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16. Abstract

The existing rigid pavement deflection data contained in the PES database are evaluated and found to be inadequate for any network-level study of the structural behavior of rigid pavements. The PES data were evaluated by comparing them with existing data contained in the CTR rigid pavement database. Having found the data to be questionable, we performed no further analysis. For the network evaluation of the structural behavior of rigid pavements, we provide recommendations for future FWD data collection for rigid pavement at the network level. The optimum sample size, the testing procedures, and a cost estimate for the data collection plan are given.

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NETWORK-LEVEL DEFLECTION DATA COLLECTION FOR RIGID PAVEMENTS

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W. R. Hudson

Research Report 1908-3

Research Project 3-18-92/3-1908

Texas Pavement Management System

conducted for the

Texas Department of Transportation

by the

CENTER FOR TRANPORTATION RESEARCH

Bureau of Engineering Research

THE UNIVERSITY OF TEXAS AT AUSTIN

July 1994

IMPLEMENTATION STATEMENT

Because an evaluation of the existing rigid pavement deflection data in the PES database shows these data to be of questionable quality, it is suggested that they be removed from the database. The FWD data collection procedure for rigid pavement developed in this study provides a basis for collecting the data items needed for network evaluation of the structural behavior of rigid pavements.

Prepared in cooperation with the Texas Department of Transportation.

DISCLAIMERS

W. R. Hudson, P.E. (Texas No. 16821)
Research Supervisor

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SUMMARY

The existing rigid pavement deflection data contained in the PES database are evaluated and found to be inadequate for any network-level study of the structural behavior of rigid pavements. The PES data were evaluated by comparing them with existing data contained in the CTR rigid pavement database. Having found the data to be questionable, we performed no further analysis. For the network evaluation of the structural behavior of rigid pavements, we provide recommendations for future FWD data collection for rigid pavement at the network level. The optimum sample size, the testing procedures, and a cost estimate for the data collection plan are given.

CHAPTER 1. INTRODUCTION

1.1 BACKGROUND

The nation's highway infrastructure requires considerable investment annually to preserve its condition and to improve the service it provides to the traveling public. And because pavements are a major part of the highway system, a large portion of this investment is used for the construction, maintenance, and rehabilitation of pavements. In the state of Texas alone, approximately $1 billion is spent annually on pavements (Ref 1). Accordingly, the need to allocate these pavement-related funds wisely has led to much pavement management research being undertaken over the past 30 years (Ref 2). Chief among the results of such research has been the development of pavement management systems (PMS).

A pavement management system is designed to assist transportation administrators in allocating funds to design, construct, and maintain pavements. The need for the efficient management of the more than 112,700 km (70,000 miles) of pavements in Texas has prompted the Texas Department of Transportation (TxDOT) to develop the Pavement Management Information System (PMIS), which will perform the tasks required of a PMS. This PMIS will maintain a pavement database that supports such analyses as needs estimates, network optimization, and maintenance program and funding level impacts (Ref 1).

A PMS is typically operated at two levels: the network level and the project level. A network-level PMS regards all pavements in the network as a group. The network-level PMS helps to optimize the allocation of funds for maintenance, rehabilitation, and construction to maximize the resulting benefits over a set or group of projects. To accomplish this, data are collected from the pavements in the network to identify trends in pavement performance; these trends are then used to predict the condition of the network under various funding levels and/or maintenance and rehabilitation strategies.

The project-level PMS deals with individual pavement sections, analyzing each to select the most cost-effective design or maintenance procedure. A major contrast between the two levels is the detail of the data required. At the project level, detailed data about the selected pavement are needed to assure good design. At the network level, sampled data covering a representative portion of the network can be used to estimate the condition of the network and to predict performance.

Required for the network-level PMS is information on the structural behavior of the pavements. This behavior is most easily and economically defined through deflection measurements. The deflection of the pavement can be measured with many different instruments, among them the Falling Weight Deflectometer (FWD). FWD data can be an important tool in network-level pavement management.

1.2 STUDY OBJECTIVES

After presenting background information relating to the measurement of rigid pavement deflections using the FWD, we evaluate existing network-level data available in the Texas

Pavement Evaluation System (PES) database. Using the background and evaluation of the existing data, we then make recommendations for the implementation of a productive, cost-effective, network-level FWD data collection plan for rigid pavements in Texas.

1.3 SCOPE AND STUDY ORGANIZATION

The Center for Transportation Research (CTR) at The University of Texas at Austin is participating with TxDOT in the development of a statewide PMS. This report is part of a major ongoing research project whose objective is to assist TxDOT in the implementation of the PMS for rigid pavements in Texas.

Chapter 1 of this report presents the background, scope, and objectives of this study. Chapter 2 reviews the background for the measurement of rigid pavement structural behavior, discussing in particular the significant factors that affect deflection measurements of rigid pavements (e.g., temperature and load placement). It shows that these factors are essential for the understanding and analysis of deflection measurements for portland cement concrete pavements. Also discussed are typical analysis techniques used for rigid pavement deflection data.

Chapter 3 summarizes evaluations of existing rigid pavement deflection data available in the Texas PES database for use in the analysis of rigid pavement behavior and structural capacity. Chapter 4 then presents a data collection plan for a network-level rigid pavement FWD data collection program. The program will account for the significant factors discussed in Chapter 2, so that the data can be used for comprehensive, network-level analyses of rigid pavement behavior in the future. Recommendations for sampling methods, optimum sample sizes, and testing procedures are also made.

Chapter 5 presents a cost estimate for the data collection plan recommended in Chapter 4. Finally, Chapter 6 presents the study's conclusions and recommendations.

CHAPTER 2. STRUCTURAL EVALUATION OF RIGID PAVEMENTS AT THE NETWORK LEVEL USING THE FALLING WEIGHT DEFLECTOMETER

2.1 INTRODUCTION

The primary purpose of a network-level PMS is to adequately prioritize the funds used to maintain, construct, plan, and rehabilitate a pavement network for the maximum possible benefit. This network-level PMS can be divided into three important subsystems: the information subsystem, the network analysis subsystem, and the implementation subsystem (Ref 2).

The information subsystem is the necessary first step that collects and maintains the data needed to support the second step, the analysis subsystem. These data pertain to condition surveys, traffic, construction and maintenance costs, geometry, and inventory. In addition, the information subsystem involves the identification of homogeneous pavement sections, and the determination of limiting values of criteria for decisionmaking (Ref 2).

The analysis subsystem uses the information subsystem to make economic and technical analyses of various maintenance, design, and rehabilitation strategies. The analysis subsystem should project, for various analysis periods and alternative maintenance and rehabilitation strategies, the economic and non-quantifiable benefits of these programs (in a way that allows comparison by the decisionmaker). The data and analyses used in this subsystem can be more approximate in nature than those used in a project-level system (Ref 2). For that reason, data collection and storage, because they are performed on such a large scale, must be well-organized and should yield accurate, dependable data.

The implementation subsystem includes budget constraints to assist the decisionmaker using the PMS. Its outputs are the alternative final programs and schedules for new construction, maintenance, and rehabilitation projects within the analysis period (Ref 2).

The discussion so far should indicate that the maintenance and collection of quality data are at the core of the PMS. Typically, the data collected are used to create prediction models for the pavement network. Some models relate to the prediction of serviceability and roughness trends, while others are prediction models for distress. For rigid pavements, especially CRC sections, roughness has not been found to be a good indicator of pavement performance (Ref 3). Therefore, more effort has focused on finding and using other methods of evaluating rigid pavement performance.

At present, the Texas Department of Transportation collects FWD deflection measurements at the network level for flexible pavements only. These measurements are used to calculate the Structural Strength Index (SSI), an index meant to represent the section's susceptibility to load-related damage (Ref 4). This index was created to address a shortcoming in the PES that allowed pavements that are structurally weak to be eliminated as candidates for rehabilitation (i.e., stopgap maintenance procedures — among them seal coats and thin overlays — had maximized pavement scores based on distress counts; see Ref 4). The use of the SSI for flexible sections has enabled the pavement manager charged with programming rehabilitation strategies to account for structural deficiencies in the pavements.

The collection of deflection data at the network level can also be a valuable tool for rigid pavements as well. These data can provide the means for constructing a structural adequacy index similar to the SSI used for flexible sections. It can also be used for future research on, and improvement of, present PMS functions. This chapter describes important factors affecting the collection of FWD data on rigid pavements — factors that need to be addressed in the design and use of a large-scale deflection data collection program.

2.2 COMPARISON OF FWD TO DYNAFLECT FOR NETWORK-LEVEL DATA COLLECTION

Non-destructive testing (NDT) devices can indicate how the pavement will behave under a load. The advantage of a non-destructive test is that the pavement structure is not disturbed during the test. The many NDT devices used for pavement evaluation range from instruments that measure deflection of the pavement under a static or slow moving load (e.g., the Benkelman beam), to instruments that produce impulse loading (e.g., the FWD). To determine which should be used for a data collection program, it is necessary to compare the benefits of the considered instruments.

Two NDT devices that have been used in Texas are the Dynaflect and the FWD. Geophones (or velocity transducers) serve as the deflection measuring sensors on each machine. The major difference between the FWD and the Dynaflect is the method by which the load is produced. The Dynaflect contains two counter rotating masses that produce steady-state sinusoidal vibrations. The peak load difference transferred to the pavement is 654 kg (1000 lb) at a frequency of 8 Hz. The FWD operates by dropping a known mass from a known height onto a spring buffer system that produces an impulse load. The impulse load is measured using a load cell at the center of the loading plate.

In comparing these two devices for use on rigid pavements, Eagleson (Ref 5) noted that, at the time of his evaluation, the major difference between the operation of the two devices was the need for manual sensor placement of the geophones on the FWD (the FWD has since been automated). Eagleson's comparison also yielded the following:

1. The Dynaflect and FWD produce nearly equivalent results when evaluated on the basis of operational characteristics and cost.
2. The Dynaflect's major advantage is the large existing empirical database relating Dynaflect measurements to performance (November 1982).
3. The major advantages of the FWD lie in its larger load magnitude and its ability to produce a variable load.
4. The variable load of the FWD potentially enables the detection of stress sensitivity of the subgrade in the field.
5. The Dynaflect and the FWD were highly correlated, indicating that the two devices would yield similar results.

As experience in Texas has determined, the FWD has other advantages over the Dynaflect: Because the FWD has sensors at locations further away from the point of load application, it can provide a better estimation of subgrade strength for rigid pavements. Moreover, since the publication of Eagleson's comparison (1982), TxDOT has acquired much data and experience with the Dynatest FWD, having purchased over the years thirteen of these systems for pavement evaluation. Using these data, TxDOT has undertaken significant research on FWD rigid pavement deflection measurements in Texas. As a result, the FWD has been adopted for network-level monitoring of flexible pavements in Texas, and deflection testing on all pavement types in the Strategic Highway Research Program Long-Term Pavement Performance program (SHRP LTPP). Given the general acceptance and availability of the FWD, we recommend that it be used as the NDT device for any network-level rigid pavement deflection data collection effort.

2.3 SIGNIFICANT FACTORS AFFECTING RIGID PAVEMENT DEFLECTION MEASUREMENTS

Deflection measurements on rigid pavements are influenced by many factors, among them subgrade support, presence of voids, load transfer across joints and cracks, environmental effects (such as temperature and moisture variations within the pavement slab), and the location on the pavement where the deflection basin is measured (Ref 6). Since the purpose of deflection testing is to estimate subgrade support, locate voids underneath the pavement, test for load transfer efficiency, and/or determine the structural capacity of a pavement, it is desirable to account for the effects of such extraneous factors as temperature, moisture, and load placement in the deflection measurement. It is necessary, therefore, to develop testing procedures that either minimize or quantify these effects.

2.3.1 Effects of Daily Temperature Variation on Measured Deflections

Morales-Valentin (Ref 6) found that daily temperature variations within the concrete slab can significantly influence deflection measurements. As he noted, the deviation of temperature throughout the day causes a temperature gradient through the slab. The gradient causes the pavement slab to curl at the slab edges and corners, an action that can change the measured deflections at these locations owing to changes in subgrade support. The temperature differential is defined as the algebraic difference between the temperature at the top and the bottom of the slab. Shown in Figure 2.2 are the curling effects of both positive and negative temperature differentials. These differentials can cause larger or smaller deflection measurements, depending on the time of day and the prevailing weather conditions. Morales-Valentin reported that maximum temperature differentials occur during the day in the spring and summer (Ref 6).

