TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/TX-91+460-2		

4. Title and Subtitle
ASSESSING LOAD TRANSFER ACROSS JOINTS AND CRACKS IN RIGID PAVEMENTS USING THE FALLING WEIGHT DEFLECTOMETER

5. Report Date
May 1990

6. Performing Organization Code

7. Author(s)
Emmanuel B. Owusu-Antwi, Alvin H. Meyer, and W. Ronald Hudson

8. Performing Organization Report No.
Research Report 460-2

9. Performing Organization Name and Address
Center for Transportation Research
The University of Texas at Austin
Austin, Texas 78712-1075

10. Work Unit No.

11. Contract or Grant No.
Research Study 3-8-86-460

12. Sponsoring Agency Name and Address
Texas State Department of Highways and Public Transportation; Transportation Planning Division
P. O. Box 5051
Austin, Texas 78763-5051

13. Type of Report and Period Covered
Interim

14. Sponsoring Agency Code

15. Supplementary Notes
Study conducted in cooperation with the U. S. Department of Transportation, Federal Highway Administration. Research Study Title: "Assessment of Load Transfer Across Joints and Cracks in Rigid Pavements Using the Falling Weight Deflectometer"

16. Abstract

Loss of load transfer across joints and cracks in rigid pavements is a major factor in rigid pavement distress. And because loss of load transfer has a significant influence on performance and, thus, on the life-cycle cost of rigid pavements, it is therefore necessary to detect such defects as early as possible so that the proper rehabilitation measures can be applied. Accordingly, the Texas State Department of Highways and Public Transportation has purchased a number of Falling Weight Deflectometers for use in evaluating pavements in Texas. This study was instituted primarily to develop techniques for assessing the load transfer efficiency of joints and cracks in rigid pavements using the Falling Weight Deflectometer (FWD) and to determine the factors influencing load transfer efficiency across such discontinuities.

This report describes the evaluation of procedures for assessing the load transfer efficiency of transverse joints and cracks using FWD deflection data collected on a controlled jointed reinforced concrete pavement research facility in a laboratory study. The application of the procedures—on a number of jointed reinforced concrete pavement (JRCP) and continuously reinforced concrete pavement (CRCP) test sections on in-service rigid pavements, in order to determine their suitability for implementation in the field—is also discussed. The results presented indicate that the Falling Weight Deflectometer can indeed be used effectively to evaluate the load transfer efficiency of joints and cracks in rigid pavements and that the results from such evaluations can be used to establish maintenance and/or rehabilitation priorities. The procedures developed were also adapted for use in the evaluation of the efficiency for longitudinal joints in rigid pavements, particularly rigid shoulder joints. Again, the results presented show that with the FWD sensor configuration suggested, deflection measurements can be obtained for use in effectively evaluating the joint efficiency of the types of longitudinal joints found in rigid pavements.

17. Key Words	18. Distribution Statement
rigid pavement, structural evaluation, nondestructive testing, Falling Weight Deflectometer, load transfer efficiency, transverse joints and cracks, longitudinal cracks, PCC shoulders, temperature differential	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	86	

Form DOT F 1700.7 (8-69)

ASSESSING LOAD TRANSFER ACROSS JOINTS AND CRACKS IN RIGID PAVEMENTS USING THE FALLING WEIGHT DEFLECTOMETER

by

Emmanuel B. Owusu-Antwi
Alvin H. Meyer
W. Ronald Hudson

Research Report Number 460-2

Research Project 3-8-86-460

Assessment of Load Transfer Across Joints and Cracks in Rigid Pavements Using the Falling Weight Deflectometer

conducted for

Texas State Department of Highways and Public Transportation

in cooperation with the

U.S. Department of Transportation Federal Highway Administration

by the

CENTER FOR TRANSPORTATION RESEARCH

Bureau of Engineering Research

THE UNIVERSITY OF TEXAS AT AUSTIN

May 1990

PREFACE

This is the second report on the work accomplished under Research Project 3-8-86-460, "Assessment of Load Transfer Across Joints and Cracks in Rigid Pavements Using the Falling Weight Deflectometer." This research project is being conducted by the Center for Transportation Research, The University of Texas at Austin, as part of the Cooperative Highway Research Program sponsored by the Texas State Department of Highways and Public Transportation. The evaluation and verification of procedures for assessing load transfer across joints and cracks in PCC pavements using the Falling Weight Deflectometer are presented in this report.

The authors gratefully acknowledge the technical assistance and support of the staff of the Center for Transportation Research. We especially would like to thank Mr. Carl B. Bertrand, whose help in the collection of the necessary data is greatly appreciated. We would also like to thank the Texas State Department of Highways and Public Transportation for their sponsorship of, and assistance with, this project.

Emmanuel B. Owusu-Antwi
Alvin H. Meyer
W. Ronald Hudson

LIST OF REPORTS

Research Report 460-1, "Temperature Differential Effect on the Falling Weight Deflectometer Deflections Used for Structural Evaluation of Rigid Pavements," by Gustavo E. Morales-Valentin, A. H. Meyer, and W. R. Hudson, presents a recommended methodology for either avoiding or removing the effect of a vertical temperature differential within a pavement slab on measured Falling Weight Deflectometer deflections. February 1987.

Research Report 460-2, "Assessing Load Transfer Across Joints and Cracks in Rigid Pavements Using the Falling Weight Deflectometer," by Emmanuel B. Owusu-Antwi, Alvin H. Meyer, and W. Ronald Hudson, presents the evaluation and verification of a procedure developed for assessing the load transfer efficiency of joints and cracks in PCC pavements. Conclusions and recommendations are presented on the use of the procedure for load transfer efficiency evaluation on pavements in service as part of a comprehensive structural evaluation program of PCC pavements. May 1990.

ABSTRACT

Loss of load transfer across joints and cracks in rigid pavements is a major factor in rigid pavement distress. And because loss of load transfer has a significant influence on performance and, thus, on the life-cycle cost of rigid pavements, it is therefore necessary to detect such defects as early as possible so that the proper rehabilitation measures can be applied. Accordingly, the Texas State Department of Highways and Public Transportation has purchased a number of Falling Weight Deflectometers for use in evaluating pavements in Texas. This study was instituted primarily to develop techniques for assessing the load transfer efficiency of joints and cracks in rigid pavements using the Falling Weight Deflectometer (FWD) and to determine the factors influencing load transfer efficiency across such discontinuities.

This report describes the evaluation of procedures for assessing the load transfer efficiency of transverse joints and cracks using FWD deflection data collected on a controlled jointed reinforced concrete pavement research facility in a laboratory study. The application of the procedures—on a number of jointed reinforced concrete pavement (JRCP) and continuously reinforced concrete pavement (CRCP) test sections on in-service rigid pavements, in order to determine their suitability for implementation in the field—is also discussed. The results presented indicate that the Falling Weight Deflectometer can indeed be used effectively to evaluate the load transfer efficiency of joints and cracks in rigid pavements and that the results from such evaluations can be used to establish maintenance and/or rehabilitation priorities. The procedures developed were also adapted for use in the evaluation of the efficiency of longitudinal joints in rigid pavements, particularly rigid shoulder joints. Again, the results presented show that with the FWD sensor configuration suggested, deflection measurements can be obtained for use in effectively evaluating the joint efficiency of the types of longitudinal joints found in rigid pavements.

KEY WORDS: rigid pavement, structural evaluation, nondestructive testing, Falling Weight Deflectometer, load transfer efficiency, transverse joints and cracks, longitudinal joints, PCC shoulders, temperature differential, voids.

SUMMARY

This report describes the evaluation and verification of a procedure for the evaluation of the load transfer efficiency of transverse joints and cracks and the evaluation of the efficiency of longitudinal joints in rigid pavements using the Falling Weight Deflectometer (FWD), a nondestructive testing device. FWD load and deflection measurements on a controlled jointed reinforced concrete pavement (JRCP) research facility were used in evaluating and verifying a load transfer efficiency determination procedure developed in a previous study. To test the suitability of the procedure for implementation, similar measurements were taken on field sections for the evaluation of load transfer efficiency across transverse joints and cracks in rigid pavements in service. The procedure was used in the evaluation of the efficiency of longitudinal joints, especially rigid pavement shoulder joints. The application of the procedure for use in establishing baseline values for future pavement evaluation, maintenance, or rehabilitation priorities is described in the report. The benefits obtained by attaching PCC shoulders to existing rigid pavements—including the influence of tie bar size, length, and spacing on the effectiveness of tied shoulder joints—were also investigated. Inverted-tee joints were compared with ordinary tied joints to determine the more effective shoulder-joint type. The results of these investigations are presented in this report.

IMPLEMENTATION STATEMENT

A procedure for assessing load transfer efficiency of joint and cracks in PCC pavements using the Falling Weight Deflectometer has been evaluated and verified. Methods for the application of the procedure for assessing the efficiency of longitudinal joints, especially shoulder joints, has been presented. Implementation of the procedure by the Texas State Department of Highways and Public Transportation (SDHPT) for establishing baseline values for future pavement evaluation is recommended. Evidence has been provided as to the beneficial effects of PCC shoulders attached to existing rigid pavements. Guidelines are given concerning tie bar dimensions for use on such ordinary joints. It is recommended that the Texas SDHPT implement these findings to improve the performance of PCC pavements.

TABLE OF CONTENTS

CHAPTER 1. INTRODUCTION

BACKGROUND

The evaluation of pavements is important within an overall pavement management framework and must be taken into consideration by every agency or organization charged with the provision of efficient highway pavements. With much of the nation's highway infrastructure already in place, emphasis in the last decade has shifted to evaluation needed to determine where and how best to invest available resources for the upkeep of highway facilities. In this regard, the structural evaluation of rigid pavements requires careful attention in any comprehensive pavement evaluation process and is essential for determining the maintenance, rehabilitation, and reconstruction measures necessary for extended performance. Accordingly, the Texas State Department of Highways and Public Transportation has, in the last decade, sponsored a number of studies to develop proper structural evaluation techniques for rigid pavements.

Since many miles of pavements must be evaluated under a comprehensive program, the structural evaluation techniques developed require nondestructive testing methods. Nondestructive testing methods are rapid, do not damage pavements, and provide useful data in the three key areas of (1) structural capacity evaluation through in-situ material characteristics estimation; (2) joint load transfer evaluation; and (3) loss of support or void detection. As part of this overall program, a procedure for determining the presence of voids using discriminant analysis based on nondestructive testing dynamic deflections and field observation of pumping was suggested in one study (Ref 1). In another extensive study, a self-iterative procedure based on dynamic deflections was developed to estimate the in-situ moduli of rigid pavements by matching theoretical and measured deflection basins through an inverse application of layered elastic theory (Ref 2). A preliminary investigation of the use of deflections for determining joint efficiency and the detection of voids on field test sections was also conducted in a third study (Ref 3).

The work reported here extends previous studies with further evaluation of the joint systems inherent in rigid pavement structures. The term "joint systems" here refers to the constructed transverse and longitudinal shoulder joints, the transverse cracks which ultimately develop in both jointed and continuously reinforced concrete pavements, and the longitudinal joints between travel lanes. These joint systems or discontinuities often represent the "weakest link" in rigid pavement structures and, consequently, tend to develop extensive distress. The ability of a joint system to transfer load across the discontinuities efficiently, however, greatly influences the occurrence of distress. Such load transfer has a significant influence on the performance and life-cycle costs of rigid pavements and, moreover, has an impact on maintenance and rehabilitation performance. With inadequate load transfer, the application of load on one side of a joint or crack will produce high slab deflections and excessive stresses and strains which can lead to pavement distress such as spalling, pumping, faulting, and corner cracking. Insufficient foundation support at the joint or crack further compounds the problem and contributes to higher stresses, strains, and distress. Consequently, installed load transfer devices, good aggregate interlock, or both should be used at joints and cracks in rigid pavements to ensure good performance.

Since stress and strain are limiting criteria for distress, stress and/or strain provide suitable pavement response parameters for load transfer assessment. In the absence of efficient and economical instruments for directly measuring stress and strain in the field, differential deflection appears to be a parameter suitable for large-scale pavement evaluation. With the steady improvement of equipment that can accurately measure pavement deflection in the field, a comparison of the measured deflections under load of the slabs adjacent to a joint or crack has evolved as a measure of load transfer efficiency across joints and cracks. The current study was initiated to develop field procedures for assessing load transfer across joints and cracks of rigid pavements using the Falling Weight Deflectometer (FWD). This report describes the development and evaluation of procedures for assessing the load transfer efficiency of joints and cracks in Portland Cement Concrete (PCC) pavements. Application of these procedures in pavement structural evaluation can yield a better definition of maintenance, rehabilitation, and reconstruction needs, and can produce efficient and cost-effective strategies for implementation.

OBJECTIVES

The major objective of this study is to develop, evaluate, and implement procedures for assessing the efficiency of load transfer across joints and cracks in rigid pavements using the FWD. Specific objectives of the study are to:

(1) develop procedures for evaluating load transfer at transverse joints in rigid pavements;
(2) develop a method for evaluating load transfer efficiency across cracks; and
(3) develop a method for evaluating longitudinal joints, especially shoulder joints.

To meet these objectives, several specific tasks were carried out to:

(1) evaluate the repeatability of the FWD and determine the FWD load levels most suitable for deflection measurements;
(2) document the effects of daily and seasonal environmental variations on slab curling and warping;
(3) evaluate the effects of daily and seasonal temperature variations on rigid pavement deflection response;
(4) determine the effects of voids under PCC pavements;
(5) evaluate and verify the deflection-based load transfer efficiency procedure developed in Ref 3 for transverse joints and cracks;
(6) evaluate the effects of environmental variations on load transfer efficiency determined with the procedure;
(7) field test the procedure on in-service pavements;
(8) adapt the transverse joint and crack evaluation procedure for evaluating longitudinal shoulder and center line joints;
(9) evaluate the beneficial effects of PCC shoulders;
(10) determine the effectiveness of inverted-tee shoulder joints; and
(11) evaluate the influence of tie bar size, length, and spacing on longitudinal shoulder joint efficiency.

SCOPE

To address these objectives, the work in this report has been divided into nine chapters. Chapter 2 is a review of current literature on nondestructive evaluation of rigid pavements using the FWD. Chapter 3 outlines a laboratory program carried out to collect data for load transfer efficiency under controlled conditions on a research rigid pavement facility. The effects of daily and seasonal temperature and moisture variations, including loss of slab support on the response of the research pavement, are discussed in Chapter 4. Chapter 5 details the verification of procedures for assessing load transfer efficiency across joint systems. Chapter 6 describes a field research program conducted to test the procedures and to evaluate the beneficial effects of PCC shoulders attached to existing rigid pavements. The results of the analysis of the field data are presented in Chapter 7. Chapter 8 is a discussion of the major findings of this study, and the summary, conclusions, and recommendations of the study are presented in Chapter 9.

