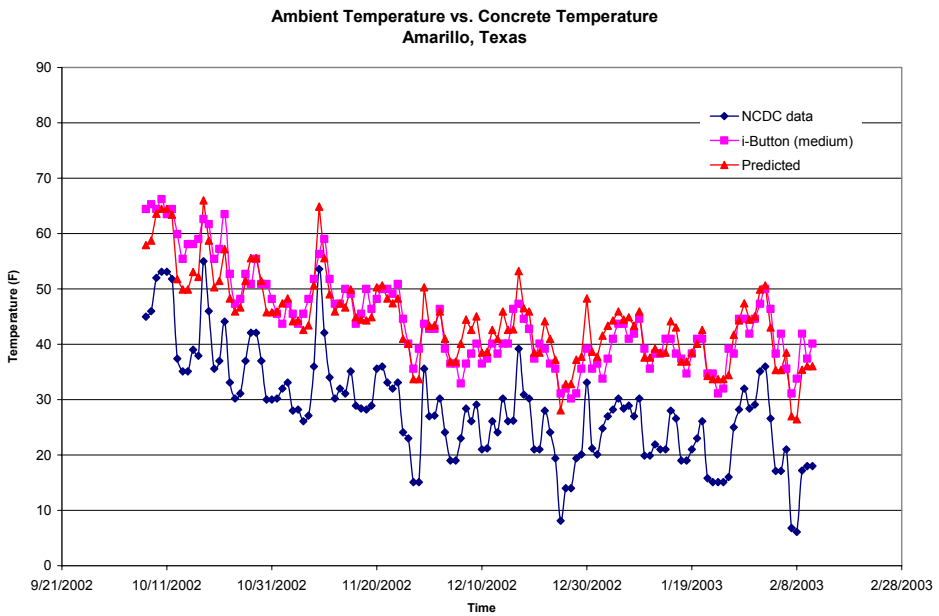


Figure 6.7 Ambient, i-Button, and predicted temperatures (mid-depth)

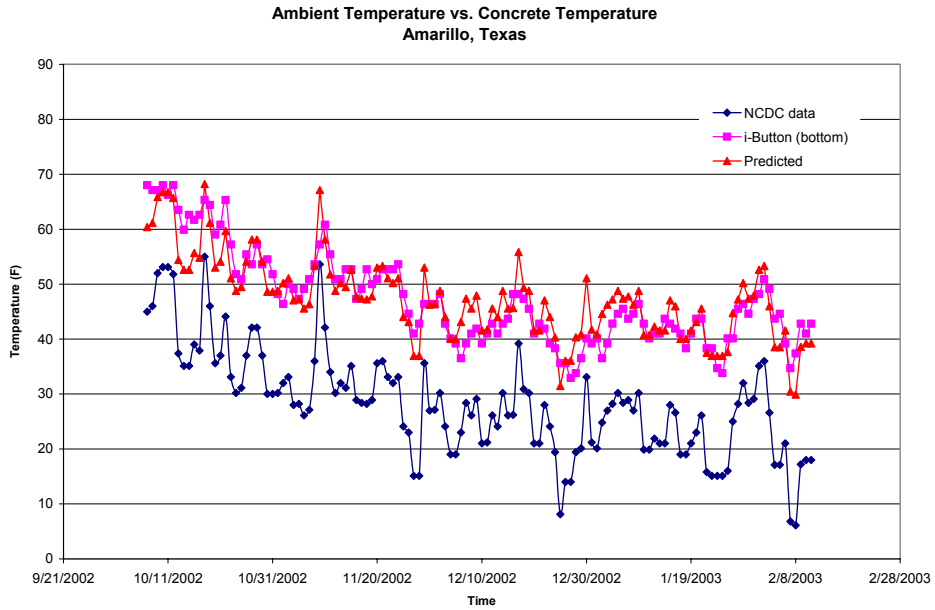


Figure 6.8 Ambient, i-Button, and predicted temperatures (bottom)

From Figures 6.6–6.8, it can be seen that the NCDC ambient temperature curve remains unchanged in position, but the i-Button and the predicted concrete temperature curves shift upward with an increase in depth. Table 6.1 presents the linear regression equations used for prediction of the concrete temperature and their corresponding correlation coefficient values.

Table 6.1 Equations and correlation coefficients R^2 for temperature prediction

| Slab Location | Regression Equation | Correlation Coefficient (R^2) |
|---------------|-------------------------|-----------------------------------|
| Top | $y = 0.9535 x + 12.424$ | 0.8365 |
| Mid-Depth | $y = 0.8094 x + 21.492$ | 0.8148 |
| Bottom | $y = 0.7836 x + 25.112$ | 0.7732 |

6.1.2 Multivariate Model

This model uses other climatic variables in addition to the ambient temperature variable used in the previous example. The parameters included in this multivariate model are ambient temperature, wind speed, and solar radiation, which have the most significant effects on hardened concrete pavement temperature. Because this model uses more input data, it is in fact more accurate; however, it is more difficult to adapt to specific geographic locations where no climatic data is available. The results provided by this model are better than those obtained from the model that uses only air temperature; unfortunately, solar radiation values are not available for many locations in Texas.

In the analysis presented here, solar radiation data were not available for the entire period of time analyzed, from October 2002 to February 2003; however, data were available for October 2002. These values were retrieved from the Texas Solar Radiation Database website (Ref 13). The analysis estimated the minimum concrete temperature evolution for the pavement slab instrumented in Amarillo. The input parameters included minimum daily ambient temperature, mean daily wind speed, and global solar radiation. Global solar radiation equals the sum of incident diffuse radiation plus the direct normal irradiance projected onto the horizontal surface of the earth (Ref 14).

Table 6.2 shows the parameters used to estimate the minimum concrete temperature of the pavement at the top fiber of the slab. Column 1 displays the date for which the parameters were retrieved or calculated, Column 2 contains the minimum daily concrete temperature as reported by the NCDC database, Column 3 corresponds to the mean daily wind speed also obtained from the NCDC database, Column 4 is the global solar radiation retrieved from the Texas Solar Radiation Database, Column 5 is the filtered minimum daily concrete temperature recorded with the i-Button, Column 6 corresponds to the minimum concrete temperature estimated or predicted by the multivariate model, and Column 7 represents the minimum concrete temperature predicted by the linear model, with the ambient temperature as input.