To use deflection data at the network level, it is necessary for the data to reflect the structural capacity of the pavement accurately. Accordingly, it is necessary to remove or minimize the effects of temperature differentials and slab curling/warping on the measured deflections. This is accomplished best by using the FWD at times when the potential for differential effects is at a minimum, and by testing away from the slab corners and edges (Ref 6).

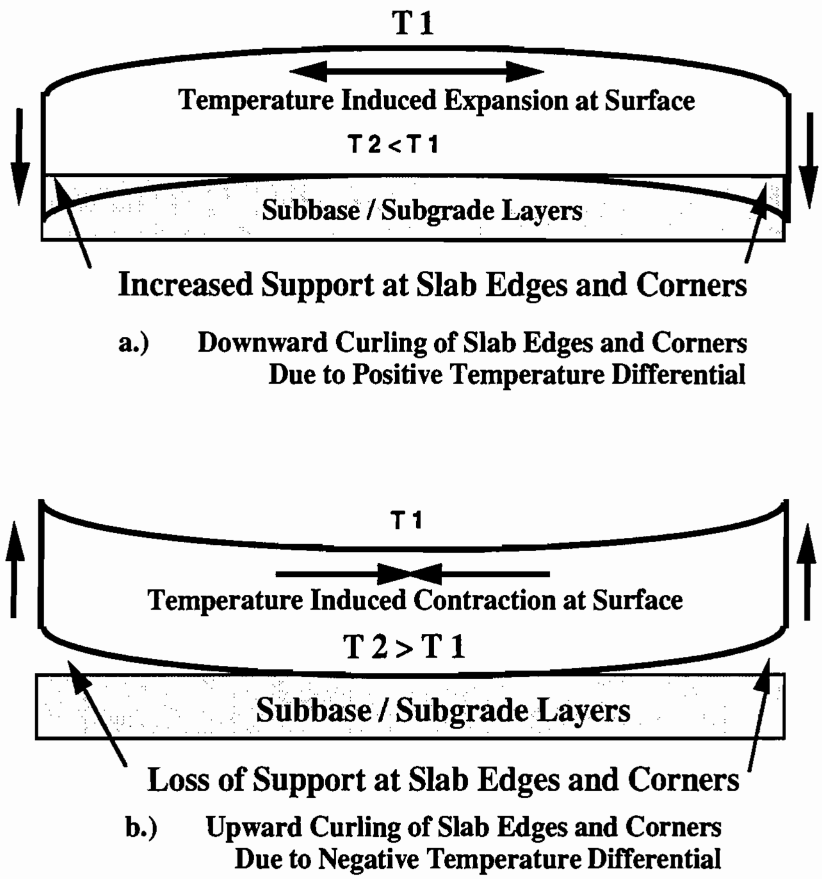

Figure 2.1 Curling effects of positive and negative temperature differentials

To measure the capacity of the pavement structure, the materials in the pavement should be characterized. This can be done by determining the relative strengths of the pavement layers (i.e., quantify their respective material properties). Morales-Valentin suggested that deflection testing for materials characterization should be performed away from the edges and corners of the pavement slab, since at these locations daily temperature variations have little or no effect on measured deflections. Deflection testing near joints and cracks for load transfer studies should be performed in the morning hours to minimize the effect of daily temperature variation (Ref 6).

2.3.2 Long-Term (Seasonal) Effects of Climate on Deflection Measurements

Distortion of the pavement slab caused by seasonal climate changes can either compensate or exaggerate the temperature curling discussed in the previous section. Torres-Verdin (Ref 5) reported that seasonal changes in deflection measurements are due to changes in average temperature and moisture content in the pavement layers. Moisture-induced volume change in concrete layers is not as detectable in a daily cycle as the volume change that results from temperature differential, though it is apparent on a long-term, or seasonal, basis (Ref 5)

Because seasonal changes in deflection may be attributed to changes in moisture, these effects should also vary from region to region within Texas; that is, in some areas the seasonal effect should be more significant than in others. In areas having wet winters, measured deflections will generally be larger than those measured during drier seasons, a result primarily of the soft and wet subgrades, low effective modulus in the surface layer (caused by concrete shrinkage), and the larger crack widths that cause poor load transfer. During dry summers, conversely, measured deflections may decrease, especially near the point of load application, owing to stiffer subgrades and smaller crack widths that lead to better load transfer in the surface layer (Ref 5). Although the curling effect of moisture changes do not correspond to a daily cycle, the magnitude of this curling should not be neglected when deflections are compared over long periods.

Another effect of long-term weather variation is the horizontal expansion and contraction of the concrete as the average seasonal temperature changes. This volume change in the slab affects the opening of the joints and cracks in the pavement, as well as the amount of friction between the slab and subgrade. The expansion of the slab that occurs as a result of summer heat reduces the size of the joint openings, thereby increasing load transfer efficiency and reducing the curling produced by daily temperature variation. Just the opposite occurs during colder periods, when the concrete slab has contracted and joint and crack widths are comparatively larger.

Although the effect of seasonal changes on FWD measurements has not yet been quantified, the collection of network-level deflection data during all seasons will allow the study of seasonal variation of deflection measurements for materials characterization.

2.3.3 Effect of Load Placement on Rigid Pavement Deflections

Another factor that significantly affects the results of deflection testing on rigid pavements is the placement of the load on the pavement slab. In discussing this effect on deflections measured with the Dynaflect, Uddin (Ref 7) reported that deflections measured at varying distances from the pavement edge represent data collected from different populations. Therefore, if statistical inferences are to be made from deflection data collected on rigid pavements, the positioning of the deflection measuring device with respect to the pavement edge must be consistent for all measurements.

Because network-level data are often used to make inferences about the structural capacity of the pavement network, it is vital that the position of the FWD be consistent for all measurements. According to Uddin, the placement error should never be more than 12.7 cm (5 in.) (Ref 7). Therefore, careful attention is required when positioning the FWD on the rigid pavement slab.

2.4 ANALYSIS OF FWD DEFLECTION DATA

Once FWD data have been collected, it is possible to use many different analysis techniques to determine the structural condition of the pavement network. Among these are modulus backcalculation and empirical structural adequacy indices (e.g., the SSI discussed above). This section reviews these methods as they apply to network-level deflection testing.

2.4.1 Modulus Backcalculation

Modulus backcalculation is a method derived from the inverse application of elastic layer theory to determine the approximate elastic moduli of the pavement layers. The method involves assuming an initial estimate of pavement layer moduli and calculating a theoretical deflection basin using elastic layer theory. This basin is compared with the actual basin measured by the FWD; adjustments are then made to the initial assumptions of the pavement moduli based on this comparison. Using the adjusted moduli, another basin is calculated; this is again compared with the original and further adjustments are made. This process is repeated until the calculated basin matches the measured basin within an acceptable tolerance. The moduli used for calculating the last basin are then assumed to be the moduli of the pavement layers.

Although the basic methodology used in backcalculation is simple, this process can become complex. Many factors influence the deflections measured by the FWD. In addition to those discussed above, there are other, more elaborate factors, including nonlinear behavior of the subgrade and granular layers, effects of a non-infinite subgrade, and the fact that the backcalculation process may not produce a unique solution. Therefore, to assure a reasonable degree of accuracy when using modulus backcalculation, one should know the layer thicknesses, the depth to a rigid layer beneath the subgrade (if any), and the materials used in the construction of the pavement. A review of the necessary inputs and a methodology to backcalculate moduli from deflection data are presented by Uddin (Ref 8).

While modulus backcalculation can be useful for characterizing pavement layer properties at the project level, at the network level the required inventory data are not always available. And to define all the construction factors necessary for each section would be a large undertaking. For this reason modulus backcalculation is not generally suited for analysis of pavements at the network level.

2.4.2 Structural Capacity Indices

Better suited for use in a large-scale deflection testing program are analyses of a more approximate nature, namely, structural capacity indices. Parameters calculated from the deflection basin can give a wealth of information about the structural capacity of the pavement. Figure 2.2 is a typical deflection basin showing the Surface Curvature Index (SCI), which is calculated to indicate the strength of the surface layers in the pavement structure. By using such quantities as the SCI, the surface strength of the pavement can be estimated for comparison with other pavements in the network. Other quantities, such as the maximum deflection and deflections measured at the outer geophones, can be used to estimate structural strength as well.

It has been shown by various researchers that deflections measured near the center of the deflection basin (the SCI [W1-W2]) and maximum deflection (W1) are influenced largely by the surface stiffness. Conversely, deflections measured far away from the center of the basin (W7) are influenced largely by the subgrade layer stiffness. Uddin (Ref 8) and Scullion (Ref 4) provide excellent reviews of the many different deflection basin parameters used by engineers and researchers.

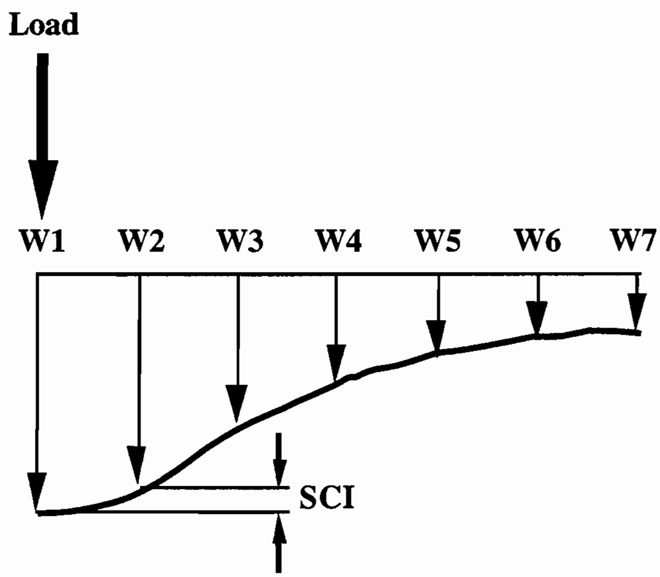

Figure 2.2 A typical FWD deflection basin

Pavements that exhibit low layer stiffness using some of the parameters calculated from the deflection basin can then be selected for more detailed evaluation of the pavement structure, perhaps using modulus backcalculation techniques.

At the network level, the FWD is used to estimate the structural capacity of a pavement. This estimate reflects the capacity of the pavement to carry its expected traffic in the future; in effect, then, the structural capacity is a measure of the pavement's remaining life. Accordingly, an index calculated from deflection measurements should reflect a pavement's capacity to withstand future traffic loading. Haas and Hudson (Ref 2) described a Structural Adequacy Index (SAI) that relates the measured or design deflection to a maximum tolerable deflection needed to withstand expected traffic for a particular analysis period. Pavements with measured deflections greater than a maximum tolerable deflection would score low on the index because the pavement was assumed to be incapable of withstanding the traffic loads for the whole period (without some type of rehabilitation). The pavement is rated on a scale from 1 to 10, with 5 being barely adequate (i.e., maximum tolerable deflection equals the measured or design deflection) (Ref 2). This index can be used to identify sections with structural deficiencies before major reconstruction becomes necessary.

Another structural capacity index in use at present in Texas for flexible pavements is the Structural Strength Index (SSI). While incorporating the same basic premise as the SAI described above, the SSI differs in that it includes several environmental factors. In using this index, the SCI and the W7 deflection are measured for the pavement; based on these values a preliminary value for the SSI is assigned by comparing these with measurements made on the same pavement type and thickness by researchers at the Texas Transportation Institute (Ref 4). This preliminary SSI is then weighted for traffic and environmental factors, with pavements in particularly harsh climates with heavy traffic penalized the most. Traffic factors used to weight the SSI are calculated for a 20- year period, given the expected applied 8170 kg (18 kip) ESALs for the whole period. The SSI rates the pavement not only for its ability to carry expected traffic, but also for its ability to resist environmental damage as well. Using network-level deflection data for rigid pavements in a manner similar to that described above can be useful in managing rigid pavements.

2.5 SUMMARY

This chapter discussed the significant factors that affect the collection of deflection data on rigid pavements at the network level (including the analyses that can be used with the data). The use of deflection data at the network level requires that these factors be accounted for in the collection of the data and in any subsequent analyses.

The next chapter evaluates the quality of data presently available in the Texas PES database. The existing data will be evaluated based on the information presented in this chapter and on its potential for use in network-level analysis of rigid pavement behavior. Chapter 4 of this report will present a data collection plan that accounts for the factors and analyses discussed here for future network-level evaluation of rigid pavement behavior in Texas.