CHAPTER 2. NONDESTRUCTIVE EVALUATION OF RIGID PAVEMENT JOINT SYSTEMS USING THE FALLING WEIGHT DEFLECTOMETER

BACKGROUND

Rigid pavements are designed to carry traffic loads through bending of the concrete slab. Structural continuity is therefore essential to rigid pavements and needs to be given a high priority during their design and subsequent evaluations. Because of temperature stresses, however, rigid pavements either have discontinuities designed as part of their structure (joints) or ultimately end up with cracks developing in them. The designed joints are built to facilitate construction or to reduce the temperature stresses which cause cracking. Without control, such cracks occur at random locations in different sizes and alignments and reduce the load-carrying capacity of the pavement. Controlled discontinuities, in the form of transverse and longitudinal joints of specific sizes and alignments, are built to curtail such cracking and streamline maintenance requirements. Collectively referred to as joint systems, the joints and cracks can still be a weak link in the concrete structure, and they are often the location of most of the failures encountered in PCC pavements. Consequently, a major aspect of rigid pavement structural evaluation is the evaluation of joint systems. With these joint systems occurring every few feet to a hundred feet, the use of efficient and cost-effective evaluation techniques becomes essential. The application of the proper evaluation techniques may point to a need for nothing more than localized maintenance or repair (in lieu of major pavement rehabilitation) and may result in considerable savings. In this chapter, the types and functions of joint systems common to rigid pavements, including the factors to be considered in their evaluation, are discussed.

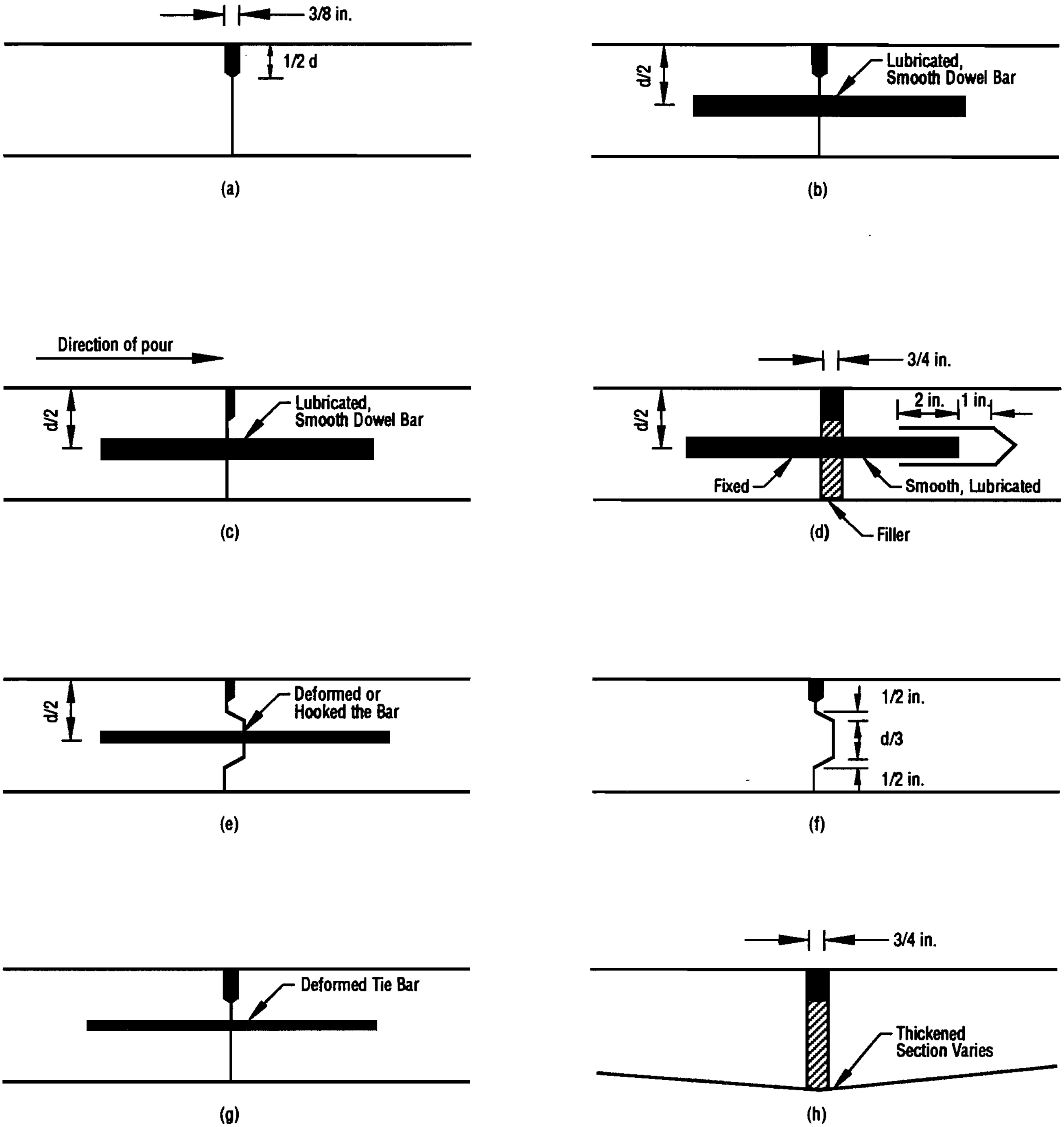
TYPES AND FUNCTIONS OF JOINT SYSTEMS

Most PCC pavements can be classified as jointed plain concrete pavements (JPCP), jointed reinforced concrete pavements (JRCP), or continuously reinforced concrete pavements (CRCP). The major differences between these types are the use of reinforcement and joints to limit temperature stresses within the pavements. Briefly, JPCP are designed as unreinforced concrete slabs of relatively short length to eliminate the likelihood of the formation of transverse cracks within a single slab unit. Typically the slabs are less than 20 feet long. The joints between adjacent slabs may or may not be doweled. JRCP are jointed PCC pavements reinforced in the longitudinal direction to allow the use of longer slabs. Transverse joint spacings generally range from 20 to 100 feet and the joints are usually doweled. CRCP consist of long lengths of PCC pavements with transverse joints provided only at extreme ends, at junctions with other structures, and at work stoppage locations. Adequate longitudinal reinforcement is provided to withstand temperature stresses, although closely-spaced transverse cracks ultimately form. The reinforcement also helps keep crack openings tight. PCP are constructed with slabs mainly post-tensioned with steel. Average slab lengths are in the range of 400 feet, but joint spacings of up to 760 feet in the United States and over 1,000 feet in Europe have been reported (Ref 4). Transverse steel reinforcement may be included in all of the last three reinforced pavement types.

The types and functions of the joints and cracks in these pavements are discussed in the following sections. Although all types of rigid pavements have been used in Texas, JRCP and CRCP are the most common. Consequently, this discussion is limited to the joint systems which occur in these two pavement types. Most of the joints systems are classified under contraction, expansion, construction, warping, or longitudinal joints. Figure 2.1 shows some of the typical joints found in rigid pavements.

TRANSVERSE JOINT SYSTEMS

Transverse joints and cracks fall into three categories: contraction joints and cracks, expansion joints, and construction joints (Refs 4, 5 and 6). Contraction joints are used in JPCP and JRCP to permit slab contractions in the horizontal direction and to relieve tensile stresses caused by frictional forces between the pavement and the underlying layer. The joints also relieve warping and curling stresses. The transverse cracks which form in CRCP are similar in nature. Expansion joints are built into pavements to provide space for expansion because of horizontal movements resulting from temperature and moisture variations. Such joints are usually provided at junctions that have obstructions such as bridges and other intersecting rigid pavements. In some cases, where there is the problem of joints' opening up considerably at certain times of the year, expansion joints may also be provided in rigid pavements at some regular interval in place of contraction joints. The expansion joints limit the high compressive stresses which can occur owing to incompressible material infiltration into the joints leading to the transfer of high compressive forces through adjoining slabs (Refs 5 and 6). Construction joints are provided at locations in rigid pavements where there is a break in construction, such as at the end of a work day. Efforts



(a) dummy-groove contraction (b) dummy groove, doweled, contraction (c) butt construction (d) expansion (e) keyed longitudinal, tied construction (f) keyed hinge or warping construction (g) tied longitudinal warping (h) thickened edge expansion (airfields).

Fig 2.1. Some typical types of rigid pavement joints (Ref 6).

are often made to ensure that construction joints coincide with contraction or expansion joints.

LONGITUDINAL JOINT SYSTEMS

Longitudinal joints are comprised of the construction joints dependent on the width of paving and the warping or hinge joints which separate multiple lanes cast together (Refs 4 and 6). The latter are usually formed by sawing to a fraction of the depth of freshly-placed concrete for pavements at some time between setting and hardening to induce cracking at a specific location. The resulting pavement lane widths, usually between 10 and 13 feet, reduce transverse temperature and moisture stresses to manageable levels. Tie bars are generally used to hold the adjacent lanes together to prevent lateral movement. Construction longitudinal joints are usually butt or keyed joints (Fig 2.1).

PROVIDING LOAD TRANSFER

Each joint or crack in a rigid pavement is a potential location of distress. Typical distresses that occur in these areas include spalling, faulting, pumping, punchouts, blowups, and corner breaks. Figure 2.2 illustrates some of these common distress types. Although these distresses may be due to a host of factors including loss of slab support, poor drainage, excessive loads, poor material quality, bad joint design, and combinations thereof, loss of load transfer at joints and cracks, even if not initially responsible for distress, will result in further deterioration. Consequently, the provision of load transfer at joints and cracks has been an integral part of rigid pavement design, beginning with the construction of the first PCC pavement which utilized load transfer devices at transverse joints in Newport News, Virginia, in 1917 (Ref 7). Since then, various devices have been used to allow load transfer across rigid pavement discontinuities.

LOAD TRANSFER DEVICES AND MECHANISMS

Basically, the load transfer provided in rigid pavements is accomplished in one of three ways (or by some combination of the three): load transfer by aggregate interlock; load transfer by a mechanical device; and load transfer through the support offered by a stabilized base. In all cases load transfer is accomplished mostly by shear transfer. Load transfer solely by aggregate interlock is accomplished where protrusions at a joint or crack face and the interlocking of aggregates resist vertical movement (Ref 8) and load is transferred across the discontinuity by shear. This process is effective only if the width of the crack or joint opening is small enough to allow the adjacent faces to remain in contact.

Engineers have also used mechanical devices of all sizes and shapes, such as steel dowels, tie bars, steel plates, and key joints (Refs 7 and 9), to accomplish load transfer across joints and cracks. In most instances these devices span the discontinuities and transfer load by shear—this is especially true for two-member load transfer devices. In a few instances, some moment transfer, in addition to shear transfer, is possible, as in the case of load transfer across cracks held tight by reinforcement, such as in CRCP and across tied longitudinal joints (Ref 10). At transverse joints, it is important to ensure that at least one end of the load transfer device is unbonded to permit horizontal movement of the pavement slabs at the joint system.

Support by a stabilized base, such as cement-treated or lime-treated base, also contributes to load transfer (Refs 8 and 11). By adding to the support subgrades give to pavements, stabilized bases supply some extra rigidity to pavements and improve load transfer at joints and cracks. In most instances, a combination of all these mechanisms of load transfer comes into play at joints and cracks. Aggregate interlock and dowel shear transfer can provide load transfer in a plain jointed concrete pavement with doweled transverse joints, while tied shoulder joints in CRCP with a stabilized base, in effect, involve load transfer by shear and moment transfer.

FACTORS WITH DIRECT EFFECTS ON PERFORMANCE OF LOAD TRANSFER DEVICES

Many studies have been conducted regarding the factors which directly affect the performance of load transfer devices. Findings of these studies are discussed in this section. A distinction is made at this point between direct factors and factors which *in*directly influence the performance of load transfer devices. For example, temperature and moisture variations affect some characteristics of pavement slabs which in turn may influence the performance of load transfer devices. Those effects are classified as indirect effects and are discussed in a later section.

In a comprehensive study on load transfer by aggregate interlock, Colley and Humphrey (Ref 8), using repetitive loading of pavement slabs, found that effective load transfer by aggregate interlock depends on joint opening, magnitude of the applied load, number of load repetitions, aggregate angularity, slab thickness, and subgrade support. Since load transfer is accomplished through shear, an increased joint opening means an increasing loss of contact between joint faces which decreases joint effectiveness. More particle angularity increases the joint effectiveness by enhancing contact between the joint faces. The effects of the other factors are in keeping with good pavement design practices. Thicker slabs and good subgrade support enhance joint effectiveness, while higher load magnitudes and increasing load repetitions decrease joint effectiveness.