Table 6.2 Climatic variables used to predict minimum concrete temperature

| 1 | 2 | 3 | 4 | 5 | 6 | 7 |
|------------|----------------------------------|-----------------------|--|-----------------------------------|--|--|
| Date | Minimum Ambient Temperature (°F) | Mean Wind Speed (mph) | Global Solar Radiation (W/m ²) | Minimum Temperature i-Button (°F) | Predicted Temperature Multiple Regression (°F) | Predicted Temperature Simple Regression (°F) |
| 10/7/2002 | 45.0 | 9.7 | 113 | 60.80 | 57.05 | 55.33 |
| 10/8/2002 | 46.0 | 8.2 | 93 | 62.60 | 57.86 | 56.29 |
| 10/9/2002 | 52.0 | 6.5 | 108 | 62.60 | 62.93 | 62.01 |
| 10/10/2002 | 53.1 | 8.4 | 80 | 62.60 | 62.95 | 63.05 |
| 10/11/2002 | 53.1 | 13.8 | 199 | 61.70 | 63.41 | 63.05 |
| 10/12/2002 | 51.8 | 15.6 | 121 | 60.80 | 60.96 | 61.82 |
| 10/13/2002 | 37.4 | 7.8 | 158 | 56.30 | 52.42 | 48.08 |
| 10/14/2002 | 35.1 | 9.7 | 209 | 50.00 | 51.00 | 45.89 |
| 10/15/2002 | 35.1 | 7.5 | 203 | 53.60 | 51.40 | 45.89 |
| 10/16/2002 | 39.0 | 13.4 | 198 | 53.60 | 52.94 | 49.61 |
| 10/17/2002 | 37.9 | 9.8 | 157 | 55.40 | 52.34 | 48.56 |
| 10/18/2002 | 55.0 | 11.8 | 141 | 60.80 | 64.47 | 64.87 |
| 10/19/2002 | 46.0 | 10.4 | 121 | 58.10 | 57.77 | 56.29 |
| 10/20/2002 | 35.6 | 6.8 | 180 | 50.90 | 51.60 | 46.37 |
| 10/21/2002 | 37.0 | 10.5 | 186 | 52.70 | 51.93 | 47.70 |
| 10/22/2002 | 44.1 | 11.5 | 149 | 61.70 | 56.49 | 54.47 |
| 10/23/2002 | 33.1 | 14.2 | 39 | 47.30 | 46.11 | 43.98 |
| 10/24/2002 | 30.2 | 10.1 | 50 | 42.80 | 45.00 | 41.22 |
| 10/25/2002 | 31.1 | 4.8 | 150 | 45.50 | 48.26 | 42.08 |
| 10/26/2002 | 37.0 | 10.7 | 34 | 50.00 | 49.75 | 47.70 |
| 10/27/2002 | 42.1 | 11.2 | 121 | 48.20 | 54.67 | 52.57 |
| 10/28/2002 | 42.1 | 7.0 | 42 | 53.60 | 54.49 | 52.57 |
| 10/29/2002 | 37.0 | 9.0 | 177 | 46.40 | 52.12 | 47.70 |
| 10/30/2002 | 30.0 | 12.5 | 118 | 46.40 | 45.28 | 41.03 |
| 10/31/2002 | 30.0 | 9.4 | 41 | 43.70 | 44.89 | 41.03 |

The regression equations that were used to calculate Columns 6 and 7 are as follows.

Ambient temperature based model:

$$y = 0.9535A_T + 12.424 \quad (6.1)$$

Multivariate model:

$$y = 0.7485A_T - 0.2217W_S + 0.0140S_R + 23.9411 \quad (6.2)$$

where

A_T = minimum ambient temperature, °F

W_S = wind speed, mph

S_R = solar radiation, W/m²

Figure 6.9 displays the evolution of minimum daily temperatures during October 2002. The pink curve represents the ambient temperature and, as can be seen, its value is always lowest in comparison with the other curves. The dark blue curve corresponds to the i-Button-measured temperature, the light blue curve represents the temperature predicted with only the ambient temperature as input, and the orange line corresponds to the temperature predicted by the multivariate model.

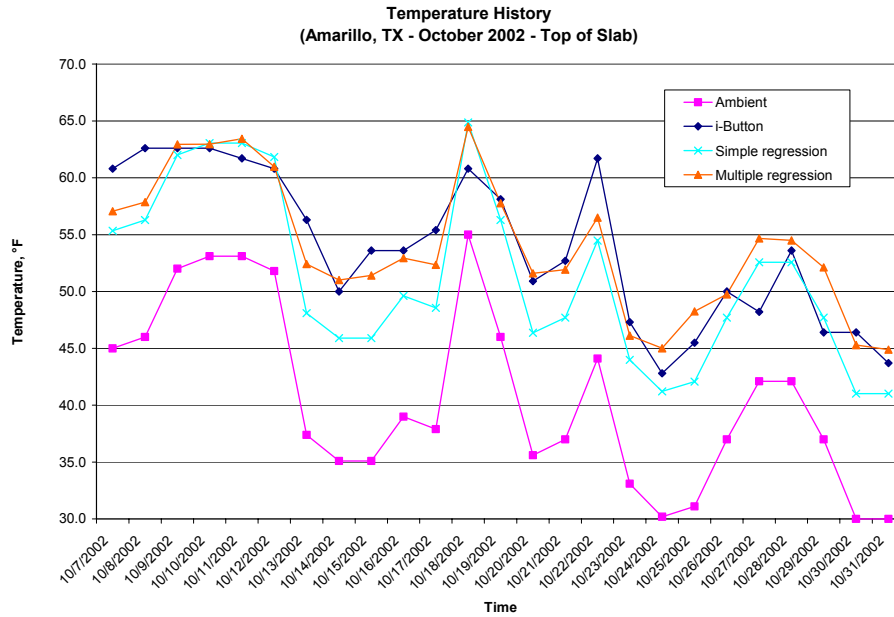


Figure 6.9 Concrete temperature history for top of slab (October 2002)

It can be inferred from Figure 6.9 that the following issues are relevant.

1. The ambient temperature obtained from the NCDC database is considerably lower than the real minimum concrete temperature as measured by the i-Buttons. This means that for design purposes, the use of the ambient temperature as a basis to estimate the value of the $\Delta T_{maximum}$ will result in oversized reinforcement steel contents that would account for that differential.
2. Both models—the one using only ambient temperature and the multivariate model—provide reasonable predictions of the concrete temperature.
3. The multivariate model provides the best representation of the real concrete temperature, especially when the slope of the temperature curve changes from negative to positive (sag locations).

Appendix B contains a set of graphs showing the findings for other all the North Texas locations. In all cases, the solar radiation value was the one from Amarillo.

6.2 Concrete Pavement Spalling

Chapter 4 in this report presented a model for estimating the DI of a pavement. In this model, spalling was incorporated into an existing model developed by Chou (Ref 8). To complement the information previously provided about the proposed model and to test its reliability, this section presents an example in which six pavement sections sampled from the Yoakum district were analyzed. The sections were surveyed during the 2002 collection effort, and their locations and characteristics (inventory information) are summarized in Table 6.3. In the table, *District 13* refers to the Yoakum District; *CFTR* corresponds to the CTR control number of the section in the RPDB; *County* is self-explanatory; *Hwy* is the highway where the sections are located; *RM1* and *RM2* correspond to the starting and ending reference markers, respectively; *DISP* is the displacement in miles; and *GPSLAT* and *GPSLON* are the geographic coordinates latitude and longitude, respectively. These acronyms and more can be found in previous reports related to this research project.

Table 6.3 Inventory data for sections sampled in the Yoakum District

| DISTRICT | CFTR | COUNTY | HWY | RM1 | DISPL | RM2 | DISPL | GPSLAT (N) | GPSLON (W) |
|----------|---------|----------|------|------|-------|------|-------|------------|------------|
| 13 | 13071-1 | FAYETTE | SH71 | 634 | 1.8 | 634 | 2 | 29.90944 | 96.90333 |
| 13 | 13071-2 | FAYETTE | SH71 | 636A | 0 | 636A | 0.2 | 29.91083 | 96.90056 |
| 13 | 13071-3 | FAYETTE | SH71 | 638A | 0 | 638A | 0.2 | 29.92333 | 96.56278 |
| 13 | 13071-4 | FAYETTE | SH71 | 646 | 0 | 646 | 0.2 | 29.87667 | 96.75389 |
| 13 | 13071-5 | COLORADO | SH71 | 652 | 0.2 | 652 | 0.4 | 29.81917 | 96.68917 |
| 13 | 13071-6 | COLORADO | SH71 | 654 | 0 | 654 | 0.2 | 29.80139 | 96.66806 |

The analysis of these sections was conducted in such a way that the resulting DI values could be compared with each other, assuming various conditions. The proposed model shown in Equation 4.6 was used, with the following assumptions.

1. All the spalls were minor.
2. All the spalls were severe.
3. The spalls were a mixture of both minor and severe (i.e., real conditions).

The results of the analysis are shown in Table 6.4. As has been explained previously, when the calculated PDI is equal to zero or negative, rehabilitation is recommended. None of the sections in Table 6.4 presented minor or severe punchouts; all the sections except one (13071-6) manifested patches, minor spalls, severe spalls, or a combination of all distresses.