CHAPTER 3. EVALUATION OF EXISTING RIGID FWD DATA CONTAINED IN THE PES DATABASE

3.1 INTRODUCTION

One of the objectives of this research project was to evaluate any rigid deflection data contained in the Texas PES database for a network-level analysis of rigid pavement behavior in Texas. The Texas PES was designed to allow consistent review and evaluation of pavement conditions across the state. This system includes a database that is used to provide both summary and section-specific information about the condition of the state's pavements. To accomplish this, researchers have collected data relating to (1) visual distress, (2) ride quality, (3) structural strength, and (4) skid resistance.

The PES divides the state's highways into approximately 3.22-km (2-mile) segments using the PES reference marker system. In 1990, data were collected on 44.4 percent of the sections for ride and distress surveys, and 18 percent of the network for structural surveys using the FWD. Using the data collected, the PES system produces scores that summarize the condition of the pavement section (Ref 9).

3.2 THE PES DATABASE

The PES database contains approximately 35,000 records per year. Each record is 1,399 characters long and each contains data that pertain to a given PES section. For each section, the record is divided into six categories. These categories include:

1. Section Identification (Inventory) Data,
2. Visual Evaluation (Distress) Data,
3. Ride Quality Data (Serviceability Index),
4. Skid Data (Skid Number),
5. Deflection Data (FWD), and
6. Computed Pavement Scores.

Deflection data contained in the PES database are collected using the FWD and are stored as both a Structural Strength Index (SSI) and as the direct geophone readings or deflection basins. Also included is the testing date, the lane tested, and the ambient air temperature (although this is not usually collected). It is important to note that the pavement type of the lane where the FWD test was conducted is not stored along with the FWD data.

3.3 PES DEFLECTION DATA COLLECTION

In the PES, a standard method for FWD testing is used throughout so that the data are consistent over various testing sites. Specified in the 1990 PES FWD Manual (Ref 10) are the following guidelines for the operators of the FWD:

1. FWD tests are to be performed with one geophone placed underneath the load plate and six more spaced 0.3 m (1 foot) apart in front of the load plate.
2. Testing is to be done in the outer (closest to the shoulder) wheelpath of the pavement.
3. Applied load should be approximately 4080 kg (9000 lb).
4. Two drops should be completed for each test point, the first to seat the load plate fully on the pavement, the second for storage in the database.
5. Five test points are to be used for each section tested, starting at the first reference marker and spaced equally along the section from beginning to end.
6. Testing should be performed in the outside lanes unless otherwise specified.

For PES deflection data collection, there is no specification as to the placement of the FWD in relation to the pavement edge or joints and cracks. These instructions were drafted for flexible pavement FWD data collection only. Therefore, it is not known where the FWD was positioned with respect to the pavement edge for any of the rigid data found in the PES.

3.4 QUANTITY OF RIGID PAVEMENT DEFLECTION DATA IN THE PES DATABASE

Table 3.1 summarizes the quantity of rigid deflection data in the PES database maintained by the TxDOT. It should be noted that before 1987 there are no deflection data in the PES database for rigid pavements. For each of the three rigid pavement types in the PES (continuously reinforced, or CRCP, jointed reinforced, or JRCP, and jointed plain, or JPCP), there is a listing of the number of sections that have been visually evaluated or maintained for evaluation in the PES database. Of these sections, those that have been tested using the FWD are listed along with the percentage of the total number of sections for each rigid pavement type. Given also are the totals for all four years combined.

In order to determine which rigid sections in the PES contained FWD data, it was necessary to first determine the pavement type from the visual evaluation category of the records, cross-matching these pavement types with both the lane in which the FWD measurements were taken and the presence of geophone readings. In some cases, the pavement type of the section was identified for the opposite side of the road from where the deflection data were collected, leaving unspecified the pavement type of the section where the data were actually collected. It will take further investigation to determine if, in fact, these sections are portland cement concrete pavements.

3.5 PES RIGID DEFLECTION DATA QUALITY

During the years that the rigid data found in the PES database were collected, TxDOT had no policy for collecting deflection data for rigid pavements. Therefore, the procedures outlined for data collection described above were intended only for flexible sections. For this reason, it was originally thought that the rigid data found might have been stored into the PES database erroneously, putting into question the origin of the data.

Table 3.1 Deflection data for pavements labeled rigid in the PES database (1987 -1990)

Year	Pavement Type	Number of PES Test Sections	PES Sections with Deflection Data	% of Total	Number of Deflection Basins
1987	CRCP	159	11	6.9	94
	JRCP	150	12	8.0	93
	JPCP	16	0	0.0	0
1988	CRCP	325	7	2.2	53
	JRCP	341	13	3.8	102
	JPCP	90	0	0.0	0
1989	CRCP	176	19	10.8	156
	JRCP	210	4	1.9	26
	JPCP	17	0	0.0	0
1990	CRCP	343	32	9.3	304
	JRCP	247	7	2.8	62
	JPCP	128	1	0.8	4
1987-1990	CRCP	1003	69	6.9	607
	JRCP	948	36	3.8	283
	JPCP	251	1	0.4	4

3.5.1 Correspondence with TxDOT

Given the amount of data that was found *and* the absence of a data collection program, we considered the deflection data contained in the PES database for rigid pavements to be somewhat suspect. A meeting was held with Doug Chalman of TxDOT to solicit his comments concerning the data in question. The meeting produced the following conclusions regarding any rigid deflection data found in the database:

1. TxDOT does not have a regular network-level FWD data collection program for rigid pavements at this time and did not have such a program during the years when the PES data were collected (6/24/93);
2. It is very likely that the few deflection basins designated as rigid in the PES database were actually measured on flexible or composite sections and have been misidentified at some step in the data collection and storage process.

Based on this information, we decided not to use deflection data found in the PES database for rigid pavements for any analysis. Although the data will not be used for analysis, we decided to identify which basins contained in this data were actually misidentified flexible deflection measurements. To do this, we compared the PES deflection data with data that were considered to be more reliable from the CRCP database maintained by CTR. In the process, a method for screening rigid FWD data for "abnormal" deflection basins was developed.

3.5.2 The CTR CRCP Database

Since 1974, CTR has been periodically collecting data on the state's CRC pavements. In 1989, CTR compiled these data into a large database (Ref 3). During the summer of 1988, a statewide deflection survey of CRC pavements was completed by CTR. These data were reviewed extensively before storage into the CTR database. The deflection data were collected at various points on the CRC pavements (edge and interior, as well as both sides of cracks) and at many different load levels. Given the quality and availability of the data contained in the CTR database, we decided to allow these deflection data to represent the population of deflection measurements taken on CRC pavements in Texas.

3.5.3 Examination of the PES and CTR Data

In evaluating the deflection data of the CTR and PES databases, we first compared the geophone measurements from the deflection basins. Since all data from the CTR database pertain to CRC pavements, only the CRC pavements from the PES were used for comparison. From Table 3.2 above there are 607 deflection basins in the PES that are labeled "CRCP." All deflection measurements in the PES are "normalized" to a 4080 kg (9000 lb) load. The normalization process uses the ratio of the measured deflection to the applied load to determine the deflection at 4080 kg (9000 lb). Therefore, only deflections measured between a 3630 kg (8,000 lb) and 4540 kg (10,000 lb) applied load were used from the CTR database for this comparison. These deflections were also "normalized" to 4080 kg (9000 lb).

Figure 3.1 plots the mean deflection measured at each sensor for the two databases. A sharp contrast between the slope of the two deflection basins near the point of load application is apparent. We decided to review the literature to substantiate this finding. Uddin (Ref 8) also obtained this shape in a plot of deflection basins resulting from an applied load of 450 kg (1000 lb) for typical rigid and flexible pavements calculated from the elastic layer theory program ELSYM5 (Ref 11). For this study, a sample run of the elastic layer theory program, BISAR (Ref 12), at a 4080 kg (9000 lb) applied load was used to obtain the deflection basins shown in Figure 3.2. The shapes of both the rigid and flexible basins calculated by the BISAR analysis are similar to those reported by Uddin. Given that the shape of the deflection basins differ mainly at or near the point of load application, the SCI will be used to discriminate between rigid and flexible deflection basins.

3.5.4 Comparison of Individual PES Deflection Basins with the CTR Data

The purpose here was to determine if each deflection basin, based on its SCI value, could be accepted statistically as being sampled from the same population (i.e., CRC pavements) as the CTR data. The statistical comparison requires that the CTR data be from a normal distribution. The distribution of the SCI values from the CTR database were checked for normality using the W test developed by Shapiro and Wilk (Ref 13). The W statistic is computed from the sample data and is interpreted by comparing the statistic to critical values of W for a particular significance level. Small values (close to 0) of W indicate that the data are non-normal. The following values were computed for the CTR data selected for this comparison:

N = 386

W statistic = .9766

Probability of a W statistic less than .9766 = .0253

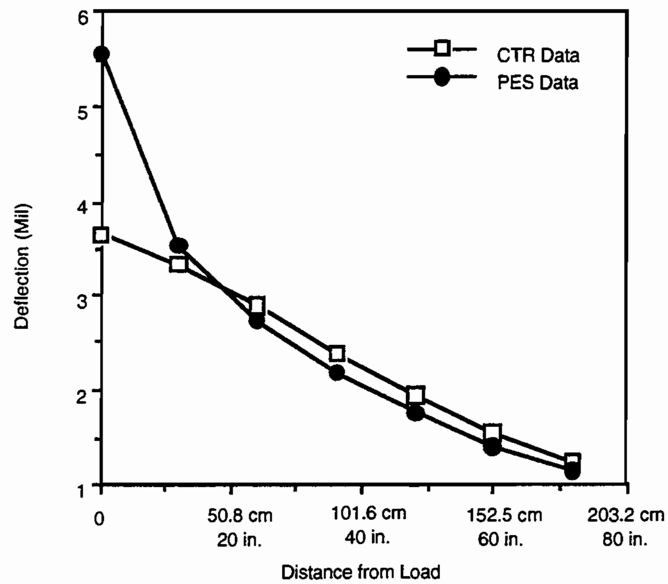

Figure 3.1 Plot of average deflection basins from CTR and PES databases (1 mil = .025 mm)

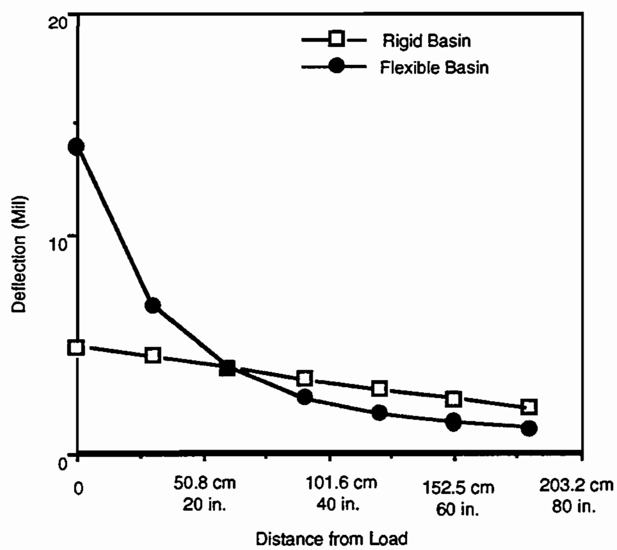

Figure 3.2 Theoretical deflection basins calculated from elastic layer theory program BISAR for typical rigid and flexible pavements (1 mil = .025 mm)

Therefore, the CTR data were considered to be normally distributed at the .02 significance level. In their discussion of the W statistic, Anderson and McLean suggest that if the data are considered to be normal above the .01 significance level, the data should be considered normal, unless there is a theoretical reason for a data transformation (Ref 14). Given this, a one-sided prediction interval for a new observation can be constructed about the mean of the CTR data. Then, if any single value of the SCI computed from the PES data is greater than the upper limit of the prediction interval calculated from the CTR data, that observation will not be considered a deflection basin from a rigid pavement. The prediction interval can be constructed using the t statistic. The t statistic is the difference between the mean value from the CTR database and the value of the new observation from the PES database divided by the standard deviation of the data from the CTR database. Because the t statistic has a known distribution, the probability that a new observation in the sample will have a given value of t can be computed. If that probability is less than a critical value selected before the test is run, the new observation is not expected to be sampled from the same population as the sample data. The required formula for this analysis is:

$$t = \frac{X_{new} - \overline{X}}{S\sqrt{1 + 1/n}} \qquad \text{(Eq. 3.1)}$$

where:

n = number of observations in the CTR database,

S = standard deviation of the CTR database,

X_{new} = value of the new observation from the PES to be tested, and

$\overline{X}$ = mean value of the variable from the CTR database.