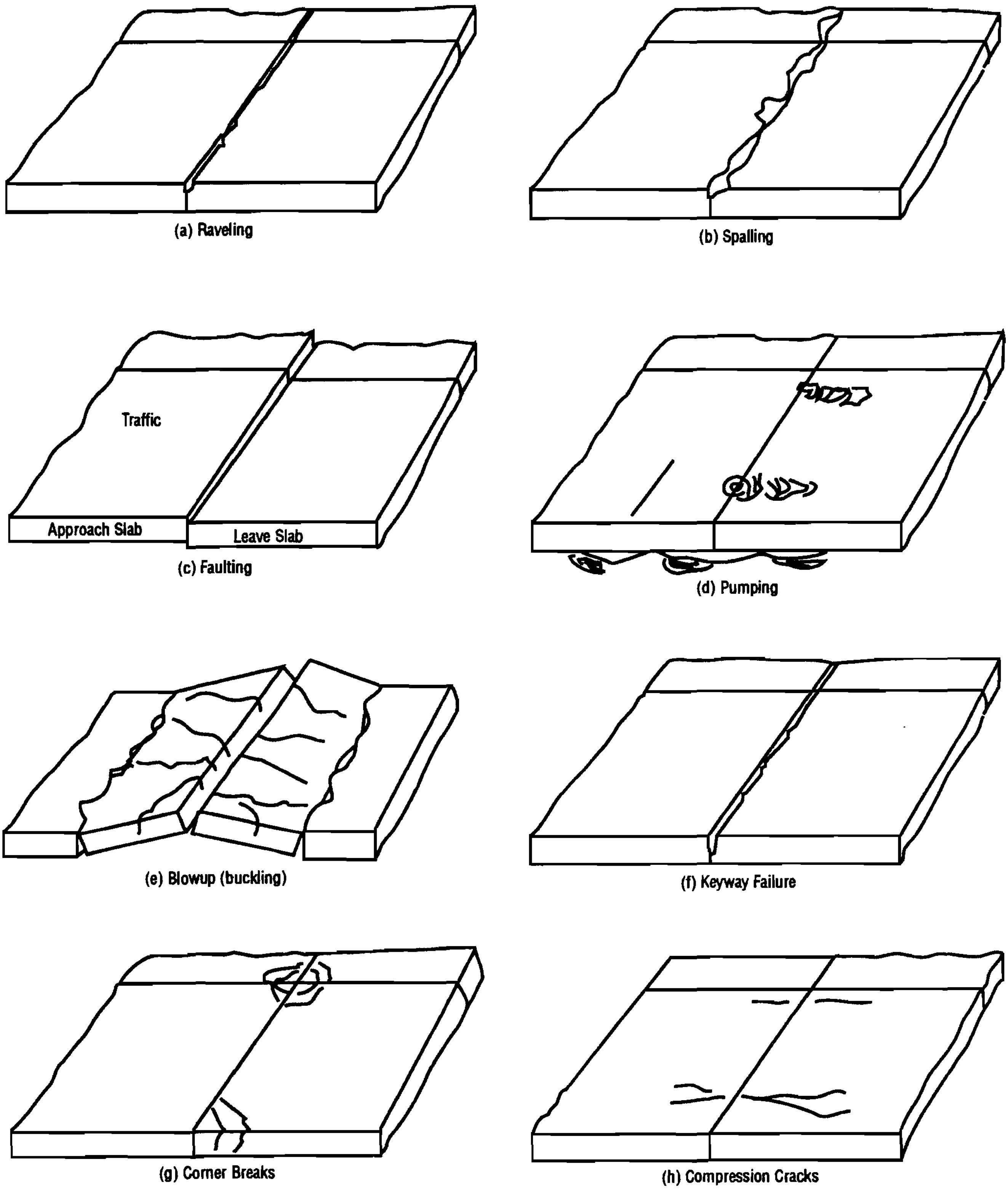


Fig 2.2. Distress at joints (Ref 5).

Extensive theoretical and laboratory studies have been conducted on the effectiveness of mechanical load transfer devices. Westergaard, in a theoretical analysis (Ref 12), presented results which show that, for a given slab thickness and subgrade support, dowel spacing significantly affects stress reduction at a joint, while increasing dowel spacing reduces the effective load transfer at a joint. In a theoretical analysis of doweled joints, Bradbury (Ref 13) also obtained results which indicate that the size, length, and spacing of dowels significantly affect load transfer at a joint, with shorter length, large diameter, and closely-spaced dowels improving load transfer. In extensive tests conducted under the auspices of the Bureau of Public Roads on the structural design of rigid pavements, Teller and Sutherland (Ref 7) obtained results in agreement with the findings of these theoretical analyses with respect to dowel size, spacing, and length.

From the results of Westergaard's theoretical analysis of stresses in rigid pavements, Friberg (Ref 14) observed that, for a load at an edge and over a dowel and at a considerable distance from a corner, the effective dowel shear decreases from a maximum at the point of loading to zero at a distance of about 1.8 times the radius of relative stiffness. (Recent research documents that the constant in Friberg's equation should be modified from 1.8 to 1.0; see Ref 36.) Consequently, dowels spaced more than this distance from a point of loading do not contribute to stress reduction, and this distance constitutes the maximum spacing for dowels at joints. Experimental data from laboratory tests by Friberg (Ref 15) on single dowels embedded in concrete also indicate that dowels do not gain any additional effectiveness with an increase in embedment length over a maximum limit. Friberg also found that the modulus of dowel reaction, the rate at which concrete reacts under a deflecting dowel, increases with concrete strength and decreases with decreasing concrete cover over the dowel.

In extensive repetitive loading laboratory tests of transverse joints, Teller and Cashell (Ref 9) also obtained important results concerning the effect of dowel diameter, length of dowel embedment, and joint opening on load transfer, confirming the findings of the previous studies. Significant results, still a mainstay of present practice, indicate that a minimum dowel diameter of one-eighth the slab thickness and an effective embedded dowel length of eight diameters for 3/4-inch dowels and six diameters for 1-inch and larger dowels are required for adequate load transfer across the joints of rigid pavements. Teller and Cashell also determined that under constant conditions load transfer exponentially increased with dowel diameter and that decreasing joint width enhanced load transfer. Repetitive loading decreased load transfer, as did dowel looseness, which had to be taken up by load before effective load transfer was attained.

EVALUATING JOINT SYSTEMS FOR LOAD TRANSFER EFFICIENCY

A number of analysis methods have evolved over the years from attempts to evaluate load transfer efficiency. In the studies cited above, a number of load transfer efficiency analysis methods were developed to enable comparisons of effectiveness. Bradbury (Ref 9), in an early theoretical analysis of dowel shear and moment transfer, developed expressions for the load transfer capacity of an infinitely long dowel in terms of shear resistance and moment resistance as limited by concrete bearing stress. Teller and Sutherland (Ref 7), in tests on the structural action of several types of transverse and longitudinal joints, used a number of formulas as criteria for comparing the effectiveness of joints. A stress-based formula for determining the ability of a joint to reduce critical load stresses compares the joint stresses in a pavement to the minimum and maximum stresses obtained at an interior location and a free edge. The joint efficiency, JE_s, expressing the ability of a joint to reduce stresses from the worst condition of a free edge to the best condition of an interior location is given by:

$$JE_s = f(e_f - e_i, e_f - e_j) \tag{2.1}$$

where

e_f = the critical stress for load applied at the free edge,

e_i = the critical stress for load applied at an interior position, and

e_j = the critical stress for the load applied at the joint.

Other formulas were suggested for use in a comparison of joints on the basis of their ability to reduce deflection to an acceptable level. In a formula used by Colley and Humphrey (Ref 8) in the investigation of load transfer by aggregate interlock, joint efficiency based on deflection, JE_d, is defined as:

$$JE_d = f(2d_u, d_i + d_u) \tag{2.2}$$

where

d_u = the unloaded slab deflection at the joint, and

d_i = the loaded slab deflection at the joint.

Friberg (Ref 14) suggested a formula for calculating load transfer which was used by Teller and Cashell (Ref 9) in laboratory tests to determine the performance of doweled joints under repetitive loading. A deflection based joint efficiency, JE_d, signifying the ratio of dowel shear to the applied load, or proportion of load transferred, is given by:

$$JE_d = f(y_p, 2y_p + y_d) \tag{2.3}$$

where y_d and y_p are the dowel and pavement deflections, respectively.

Westergaard (Ref 16), in a theoretical analysis of stresses in an airfield pavement, suggested, for a load at a joint with load transfer devices which are either continuous or closely spaced along the joint, an approximate formula for determining load transfer efficiency, JE_d, from deflection data given by:

$$JE = 1 - f(z_i - z'_i, z_e \; z'_e) \quad (2.4)$$

where z_j and z'_j are the deflections of the unloaded slab at any point along the joint, and z_e and z'_e are the corresponding deflections that would occur at the same place if the joint had no capacity to transfer load. In recent practice, a method for calculating load transfer that has been commonly used is the one recommended in the AASHTO design manual (Ref 4), in which load transfer efficiency, JE_d, based on deflection reduction at a loaded joint, is given as:

$$JE_d = d_u/d_i \times 100 \quad (2.5)$$

where d_u and d_i are the deflections of the unloaded and loaded slabs at the joint, respectively. The corresponding load transfer efficiency , JE_s, based on stress reduction is calculated as:

$$JE_s = s_u/s_i \times 100 \quad (2.6)$$

where s_u and s_i are, respectively, the stresses in the unloaded and loaded slabs of the pavement at the joint.

In previous research, Ricci (Ref 3) recommended a variant of the deflection-based load transfer formula (Eq 2.5) as a result of experience with field data, where d_u has sometimes been found to be greater than d_l. JE_d is still calculated as the ratio of the deflections of the pavement slabs at a joint or crack, with the condition that the final value is less than 100 percent. JE_d is calculated for both upstream and downstream load positions, and the average, called the joint deflection ratio, is used to characterize load transfer efficiency at the joint.

SELECTING THE RESPONSE PARAMETER AND DEVICE FOR LOAD TRANSFER EVALUATION

For load transfer analysis, deflection and stress, as indicated, are commonly used. In selecting the parameter to use for evaluation, however, a number of factors have to be taken into account. Pavement deflection as an overall measure of pavement response to loading is arguably not as critical a distress parameter as stress or strain. Clearly, two pavements with the same deflection response but different thicknesses and, therefore, radii of curvature will experience different stresses and strains. Stress is therefore a better pavement response parameter to use for load transfer analysis. With no commercially available devices for the measurement of stress or strain in the field, with the large number of joints and cracks to evaluate in a typical pavement, and with the general inappropriateness of destructive testing, deflection measurements from nondestructive testing have become the most efficient and effective pavement response parameter used.

The nondestructive testing (NDT) device used must be able to simulate pavement responses for loads in the range of the impact forces expected on highways. An investigation of the dynamic aspects of pavement loads and ways in which vehicle and road characteristics influence them indicates that, even at low speeds (20 and 40 mph), the maximum impact forces applied can be as high as 47 and 64 percent above the applied static weight, respectively, with road and other vehicle characteristics kept constant (Ref 17). With speed and tire pressure kept constant, impact forces above the applied static load varied by as much 22 to 65 percent because of increased pavement roughness. In similar tests by Al-Rashid (Ref 18) in Texas, average dynamic load increases over static load of 6, 15, and 19 percent for vehicles operating at speeds of 10, 30, and 60 mph, respectively, were recorded. Increases of over 100 percent were also recorded for vehicle tires going over obstructions on the order of 3/8-inch high on pavements. (Dynamic load increases of as much as 250 percent over static load were observed on a bridge deck with similar obstructions.) Although results to the contrary from the AASHO Road Test (Ref 19) and reported by Bohn (Ref 20) indicate a slight decrease in deflections on some pavements with increase in vehicular speed, the findings from the earlier-mentioned studies are widely accepted. Consequently, for pavement evaluation, the NDT device used must be capable of generating impact forces over 100 percent above expected static wheel loads and must have a load duration and magnitudes of deflection, stress, and strain closely resembling those of a moving wheel load. The Falling Weight Deflectometer (FWD) was selected for use in this study because it meets such criteria, in addition to its being easy to use and generating easily-interpreted data.

DESCRIPTION AND OPERATING CHARACTERISTICS OF THE FWD

The FWD was developed from research performed in France by S. Bretonniere in the early 1960's. The FWD is intended to generate responses in pavements similar to the responses generated by a wheel load of a moving truck. The following is a brief description of the Dynatest 8000 Falling Weight Deflectometer used for testing in this study. Further details are available in Refs 2 and 21.

GENERAL DESCRIPTION

The FWD is basically a dynamic plate loading test device that generates a transient force impulse on pavements. Its three main components are (1) a trailer consisting of a force-generating apparatus and load and deflection measuring sensors, (2) a microprocessing system, and (3) a desktop personal computer. The last two are housed in a vehicle which tows the trailer. In operation, an impact force is generated by dropping a weight setup down a guide shaft onto a specially-designed rubber spring system resting on a circular loading plate. This circular plate, 11.81 inches (300 mm) in diameter, with a 0.22-inch-thick (5.5-mm) ribbed rubber sheet under-padding to ensure even distribution of the load impulse, rests on the surface to be tested. The force pulse generated is approximately a half-sine shape and about 25 to 30 msec in duration. In tests conducted on a Danish test road, Bohn (Ref 20) found that this corresponded closely to the duration of load effects in the upper layers of pavements owing to the wheel loads of heavy trucks travelling at speeds between 25 and 40 mph. (In lower layers, however, Bohn found that while the FWD load-effect duration was practically the same, the effect of moving trucks was of greater duration on the order of 200 msec). The magnitude of the FWD weight assembly dropped on the spring system, and the drop height, can be varied to give reproducible peak forces ranging from 1,500 to 24,000 pounds force (lbf). Again, Bohn (Ref 20) found in tests in Denmark and Holland that the forces generated with the FWD gave surface deflections with a high correlation to moving wheel surface deflections, and he obtained good agreement between FWD and moving wheel stress and strain magnitudes.

A load transducer and seven velocity transducers (geophones) are used to measure the peak force and deflections, respectively. The load transducer is a strain-gage type load cell specially built as an integral part of the loading plate. To accommodate measurement of the deflection at the center of the loading plate, this load cell has a 1-3/8-inch center bore through which a deflection measurement rod with a transducer attached at its end can reach the test surface. The six other transducers in movable holders can be arranged along an 8-foot raise/lower bar. Since the velocity transducers can measure vertical movements accurately only when mounted vertically on a horizontal surface, they have a magnetized base and are held in spring-loaded holders to ensure good contact between the sensors and the test surface. An output voltage from each velocity transducer is generated by a coil moving through a magnetic field which is directly proportional to the velocity of motion of the test surface. The impact loading of the test surface generates signals which are sent through 16-1/2-foot (5-meter) cables to the system processor.

The modular microprocessor-based system processor, interfaced with both the FWD trailer and the desktop computer, is used to scan, condition, digitize, and transmit the signals from the eight transducers to the computer. The processor also scans and stores the time histories of the signals from the transducers and calculates peak values. In addition, it controls the FWD trailer hydraulics and continuously tests the system's performance to reveal any functional or operational errors that might occur. A single "multi-signal cable" connects the microprocessor unit to the FWD trailer. The desktop computer is used for input of control and test identification data, as well as for displaying, printing, storing, editing, sorting, and even, where necessary, for further processing of the FWD data.