Table 6.4 Estimation of PDI for different assumed and real conditions

| Section CFTR | Minor Punchouts | Severe Punchouts | All Patches | Minor Spalls | Severe Spalls | Calculated PDI, spalls being: | | |
|-----------------|--------------------|---------------------|----------------|-----------------|------------------|-------------------------------|--------|-------|
| | | | | | | Minor | Severe | Real |
| 13071-1 | 0 | 0 | 0 | 2 | 0 | 0.86 | 0.76 | 0.86 |
| 13071-2 | 0 | 0 | 1 | 0 | 0 | 0.71 | 0.71 | 0.71 |
| 13071-3 | 0 | 0 | 0 | 17 | 2 | 0.61 | 0.33 | 0.38 |
| 13071-4 | 0 | 0 | 1 | 12 | 9 | 0.30 | 0.02 | -0.14 |
| 13071-5 | 0 | 0 | 1 | 19 | 11 | 0.26 | -0.05 | -0.24 |
| 13071-6 | 0 | 0 | 0 | 0 | 0 | 1.00 | 1.00 | 1.00 |

From the results included in Table 6.4, the following observations are significant.

1. In all cases, the displayed occurrences of distresses were real observed values.
2. If all the spalls are assumed to be minor, the calculated PDI is always greater than zero, meaning that no rehabilitation is required or recommended for any of the sections.
3. In contrast, if all the spalls are assumed to be severe, the calculated PDI is almost always greater than zero, meaning that no rehabilitation is required or recommended. In this case only Section 13071-5 would have a PDI = -0.05, meaning that rehabilitation would be required.
4. When the real distress occurrences are put into the model, with both minor and severe spalls, four sections require no further treatment, but two sections (13071-4, PDI = -0.14, and 13071-5, PDI = -0.24) require rehabilitation.

It is thus observed that the model proposed for the estimation of the DI is sensitive to the ranking of the spalling. In other words, it matters whether a pavement has minor or severe

spalling, and furthermore, spalling is indeed a parameter that has to be included in the evaluation of pavement condition.

As complementary graphical information, Figures 6.10–6.13 display images of the conditions of the two sections that required rehabilitation, according to the proposed model.

Figure 6.10 shows a zone of Section 13071-4, which features a couple of cracks with minor spalling accumulated along the crack. Likewise, Figure 6.11 displays the average condition of severe spalls found in the same pavement section. Similarly, Figures 6.12 and 6.13 display two different views of the typical severe spalls found in Section 13071-5.



Figure 6.10 Panoramic view of Section 13071-4



Figure 6.11 Severe spalls found in Section 13071-4



Figure 6.12 Typical severe spalls found in Section 13071-5



Figure 6.13 Another view of the spalls found in Section 13071-5

In the analysis discussed and the images presented here, it is important to highlight that although it is not considered in current PDI models, spalling indeed represents a distress that negatively affects pavement performance. It was demonstrated that there is a great difference between minor and severe spalls in defining rehabilitation tasks for a given pavement. In addition, it is important to keep in mind that the combination of spalls and other types of distresses accelerate the deterioration of pavements. Further research is needed regarding the evolution of minor spalling and the factors that turn it into a severe distress.

6.3 Pavement Deflection: Case Study

Chapter 5 provided background about deflection of pavements, deflection measurement, and deflection analysis. This section focuses on the application of an analysis of deflection data collected for a particular section in the Atlanta District. The example is based on stress calculations and deflections measured at pavement cracks and at the midspan of pavement slabs. For the analysis, a stress ratio and a deflection ratio were computed in the manner described in Chapter 5. The stress ratio was calculated by using the concept of elasticity, applying the pavement layered theory, and includes the estimation of the stress

for two conditions: with the consideration of an overlay and without it. Regarding the deflection ratio, a comparison is made between deflections at cracks and deflections at midspans. This comparison allows an evaluation of the LTE of the pavement.

The section analyzed is a 3.4-mi. long CRCP section located on IH 20 in Harrison County, near Marshall, in the Atlanta District. Performance data from this particular section was used extensively in TxDOT Project 4398. The CTR has valuable information about this section, which is considered part of a satellite factorial. The deflection data for this section are especially valuable. It is known that the pavement has been overlaid with AC on several occasions; it is also known that one of those overlays was removed sometime in 2001. Likewise, it is documented that a new AC overlay was placed in December of that year. Fortunately for the purposes of the present study, deflection measurements were taken three different times during this period: before the old overlay was removed, while the CRCP was exposed after the overlay was removed, and right after the new overlay was placed. As a result, the cross-section of the pavement structure consists of an 8-in. thick CRCP, placed on top of a 7-in. thick layer of cement-stabilized subbase, placed on 6 in. of cement-treated subgrade. The AC overlay placed in December 2001 is 4 in. thick. Figure 6.14 illustrates the cross-sectional view of the structure.

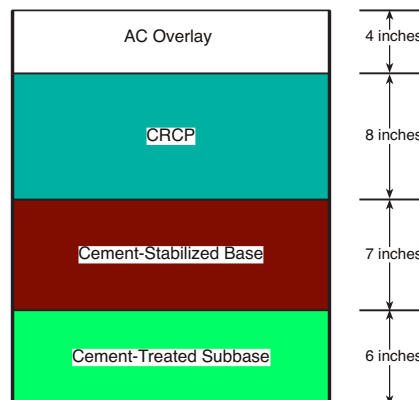


Figure 6.14 Cross-section of the pavement on IH 20 in Harrison County

Table 6.5 presents the layer properties of the pavement and the stresses calculated by using the elasticity theory for stress calculations in pavements. These calculations were

performed using the ELSYM-5 program (Ref 15); the seed values for moduli of elasticity for the layers were backcalculated from FWD measurements. The calculation of stress and deflection ratios is summarized in Table 6.6.

Table 6.5 Backcalculated properties of pavement using ELSYM-5

| Layer | Thickness (in.) | Modulus of Elasticity (ksi) | Poisson's Ratio | Stresses With Overlay (psi) | Stresses Without Overlay (psi) |
|------------|--------------------|-----------------------------|-----------------|-----------------------------|--------------------------------|
| AC overlay | 4 | 200 | 0.35 | 21.5 | 27.46 |
| CRCP | 8 | 1,000 | 0.22 | | |
| CTB | 7 | 500 | 0.30 | | |
| CTSB | 6 | 150 | 0.35 | | |
| Fill | 6 | 30 | 0.40 | | |
| Subgrade | $\frac{\infty}{2}$ | 15 | 0.40 | | |

Table 6.6 Summary of stress and deflection ratios calculation

| Stress Ratio | | |
|-------------------------------|-------|--------------|
| <i>Stresses (psi)</i> | | <i>Ratio</i> |
| Stresses without overlay | 21.50 | 1.28 |
| Stresses with overlay | 27.46 | |
| Deflection Ratio | | |
| <i>Deflection (mils)</i> | | <i>Ratio</i> |
| WB Outside Lane: | | |
| Average deflection at cracks | 3.08 | 1.06 |
| Average deflection at midspan | 2.90 | |
| WB Inside Lane: | | |
| Average deflection at cracks | 3.24 | 1.15 |
| Average deflection at midspan | 2.83 | |

The results from Table 6.6 were plotted in Figure 6.15 with the value of 1.0 as a reference for both the stress ratio and the deflection ratio. Any departure from these reference values suggests the need for a more structural solution to the rehabilitation decision (Ref 11). For

this highway section of IH 20, the deflection ratios for both the outside and inside lanes were fairly close to 1.0 (reference value), suggesting that the load transfer in the CRCP is good and the pavement is structurally sound. Likewise, the stress ratio of 1.28 displayed in Table 6.6 indicates that the structural contribution of the overlay to the existing CRCP will be very small.