The standard deviation (adjusted for a finite population), not the standard error of the difference, is used in this test because the test is for a single observation, not another sample mean. Using this formula for calculating the t statistic is equivalent to constructing a one-sided prediction interval about the mean of a sample of data. The critical value of the t statistic is found by selecting a significance level of the test. A 95 percent one-sided prediction interval was selected for this study. Given this, a critical value of 1.645 for "t" is found from tables of probability for a random t variable (Ref 15).

To facilitate this comparison in the computer, a simple rearrangement of the Equation 3.1 above was made to find an upper limit value for the SCI of a CRC pavement. This upper limit represents the value below which 95 percent of new observations are expected to fall if the new observations are taken from the same population that yielded the CTR data. This is done by substituting the critical value of t, t_{cr}, into the equation and solving the equation for the critical value of a new observation, X_{cr}.

$$X_{cr} = \overline{X} + t_{cr}\, S \sqrt{1 + \frac{1}{n}} \qquad \text{(Eq. 3.2)}$$

where the other variables are the same as defined above. The following were calculated from the CTR data:

Mean SCI value ($\overline{X}$) = 0.008 mm (0.32 mil),

Standard Deviation (S) = 0.003 mm (0.118 mil), and

Number of Observations (n) = 386.

Substituting these and the critical value of t given above, the critical value for a new observation is:

$$X_{cr} = 0.013 \text{ mm } (0.52 \text{ mil}).$$

Therefore any SCI value greater than 0.013 mm (0.52 mil) found in the PES database will be considered questionable. Shown in Figure 3.3 is the distribution of CTR SCI values with the mean and upper limit for the prediction interval. Using this distribution, each individual PES observation was compared with the critical SCI value calculated above. Using this procedure, Figure 3.4 was produced showing the total number of PES basins and the number of these that had SCI values greater than 0.013 mm (0.52 mil). There is a significant number of basins that could not be accepted as CRCP using this analysis over the four years. In all, 242 of the 607 basins (approximately 40 percent) examined in the PES database could not be considered to be sampled from the same population as the CTR data. If the PES and CTR data were from the same population, then approximately 95 percent of the basins from the PES would be expected to have SCI values less than the critical value calculated above. The fact that only 60 percent of the data had acceptable SCI values indicates that the PES data and the CTR data were not sampled from the same population.

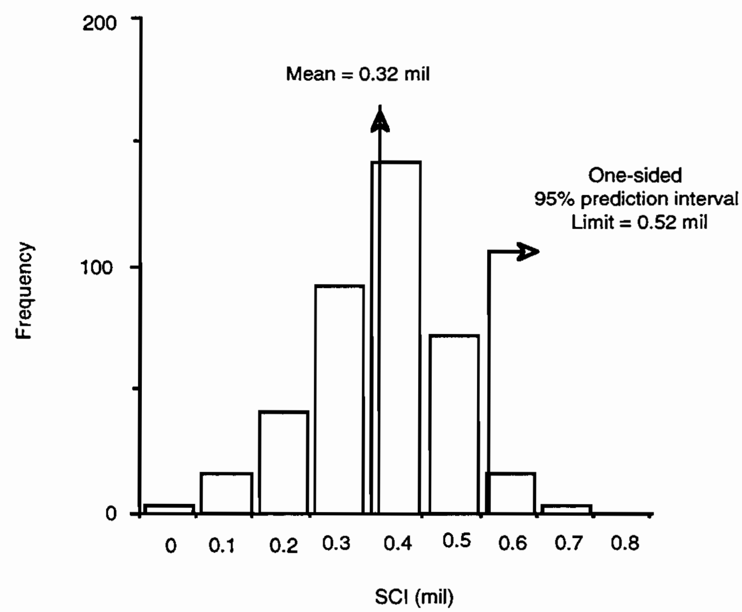

Figure 3.3 SCI distribution from CTR database (1 mil = .025 mm)

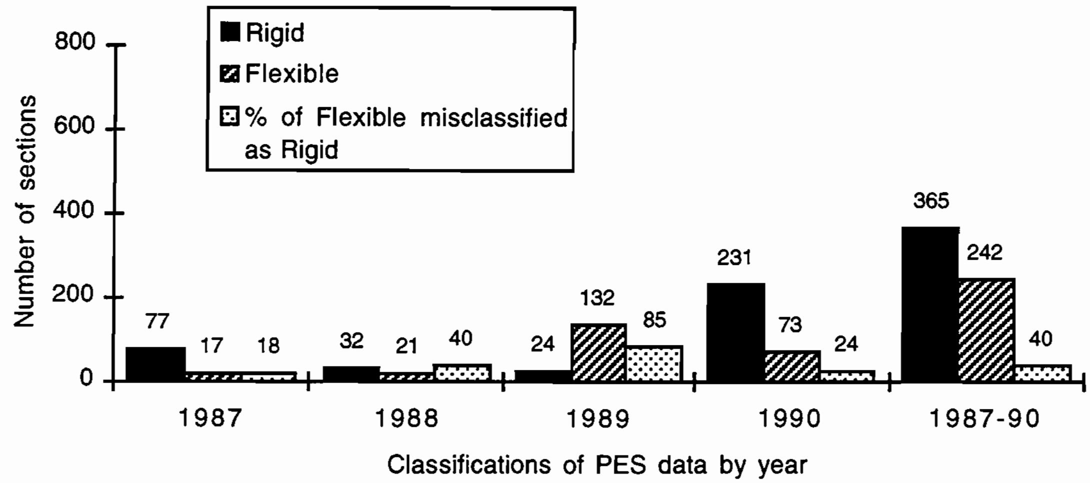

Figure 3.4 Results of comparison between PES and CTR data

3.6 CONCLUSIONS

The following are the findings of this study thus far:

1. It has been shown that the PES data for CRCP sections, when compared with similar data in the CTR database, differ significantly from the expected deflection patterns observed from CRC pavements.
2. This difference suggests that the data were actually measured on flexible pavement sections and, at some point, misidentified as rigid sections.
3. There is currently no specification for FWD testing on rigid pavements used by TxDOT. Therefore, if the PES data were actually rigid deflection basins, the loading, layer thicknesses, and the testing procedures used by the operators of the FWD are unknown. As discussed in Chapter 2, information concerning these factors is very important when analyzing rigid deflection data.
4. The method described for constructing a prediction interval about the mean of deflection basin parameters was an effective procedure for checking the quality of deflection data.

Given these findings, we recommend that the rigid deflection data in the PES database be removed (or at least designated as questionable). It is also recommended that the method for screening PES data be expanded to include other pavement types for use in future large-scale deflection data collection efforts. This expansion would involve computing prediction intervals from existing data for both rigid and flexible pavements. Using this information, incoming data can be verified for accuracy before their storage in a database.

CHAPTER 4. RECOMMENDED DATA COLLECTION PLANS FOR FUTURE NETWORK LEVEL FWD TESTING ON RIGID PAVEMENTS

4.1 INTRODUCTION

The discussion of the structural behavior of rigid pavements (Chapter 2) showed that the measurement of deflections using the FWD is greatly affected by such factors as load placement and daily and seasonal temperature variation of the PCC slab. Since it was found that the usefulness of the rigid pavement deflection data contained in the PES is marginal at best (Chapter 3), this chapter will outline provisions necessary to implement a cost-effective network-level rigid pavement deflection data collection program that considers the factors discussed in Chapter 2.

A network-level FWD data collection plan should address the following items: first, the type of sampling that should be used; second, the optimum size of that sample; and third, the testing procedures to be used for data collection. This chapter presents the recommended sampling method for FWD data collection, outlines the determination of an optimum sample size, and recommends testing procedures for data collection.

4.2 RECOMMENDED SAMPLING SCHEME FOR FWD DATA COLLECTION ON RIGID PAVEMENTS

Various types of random samples could be used for collecting network-level pavement data. These range from a simple random sample to cluster samples and stratified samples. In a report describing recommended sampling procedures for network-level pavement performance data collection in Texas, Mahoney and Lytton concluded that a stratified random sample is best (Ref 16). The stratified random sampling method described therein was found to be the best method for a number of data items, including deflection measurements for flexible pavements.

A stratified random sample exercise begins by dividing the population into distinct strata or divisions before the random sample is drawn. A random sample is then taken from each of the strata. The advantage of this type of sample is that, if the population is relatively homogeneous within the strata, but differs considerably between the strata, the precision of the estimates of the parameters from the sample is increased. In addition, stratified random sampling ensures the representation of each of the strata in the sample.

The two-stage stratified random sample described by Mahoney and Lytton is well-suited for application in states like Texas, where districts are required to provide estimates of pavement condition and performance. The two-stage stratified random sample for the collection of rigid FWD data involves randomly sampling counties from each of the districts (first stage) and then randomly selecting pavement sections from within the selected counties (second stage). This scheme should also be used for any collection of network-level rigid FWD data in the future.

4.2.1 Sampling to Evaluate Seasonal Variation of Deflection Measurements

Using this sampling strategy throughout the year and in all districts can yield the seasonal variation of the deflection measurements for each district. Because many different pavement sections will be included in the database, a large number of degrees of freedom will be available for studying seasonal effects and their interaction with the climatic regions in Texas. These data can be used in a statistical evaluation of the seasonal variation of mid-slab deflections.

4.3 OPTIMUM SAMPLE SIZE DETERMINATION FOR RIGID PAVEMENT DEFLECTION DATA COLLECTION

As explained in Chapter 2, network-level data collection involves selecting a random sample of pavement sections from the pavement network for data collection, and then using these random samples to infer the condition, behavior, and performance of the pavements within the network. Two factors influence the optimum sample size required, namely, the precision of the sample and the cost of the data collection effort. Because both of these factors increase with an increase in sample size, it is therefore desirable to collect only the amount of data sufficient to assure reasonable precision of the sample. The sampling plan used for data collection should thus recommend an optimum sample size.

In sampling from a population, the amount of variation in the observed sample means (the standard error) of the sample increases as the sample size decreases. It is desirable to minimize this variation by increasing the sample size (which consequently increases the data collection cost). Thus, a balance between a reasonable standard error of the sample and an affordable total cost for data collection determines the optimum sample size.

Because the sample standard error and the data collection cost are two different quantities, it is necessary to combine these two variables in a measure that represents the advantage of a given sample size over any other sample size. Mahoney and Lytton (Ref 16) made use of utility theory to accomplish this when determining an optimum sample size for flexible pavement data collection in Texas. Utility theory is a decision-making tool used to associate factors that cannot be combined directly because of incompatible units.

4.3.1 Applying Utility Theory to Determine Optimum Sample Size

A utility is a value that ranges from 0 (least preferable) to 1 (most preferable) and represents the relative preference given to one sample size with respect to all others. The first step in applying utility theory for this task was to create the utility functions or curves for both of the decision criteria (cost and sampling precision). In terms of cost, the utility of a small sample size would be higher than that for a large sample. Just the opposite would be true if the precision of the sample data was the only criterion being considered. The curves for each of the decision criteria were then combined using weighting coefficients to create a combined utility for a given sample size. The optimum sample size is then given by the sample size with the maximum combined utility.

4.3.2 Development of a Utility Curve for Sampling Precision

The precision of a sample is defined by the amount of variation that is present in the observed sample means computed from separate samples of equal size. If a large number of equally sized random samples were drawn from the same population, the means calculated from each of the samples would be different. The standard deviation of those observed sample means, which is an estimate of the sample standard error, can be used as a measure of the precision for that sample size. As the sample size increases, the amount of variation in the observed sample means will decrease. Therefore, the utility function should reflect the relationship between the variance in the sample means and the sample size.

Sampling Simulation Study. A sampling simulation study was conducted to determine the relationship between the sample standard error and sample size. The largest amount of data available to study this relationship was found in the CTR database. A range of sample sizes was selected from it and the standard error was estimated for each. These estimates were then used to find the relationship between sample precision and sample size. An SAS computer program was coded to sample sections from the CTR database. Appendix A lists the program code, the sample sizes, and the estimates of the standard error from the simulation. Only 200-mm (8-in.), non-overlaid CRC pavements were considered for this study, since the database included too few pavements having a greater thickness to allow a reasonable range of sample sizes to be selected. In all, there were 135 pavement sections in 8 TxDOT highway districts used for the simulation.