OPERATING CHARACTERISTICS

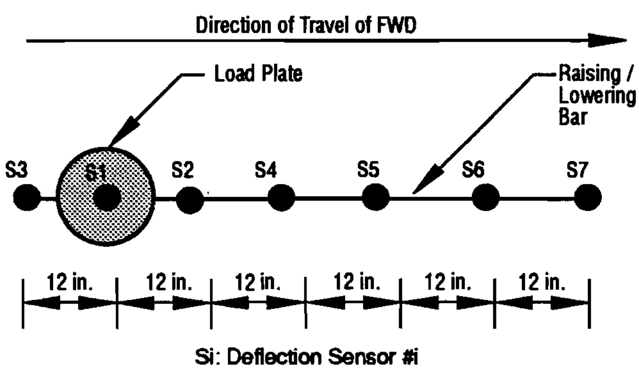
The entire operation of the FWD can be remotely controlled by a single operator using the desktop computer in the FWD towing vehicle. (Manual control of the FWD operation from a trailer connection box which prevents remote control from the microprocessing system in the towing vehicle can be used where necessary). By adding a set of two, six, or ten detachable 55-pound weights, and two, four, or six detachable rubber buffers, respectively, to the basic FWD weight of 110 pounds, weight assemblies of 110, 220, 440, or 660 pounds can be set up for a test. The force pulse generated by each of these setups will still have a duration of approximately 25 to 30 msec, and the setup selected depends on the peak load force range required. The peak load force ranges obtained by dropping each weight setup from four preset falling heights between 1-1/2 and 15-1/4 inches are as follows:

Falling Weight (lb)	Peak Loading Force (lbf)
110	1,600 - 3,800
220	3,400 - 7,700
440	5,000 - 20,000
660	11,600 - 23,500

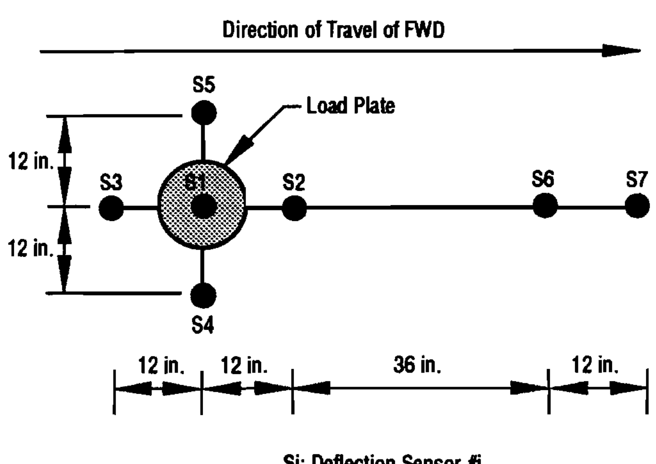
A newer version of the FWD, the Dynatest 8002 FWD, was also used in this study. With a weight setup of 770 pounds in place of the 660 pounds, this version could deliver a force in the 10,000 to 27,100 lbf range.

In routine testing, the FWD with the required weight setup is positioned on the test pavement such that the loading plate is located directly above the marked test location and the deflection transducers are in the positions where deflection measurements are required. Where necessary, extension bars can be attached to the FWD to permit deflection measurements along the 90 and 270-degree axes to the raise/lower bar. The operator seated at the keyboard of the desktop computer in the towing vehicle can select from a number of set test sequences that specify the drop heights and the number of drops from each height per test location and allow input of the test site identification information. These test sequences can also be modified to obtain custom-designed test sequences. By keying in the necessary commands, the FWD loading plate and raise/lower bar assembly are lowered to the pavement surface, and the weight setup is dropped from each of the preset heights. The whole assembly is raised automatically after the last drop, and the FWD can be transported to the next test location. With the necessary site preparation, a test of single drops from four heights at a particular location takes about a minute to complete.

The data (stored on floppy disks) can be retrieved for sorting and analysis. A hard copy of the data is also produced during testing via a printer interfaced with the desktop computer. Since the FWD geophones for measuring the deflections operate on the basis of a single-degree-of-freedom system, no reference point is needed, and very accurate deflection measurements can be obtained. Typically, the deflections are measured with an absolute accuracy of better than 2 percent ± 0.08 mils. This enables the successful use of the data obtained for the evaluation of pavements.



(a) **Transverse joint and crack test arrangement.**



(b) **Longitudinal joint test arrangement.**

Fig 2.3. FWD load and sensor arrangement.

USING THE FWD FOR JOINT SYSTEM EVALUATION

With the attributes described above, the FWD is particularly suitable for joint evaluation. The 8-foot raise/lower bar allows the arrangement of the deflection sensors at varying distances. An added feature of later versions of the FWD, illustrated in Fig 2.3(a), is a raise/lower bar split 1 foot before the load center and 7 feet after, in the direction of travel. Figure 2.3(a) also illustrates the FWD load and sensor arrangement used for measurements at transverse joints and cracks. Figure 2.3(b) shows the extension bars which can be attached perpendicular to the raise/lower bar, and the load and sensor arrangement used for measurements at longitudinal joints. In some instances sensors six (S6) and seven (S7) were moved closer to the load plate, to distances of 12 and 24 inches, respectively, from sensor two (S2). With these two arrangements, deflection data can be collected at joints and cracks for use in the load transfer analysis methods described previously.

CONSIDERATION OF ENVIRONMENTAL FACTORS

As indicated previously, temperature changes during the day result in temperature gradients in PCC pavements which cause slabs to curl, most noticeably at the edges and corners of pavements. Temperature expansion and contraction of slabs in the horizontal direction also result in joint width fluctuations. Similarly, environmental moisture variations, which result in moisture gradients in pavements, cause warping of PCC pavement slabs. The shrinkage and expansion of the pavement slabs also affect joint widths. Consequently, such environmental factors, which may indirectly affect the load transfer performance at joint systems, have to be taken into account when FWD deflection data are used for load transfer evaluation. Morales-Valentin (Ref 21) suggested testing procedures to correct such effects of temperature variations. Shahin (Ref 22) and Foxworthy and Darter (Ref 23) also suggest methods which involve the development of load transfer correction charts, from which load transfer factors can be used to correct calculated load transfer efficiencies. A full treatment of this subject as it pertains to this study is dealt with in a subsequent chapter.

SUMMARY

The majority of the distresses associated with PCC pavements occur at joints and cracks as a result of the reduction in pavement rigidity that accompanies such discontinuities. Load transfer devices are provided at such cracks and joints as a way of providing the pavement some rigidity. Ostensibly, load transfer reduces stresses from applied loads and thereby increases the load-carrying capacity of the pavement as a whole. This chapter presents a review of load transfer at pavement discontinuities. The factors which affect the performance of such devices—and, consequently, have to be taken into account during the evaluation of the load transfer devices—are described. A brief description of the Falling Weight Deflectometer is given. Some of the factors that have to be taken into account in the interpretation of FWD deflection data for load transfer assessment at PCC discontinuities are discussed.

CHAPTER 3. LABORATORY TEST PROGRAM AND DATA COLLECTION

BACKGROUND

To investigate load transfer efficiency evaluation across joints and cracks using deflections obtained with the FWD, a two-part research study was instituted. The first part involved controlled laboratory work on an instrumented research pavement facility constructed at the Balcones Research Center (BRC) of The University of Texas at Austin. This research facility allowed the various factors which affect joints and cracks to be closely monitored and was used to acquire laboratory data for the development of procedures for the evaluation of load transfer efficiency of joint systems. Data were collected for a period of about four years and included FWD deflection measurements on the slab, slab temperature data, environmental data, and measurements of slab movements due to the effect of thermal gradients in a number of experiments. This chapter details the work done at the laboratory research facility and describes the data collected.

DESCRIPTION OF LABORATORY RESEARCH FACILITY

The instrumented rigid pavement research facility used in the laboratory study is a multi-purpose pavement facility constructed at BRC. This rigid pavement research facility was designed and constructed for testing various aspects of rigid pavements under controlled conditions. Its design and construction followed laboratory tests on a pilot slab to test some of the planned concepts of investigation and incorporated the provisions necessary to allow the monitoring of variables that have a significant effect on pavement deflections. These variables selected for monitoring included load transfer across a joint, slab temperature gradient, void or loss of support effect, and sub-surface moisture content. Details of the design and construction of this test facility are available in Refs 24 and 25. A brief description of this instrumented rigid pavement research facility is given here with emphasis on the factors which pertain to this study.

PAVEMENT DESIGN

A plan view of the multipurpose instrumented JRCP facility for controlled laboratory testing is shown in Fig 3.1. It is comprised of a 10-inch-thick jointed reinforced concrete pavement placed on a 3-inch asphalt cement concrete base course and a 6-inch-thick compacted crushed stone subbase, over an embankment of clayey gravely sand with limestone cobbles constructed on an underlying natural rock layer approximately 10 feet below the resultant subgrade. The PCC pavement was constructed with Class A concrete and reinforced with No. 4 bar-size steel spaced 36 inches center-to-center in both the transverse and the longitudinal directions, in accordance with Texas State Department of Highways and Public Transportation (SDHPT) 1982 Standard Specifications (Ref 26). Crushed limestone coarse aggregate was used for the concrete to limit stresses due to thermal expansion and contraction (Ref 4). The pavement consists of a 39-foot-10-inch by 12-foot-6-inch large slab anchored in place at the end across from the joint, and an 18-foot-10-inch by 12-foot-6-inch movable small slab equipped with a loading frame and hydraulic ram system to facilitate mechanical longitudinal movement of the slab. Three sheets of 4-mil polyethylene were placed under the small slab to reduce friction between the asphaltic concrete base course and the concrete slab and permit its movement with the hydraulic ram system.

JOINT DESIGN

The transverse joint between the two slabs is a specially-constructed dowelled joint with two-member dowel units designed to provide variable load transfer at the joint. In all there are 13 of the two-member stainless steel bar dowel units installed at the transverse joint, spaced at 1-foot intervals. Each unit consists of a 13-1/2-inch male bar one inch in diameter and tapered at one end to a slight conical shape, to allow it to fit snugly into an end of a 1-1/4-inch diameter female bar 12 inches long reamed out to a similar taper. These two bars fit together to give a dowel unit 2 feet long with an overlap of 1-1/2 inches. This dowel unit design was selected after extensive analysis detailed in Ref 24. The hydraulic ram system attached to the small slab is used to adjust the horizontal gap at the transverse joint, which in turn varies the vertical distance in between the tapers of the male and female dowel units. This arrangement allows the dowels to provide (1) zero load transfer when the joint is fully opened and there is no contact between the members of each dowel unit; (2) full load transfer when the joint is fully closed and there is full contact between the tapers; and (3) partial load transfer in the intermediary stage between these two extremes.

OTHER FEATURES OF RESEARCH FACILITY

To study the effect of a lack of support beneath pavement slabs on load transfer and to develop void detection techniques using the FWD, two 3-foot-by-3-foot voids were created at the locations shown in Fig 3.1 at the joint and edge of the larger slab. The voids were formed by placing 1-inch-thick foamed styrene layers into depressions formed in the asphalt concrete base course prior to the placement of the portland concrete cement pavement.

The foamed styrene layers were then subsequently dissolved with a solvent to obtain the required voids which are approximately 1 inch deep.

For slab temperature measurements, thermocouples installed at depths of 1 inch from the top, 1 inch from the bottom, and at mid-depth at each of the locations T_1, T_2, and T_3 in Fig 3.1, were used to monitor the temperature within the larger slab. These thermocouples were selected after a thorough search and were fabricated and calibrated in-house. To minimize the possibility of corrosion copper, constantan thermocouple wire (Type T) with a polyvinyl insulating cover was used. These thermocouple wires are connected to an instrumentation panel and a Hewlett-Packard (HP) data acquisition system for automatic data collection and recording. This portable acquisition system is made up of an HP Model 3497A data acquisition unit and an HP Model 150 personal computer, which records the data on 3.5-inch microfloppy disks and can also give a hard copy of the data via a printer interfaced with the computer. The system used had 39 channels for reading and processing instrument signals.

Equipment for collecting data on environmental conditions, such as humidity, ambient temperature, solar radiation, and wind speed at the time of testing, was also provided. The equipment included a meteorological station (Fig 3.1) housing a Belfort hygrothermograph for continuous recording of the ambient temperature and relative humidity. A Dwyer wind speed indicator installed outside the instrumentation building was available for measuring the prevailing wind speed, and a microprocessor-controlled Licor Model LI-1776 solar monitor was used to record solar radiation during testing.

LABORATORY TEST PROGRAM

Three major groups of tests were conducted on the BRC test pavement. These were (1) deflection measurements for load transfer efficiency studies; (2) measurements of slab movements in a study to determine the effects of variations in environmental factors on pavement curling and warping; and (3) measurements for an investigation into the direct effect of environmental and other factors on deflection. The first and last groups of tests were essentially combined into tests for load transfer efficiency, and the tests to determine the effect of environmental factors on pavement curling and warping were conducted separately.

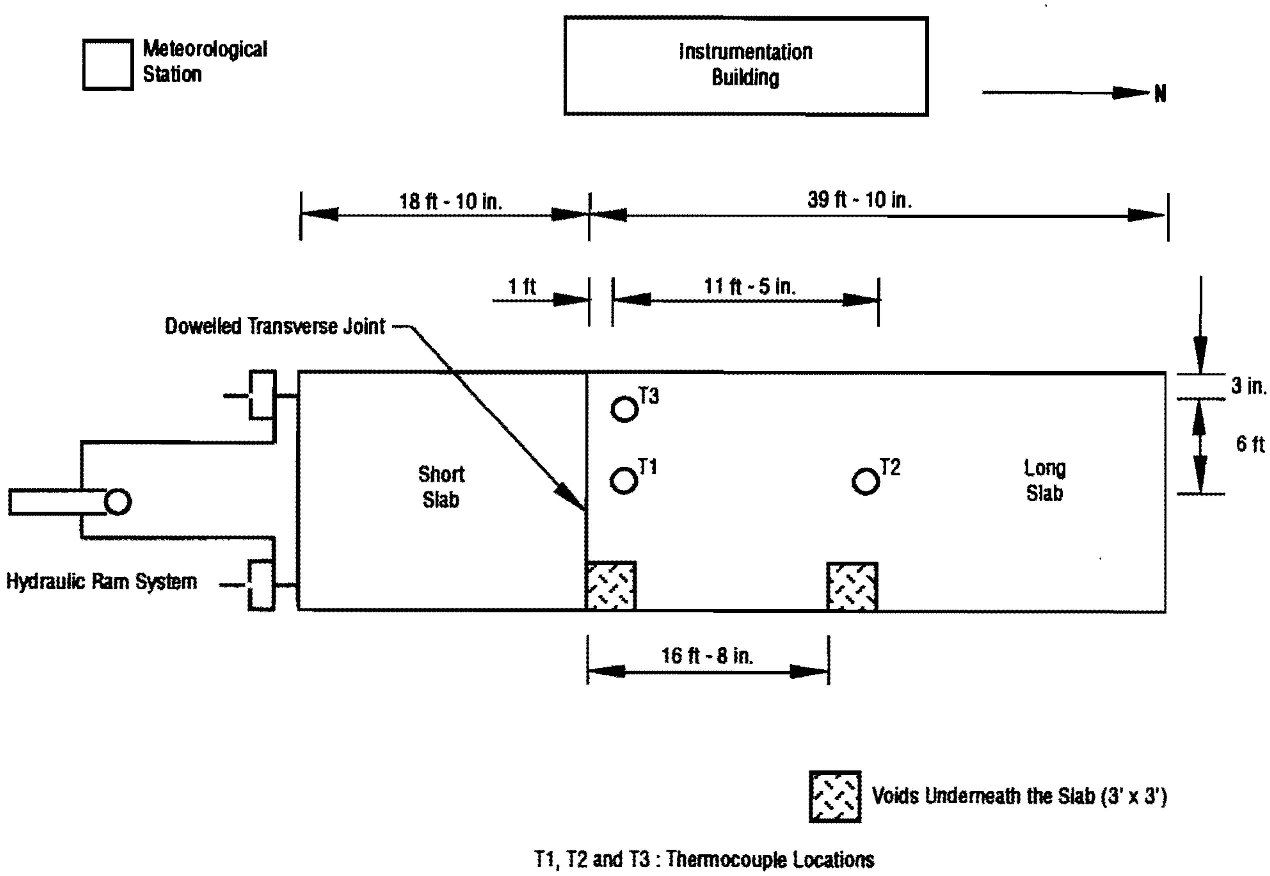


Fig 3.1. Layout of test pavement research facility.