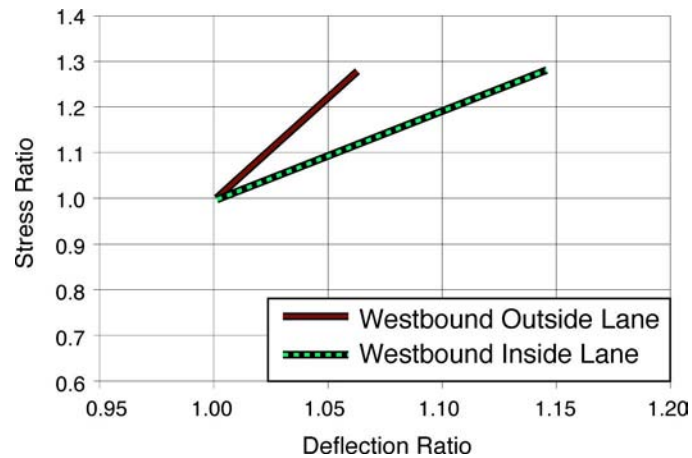


Figure 6.15 Stress and deflection ratios for IH 20 section near Marshall

It is acknowledged that the one case study presented here is not sufficiently statistically significant to establish a general guideline for pavement deflection evaluation. However, from this analysis, it was found that the results from the IH 20 section studied in this project correspond to what was expected for a thin AC overlay deflection criterion. Therefore, for this particular case, the deflection criterion threshold values for an AC overlay can be defined with the amount of information available to this point as a stress ratio between 1.0 and 1.3 and a deflection ratio between 1.0 and 1.2. Ratio values greater than these will require the consideration of either a BCO or, probably, an unbonded concrete overlay.

The deflection decision criterion presented here implies a structural evaluation of the existing pavement, but it also engages a theoretical assessment of a future overlay and its structural contribution, with the computation of stresses. This deflection criterion will indicate whether an AC overlay would be a good solution, if both the deflection ratio and

the stress ratio are close enough to 1.0, or whether a more structural remedy is necessary. In general, a considerable departure from a value of 1.0 for both ratios signifies that the pavement needs an overlay that can provide more structural benefits than an AC overlay.

7. Conclusions and Recommendations

7.1 General Conclusions

The activities conducted for this project have been limited during the last fiscal year, basically owing to budgetary constraints. As it was originally planned and stated in a previous report, more recently constructed CRCP sections with thicknesses over 10 in. were to be added to the RPDB to represent those pavements; however, not many have been surveyed. Fortunately, the few field trips that have been scheduled have served to collect new information and data that improve the quality of the database in many ways and also provide supporting information for other research projects.

Although the field tasks for the project have been limited, the use of the information collected for the project has been extended. This report presents a set of concepts developed and proposed by the RPDB project. These concepts are fully described in the report, and examples are provided to illustrate the usefulness of the data contained in the database.

This report is presented in an organized scheme in which the topics that are to be treated are first explained in a single chapter, and then each of those topics is covered in a subsequent chapter, in which deeper information is revealed. Chapter 2 provides an introduction to three different concepts that have been of particular interest to TxDOT. First, the effect of climatic variables on concrete pavement performance was discussed. The effect of ambient temperature and other variables in the evolution of concrete pavement temperature was analyzed, because it is a critical design parameter for rigid pavements. Second, a revision of the currently available distress prediction model was performed. The proposed new model includes the effect of minor and severe spalls on the pavement. This distress has been found to deteriorate concrete pavements across the state, and therefore, it was necessary to include it in the objective pavement rating model used previously. Finally, the third concept is related to pavement deflection and its consideration for rehabilitation of rigid pavements. None of these three concepts could have been developed without the information provided by the RPDB.

7.2 Summary of Research Findings

The work developed under this research project and presented in this report has provided excellent tools that will aid in a better understanding of concrete pavement behavior. The following are some of the most relevant findings of this work.

- Climatic variables such as ambient temperature, wind speed, and solar radiation have a tremendous effect on the performance of concrete pavements, in addition to other material, construction, and loading conditions.
- It is possible to use prediction models to estimate the minimum concrete pavement temperature that will be experienced in the medium and long terms. The minimum concrete temperature has an influence on design results and, therefore, it should be estimated by using either the algorithms proposed in this report or any other method. The use of only conservative “assumed” minimum temperature values will continue to result in more expensive, oversized pavements.
- Two models were proposed in this report to predict the minimum concrete design temperature. Both models are reliable, and they are recommended for use on the basis of the available climatic data for a particular design. Furthermore, these models can be implemented on a statewide basis, and three climatic regions were proposed within this report.
- A new pavement DI was presented. It is an improvement over the existing model developed in the 1980s. The proposed model considers the effect of spalling in severity and extent. This index can be applied at a project level and can be used as a good guideline for pavement rehabilitation decisions.
- It is important to differentiate minor spalling from severe spalling. This report proposed a criterion to differentiate between the two types of distress on the basis of dimensional characteristics—something not found in the literature.
- It was determined that the deflection data contained in the database is useful, though outdated. Deflection basins are available mostly for 8-in. thick

pavements. By using these data, it is possible to determine whether the structural support for a CRCP is good or poor for any given district.

- Interpretation of deflection data was performed, and the concepts of deflection ratio and stress ratio were introduced. The deflection ratio is computed by using the deflection values at the crack and at the midspan of a slab. The stress ratio is the relationship that exists between the stresses that act on a pavement without an overlay and the stresses on the pavement with an overlay. The combination of these two ratios aids in determining whether a pavement requires an overlay and in determining which type of overlay might be required.

7.3 Recommendations

The success of the RPDB is based on its dynamic nature. Many data-collecting activities are periodic and repetitive; however, the way in which the data are analyzed makes a great difference. In the course of this project, many research tasks have been proposed, and various ways to analyze the data in the RPDB have been discussed. Because of the way technology advances and the needs of pavements change, there are still some activities that could be performed that would result in the improvement of this project and its products.

Depending on the duration of this project and its possible extension, various interesting activities could be conducted in the near future to improve the quality of the database and its applicability. The following are some plans that could be accomplished in the short term.

1. It might be necessary to reorganize the entire database. The current RPDB is organized into two main components: JCP sections and CRCP sections. Although JCP construction is not so common anymore, it is important to keep the information about those sections in the database. For CRCP, it is important to continue adding more recently constructed pavements that are over 8 in. in thickness. Most continuous pavement construction these days

includes 12- to 14-in. thick pavements, and it would be necessary to fairly represent those pavements in the database. The old thin CRCPs do not simulate the performance of the thick pavements constructed more recently.

2. The information that is available about the sections is varied. The majority of the sections in the RPDB are identified by only inventory and performance data collected from time to time. Detailed information is available for selected sections that are part of the so-called *satellite factorials*. It might be necessary to differentiate between those types of sections in the database. Probably levels of information should be adopted, and the level of the section would indicate the type of information that is available for that section. For instance, a Level 1 section would be one for which very detailed information is available, and a Level 3 section would be a section for which only basic information was collected.
3. Information that would improve the quality of the data include deflection, traffic, and crack width. These three parameters could be collected for selected sections in addition to the conventional information gathered.

This report describes a series of tasks conducted under TxDOT Project 1778. The information contained in this report is the result of the work performed by researchers working on this project, and the ideas presented here are based on discussions held with TxDOT staff. This report is the fifth of a series of six, and it supplements the information provided in previous reports. For a better understanding of the development of this project, the reader is referred to the available literature.