In the CTR database, FWD deflection basins are taken at various locations on the pavement slab (the corner, edge, and midslab) every 61 m (200 feet). Because deflection data are used at the network level for materials characterization, we decided to study only the midslab deflections, as discussed in Chapter 2. Given that there are several deflection basins at the midslab location in the CTR database for each pavement section, we also decided to investigate the benefit of using data from only one drop per section (versus five drops per section). The simulation was run twice, first using five of the basins from each of the sampled CTR pavement sections, and then using only the first deflection basin from each of the sampled sections.

Owing to the amount of data available, there were some problems encountered in the sampling simulation. Since there are only 135 sections available in the database, if sections are sampled from every district (as required by the previously recommended two-stage sampling scheme), the smallest sample size that can be taken is 5.9 percent (8/135). It was felt that smaller sample sizes would be needed to define the relationship between sampling precision and sample size fully. The only possible way to accomplish this, given the data available, was to randomly sample districts as well.

It was desirable to sample over a range of sample sizes so that a reasonable estimate of the sampling precision could be obtained. The sample sizes ranged from a total of two pavement sections sampled from a single district (1.5 percent), to a total of forty pavement sections sampled five at a time from each of the eight districts (29.6 percent) in the database. This range was selected because it was felt that a smaller sample size would not allow a representative sample to be drawn, while a larger sample size would not increase the precision of the sample significantly.

4.3.3 Analysis of the Deflection Basin

Since the highest variation is found in the maximum deflection of a FWD deflection basin, the means and standard errors were computed using the maximum deflection as the critical variable. We decided that if an optimum sample size was determined based on the maximum deflection, that optimum size would be sufficient for all other geophones as well. Three hundred samples were drawn from the database at each sample size, and the standard deviation of those 300 mean maximum deflections were computed to estimate the standard error.

Calculation of the Sample Means and Standard Error. To calculate the grand mean from a two-stage stratified random sample, we used the following procedure. The district mean maximum deflection was found by weighting the mean deflections measured at the pavement section by the total number of pavement sections in the county containing the pavement section. This is expressed by (Ref 16):

$$\overline{D} = \frac{\sum_{i=1}^{n} M_i D_i}{\sum_{i=1}^{n} M_i} \qquad \text{(Eq. 4.1)}$$

where:

$\overline{D}$ = the mean maximum deflection for the district,

M_i = the number of eligible sections in the county containing pavement section i,

D_i = the mean maximum deflection for county i, and

n = the number of counties selected within the district for a given sample size.

The grand mean for the network was then computed by weighting the district mean maximum deflection by the number of counties within the district. This is given by (Ref 16):

$$\overline{\overline{D}} = \frac{\sum_{i=1}^{N} C_i \overline{D_i}}{\sum_{i=1}^{N} C_i} \qquad \text{(Eq. 4.2)}$$

where:

$\overline{\overline{D}}$ = the grand mean maximum deflection,

C_i = the number of eligible counties in the ith district,

$\overline{D_i}$ = the district mean maximum deflection for district i, and

N = the number of districts sampled for a given sample size.

The standard error for each sample size was then computed by calculating the standard deviation of the 300 grand means from Eqs. 4.1 and 4.2 above. This value is given by:

$$S_{\bar{x}} = \sqrt{\sum_{j=1}^{N} \frac{\left(\overline{\overline{D_j}} - \overline{\overline{\overline{D}}} \right)^2}{N-1}} \qquad \text{(Eq. 4.3)}$$

where:

$\overline{\overline{\overline{D}}}$ = the average of all the grand means from the 300 samples,

$\overline{\overline{D_j}}$ = the grand mean from the jth sample,

N = the number of samples taken (300 for this study), and

$S_{\bar{x}}$ = the standard error for that sample size.

4.3.4 Simulation Results

The standard error, a measure of the sample precision, is not a dimensionless quantity. And for utility theory, it is necessary to use a dimensionless quantity to create the utility curves. Therefore, the coefficient of sampling variation (COSV) was used to create the utility function. Mahoney and Lytton (Ref 16) defined the COSV as the standard error ($S_{\bar{x}}$) computed in Equation 4.3 from the previous section divided by the average of the 300 sample means ($\overline{\overline{\overline{D}}}$) multiplied by 100. The COSV, analogous to a coefficient of variation, is a dimensionless measure that can be used to compare the precision of different sample sizes.

Figure 4.1 shows a plot of the COSV versus sample size, as estimated from the simulation study for both one and five basins per section. As expected, the COSV decreased as the sample size increased, a finding indicating that the sample precision increased with sample size. Also shown is the effect of using a sample of the highway districts. For example, at a sample size of 5.9 percent using five basins per section, if all the districts are sampled (i.e., one pavement section from each of the eight districts), the COSV is 5.5 percent; however, if the same size sample is drawn from only four districts (two pavement sections from each district), the COSV increases to 8.2 percent. This was found to be true for any of the sample sizes used in the study. Therefore, the precision of the sample is affected greatly by the exclusion of any of the strata in the sample.

4.3.5 Effect of Sampling Multiple Basins per Section

Also shown in Figure 4.1 is the difference in the computed COSV using only one drop per section, as compared with using five drops per section. A curve is shown on the graph for both one and five basins per section. Although the difference is apparent, the sample's precision is improved more by measuring one basin on more pavement sections (rather than by simply increasing the number of basins measured per section). Therefore, it is recommended that for any future rigid pavement deflection data collection efforts, only one midslab deflection basin be measured per section.

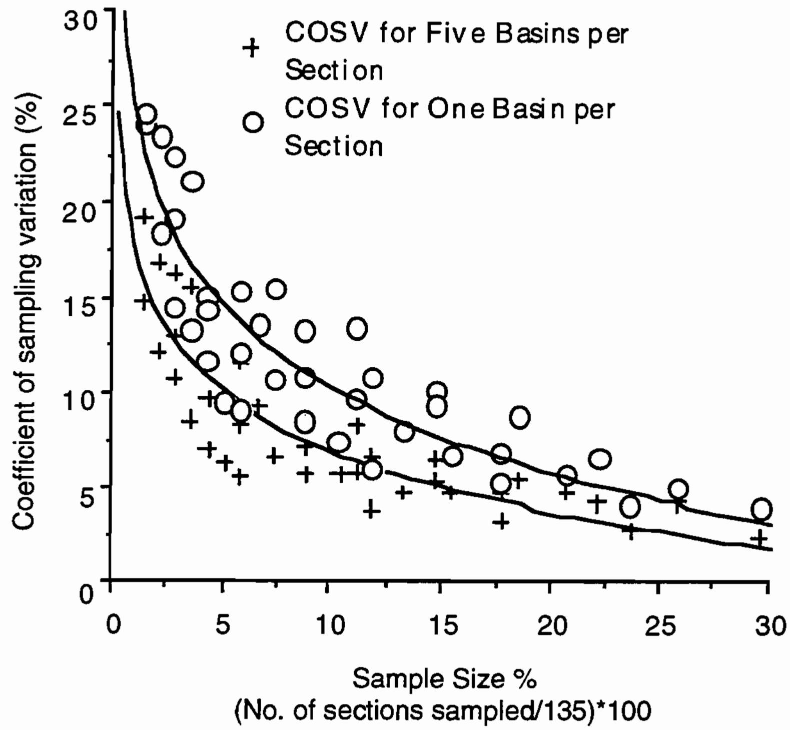

Figure 4.1 Coefficient of sampling variation vs. sample size for one and five basins analyzed per section

4.3.6 Sampling Precision Utility as a Function of Sample Size

To define the utility function for sampling precision, it is necessary to define the boundary conditions. If the sampling precision were the only decision criteria for finding an optimum sample size, then the maximum utility would occur at the largest sample size possible. Conversely, the minimum should occur at the smallest possible sample size. For this study, these extremes are located at the endpoints of the sample range studied. Thus, in terms of sampling precision, the maximum utility (a value of 1) occurs at a sample size of 29.6 percent, while the minimum utility (a value of 0) occurs at a 1.5 percent sample size.

To define the curve between the endpoints, a simple rearrangement of the COSV versus sample size plot was made so that the values could satisfy the boundary conditions. This value, called the Precision Utility (PU), is given by:

$$PU = 1 - \frac{COSV - COSV_{min}}{COSV_{max} - COSV_{min}} \quad \text{(Eq. 4.4)}$$

where:

$COSV_{max, min}$ = the largest and smallest values of the COSV computed from the simulation respectively,

COSV = the value of the COSV at any given sample size, and

PU = the Precision Utility value that is equal to 0 at the smallest sample size and 1 at the largest sample size studied.

Now that the PU is defined, a function of PU with sample size must be found. This function must also satisfy the boundary conditions (i.e., equal to 0 at the smallest sample size and 1 at the largest), so it should take the following form:

$$PU = \left(\frac{(SS - SS_{min})}{(SS_{max} - SS_{min})}\right)^X \quad \text{(Eq. 4.5)}$$

where:

SS = sample size,

$SS_{max, min}$ = the maximum (29.6 percent) and minimum (1.5 percent) sample sizes respectively, and

X = a regression coefficient that gives the best fit to the points on a plot of the calculated PU values from Eq. 4.4 versus sample size.

A plot of the PU values calculated from Equation 4.4 and the function defined by Equation 4.5 (with the value of the regression coefficient X that gives the best fit to those points) are shown in Figure 4.2. The curve fit equation for the PU values is given by:

$$PU = \left(\frac{(SS - 1.5)}{28.1}\right)^{0.327} \quad \text{(Eq. 4.6)}$$

4.3.7 Development of a Utility Curve for Cost

To create a utility curve for cost, it is necessary to begin by assuming a model that predicts cost as a function of sample size. It was assumed that the cost for a given sample size would include both a fixed overhead cost, which is constant for all sample sizes, and a cost for data collection that is a linear function of the sample size. This model is given by:

$$\text{Total Cost} = \text{Fixed Cost} + C1 * SS \quad \text{(Eq. 4.7)}$$

where C1 is the cost per percent sample size.

Note that the fixed cost in Equation 4.7 can be considered the cost required to begin the sampling program. This is given by the costs at the smallest allowable sample, or 1.5 percent for this study. The cost model then becomes:

$$C(SS) = C(1.5\%) + C1 * (SS-1.5\%) \qquad \text{(Eq. 4.8)}$$

where:

C(1.5 percent) = a constant cost that includes the initial costs and the costs for sampling the first 1.5 percent, and

C(SS) = the total cost as a function of sample size.

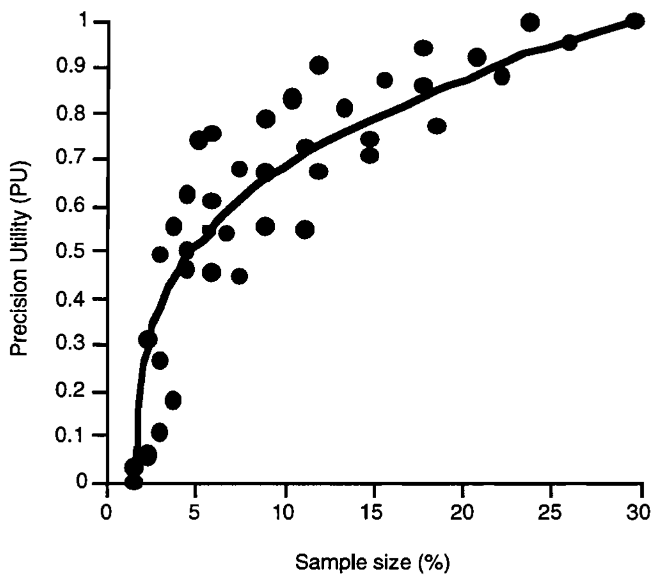

Figure 4.2 Plot of calculated precision utility values vs. sample size with appropriate curve fit

As before, it is necessary to translate this cost into a dimensionless number so that it may be used with the utility theory. To do this, a cost ratio will be defined. The cost ratio will be the ratio of the cost at a given sample size to the maximum cost (i.e., the cost at the maximum sample size). Because in the simulation study discussed above the minimum allowable sample size was 1.5 percent, the cost ratio will be defined as follows:

$$CR = \frac{C(SS) - C_{min}}{C_{max} - C_{min}} \qquad \text{(Eq. 4.9)}$$

where:

CR = the cost ratio,

$C(SS)$ = the cost at a particular sample size, SS,

C_{min} = the cost at the minimum allowable sample size, C(1.5 percent), and

C_{max} = the cost at the maximum allowable sample size, C(29.6 percent).