LOAD TRANSFER EFFICIENCY STUDIES

The most important of the tests conducted on the research pavement facility were extensive tests for load transfer efficiency studies. Fifteen such tests were conducted in the period between December 1985 and May 1989. Tests were conducted in each of the four main seasons in Texas—fall, winter, spring, and summer. Although there were slight variations between some of the tests conducted, a majority were of the same kind and the data collected were similar. The Dynatest Model 8000 FWD described in Chapter 2 was used for measuring the deflections required for these tests. Figure 3.2 shows the test paths and selected locations on the slab facility where these tests were conducted. In general, the tests consisted of measurements at these locations over extended time periods, during which supplementary data in the form of environmental data were also collected. Consequently, the deflection measurements are also useful for investigations into the direct effects of environmental and other factors on rigid pavement deflection, and no distinction is made between the tests in this regard. The tests conducted can be classified into two groups. One group consists of tests required for what are called closed-joint studies, and the other group consists of tests for open-joint or variable load transfer studies.

Closed-Joint Studies

The closed-joint tests were conducted to collect data for load transfer studies at the transverse joint of the research slab facility simulating conditions at an ordinary transverse joint. In these tests, FWD deflection measurements at the test locations shown in Fig 3.2 were taken with a 440-pound-weight setup dropped from four preset heights to give four peak loads ranging from about 4,500 to 20,000 pounds. The FWD sensor arrangement shown in Fig 2.3(a) was used for all of these tests on the controlled research pavement. As indicated, with an FWD load plate diameter of 11.81 inches (30 cm), the joint was approximately midway between sensors S1 and S2, and sensors S3 and S1, respectively, for tests with the load upstream and downstream of the joint. Most of the FWD testing of this kind on the controlled slab facility consisted of tests along the three test paths shown in Fig 3.2 in three-day periods during which no major change in the weather condition was expected. Tests were conducted on one test path the first day, on another on the second day, and on the last test path on the third day. As much as possible, tests on the test paths along the edges, that is, on the no-void edge test path and on the void edge test path of the pavement (Fig 3.2), were conducted on consecutive days.

The tests on a path included FWD tests at the upstream, downstream, and midspan locations shown in Fig 3.2 for the joint in an open condition and followed by similar tests at the same locations with a 45-psi pressure applied across the joint (closed condition). The whole sequence was then repeated at hourly intervals. The seven deflections for each of the four peak loads were obtained at each test location for these two joint conditions in a typical test.

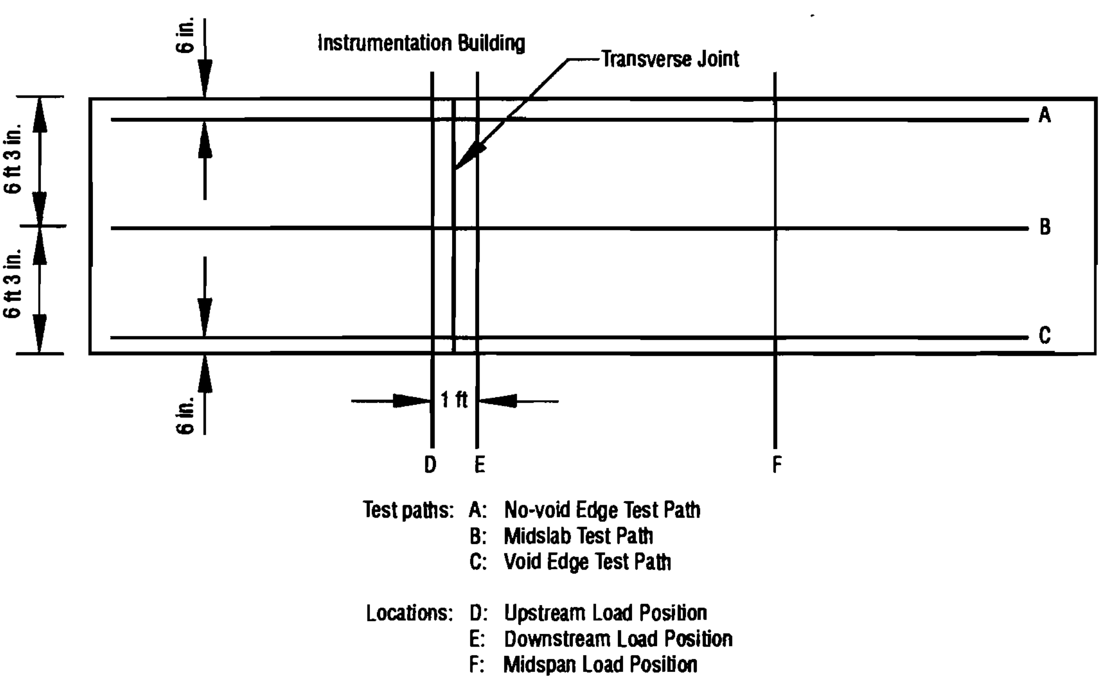


Fig 3.2. Test paths and test locations on test pavement.

During the time of testing, pavement temperature data were also obtained using the thermocouples embedded in the long slab. With the data acquisition system the pavement temperature was automatically collected and recorded at 30-minute intervals for the duration of testing. Supplementary data in the form of wind speed, solar radiation, and pavement surface temperature, measured with both a portable surface thermometer and a hand-held Omegascope Model OS-2000A infrared pyrometer, were also collected during testing. In addition, the Belfort hygrothermograph in the meteorological station was set up to continuously record the ambient temperature and relative humidity during testing.

Thus, for a typical test, the data collected include FWD deflection measurements for two joint conditions at hourly intervals; temperature data from the thermocouples within the pavement at 30-minute intervals; a continuous record of the ambient temperature and relative humidity; and wind speed, solar radiation, and pavement surface temperatures at 30-minute intervals.

Variable Load Transfer Efficiency Studies

As indicated, the doweled joint of the research pavement was built with features that allow the load transfer at the joint to be varied. The tests for which the width of the joint was varied to obtain variable load transfer are described in this section. The tests were conducted at the test locations along the three test paths shown in Fig 3.2, for joint gaps varying in 1/8-inch increments between 0 and 3/4 inch. The procedures used in these tests were similar to those used in the tests for the closed-joint studies. However, the FWD measurements could not be taken for all the joint gaps tested at fixed time intervals as before. For any particular joint gap, tests were conducted at all the required locations on the pavement before the next decrement, and then the tests were repeated. Supplementary environmental data comprised of ambient temperature, wind speed, solar radiation, and pavement surface temperature were also collected at hourly intervals at the time of each test; the temperatures within the slabs were obtained at 30-minute intervals via the embedded thermocouples.

The data collected for the joint gap studies, therefore, include FWD deflection data for different joint gaps at the locations tested on the pavement; temperature data from the thermocouples embedded in the pavement at 30-minute intervals; and ambient temperature, wind speed, solar radiation, and pavement surface temperature at hourly intervals at the time of the tests.

PAVEMENT CURLING TESTS

The other group of important tests conducted on the test pavement were curling and warping tests to measure slab movements resulting from temperature and moisture variations, respectively. The tests were conducted between March 1986 and January 1987 and covered the spring, summer, fall, and winter seasons. To measure the vertical movement of the pavement slab, a number of linear variable displacement transducers (LVDTs) mounted on a wooden beam suspended across the slab parallel to the transverse joint were used. The LVDTs used were Trans-Tek, Series 240, transducers which can automatically monitor displacements of up to 1 inch. The wooden beam was specially treated and painted to avoid excessive moisture variation and, therefore, any appreciable warping. Since temperature effects on wood are negligible, no serious curling of the wooden beam was expected.

Two types of curling tests were conducted. The first were to measure curling along the transverse joint of the large slab of the research pavement. In the second group of tests, in addition to the transverse curling measurements, curling movements along the edges of the large slab of the pavement up to a distance of 9 feet from the transverse joint were also measured. Figure 3.3 shows the LVDT arrangement for the two groups of tests. In the second group of tests, the LVDTs placed along the pavement edges to measure longitudinal curling at the pavement corner, were each held in place by a reinforcement bar with one end driven into the ground and a clamp at the other end holding the LVDT.

In a test, the LVDTs and the thermocouples for measuring the pavement temperature were connected to the instrumentation panel interfaced with the HP 3497A data acquisition system. Using calibration blocks, each LVDT was calibrated separately. The data generated from both the LVDTs and the thermocouples were then automatically collected and recorded at 30-minute intervals for the duration of the test, which typically lasted a week. The data collected from these tests for the transverse joint curling measurements, consequently, consist of five LVDT readings and nine temperature readings from the three thermocouples for the duration of the tests; and the data for the tests for measuring both transverse and longitudinal curling comprise eleven LVDT readings and nine thermocouple temperature readings. All together, seven curling tests were conducted, two of which had the LVDT arrangement for measuring longitudinal curling.

LABORATORY TEST PROGRAM DATA BASE

In summary, the data base available from the laboratory test program is comprised of four main groups of data. The first group of data consists of FWD deflection measurements for a load transfer efficiency study of the transverse joint of the pavement research facility along three test paths (Fig 3.2) for what is referred to as the closed-joint condition. These deflection data are supplemented by data on the pavement slab temperature; pavement surface temperature; and the environmental variables—ambient temperature, relative humidity, solar radiation, and wind speed, in order of their perceived importance.

The second group of data, similar to the first, is also comprised of FWD deflection data for load transfer study of the transverse joint of the pavement research facility along the three test paths shown in Fig 3.2. The deflection data are, however, for varying joint gaps obtained by using the hydraulic ram system to move the small slab of the pavement. These data are also supplemented by pavement slab temperature; pavement surface temperature; and environmental data consisting of ambient temperature, solar radiation, and wind speed.

The third group of data is inherent in the first two groups and is comprised of FWD deflection data on the pavement research facility at specific locations with certain peculiar characteristics, e.g., deflection measurement at the downstream location of a transverse joint directly over a void, supplemented by data on the environmental variables mentioned above.

The last group is comprised of curling movement and temperature data measured for two cases. The primary data consist of measurements of the transverse curling movement of the large slab of the pavement research facility with accompanying slab temperature data. Some of these data contain in addition data on longitudinal curling movements along the edges of the large slab of the research pavement at the corners with the transverse joint.

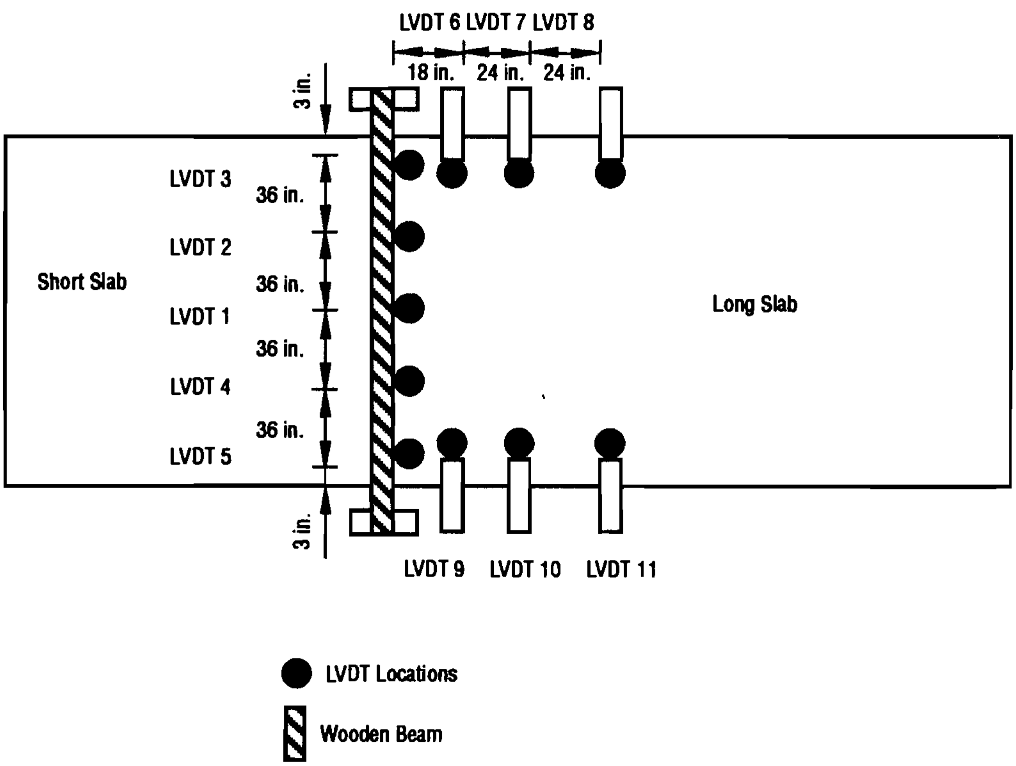


Fig 3.3. Arrangement of LVDTs for transverse and longitudinal curling measurements.