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Appendix A

Temperature Raw Data for one i-Button in Each Location in North Texas (Amarillo, Wichita Falls, and Childress)

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12/22/2002 17:52, 46.4°F
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12/22/2002 22:22, 40.1°F
12/22/2002 23:52, 39.2°F
12/23/2002 01:22, 39.2°F
12/23/2002 02:52, 38.3°F
12/23/2002 04:22, 37.4°F
12/23/2002 05:52, 33.8°F
12/23/2002 07:22, 32.0°F
12/23/2002 08:52, 32.0°F
12/23/2002 10:22, 32.0°F
12/23/2002 11:52, 27.5°F
12/23/2002 13:22, 35.6°F
12/23/2002 14:52, 45.5°F
12/23/2002 16:22, 46.4°F
12/23/2002 17:52, 41.0°F
12/23/2002 19:22, 34.7°F
12/23/2002 20:52, 32.0°F
12/23/2002 22:22, 27.5°F
12/23/2002 23:52, 26.6°F
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12/24/2002 02:52, 31.1°F
12/24/2002 04:22, 34.7°F
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01/12/200


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Mission State
-----
Mission is in progress
Sample rate: 90 minute(s)
Roll-Over Enabled: no
Roll-Over Occurred: n/a
Mission Start time:
10/06/2002 15:03
Mission Start delay: 0
minute(s)

Mission Samples: 3857
Device total samples: 317656
Temperatures displayed in:
(Fahrenheit)
High Threshold: 185.0°F
Low Threshold: -40.0°F
Temperature threshold alarm
state: Conditional(none) Alarm(none)
Current Real-Time Clock from
DS1921: Wednesday 06/04/2003
15:42:16
Current PC Time: Wednesday
06/04/2003 15:44:22
Time Alarm mode: Alarm
weekly
Alarm Time: Sunday 00:00:00
Alarm Time
state:
Conditional(yes) Alarm(yes)

-----
Format: [(HIGH/LOW),
Time/Date range]
none

Temperature Histogram
-----
Format: [Temp Range, Count]
(Fahrenheit)
-40.0°F to -37.3°F, 0
-36.4°F to -33.7°F, 0
-32.8°F to -30.1°F, 0
-29.2°F to -26.5°F, 0
-25.6°F to -22.9°F, 0
-22.0°F to -19.3°F, 0
-18.4°F to -15.7°F, 0
-14.8°F to -12.1°F, 0
-11.2°F to -8.5°F, 0
-7.6°F to -4.9°F, 0
-4.0°F to -1.3°F, 0
-0.4°F to 2.3°F, 0
3.2°F to 5.9°F, 0
6.8°F to 9.5°F, 0
10.4°F to 13.1°F, 0
14.0°F to 16.7°F, 0
17.6°F to 20.3°F, 0
21.2°F to 23.9°F, 0
24.8°F to 27.5°F, 0
28.4°F to 31.1°F, 0
32.0°F to 34.7°F, 18
35.6°F to 38.3°F, 74
39.2°F to 41.9°F, 151
42.8°F to 45.5°F, 337
46.4°F to 49.1°F, 413
50.0°F to 52.7°F, 388

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 02/11/2003 15:03, 48,2°F

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Mission State
-----
Mission is in progress
Sample rate: 90 minute(s)
Roll-Over Enabled: no
Roll-Over Occured: n/a
Mission Start time:
10/06/2002 16:02
Mission Start delay: 0
minute(s)

Mission Samples: 3910
Device total samples: 3910
Temperatures displayed in:
(Fahrenheit)
High Threshold: 185.0°F
Low Threshold: -40.0°F
Temperature threshold alarm
state: Conditional(none) Alarm(none)
Current Real-Time Clock from
DS1921: Sunday 06/08/2003 00:34:25
Current PC Time: Thursday
06/12/2003 18:46:19
Time Alarm mode: Alarm
weekly
Alarm Time: Sunday 00:00:00
Alarm Time state:
Conditional(yes) Alarm(yes)

Temperature Alarms
-----
Format: [(HIGH/LOW),
Time/Date range]
none

Temperature Histogram
-----
Format: [Temp Range , Count]
(Fahrenheit)
-40.0°F to -37.3°F, 0
-36.4°F to -33.7°F, 0
-32.8°F to -30.1°F, 0
-29.2°F to -26.5°F, 0
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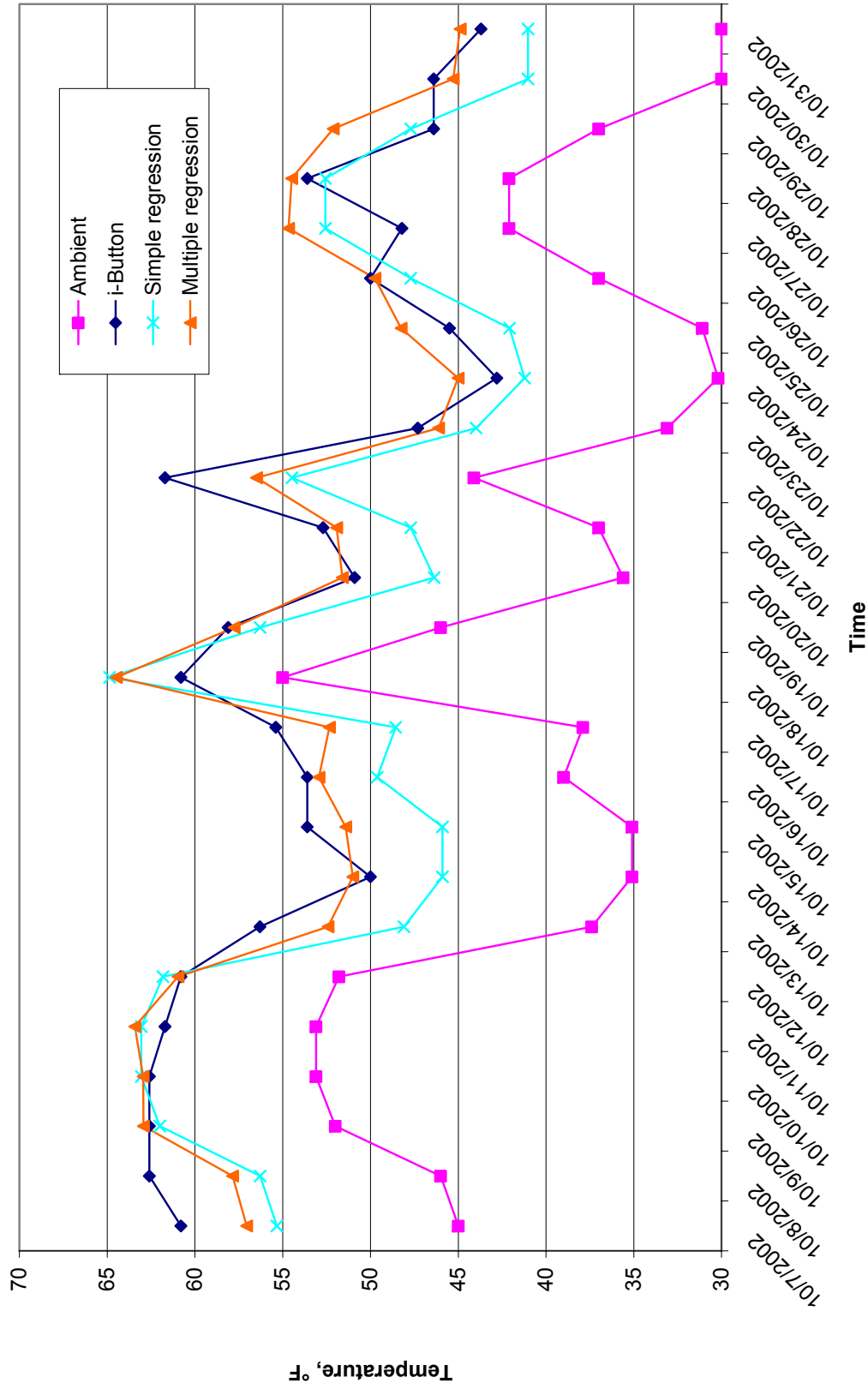
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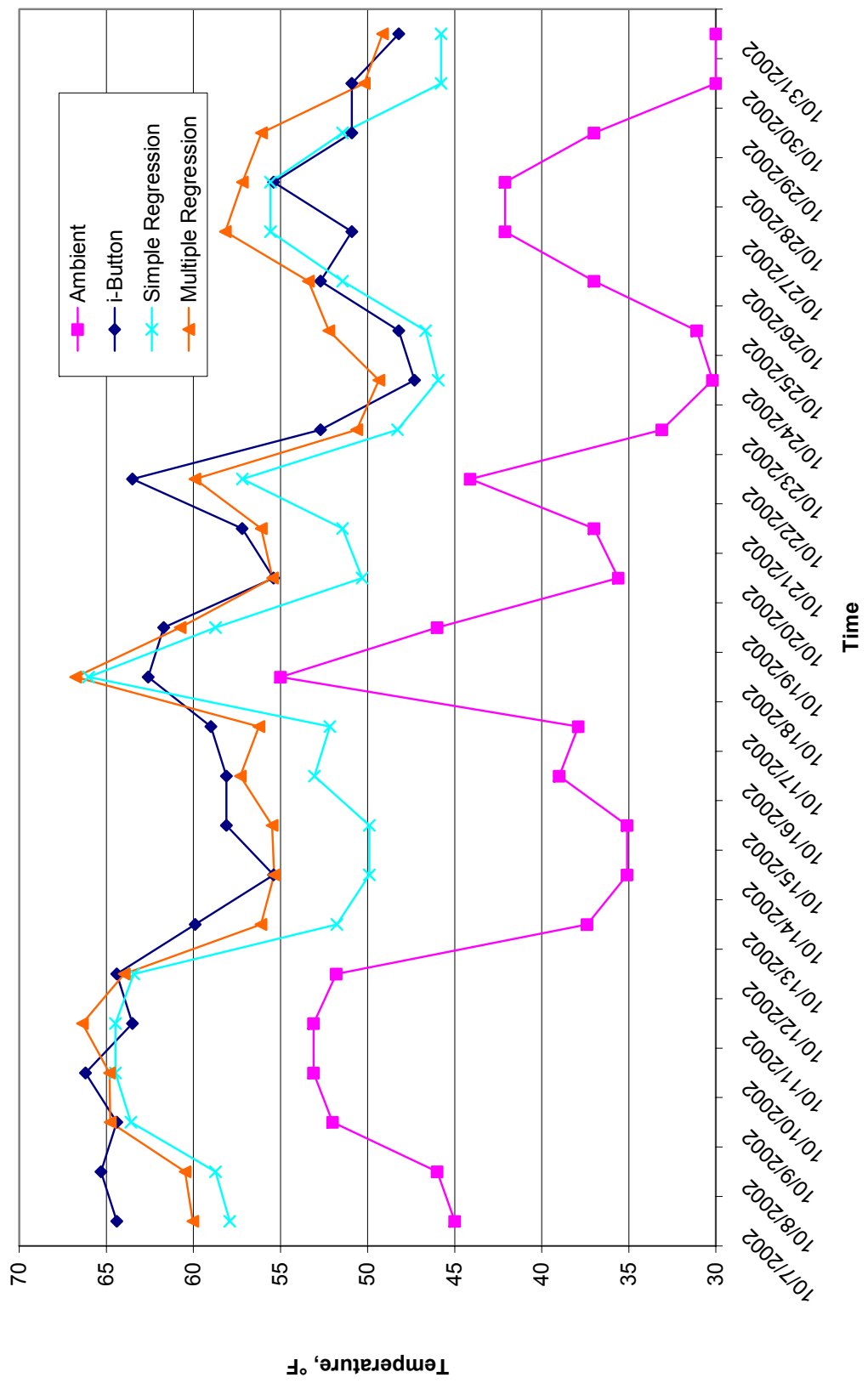
Appendix B