Arranging Equation 4.8 into Equation 4.9, it is found that the cost ratio is independent of both the initial cost (C(1.5 percent)) and the cost per percent sample size (C1). The final equation for the cost ratio is:

$$CR = \frac{SS - SS_{min}}{SS_{max} - SS_{min}} \qquad \text{(Eq. 4.10)}$$

where all the terms have been defined previously.

The cost utility function measures the relative preference of one sample size over all others when considering cost as the only criterion. As discussed earlier, the utility will be maximum at the lowest sample size and minimum at the highest sample size. Although the cost ratio function, Equation 4.10, is dimensionless and suitable for use in utility curves, it does not fit the boundary conditions necessary for a cost utility function. Therefore, the following form was used to represent the cost utility (CU):

$$CU = 1 - CR = 1 - \frac{SS - SS_{min}}{SS_{max} - SS_{min}} = 1 - \frac{SS - 1.5}{28.15} \qquad \text{(Eq. 4.11)}$$

A plot of this function is shown in Figure 4.3.

4.3.8 Development of the Total (Combined) Sampling Utility Curve

Finally, the cost (CU) and precision utility (PU) curves were combined into a total sampling utility curve (SU). This is done by using the following equation:

$$SU = W_1\, PU + W_2\, CU \qquad \text{(Eq. 4.12)}$$

where

W_1 and W_2 are weights that must sum to 1.

The values of the two weights reflect the emphasis that is placed on the two decision criteria by TxDOT. Therefore, unless there is a reason for selecting other than equal values for the weights (which gives equal importance to both the decision criteria), their values should be 0.5.

Arranging Equations 4.11 and 4.5 (with the regression coefficient X = 0.327) into Equation 4.12 , the final SU function is given by:

$$SU = W_1\left(\frac{(SS - SS_{min})}{(SS_{max} - SS_{min})}\right)^{0.327} + W_2\left(1 - \frac{(SS - SS_{min})}{(SS_{max} - SS_{min})}\right) \quad \text{(Eq. 4.13)}$$

Since the two weights must sum to 1, Equation 4.12 can be expressed as:

$$SU = W_1\left(\frac{(SS - SS_{min})}{(SS_{max} - SS_{min})}\right)^{0.327} + (1-W_1)\left(1 - \frac{(SS - SS_{min})}{(SS_{max} - SS_{min})}\right) \quad \text{(Eq. 4.14)}$$

Substituting the values of SS_{max} (29.6 percent) and SS_{min} (1.5 percent) in Equation 4.14 gives:

$$SU = W_1\left(\frac{(SS - 1.5)}{28.1}\right)^{0.327} + (1-W_1)\left(1 - \frac{(SS - 1.5)}{28.1}\right) \quad \text{(Eq. 4.15)}$$

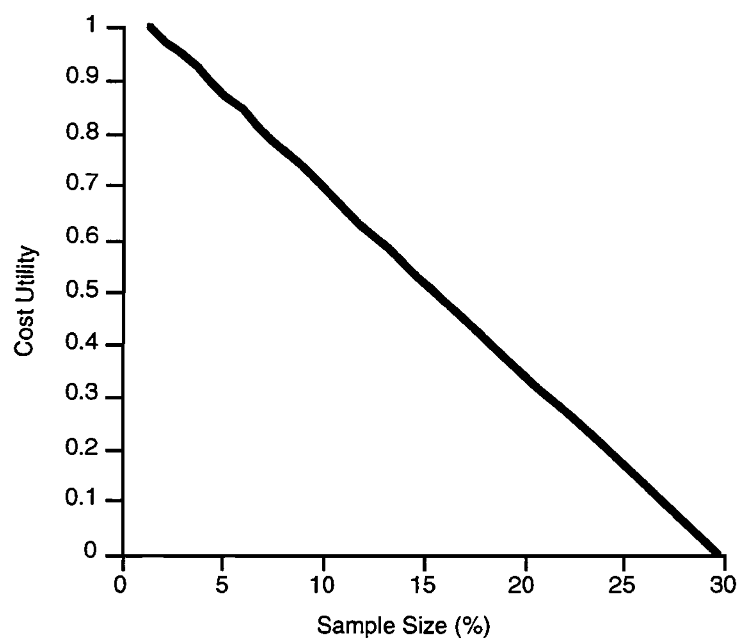

Figure 4.3 Cost utility vs. sample size

Three curves for the Sample Utility (SU) versus Sample Size (SS) are shown in Figure 4.4, using W_1 equal to 0.25, 0.50, and 0.75. The optimum sample size occurs at the sample size where the SU value is a maximum. The optimum sample sizes for the given weights are 2.5

percent, 6.8 percent, and 28.8 percent, respectively. As shown in Figure 4.4, the values of the weights affect the optimum sample size greatly.

The maximum combined utility will occur where the combined utility function (Eq. 4.15) has a slope of 0. This is expressed by:

$$SS_{opt.} = 28.1\left(\frac{0.327\ W_1}{1 - W_1}\right)^{1.486} + 1.5 \qquad \text{(Eq. 4.16)}$$

This equation can be used to compute the optimum sample size if the chosen weights are not equal.

4.3.9 Recommended Sample Size for FWD Data Collection

Developed in the previous section was a utility curve that can be used to find the optimum sample size for rigid pavement deflection data collection. It was shown that the chosen value of the weights in the utility function affects the optimum sample size greatly. Unless there is a specific justification for using unequal weights, the values of the weights should be equal to 0.5. Using equal weights, the optimal sample size for data collection is 6.8 percent

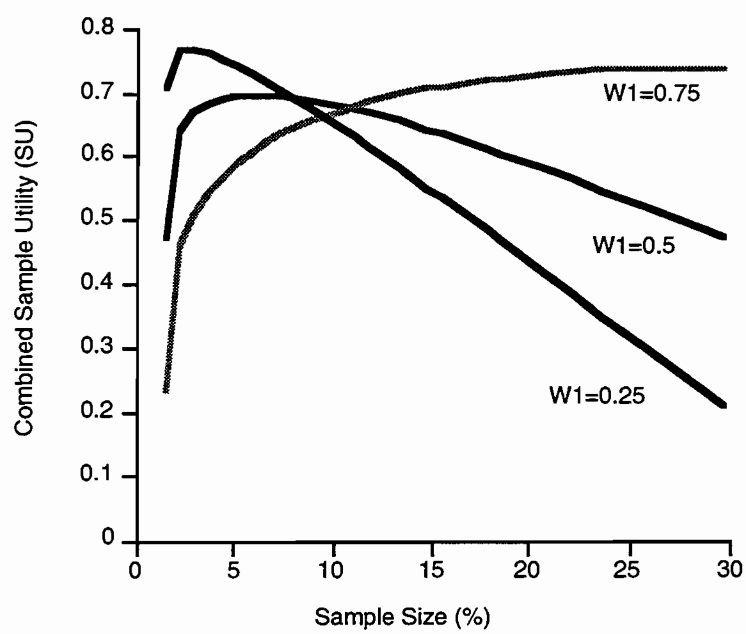

Figure 4.4 Combined sampling utility vs. sample size for W1 = 0.25, 0.5, and 0.75.

4.3.10 Translation of the CTR Sample Size into the PMIS

One important consequence of using the CTR data for computing an optimum sample size is that sections in the CTR database are defined in one direction only. Therefore, in the simulation study, sections located on opposite sides of the same mile marker were separate.

Because the PMIS defines a single section as all the lanes in both directions, the question remains as to how to translate the optimum sample size from the CTR definition into the PMIS definition. Two alternatives are available: Testing for the PMIS could be performed in one direction only, or testing could be performed in both directions for each PMIS section.

Data collection using the FWD for asphalt pavements in Texas is performed in the outside lanes only. This is due to safety concerns and to the fact that truck traffic is heaviest in these lanes. Testing for rigid pavements should also be performed in the outside lanes only. If testing is undertaken in one direction, there are two possible lanes that can be selected for testing when a PMIS section is chosen in the sample: the outside lane in either direction. For PMIS sections, the number of available lane kilometers for testing per section is twice the number of centerline kilometers available for selection in the sampling process. If testing is done in one direction, then only half of the available lane kilometers per section is tested. Thus, the sample size defined using centerline kilometers is twice the size of the sample defined using lane kilometers. Note that the percentage of lane kilometers sampled remains the same.

This is best illustrated by a simple example. Suppose there are 322 lane km (200 lane miles) in a population, from which a 50 percent sample is to be randomly selected. Then 161 lane km (100 lane miles) are to be tested. If the roadbed has two lanes (which is essentially the case here, since only the two outside lanes are available for sampling), then there are 161 centerline km (100 centerline miles) in the population. If 80 centerline km (50 centerline miles; a 50 percent sample defined using lane km and lane miles) are selected for testing in one direction, then 80 lane km (50 lane miles) will be tested. This is only a 25 percent sample of the 322 available lane km (200 available lane miles). Therefore, to select an equal percentage of lane km (miles) the percentage of centerline km (miles) that must be selected is twice the percentage of lane km or miles (100 percent for this example).

For the second alternative, the optimum sample size is not as easily defined. To keep the length of lane distances sampled constant, the optimum sample size in centerline distance is the same as given for direction distance, since two *lane* distances are sampled for every one *centerline* distance. This size sample will not provide the same amount of information about the pavement network. The optimum sample size calculated from the CTR data considered each pavement section to be independent; that is, the measured deflections are not correlated between pavement sections. If deflections are measured on both sides of the same road, then the measurements cannot be considered independent. Therefore, the optimum amount of deflection basins needed to obtain the same precision as a sample taken from independent observations will be larger. Unfortunately, there are no data presently available that can be used to estimate the increase needed to assure a reasonable level of precision in the sample.

When analyzing the effects of various factors on measured deflections, the deflection basins from both sides of the same road will be used to estimate the variation within the pavement sections. When making statistical inferences, the inferences are based on variation between — not within — sections. By collecting more data within each pavement section, the available number of degrees of freedom to study any factor is reduced. If one basin per section is collected from a larger number of pavement sections, the number of degrees of freedom available to study the main effects and second-level interactions of any factors is increased, although some higher level interactions (third and above) cannot be studied. The analyses will then have a larger inference space and be more appropriate for network-level use. Therefore, it is recommended that deflection measurements be taken in one direction only and that the sample size (in centerline distance) be 13.6 percent.

4.4 RECOMMENDED FIELD PROCEDURES FOR FWD DATA COLLECTION

The final recommendations for a network-level data collection plan involve the procedures to be followed in the field by the data collection personnel. TxDOT presently collects FWD data for flexible pavements at the network level. The data collected through this program are stored in the PMIS database for use in pavement management activities. The collection of rigid FWD data at the network level can be incorporated into the flexible pavement data collection program with only minor changes in the flexible pavement data collection procedures.

The 1993 PMIS manual describing data collection procedures for flexible pavements (Ref 17) lists the following items that need to be specified for the FWD operators:

1) Number of test points (basins to be collected) within a section
2) Number of FWD drops and load magnitude at each test point
3) Number of roadways tested per section
4) Lanes tested and location on lane
5) Configuration (location in relation to the load plate) for the FWD geophones
6) Any other data to be collected at the testing site

4.4.1 Number of Test Points Within a Section

The PMIS flexible data collection procedure calls for one test point per section. The testing procedure requires that it be located at or as close to the first reference marker of the .80 meter (half-mile) pavement section as possible. Discussed in section 4.3.5 were the benefits of collecting more than one basin per section sampled. There, it was recommended that only one basin per section be collected. To remain as consistent as possible with the flexible data collection procedure, the test should be taken at the beginning of the PMIS section.

4.4.2 Number of Drops and Load Magnitude at Each Test Point

Presently for flexible pavements, the load weights are to be dropped a total of three times per test point. The first two drops are for seating of the loading plate, while the third is for storage in the PMIS database. It is recommended that this procedure also be followed for rigid pavement data collection.

The load magnitude for flexible pavements should be as close to 4080 kg (9000 lb) as possible to estimate the pavement response under one tire of an 8170-kg (18-kip) single axle load. The first two drops can be used by the operator to determine which of the weight/drop height configurations should be used to attain this level of load. A 4080 kg (9000 lb) load should also be used for the rigid pavement data collection.

4.4.3 Number of Roadways Tested per Section

Many of the highways in Texas have frontage roads along the sides of the highway. Data are not being collected from the frontage roads for the PMIS. Therefore, for the rigid pavement data collection plan, there is no need to collect data on the frontage roads; only the main lanes should be tested with the FWD.