CHAPTER 4. EFFECT OF ENVIRONMENTAL FACTORS ON RIGID PAVEMENT RESPONSE AND REPEATABILITY TESTING OF THE FWD

BACKGROUND

As a first step, the data collected from the tests on the Balcones Research Center pavement facility were used to investigate the influence of environmental factors on rigid pavement response. Of specific interest were the effects of daily and seasonal variations of temperature on rigid pavements. Also of interest were the indirect effects on the pavements of moisture variation and loss of foundation support. Typically, temperature and moisture variations result in horizontal as well as vertical movements in pavements. The horizontal movements directly affect the action of joints and cracks by increasing or decreasing their shear or moment transfer, depending on the direction of the net movement. A temperature or moisture differential between the top and bottom of a concrete slab also causes curling and warping which, in addition to influencing the shear and moment transfer at joints and cracks, also contribute to a loss of support, particularly at the slab corners. The effects of such environmental variations—on rigid pavement response and, therefore, on deflection measurements with the FWD—are the main subject of this chapter. Preceding this study concerning the effect of these factors on deflection response, the reliability and repeatability of FWD deflection and load measurements are also dealt with in some detail.

TEMPERATURE AND MOISTURE EFFECTS ON PCC PAVEMENTS

Changes in environmental conditions, which take place year round, result in temperature and moisture fluctuations in PCC pavements, and there is hardly any time when the temperature or moisture distribution in pavements is uniform throughout. Due to the poor conductivity of concrete, for instance, some time is required for heat to be transferred from the surface downwards, and there is often a difference in temperature between the top and the bottom of the pavement slabs. There is also a similar time lag in the transfer of moisture through PCC slabs, and often different moisture conditions come to exist at the different depths of a pavement. As a result, these fluctuations result in temperature and moisture gradients in the pavements. The temperature and moisture gradients cause differential volumetric changes and, as a consequence, the pavement slabs tend to warp and/or curl. The curling of the pavement slabs is, however, resisted by the weight of the concrete slabs, and there is a consequent build-up of warping and/or curling stresses within the pavement. Analytical work by Westergaard (Ref 27) shows that such stresses can be significant and must be taken into consideration in the design of rigid pavements. In cases where imposed traffic loads produce stresses additive to these curling stresses, excessive stress can build up in the pavements and can eventually contribute to pavement distress. Also, in most cases, the change in profile causes the pavement to partially lose contact with the subgrade, especially near the corner and edges, and this in turn affects the magnitude of the stress due to applied load.

The tests described in Chapter 3 permit documentation of the effects temperature has on PCC pavements, and the results are presented here. The direct effects of moisture gradients on PCC pavements are not addressed. It is noted, however, that Janssen (Ref 28) presents results which indicate that moisture gradients cause PCC slabs to warp considerably and that the resistance due to the weight of the concrete produces tensile stresses at the top of the pavement and compressive stresses at the bottom. Other pertinent conclusions from the work of Janssen were that the significant moisture changes in PCC slabs occur not more than 2 inches below the surface in most cases, and that although the tensile stresses developed are not enough to cause moment failure, they can result in the formation of shallow hairline cracks at the top of the pavement.

ANALYSIS OF TEMPERATURE AND PAVEMENT CURLING DATA

As noted earlier, as part of the FWD deflection tests conducted on the BRC research pavement, slab temperature data at the locations shown in Fig 3.1 and weather data were collected. Data from the curling tests on the pavement also consisted of similar temperature data and vertical displacement data. These were the data analyzed to determine the direct effects of daily and seasonal temperature variations on rigid pavement response. The mean slab temperatures 1 inch from the top, at mid-depth, and 1 inch from the bottom of the 10-inch-thick pavement were determined by averaging the measurements obtained from the three thermocouples, and by the temperature differential calculated as the difference between the average top and bottom temperatures. To obtain the displacement or vertical movement at each LVDT location, the LVDT measurements were referenced to the reading corresponding to a zero temperature differential at each particular location. Thus, the displacement data obtained from the curling tests provide a definition of the slab profile referenced to the profile at a zero temperature differential. These data were analyzed to examine the extent of temperature variation in PCC pavements and the magnitude and trends of the vertical displacements associated with these variations.

TEST SITE WEATHER CONDITIONS

The temperature of a rigid pavement depends on the effective solar radiation and on the amount of the heat energy from this radiation that is absorbed and retained. In turn, the solar radiation reaching the pavement depends on prevailing weather conditions; on the absorption and retention abilities of the pavement; and on the physical properties of the structure such as color, surface texture, amount of shading, and the slope of the surface.

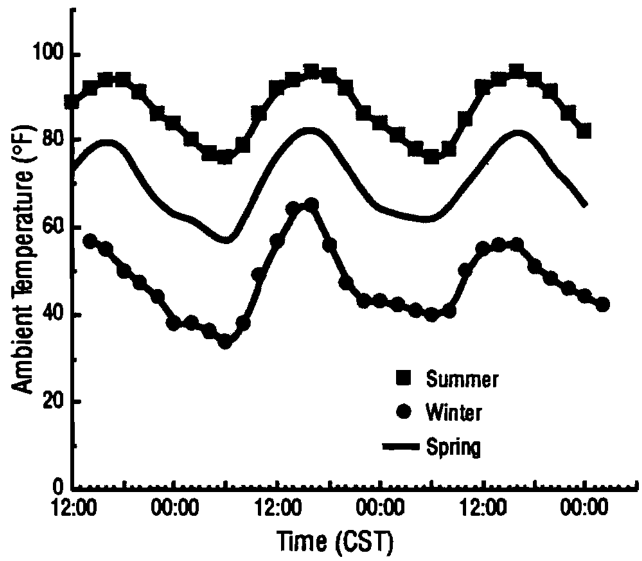


Fig 4.1. Typical daily and seasonal ambient temperature variations at test site.

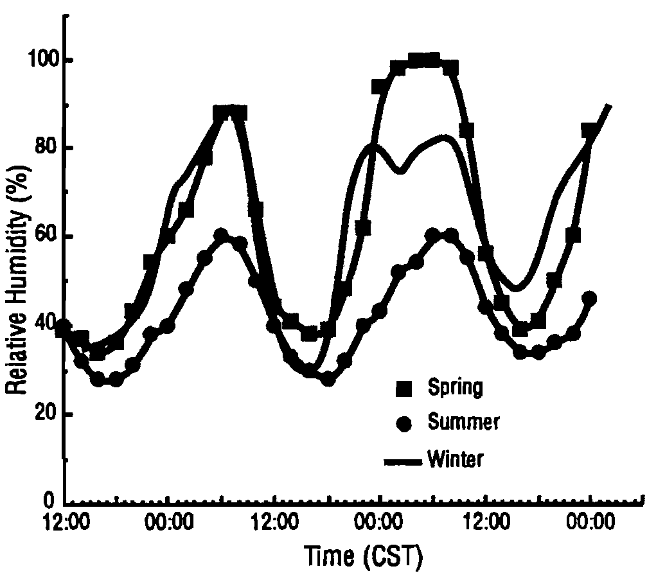


Fig 4.2. Typical daily and seasonal relative humidity variations at test site.

While the physical properties of a particular pavement are generally controllable and the best choices can be made during design and construction, weather conditions are *not* controllable and depend very much on the climate of a given locality. Consequently, the temperature of a pavement is influenced most by the climate of the area where it is located.

An inspection of the data collected indicates that, as in most parts of Texas, the weather in the vicinity of the research pavement can be divided into four seasons: a spring period from March to May; the summer months of June, July, and August; the fall season from September to November; and the winter months spanning December, January, and February. However, with conditions in the spring and fall somewhat similar as far as ambient temperature and relative humidity were concerned, the seasons could be grouped into three periods. Figure 4.1 shows typical daily ambient temperature variation for these three periods and also gives an indication of the differences between the seasons. Each graph depicts the ambient temperature variation for a period of not less than two days with clear skies and stable weather during which there was a large variation in the temperature. Figure 4.2 shows the corresponding relative humidity variation for these selected days. In all cases, the daily temperature and relative humidity variations approximately follow a sine wave pattern with the maxima of the temperature curves occurring at about the time of occurrence of the minima of the relative humidity curves. Except for cloudy or rainy days, when the temperature and relative humidity variation patterns were unpredictable, these trends were typical of all the days investigated.

Figure 4.1 shows an appreciable and distinct seasonal variation in ambient temperature, especially between the winter and summer periods, typical of the data collected at the test site. No such seasonal distinction was obvious in the case of the relative humidity as illustrated by Fig 4.2. In general, it was evident from the results that daily ambient temperature variations of up to 30°F and seasonal variations of up to 45°F were typical. Another interesting feature of the results is the occurrence of the maxima and minima of both the ambient temperature and relative humidity at about the same times on days with clear and stable weather in each of the seasons.

PAVEMENT TEMPERATURE ANALYSIS

Figures 4.3, 4.4, and 4.5 illustrate the temperature variation in the test pavement typical for the spring, summer, and winter seasons, respectively, and the relationship of the ambient temperature to the pavement temperatures. Shown are the pavement temperatures 1 inch from the surface, at mid-depth, and 1 inch from the bottom of the 10-inch-thick pavement. Again, the results shown are for days with stable weather conditions and clear skies. As can be seen from these graphs, which are typical of the general results obtained, in each case the slab temperature follows a sine wave pattern, with the trend of the top pavement temperature closely resembling that of the ambient temperature. The slab top temperature also attains its maximum value within the hour of the ambient temperature's reaching a maximum. In the warmer climate of Texas, the results also showed that the pavement temperatures were almost always higher than the ambient temperature, except in the colder winter months when the pavement temperature and the ambient temperature were on the same order of magnitude in most cases. It was also observed that during all the seasons, the warming part of the pavement temperature curves were steeper than the cooling-off part, indicating the rapid rate at which heat is supplied to the pavement in the mornings as compared with the rate at which it loses heat in the evenings. As is to be expected, on cloudy days or days of unstable weather, the pavement temperature variation did not follow any well-defined pattern; however, the trends were still very similar to those of the ambient temperature and in most cases followed a distorted sine wave pattern.

As indicated above, a temperature gradient in rigid pavements is responsible for pavement curling. Figure 4.6 illustrates the trend in the temperature differential of the research pavement typical for days with clear and stable weather in the spring, summer, and winter seasons. These temperature differential curves correspond to the pavement temperatures presented in Figs 4.3, 4.4, and 4.5, respectively, under the ambient temperature and relative humidity conditions shown in Figs 4.1 and 4.2, and they also follow a similar sine wave pattern. An important observation is the similarity in the trends of the temperature differential in all the seasons, with the temperature differential curves practically overlapping. The results obtained indicated that, in general, the trends and magnitude in temperature differential change were similar for the spring and summer seasons but were appreciably different for the winter season, especially during the daytime periods, as illustrated in Fig 4.6. As with the pavement temperature, on clear days with stable weather, the temperature differential typically increases at a relatively rapid rate in the morning, as the ambient temperature increases, until it reaches a maximum sometime between noon and 4:00 PM. It then decreases at a comparatively slower rate, as the ambient temperature falls to a minimum, anywhere between 11:00 PM and 8:00 AM. This high rate of increase in the temperature differential in the morning is the direct result of the influence of solar radiation on the pavement. The intensity of sunlight increases at a rapid rate in the morning, and the rate at which heat is supplied to the pavement—which is directly related to this former rate—changes continuously. The rate at which the pavement loses heat is, on the other hand, more controlled and influenced by stable characteristics such as material properties of the pavement on clear-weather days.

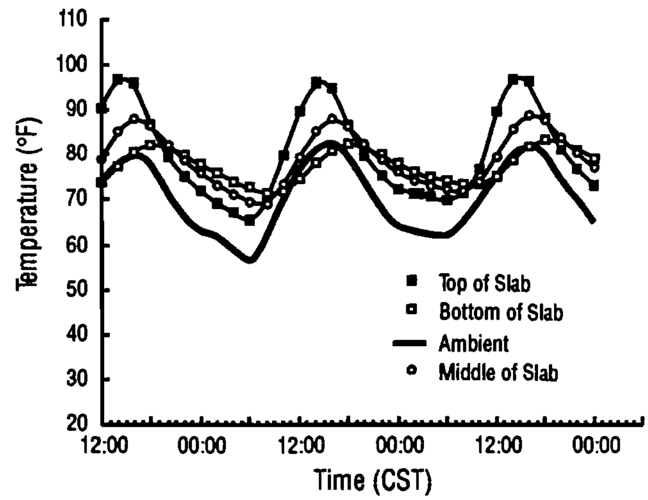


Fig 4.3. Pavement temperature trends typical of the spring.

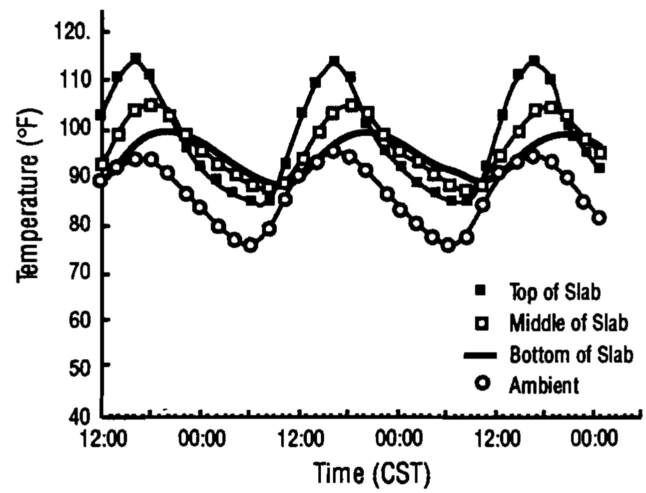


Fig 4.4. Pavement temperature trends typical of the summer.

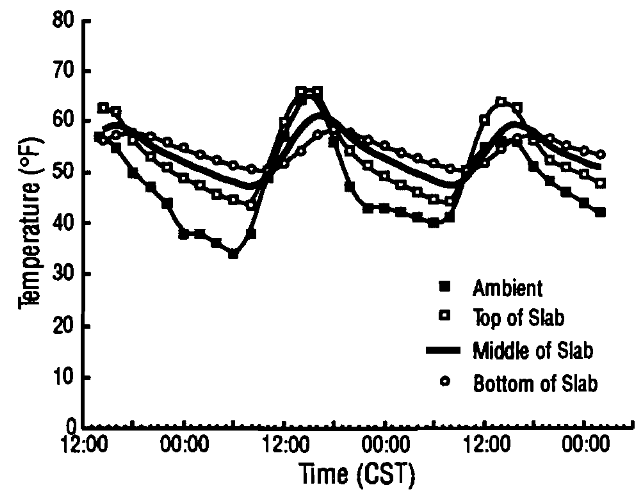


Fig 4.5. Pavement temperature trends typical of the winter.