Graphs Showing Findings of Prediction of Minimum Concrete Pavement Temperature

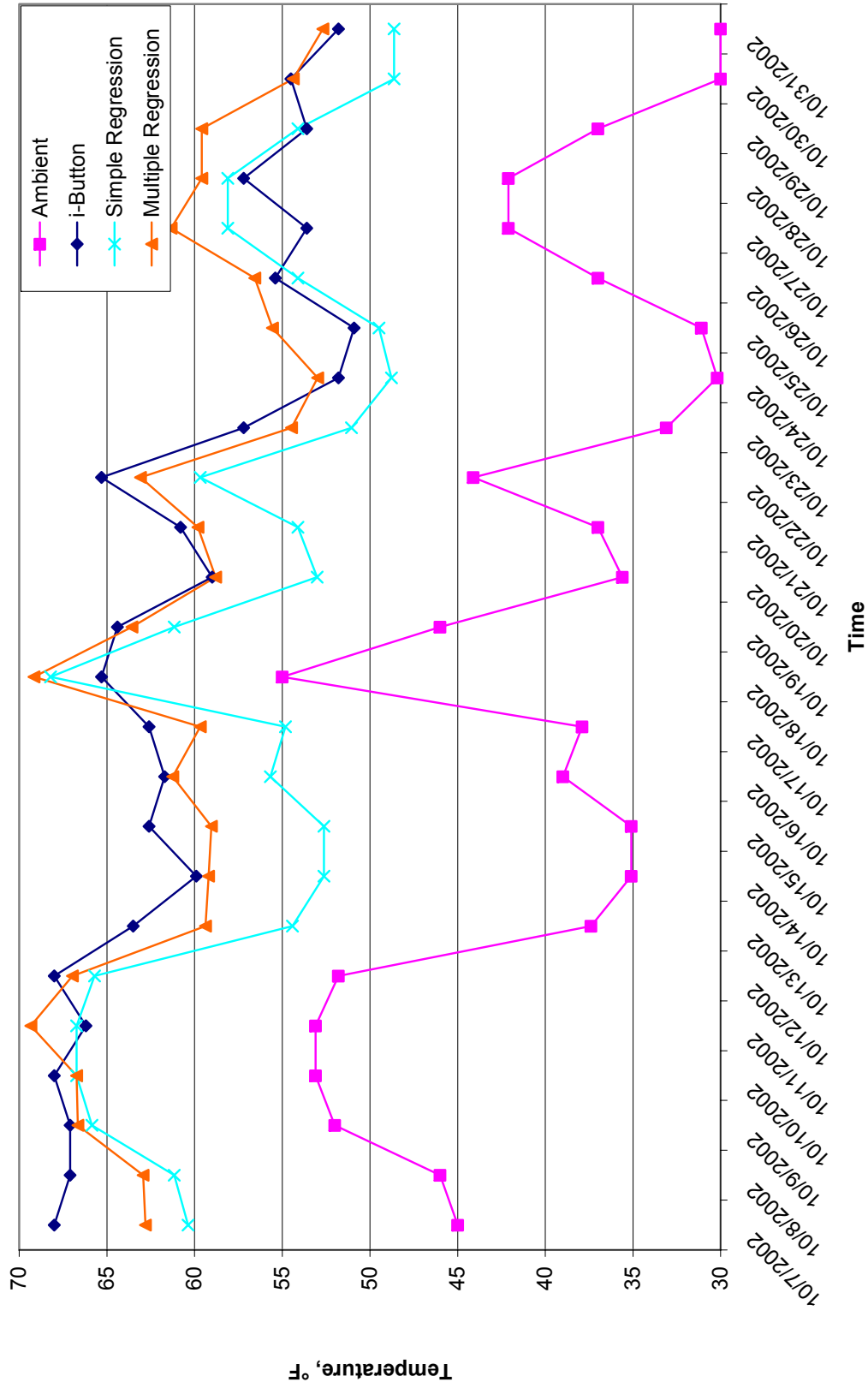
Temperature History
 (Amarillo, TX - October 2002 - Top of Slab)