4.4.4 Lane Tested and Location on Lane

To ensure safety, network-level FWD testing is normally done in the outside lanes. This requirement should also be followed for rigid pavements. One of the major differences between network-level deflection data collection for flexible and rigid pavements is the location of the FWD test on the lane. For flexible pavements, the test is usually done in the outside wheel path, since this is where most structural damage occurs. However, for rigid pavements, the goal of the network-level deflection data collection is to characterize the structural capacity of the pavement. As discussed in Chapter 2, this is done best by measuring midslab deflections. Therefore, the deflection testing should be located in the middle of the lane and as far from joints and cracks as possible. Also discussed in Chapter 2 is the effect of load placement error on the deflection basin measurements. A placement error of only 12.7 cm (5 in.) with respect to the pavement edge can affect the measurements significantly. Thus, the FWD must be consistently placed at the centerline of the lane.

4.4.5 Geophone Configuration

The location of the geophones in relation to the loading plate should remain the same as the configuration used for the flexible data collection, so that no changes are required for either rigid or flexible data collection. This configuration has one geophone underneath the load and six more spaced at 0.3-m (1-foot) intervals in front of the load plate.

4.4.6 Pavement Surface Temperature

Another consideration that should be addressed when collecting FWD data is the temperature of the pavement slab. The effects of temperature on measured deflections were discussed in Chapter 2 . The temperature of the pavement surface at the time of testing should be recorded for future analysis. The Dynatest FWDs owned by TxDOT have infrared temperature sensors that measure the surface temperature of the pavement during testing. It is imperative that these data be collected and stored in the PMIS database.

4.4.7 PCC Thickness

The thickness of the concrete slab is one of the most important variables required when analyzing rigid pavement FWD data. Therefore, a method by which this information can be obtained quickly and reliably needs to be investigated so that it may be used in future analyses.

4.4.8 Directions Sampled

The value of the optimum sample size discussed earlier is given in direction distance (twice the centerline distance). Therefore, testing needs to be performed only in one direction, given the sample size is computed from either direction distance or twice the centerline distance.

4.4.9 Time of Day and Date

The time of day that the measurement was taken can be used in the future to study the effects of temperature differential on the measured midslab deflections. For any future seasonal studies, the date of the test should also be noted.

4.4.10 Data Storage

The data to be stored in the database should include the following: (1) the deflection basin (deflections at each geophone); (2) time of day and test date; (3) pavement surface temperature; (4) pavement type; (5) direction in which the test was taken; (6) applied load; and (7) PCC thickness (when possible).

This section recommends the testing procedures that should be followed by the data collection personnel in the field. A summary of the proposed procedures and a comparison with the present procedures used for flexible pavement data collection are presented in Table 4.1.

Table 4.1 Proposed rigid and present flexible FWD data collection procedures for the PMIS

	Flexible FWD Testing Procedure	**Proposed Rigid FWD Testing Procedure**
Number of Test Points per Section	One drop at the start of the PMIS section	One drop also at the beginning of the PMIS section
Number of Drops per Section and Load Magnitude	Three drops at 4080 kg (9000 lb): two for seating the load plate, the last for storage	Three drops at 4080 kg (9000 lb): two for seating the load plate, the last for storage
Roadways Tested	Main lanes only (no frontage roads)	Main lanes only (no frontage roads)
Lanes Tested	Outside lanes only (closest to the shoulder)	Outside lanes only (closest to the shoulder)
Location of Test Point on Lane	Outside wheelpath	Centerline of lane as far as possible from joints and cracks.
Temperature Data Collection	Not required	Essential
Directions	One direction only	One direction only

4.5 SUMMARY

This chapter discussed the aspects of a network-level deflection data collection plan. Three main subjects were addressed: the type of random sampling technique that should be used for selection of pavement sections; the optimum size of that random sample; and the testing procedures that should be followed by the data collection personnel at the selected pavement sections.

First, the type of random sampling technique that should be used to select pavement sections from the network was discussed. It was found in the literature that a two-stage stratified random sample should be used for this purpose. This type of random sample would involve collecting data from all the TxDOT districts using counties as the first-stage random variable, and the pavement sections as the second-stage random variable.

The second item addressed in this chapter was the size of the sample that should be used. Using utility theory and a sampling simulation, the optimum sample size was found to be 6.8 percent of the lane distance.

The third item addressed in this chapter was the testing procedures that should be followed at the pavement sections by the operators of the FWD. Table 4.1 summarizes the recommendations made in this section.

The following chapter covers the costs of implementing the recommended data collection plan discussed in this chapter.

CHAPTER 5. COST FOR IMPLEMENTING A NETWORK LEVEL RIGID PAVEMENT FWD DATA COLLECTION PROGRAM

Before implementing the data collection plan discussed in Chapter 4, it is important to estimate the cost for the implementation of the program. This chapter will discuss the present costs for the flexible FWD data collection program and, based on these, give an estimate of the costs for the rigid data collection plan recommended in Chapter 4 and in section 5.2.

5.1 PRESENT COSTS FOR NETWORK-LEVEL FLEXIBLE PAVEMENT FWD DATA COLLECTION IN TEXAS

The costs for the network-level flexible pavement data collection program now in use for the PMIS system were recently calculated by Doug Chalman, the FWD data collection coordinator at TxDOT. The figures calculated by him were based on the average distance per day that the operators could collect data. The average cost per mile of data collection was calculated by adding the total costs per day for payroll, travel, mileage and per diem for the operators of the FWD and sign crews and dividing by the average number of miles of data collection completed per day. A summary of these cost calculations are given below.

Number of full-time employees (FTEs) needed to operate the FWD and two sign vehicles: 4
Number of Vehicles Needed: 3 (1 FWD Van & Trailer and 2 Sign)
Per diem must be paid for travel in excess of 145 km (90 miles) away from FWD region center (not including data collection mileage).
Average number of km / miles of data collection per day: 96.6 km / 60 miles
Number of km / miles traveled per day: 145 km / 90 miles
Manpower Cost: 4 FTEs x \$20 per hour x 8 hours per day
= \$ 640 per day payroll cost
Vehicle Costs: 3 vehicles x \$1.50 per mile traveled x 90 miles travel per day = \$405 vehicle costs
Total cost per day for local data collection = \$640 + \$405 = \$1045 per day
Average cost per mile (local) = \$1045 per day/60 miles data collection per day = \$17.42 per mile
Per diem cost for remote (>145 km [90 miles] from region center) data collection:
= 2 FTEs (sign crews are local) x \$80 per day
= \$160 per day for per diem
Total cost for remote data collection = \$1045 + \$160 = \$1205 per day
Average Cost per mile: = \$1205 / 60 miles per day
= \$20.08 per mile for remote data collection
(= \$12.47 per km)

The costs given above are per mile (km) in each direction. These are the average costs per mile (km) as of May 1994, for the flexible deflection data collection program. The flexible program uses a sample size of approximately 20 percent of the centerline distance per year of the flexible pavements in Texas. There are approximately 119,000 centerline km (74,000 centerline miles) of flexible pavements in Texas, of which a 20 percent sample is approximately 23,800 centerline km (14,800 centerline miles) of pavement. Therefore the total cost for flexible pavement FWD data collection is at least $250,000 per year (this is the cost if all sections were within a 145- km [90-mile] radius of their region centers) based on these calculations.

5.2 COST FOR RIGID PAVEMENT DATA COLLECTION

The cost for the rigid pavement data collection plan can be estimated based on the costs for the flexible data collection plan. The recommended testing procedures for rigid FWD testing given in Chapter 4 represent only minor changes to those procedures presently in use for the flexible pavements, as the major differences are the placement of the load on the pavement and the collection of the pavement surface temperature at the time of the test. Thus, the cost for data collection per mile (km) will be the same for both rigid and flexible sections.

5.2.1 Number of Centerline Miles of Rigid Pavements in Texas

An estimate of the number of centerline kilometers/miles of rigid pavement in Texas was made based on sample size percentages given in the 1986-1990 PES Annual Report (Ref 9). There were 3,214 centerline km (1,996 centerline miles) of continuously reinforced and 1,436 centerline km (892 centerline miles) of jointed concrete pavements in Texas in 1990.

5.2.2 Number of PMIS Sections to be Tested for the Rigid Data Collection Plan

The sample size recommended in Chapter 4 is for direction distance of pavement, since each of the pavement sections in the CTR database is in only one direction. Therefore, the recommended percentage of centerline kilometers/miles to be sampled (testing in only one direction) is twice the recommended percentage of direction kilometers/miles, or 13.6 percent. This gives a sample size of 437.1 centerline km (271.5 centerline miles) for CRCP and 195.6 km (121.5 miles) for JCP, a total of 632.7 km (393.0 miles) combined. Because PMIS sections are defined to be about .80 km (one-half mile) in length, the number of rigid PMIS sections that should be tested is approximately 786 per year.

5.2.3 Local vs. Remote Data Collection for Rigid Pavements

For use in network-level data collection, TxDOT has thirteen FWDs located at thirteen district offices throughout the state. Each of these districts is responsible for FWD data collection in its region. If the selected sections for data collection are further than 145 km (90 miles) away from these district offices, the cost per mile will increase from $17.42 to $20.08. To estimate the cost for data collection, an estimate of the amount of data collected at the higher cost per mile was needed. The 1990 PES database was searched to find the number of miles of rigid pavement in each county in the state. Then the length of the pavements in counties outside a 90-mile (145-km) radius from their FWD region center were divided by the total length of the rigid pavements of that type (CRCP or JCP). This gave the best available estimate of the percentage of data collection that requires a higher rate per mile. Table 5.1 lists the counties with rigid pavements found in the 1990 PES database that are outside a 145-km (90-mile) radius from their district centers.

Table 5.1 Counties outside a 145 km (90-mile) radius from district center

FWD Region Center	**Counties with Rigid Pavements Outside a 145-km (90-mile) Radius**
Dallas, Amarillo, Corpus Christi, San Antonio, Pharr, Bryan, Waco, Houston, Lufkin, Atlanta	None
Abilene	Wilbarger, Wichita, Clay, Montague, Cooke, Swisher, Hale and Lubbock
Odessa	El Paso and Hudspeth
Austin	Victoria, Jackson, and Wharton

Using the PES database, 6.2 percent of the JCP and 20.4 percent of the CRCP sections were located further than 145 km (90 miles) from their respective FWD region centers. Assuming that any random sample taken will also have these same percentages of remote data collection, a cost estimate for the rigid data collection plan can be made.

5.2.4 Cost Estimate for Network Level Rigid Pavement FWD Data Collection

To make an estimate of the cost for data collection, it was necessary to make the following assumptions:

1) The average cost per km/mile for both rigid and flexible data collection are the same.
2) Any overhead costs are reflected in the average cost per km/mile calculated for flexible pavements.
3) A random sample of rigid pavements will contain the same percentage of pavement sections that are further than 145 km (90 miles) away from their FWD region headquarters.
4) The collection of FWD data on rigid sections is concurrent with the collection of FWD data on flexible pavements, so that the travel time between sections (rigid or flexible) is minimized.

Using these assumptions and the recommended sample size calculated above, the total cost for data collection is given by:

$$\text{Total Cost} = \left(6.2\%\left(\frac{\$20.08}{\text{mile}}\right) + 93.8\%\left(\frac{\$17.42}{\text{mile}}\right)\right)\text{Number of JCP miles}$$
$$+\left(20.4\%\left(\frac{\$20.08}{\text{mile}}\right) + 79.6\%\left(\frac{\$17.42}{\text{mile}}\right)\right)\text{Number of CRCP miles}$$

Substituting for the number of miles of CRC and JCP sections gives:
Total Cost = $7,014 per year.

5.3 SUMMARY

This chapter presented a cost estimate for implementing the data collection plan described in Chapter 4. The final cost was estimated to be $7,014 and would probably range between $7,000 and $10,000 per year. The cost for implementing this data collection plan is quite modest compared with the expense associated with flexible data collection. However, it is important to note that the average costs per mile used for the cost estimate is based on the travel times between flexible sections. Since the number of sampled flexible pavement sections is so much greater than the number needed for the rigid plan, the cost estimate calculated here is only valid if the rigid and flexible plans are integrated so that the travel times between sections remain constant. The rigid pavement testing procedure recommended in Chapter 4 specifies the same instruments and personnel used for flexible data collection, making possible the simultaneous collection of both rigid and flexible data.

CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS

The purpose of this study was to evaluate network-level FWD data and data collection for rigid pavements in Texas. The quality of the existing data contained in the PES database was found to be too questionable for any network-level study of the structural behavior of rigid pavements. The PES data were evaluated by comparing them with data contained in the CTR rigid pavement database. Based on that comparison, the data were found to be questionable and were therefore not used for any further analysis. To allow for the network-level evaluation of the structural behavior of rigid pavements, recommendations for the implementation of a FWD data collection plan for rigid pavements at the network level were made. The optimum sample size, testing procedures, and cost estimate for the data collection plan were given.

6.1 CONCLUSIONS

The following are conclusions drawn from this study:

1.) Any analysis of rigid pavement deflection measurements must take into account the temperature of the rigid pavement slab.

2.) When deflection measurements are used for network-level evaluations, the placement of the FWD loading plate (with respect to the pavement edge, joints, and cracks) must be consistent for all measurements; otherwise statistical inferences based on the data become difficult.

3.) The PES database's rigid pavement deflection data taken between 1987 and 1990 seem to be of questionable origin.

4.) The PES database does not contain enough information about the structure of the pavement layers to allow for the analysis of rigid pavement deflection data.

5.) In sampling rigid pavement deflection data from the pavement network, a two-stage stratified sampling method was found to be the most efficient.

6.) The most cost-effective sample size was found to be 13 percent of the centerline mileage in one direction using a two-stage stratified sampling scheme. This yields approximately 786 PMIS sections per year.

7.) The FWD should be placed at the center of the lane and as far from any joints or cracks as possible when used to collect network-level deflection data.

8.) The testing procedures necessary for collecting rigid pavement deflection data are similar to those needed for flexible pavements. Therefore, both rigid and flexible data can be collected simultaneously with the same FWD device.

9.) The cost of implementing the deflection data collection program for rigid pavements outlined in this report was estimated at $7,014 per year. It is expected that the actual cost will range between $7,000 and $10,000 per year.

6.2 RECOMMENDATIONS

Recommendations for future research and the implementation of the results of the study are presented below.

1.) The Dynatest FWDs are equipped with infrared thermometers that can record the surface temperature of the pavement. However, the current thermometers have not proven to be accurate or precise enough to give reliable readings. Improved thermometers are available, and TxDOT should purchase these thermometers for use in collecting pavement surface temperature data.
2.) The thickness of the pavement layers is an important quantity in the analysis of rigid pavement deflection data. This information is not always available for network-level analyses. Further study is needed regarding the efficient determination of pavement layer thicknesses at the location of the deflection test.
3.) The PES database's rigid pavement deflection data taken between 1987 and 1990 should be removed from the database or marked as questionable so that no further analyses are made using the data.
4.) Before rigid pavement data are stored in the PMIS database, the quality of the data should be assessed using the method described in Chapter 3.
5.) The collection of network-level deflection data for rigid pavements should be incorporated into the regular deflection data collection program now used for flexible pavements in the PMIS.

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Appendix A:

Sampling Simulation SAS Code and Output

Listing of SAS Program CTRSAMP

```
*****************************************************************
Program To Draw 300 Random Samples Of FWD Data From The CTR Database
Project 1908          *Program CTRSAMP*        E. Perrone 2-28-94
Significant Variables:
        SECTID-       Individual Section Id Number Assigned By Program
                      AssignEach Section Differentiated By The Cftr Sect and
                      Dir (See Ctr R.R 472-6 For Ctr Crcp Data Base Variables)
        SAMPLE-       Sample Iteration Number (1-300)
        NDSTR-        Number Of Pavement Sections To Be Sampled From
                      Each District
        NDSAMP-       Number Of Districts To Be Sampled From
        DF1,DF7-      Deflection Readings At Each Geophone
        MF1,Mdf7-     Mean Deflection Readings For Each Pavement Section
        SDF1,SDF7-    Standard Deviation Of Geophone Readings for
                      Each Section
        NSECT-        Number Of Sections In Each Sampled County
        NCOUNT-       Number Of Counties In Each Sampled District
        STDF1,STDF7-          Standard Deviations For District Estimates Of
                      Mean Deflections At Each Geophone
        DISTRICT-     District That The Pavement Section Is In
        HEIGHT-       Drop Height At Which The Deflection
                      Measurement was Taken
*****************************************************************

DATA A; KEEP SAMPLE SECTID; DD1=5;DD2=47;DD3=30;DD4=10;
DD5=9;DD6=12;DD7=13;DD8=9;
ARRAY DIST (NN) 8 DD1-DD8;NDSTR=2;NDSAMP=8;
D1=0;
D2=0;
D3=0;
D4=0;
D5=0;
D6=0;
D7=0;
D8=0;
DO SAMPLE=1 TO 300;
   IF NDSAMP NE 8 THEN DO;
   DS=11111111;
      DO DSAMP=1 TO NDSAMP;
         RD=RANUNI(0); IF RD=0 THEN RD=1;
         DIGIT=CEIL(RD*8);
         CHECK=MOD(INT(DS/10**(DIGIT-1)),10);
         IF CHECK EQ 0 THEN
            DSAMP=DSAMP-1;
```

```
        ELSE DO;
            IF DSAMP=1 THEN D1=DIGIT;
            IF DSAMP=2 THEN D2=DIGIT;
            IF DSAMP=3 THEN D3=DIGIT;
            IF DSAMP=4 THEN D4=DIGIT;
            IF DSAMP=5 THEN D5=DIGIT;
            IF DSAMP=6 THEN D6=DIGIT;
            IF DSAMP=7 THEN D7=DIGIT;
            IF DSAMP=8 THEN D8=DIGIT;
            DS=((DS/10**(DIGIT-1))-1)*10**(DIGIT-1);
            DS=CEIL(DS);
        END;
    END;
END;
ELSE DO;
    D1=1; D2=2; D3=3; D4=4; D5=5; D6=6; D7=7; D8=8;
END;

ARRAY DARRAY (D) 8 D1-D8;

DO D=1 TO NDSAMP;

    DO SAMPLE2=1 TO NDSTR;
        NN = 1;
        X=RANUNI(0);
        PT=INT(X*(DIST-1) +1);
        IF DARRAY=1 THEN DO;
            SET SDS.D1 POINT=PT;
            IF _ERROR_=1 THEN ABORT;
            OUTPUT;
        END;
        IF DARRAY=2 THEN DO;
            SET SDS.D2 POINT=PT;
            IF _ERROR_=1 THEN ABORT;
            OUTPUT;
        END;
        IF DARRAY=3 THEN DO;
            SET SDS.D3 POINT=PT;
            IF _ERROR_=1 THEN ABORT;
            OUTPUT;
        END;
        IF DARRAY=4 THEN DO;
            SET SDS.D4 POINT=PT;
            IF _ERROR_=1 THEN ABORT;
            OUTPUT;
        END;
        IF DARRAY=5 THEN DO;
```

```
                SET SDS.D13 POINT=PT;
                IF _ERROR_=1 THEN ABORT;
                OUTPUT;
            END;
            IF DARRAY=6 THEN DO;
                SET SDS.D17 POINT=PT;
                IF _ERROR_=1 THEN ABORT;
                OUTPUT;
            END;
            IF DARRAY=7 THEN DO;
                SET SDS.D20 POINT=PT;
                IF _ERROR_=1 THEN ABORT;
                OUTPUT;
            END;
            IF DARRAY=8 THEN DO;
                SET SDS.D24 POINT=PT;
                IF _ERROR_=1 THEN ABORT;
                OUTPUT;
            END;
        END;
    END;
END;
STOP;
PROC SORT; BY SECTID SAMPLE;
/***********************************************************/
DATA B; SET SDS.FWDVAR; IF HEIGHT=4; IF D=8; KEEP SECTID
DF1-DF7 D SS HEIGHT DISTRICT COUNTY CFTR SECT DIR LBS;
 IF STATION=5;
 DF1=16000* DF1/LBS;
 DF2=16000* DF2/LBS;
 DF3=16000* DF3/LBS;
 DF4=16000* DF4/LBS;
 DF5=16000* DF5/LBS;
 DF6=16000* DF6/LBS;
PROC SORT; BY SECTID;
PROC MEANS NOPRINT;VAR DF1-DF7;BY SECTID;
 ID HEIGHT DISTRICT COUNTY CFTR
SECT D DIR;
OUTPUT OUT=SDS.MEANFWD  MEAN=MDF1-MDF7
 STD=SDF1-SDF7;*/
/***********************************************************/
DATA C; MERGE A(IN=OK) SDS.MEANFWD;BY SECTID;IF OK;
KEEP SECTID SAMPLE MDF1-MDF7
    SDF1-SDF7 NSECT NCOUNT DISTRICT;
IF INT(SECTID/100)=11 THEN NSECT=5;
IF INT(SECTID/100)=21 THEN NSECT=4;
IF INT(SECTID/100)=22 THEN NSECT=3;
```

```
IF INT(SECTID/100)=23 THEN NSECT=9;
IF INT(SECTID/100)=24 THEN NSECT=27;
IF INT(SECTID/100)=25 THEN NSECT=4;
IF INT(SECTID/100)=31 THEN NSECT=6;
IF INT(SECTID/100)=32 THEN NSECT=15;
IF INT(SECTID/100)=33 THEN NSECT=9;
IF INT(SECTID/100)=41 THEN NSECT=4;
IF INT(SECTID/100)=42 THEN NSECT=6;
IF INT(SECTID/100)=131 THEN NSECT=9;
IF INT(SECTID/100)=171 THEN NSECT=6;
IF INT(SECTID/100)=172 THEN NSECT=5;
IF INT(SECTID/100)=173 THEN NSECT=1;
IF INT(SECTID/100)=201 THEN NSECT=13;
IF INT(SECTID/100)=241 THEN NSECT=9;
IF INT(SECTID/1000)=1 THEN NCOUNT=1;
IF INT(SECTID/1000)=2 THEN NCOUNT=5;
IF INT(SECTID/1000)=3 THEN NCOUNT=3;
IF INT(SECTID/1000)=4 THEN NCOUNT=2;
IF INT(SECTID/1000)=13  THEN NCOUNT=1;
IF INT(SECTID/1000)=17  THEN NCOUNT=3;
IF INT(SECTID/1000)=20  THEN NCOUNT=1;
IF INT(SECTID/1000)=24  THEN NCOUNT=1;

PROC SORT; BY SAMPLE DISTRICT SECTID;

PROC MEANS NOPRINT; VAR MDF1;
    BY SAMPLE DISTRICT; FREQ NSECT; ID NCOUNT;
    OUTPUT OUT=Q  MEAN=MDF1 STD=STDF1;

PROC MEANS NOPRINT; VAR MDF1; BY SAMPLE; FREQ NCOUNT;
    OUTPUT OUT=R  MEAN=MDF1 STD=STDF1;

PROC MEANS; VAR MDF1;
```

Output Data from Sampling Simulation

Number of Districts Sampled	Number of Sections per District	Sample Size	5 Drops per Section	1 Drop per Section
			St. Error	St. Error
8	1	5.93%	0.41	0.67
8	2	11.85%	0.28	0.45
8	3	17.78%	0.24	0.39
8	4	23.70%	0.20	0.30
8	5	29.63%	0.18	0.30
7	1	5.19%	0.46	0.70
7	2	10.37%	0.42	0.56
7	3	15.56%	0.35	0.50
7	4	20.74%	0.35	0.42
7	5	25.93%	0.31	0.37
6	1	4.44%	0.51	0.87
6	2	8.89%	0.42	0.63
6	3	13.33%	0.35	0.59
6	4	17.78%	0.35	0.51
6	5	22.22%	0.31	0.48
5	1	3.70%	0.62	0.98
5	2	7.41%	0.49	0.80
5	3	11.11%	0.42	0.72
5	4	14.81%	0.39	0.69
5	5	18.52%	0.40	0.64
4	1	2.96%	0.79	1.07
4	2	5.93%	0.62	0.88
4	3	8.89%	0.52	0.81
4	4	11.85%	0.49	0.80
4	5	14.81%	0.48	0.74
3	1	2.22%	0.91	1.35
3	2	4.44%	0.71	1.06
3	3	6.67%	0.68	1.00
3	4	8.89%	0.61	0.96
3	5	11.11%	0.61	0.99
2	1	1.48%	1.07	1.79
2	2	2.96%	0.95	1.42
2	3	4.44%	0.87	1.10
2	4	5.93%	0.85	1.09
2	5	7.41%	0.79	1.13
1	2	1.48%	1.38	1.79
1	3	2.22%	1.23	1.70
1	4	2.96%	1.19	1.65
1	5	3.70%	1.14	1.55