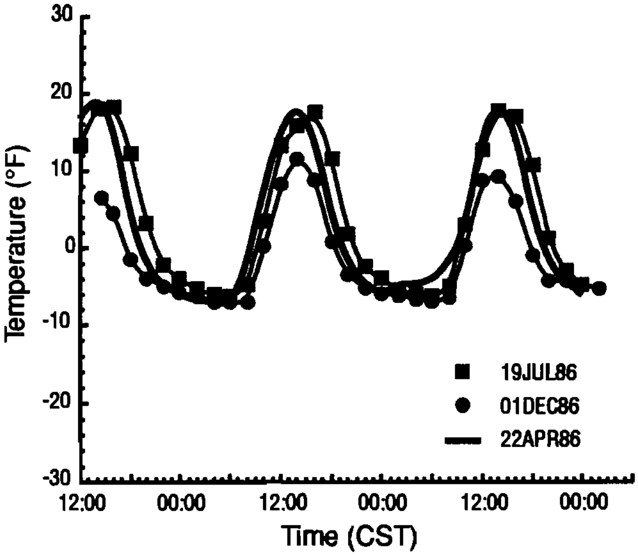


Fig 4.6. Daily and seasonal variations of slab temperature differential.

From the results obtained for a period of three years for all types of weather, the corresponding times of occurrence of the maximum positive and minimum negative temperature differentials were noon to 5:30 PM and 10:00 PM to 8:00 AM, respectively. These correspond to the times when the slab is in its maximum downward and upward curl positions, respectively, with the latter in effect representing the period of time when the pavement will offer the least resistance to traffic loads as a result of such movements.

Figure 4.7 illustrates the temperature gradients typical of the BRC pavement at different times of the year. The temperature gradients in the pavement at the same time of day are shown for different months of the year representative of the calendar seasons. Again it is apparent that although there is a shift in the magnitude of the temperature in the pavement with respect to the seasons, the temperature gradients are typically the same and are in agreement with the earlier findings. In the three years of testing a maximum temperature differential of 20.4°F and a minimum of -9.9°F were recorded in the research

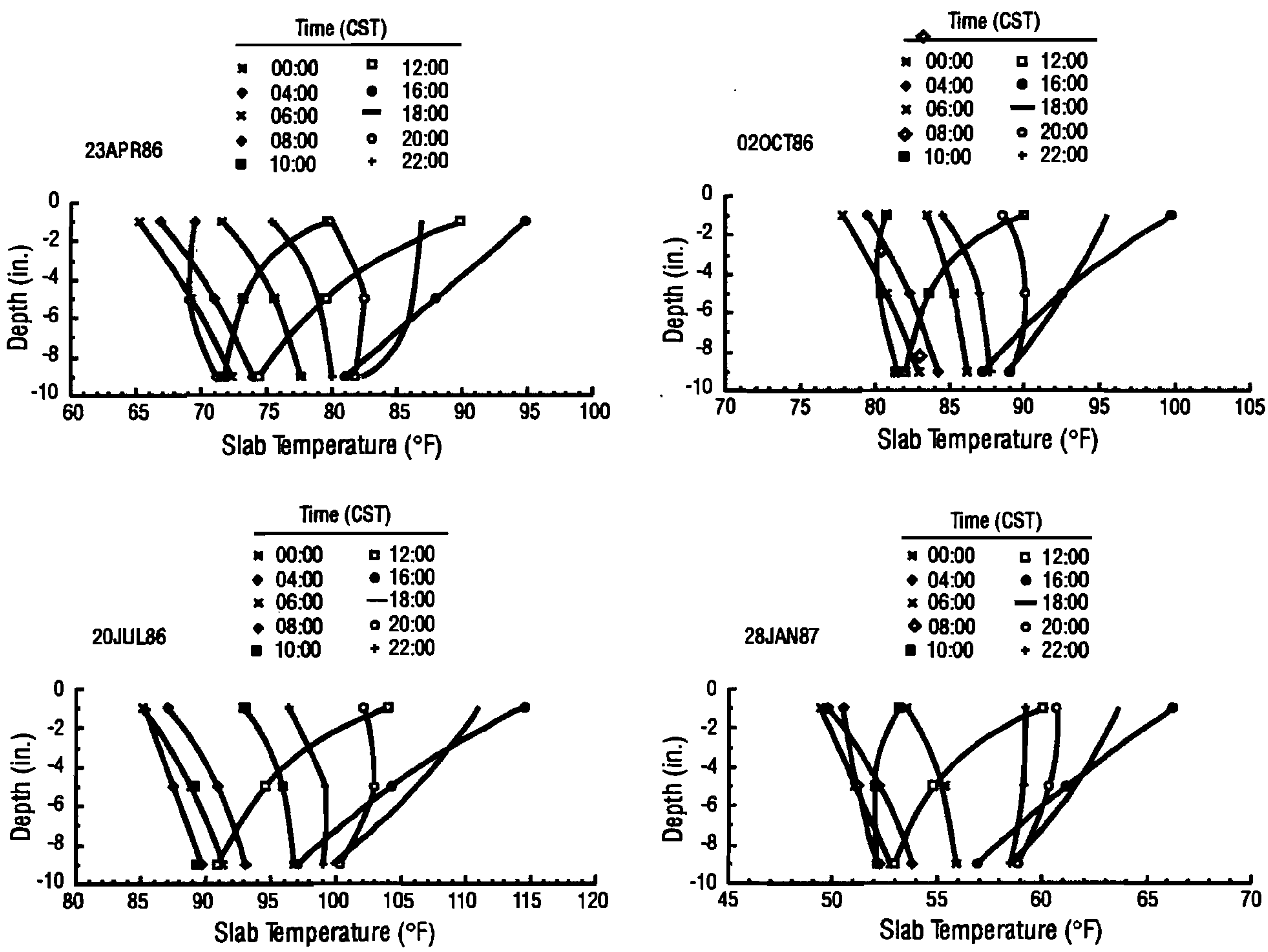


Fig 4.7. Daily and seasonal variations of slab temperature gradients.

pavement slab. As expected, on cloudy or rainy days the temperature differential in the pavement did not follow any of the well-defined trends described. Typically, cloudy and rainy weather, by decreasing the radiation provided by the sun, had a damping effect on the temperature differential curves just as it did on the ambient and pavement temperatures.

ANALYSIS OF VERTICAL DISPLACEMENT DUE TO CURLING

The trends with time of the vertical displacements caused by curling of the test pavement at selected points along the transverse joint are illustrated in Figs 4.8, 4.9, and 4.10 for periods typical of the spring, summer, and winter seasons, respectively. To understand these trends, the displacement of the slab must be examined in relation to temperature variations in the research pavement. Accordingly, the displacement trends in Fig 4.8 correspond to the ambient and pavement temperature variations in Fig 4.3, and the displacement trends in Fig 4.9 to the ambient and pavement temperature variations in Fig 4.4. Similarly, Fig 4.11 shows the ambient temperature, the pavement temperature at different depths, and the slab temperature differential variation in the winter period corresponding to the displacements in Fig 4.10. The slab temperature differential variations corresponding to the spring and summer periods are shown in Fig 4.6, and the temperature gradients on a selected day within each of these periods are shown in Fig 4.7.

Noting that the vertical displacement at the transverse joint of the research pavement is due to a combination of transverse and longitudinal curling, Figs 4.8, 4.9, and 4.10 illustrate the cyclic effect of temperature on pavement curling typical of the spring, summer, and winter seasons, respectively. The greatest vertical displacement occurs at the corners and the least at the center of the joint. Examining Fig 4.8 with reference to Fig 4.3 and the appropriate temperature differential curve for the spring in Fig 4.6, it can be noted that on any given day the pavement is displaced upward from its lowest curled-down position in the early afternoon, when the slab temperature differential is at a maximum. This continues until the pavement reaches a maximum curled-up position at about the same time the temperature differential reaches a minimum, very early in the morning of the next day. With daybreak and the accompanying sunrise, the pavement begins to curl downward with the temperature differential increasing until again it reaches its lowest curled-down position, at the time of the next occurrence of the maximum positive slab temperature differential, to complete a cycle. Figures 4.9 and 4.10 show the recorded trends typical of the summer and spring seasons, respectively, which are similar to the trends in Fig 4.8.

From a comparison of Figs 4.8, 4.9, and 4.10, it is evident that there is an overall increase in the maximum vertical displacement at the transverse joint with an

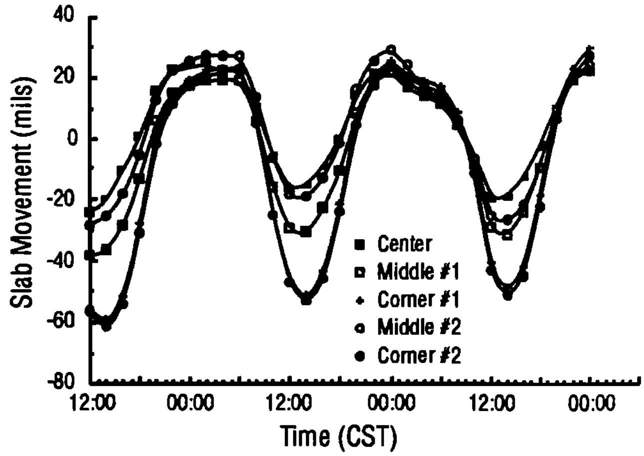


Fig 4.8. Typical vertical displacement variations at the transverse joint of research pavement in the spring.

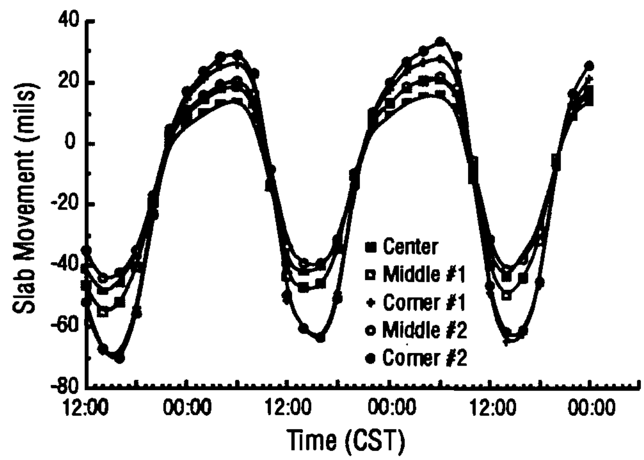


Fig 4.9. Typical vertical displacement variations at the transverse joint of research pavement in the summer.

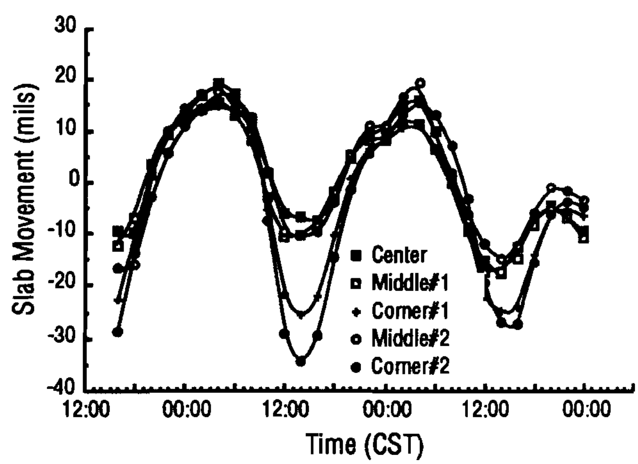


Fig 4.10. Typical vertical displacement variations at the transverse joint of research pavement in the winter.

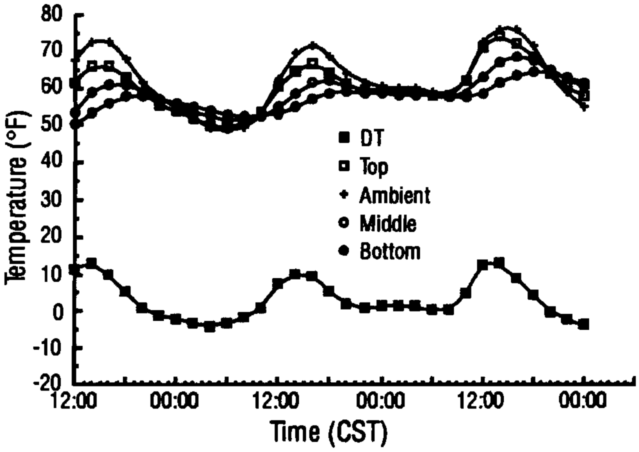


Fig 4.11. Temperature and temperature differential variations in research pavement – January 1987.

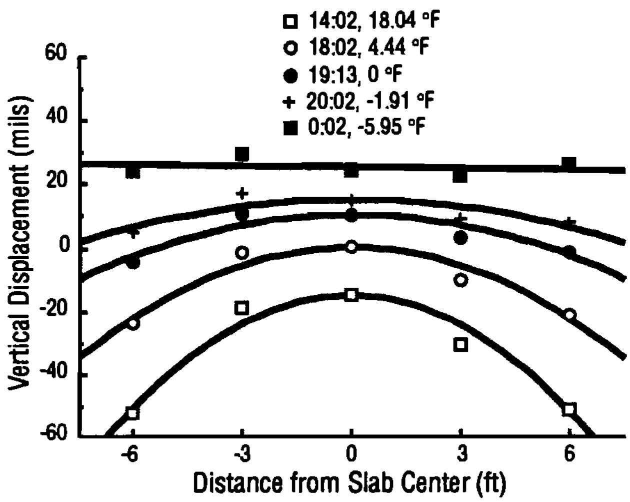


Fig 4.12. Research pavement transverse joint profile variation typical of the spring season.

increase in seasonal temperature. This is especially true of the downward vertical displacement caused by a positive temperature differential; that is, where the temperature at the top surface of the research pavement is higher than that at the bottom surface. The corresponding increase in the upward vertical displacement, caused by a negative temperature differential, is minimal. An examination of Fig 4.6 sheds some light on this result. It is clear that, for the 10-inch-thick research pavement, although there is an appreciable difference between the positive temperature differential of the winter and those of the warmer spring and summer seasons, the difference between the negative temperature differentials is minimal.