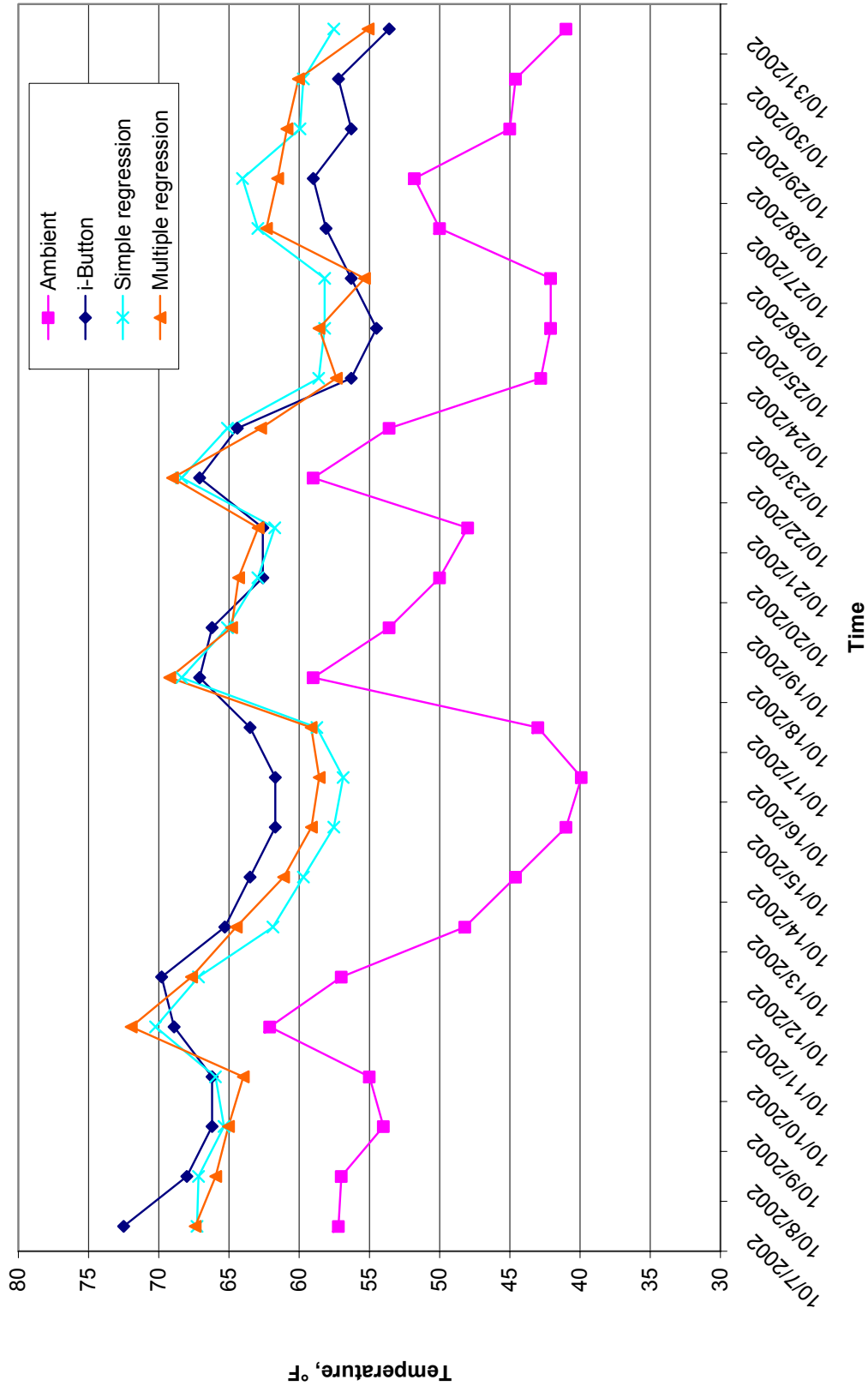
Temperature History
 (Amarillo, TX - October 2002 - Mid Depth of Slab)



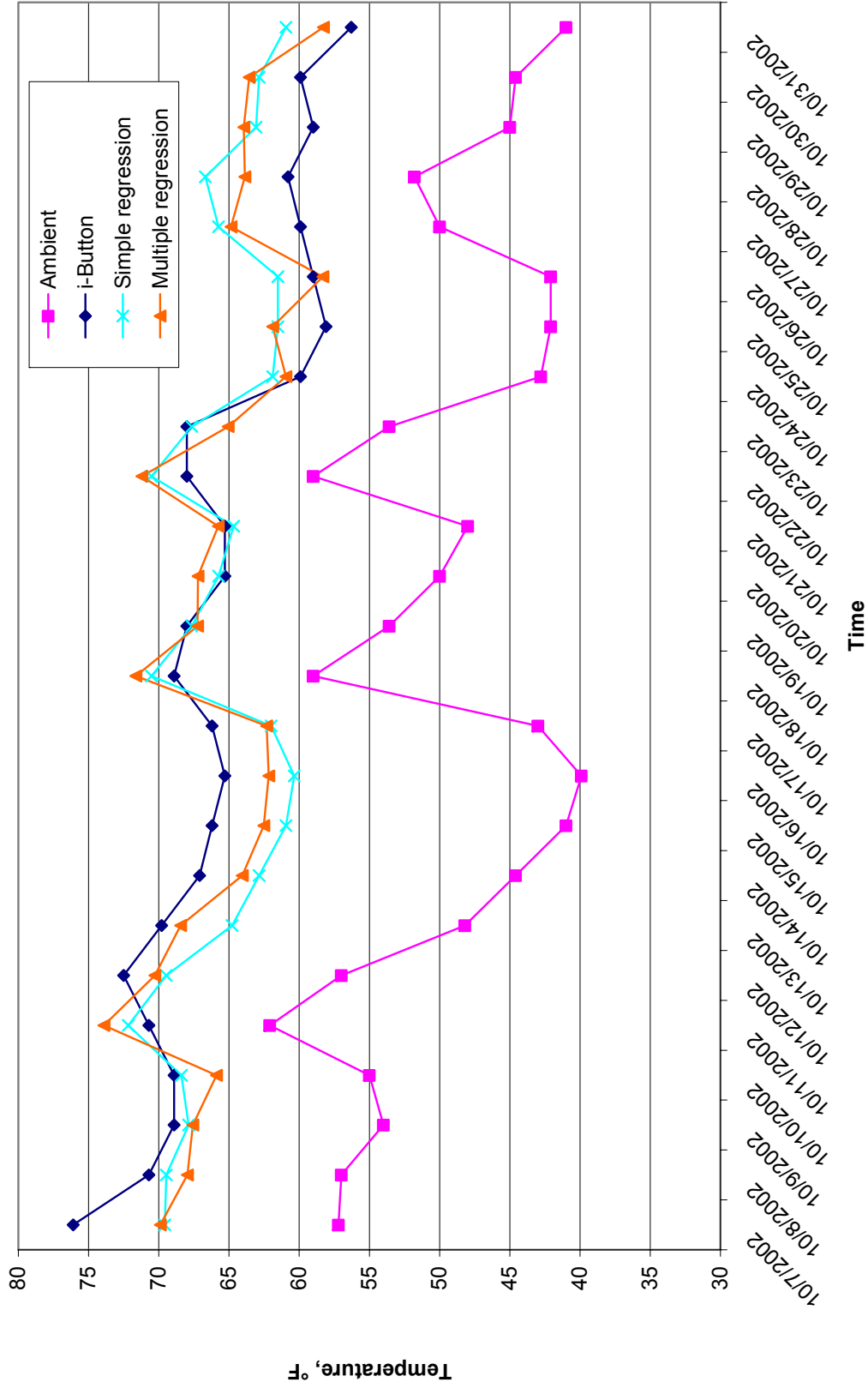
Temperature History
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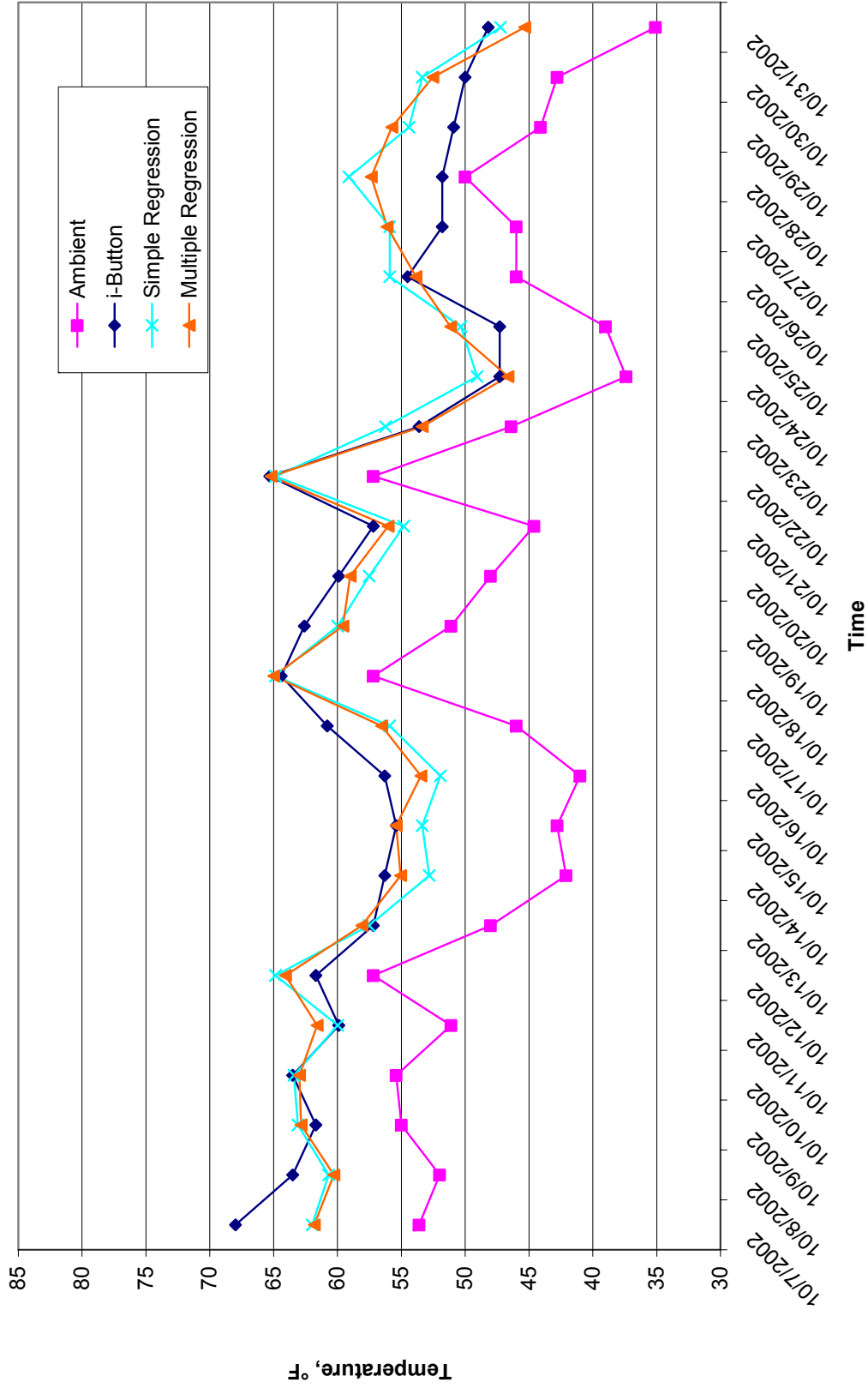
Temperature History
(Wichita Falls, TX - October 2002 - Mid Depth of Slab)



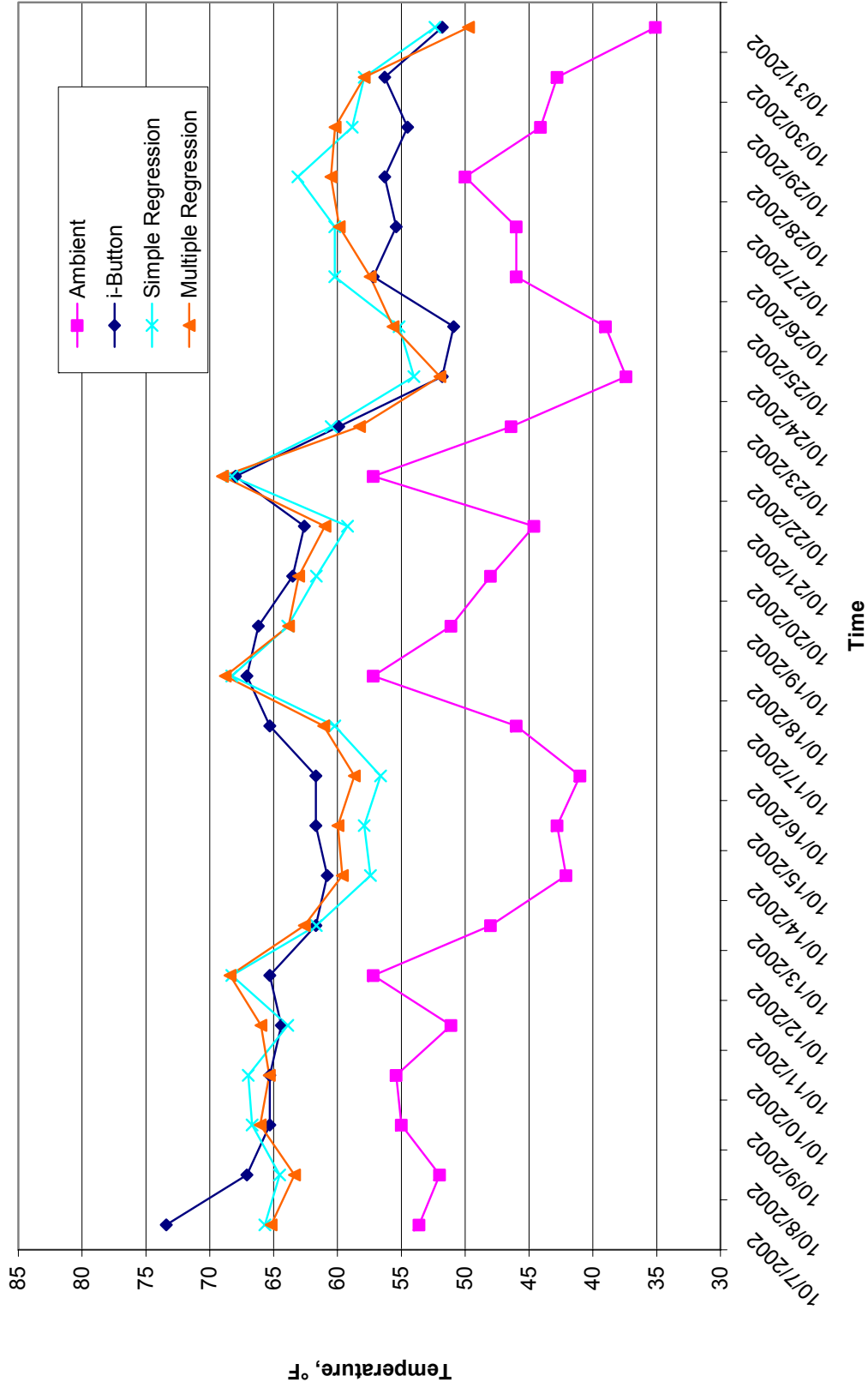
Temperature History
(Wichita Falls, TX - October 2002 - Bottom of Slab)



Temperature History
 (Childress, TX - October 2002 - Top of Slab)



Temperature History
 (Childress, TX - October 2002 - Mid Depth of Slab)



Temperature History
 (Childress, TX - October 2002 - Bottom of Slab)