The results shown in Figs 4.8, 4.9, and 4.10 are presented in a different form in Figs 4.12, 4.13, and 4.14, respectively, which illustrate the change in the transverse profile of the research pavement at the doweled transverse joint during selected parts of the cyclic change in temperature differential. The profiles attained between the extreme positive and negative temperature differentials are shown. The relative positions of the profile curves with respect to the abscissa also give an indication of the extent of the longitudinal curling of the pavement. The effects of a temperature differential change on the curling of the rigid pavement are considerable in all cases. Examining the profiles in each case, there is seen an indication, typical of the results obtained, that the magnitude of curling of the rigid pavement is higher in the summer months than in the spring and winter seasons. It is suspected that the reduced exposure of the pavement to sunlight in the spring and winter seasons and, possibly, the effects of moisture are responsible for the reduced curling of the pavement. As the figures show, in the warm climate of Texas, the magnitude of the maximum downward displacement is

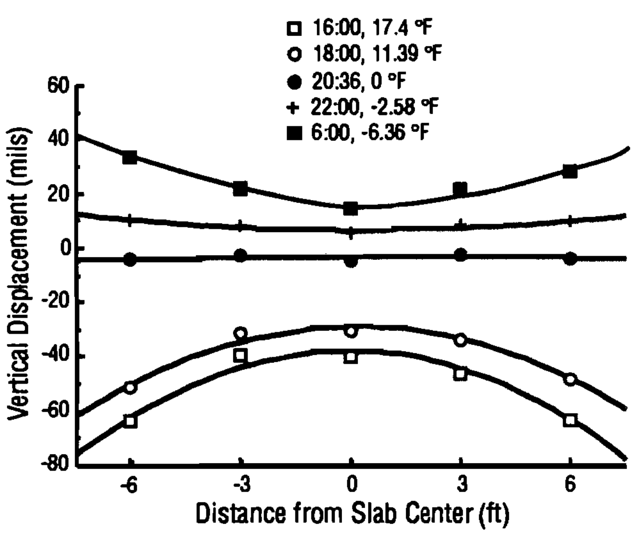


Fig 4.13. Research pavement transverse joint profile variation typical of the summer season.

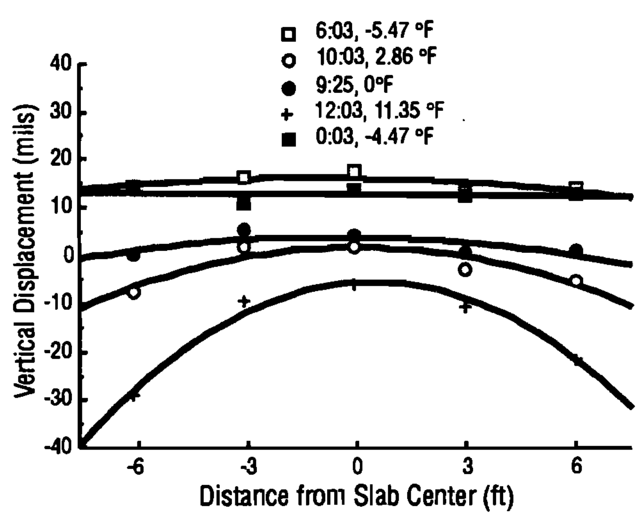


Fig 4.14. Research pavement transverse joint profile variation typical of the winter season.

considerably higher than the maximum upward displacement from the zero temperature differential position. In fact, in some cases, as illustrated in Figs 4.12 and 4.14, these effects seemed to be sufficient to cause the pavement to curl downward even when the temperature differential was negative; that is, when the bottom surface was warmer than the top surface. The results obtained also indicated that because of such effects and/or the additional effect of longitudinal curling of the slab, it was not uncommon to have different slab profiles, or unequal vertical displacements at a particular location along the joint, for the same temperature differential. Unfortunately, the results of the tests conducted to measure vertical displacement due to longitudinal curling were not conclusive enough to permit documentation of the influence of such effects. The transverse profile of the slab at the joint also was not symmetrical in all cases. Another interesting observation was that the slab was not necessarily flat in the transverse direction when the temperature differential was zero.

For this 10-inch-thick research rigid pavement, the temperature differentials on the order of -10 to 21°F that were measured resulted in vertical displacements at slab corners from one extreme to the other on the order of 100 mils. The extreme upward vertical displacements occurred in the early morning hours between 3:00 and 7:00 CST. Downward vertical displacements generally reached a maximum in the afternoon between 13:00 and 17:00 CST. The maximum vertical movements occurred in the warmer summer months when temperature fluctuations were at a maximum. A downward vertical displacement of approximately 70 mils at both corners of the large slab in the middle of July and an upward vertical displacement of 38 mils at one corner five days later, both relative to the average slab position at a temperature differential of zero degrees Fahrenheit, were the maximum recorded on the research pavement. From the results obtained from the research pavement, which is similar in design to some of the roads in the state, it is obvious that such vertical displacements have to be seriously considered in the evaluation of FWD deflection measurements on rigid pavements.

FWD REPEATABILITY EVALUATION

Before investigating the effects of environmental factors of the kind discussed in the preceding section on FWD measurements, it was necessary at the outset to determine the reliability and repeatability of FWD load and deflection measurements to ensure confidence in the results obtained in this study. Therefore, in a special experiment, FWD deflection measurements were taken at the center of the large slab of the research pavement and at its edges and corners for various combinations of load level and drop sequences for such an investigation. In all, two types of tests were carried out. One set of tests consisted of deflection measurements taken at a particular location with the FWD stationary throughout all drop sequences. The second set of tests was made up of deflection measurements during which the FWD was moved and then returned to the same location between the drop sequences. All the tests were conducted in a short period of time to ensure constant environmental conditions and thus negate the need to account for temperature and/or moisture effects on the pavement. The tests were conducted with the 440-lb-weight set-up of the FWD dropped from four heights, which gave the four load ranges of 4,500 to 6,000; 9,000 to 11,000; 13,000 to 14,500; and 16,500 to 18,500 lbf. In the repeatability analyses the load/deflection ratio, which is a reflection of the stiffness of the pavement, was selected for study. Using the load/deflection ratio has the advantage of removing error that might be due to variability in the force obtained with the FWD load dropped ostensibly from the same height, and it also allows the use of raw load and deflection data not subject to any normalization procedures to account for such variability.

Data from a series of ten FWD measurements at each of the three test locations were examined. The tests were conducted in a period of a few minutes within which environmental conditions were constant. Figure 4.15 illustrates the relationship between pavement response in terms of the load/deflection ratio at the center of the load plate and the load level at the three test locations. This figure indicates that the load/deflection ratio at each of the three locations is approximately the same at all the load levels, suggesting a linear relationship between load and deflection. This relationship is in fact borne out by Fig 4.16, which illustrates the linear relationship between FWD load and deflection measurements at the three test locations for stationary tests conducted under constant environmental conditions. Figure 4.17, however, shows

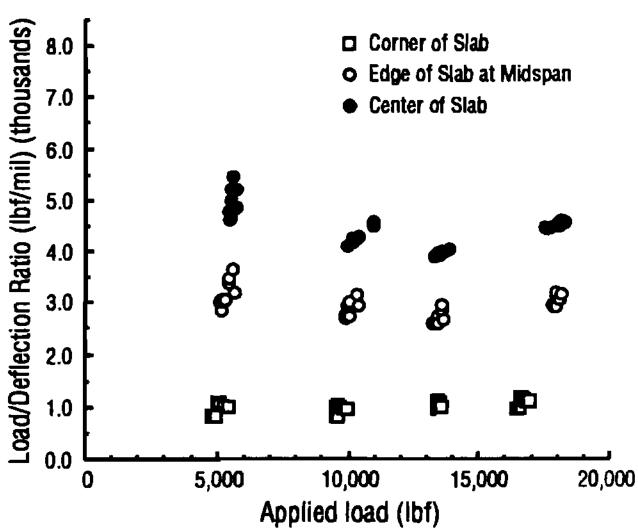


Fig 4.15. Effect of applied load level on rigid pavement response.

that the variability in the response of the pavement for each of the drop heights decreases at the higher load levels. Selecting a coefficient of variation of 10 percent as the maximum variability acceptable in the FWD measurement of the pavement response, the measurements corresponding to the first drop height (5,000 to 6,500 lbf load level) were considered unacceptable, mainly because of the results of the tests carried out at the corner of the slab. Similar results were obtained for pavement response measurements with most of the other six FWD deflection sensors. Consequently, pavement load and deflection measurements corresponding to the first drop height were not used in subsequent analysis. (Figure 4.17 also shows that the greatest variability in the measurement of the pavement response is at the corner test location, which is followed by the measurements at the edge test location, with the least variability observed in the measurements at the center test location.)

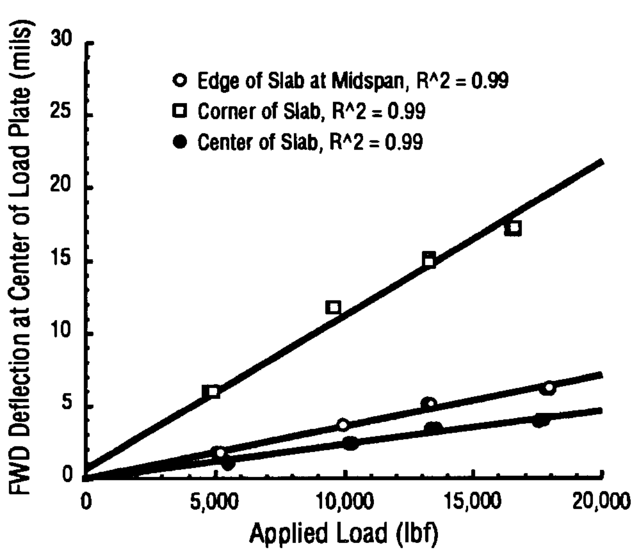


Fig 4.16. Linear relationship between FWD load and deflection measurements on rigid pavements.

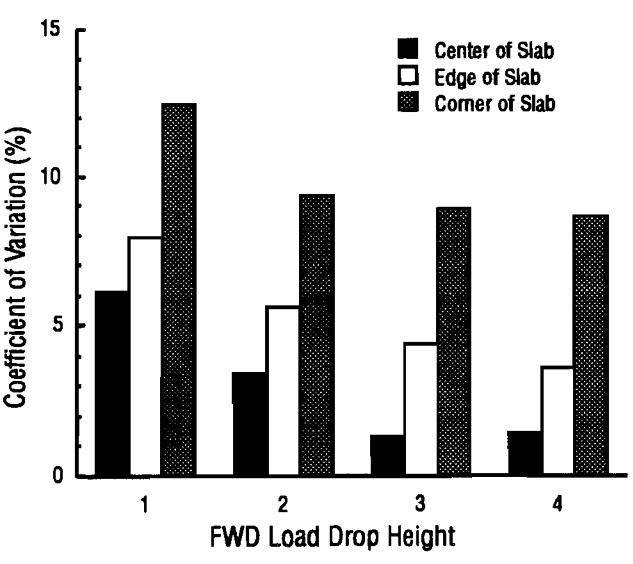


Fig 4.17. Variability in pavement response at each FWD load drop height.

Analysis of the repeatability of the FWD involved a comparison of the coefficients of variation of the load/deflection ratios for the stationary and on-off repeatability tests. Table 4.1 presents data on the repeatability of the FWD typical of stationary tests conducted at the center of the slab under constant environmental conditions. The data shown are for five tests, each conducted at the three load levels corresponding to the last three drop heights mentioned previously. The results illustrate that the coefficient of variation of the load/deflection ratios was less than 3 percent for each of the deflection sensors, and that in most cases the coefficient of variation increased with an increase in the load level. In addition, the FWD shows an excellent repeatability of the measured applied load obtained from drops from the same height, with the coefficient of variation of the five load measurements at each of the three levels not exceeding 1 percent. In this case also there is an indication that the coefficient of variation improves with increasing magnitude of applied load. Table 4.2 is a corresponding table that presents data typical of on-off repeatability tests at the center of the slab conducted under constant environmental conditions. Overall, there is a marked increase in the coefficient of variation of the load/deflection ratios determined. However, the coefficients of variation for all the sensors are below an acceptable level of 7 percent, and the coefficient of variation for the pavement response directly under the load is less than 5 percent at all three load levels. The coefficients of variation in the load measurements are also below 5 percent in all cases.

In Table 4.3, results are presented for corresponding stationary and on-off repeatability tests at the edge and at the corner of the large slab of the test pavement. Examination of this table reveals that, in both cases, these tests under constant environmental conditions also point to a high repeatability of the FWD load and deflection measurements corresponding to the load levels shown. Again, although the FWD is more repeatable when used in tests which do not involve moving the device, the results of the on-off tests also show that the FWD load and deflection measurements are very repeatable. These tests at the three locations, therefore, indicate that load and deflection measurements on rigid pavements obtained with the FWD are reliable, consistent, and repeatable. In no instance was the coefficient of variation in the pavement response—measured with the FWD in terms of load and deflection—more than 7 percent. In fact, the slight increases in variability of the on-off tests as compared with the stationary tests could even be explained as a result of changes in the actual test locations due to movement of

TABLE 4.1. RESULTS OF STATIONARY REPEATABILITY TESTS AT CENTER OF LARGE SLAB

Load Range	Test Number	Load (lbf)	Sensor Load/Deflection Ratio (lbf/mil)						
			DF1	DF2	DF3	DF4	DF5	DF6	DF7
Low	1	10,304	4,240	4,705	4,662	5,510	6,691	7,988	9,813
(9,000 - 11,500 lbf)	2	10,192	4,194	4,570	4,612	5,450	6,618	7,900	9,350
	3	10,120	4,164	4,621	4,579	5,411	6,571	7,845	9,638
	4	10,088	4.275	4,606	4,565	5,395	6,551	7,820	9,607
	5	10,096	4,278	4,610	4,568	5,399	6,556	8,077	10,096
	Mean	10,160	4,230	4,622	4,597	5,433	6,597	7,926	9,701
	C .V. (%)	0.89	1.18	1.08	0.89	0.89	0.89	1.34	2.84
Medium	1	13,568	4,014	4,434	4,363	5,101	6,139	7,666	9,357
(13,000 - 14,500 lbf)	2	13,440	3,976	4,335	4,377	5,053	6,222	7,593	9,531
	3	13,392	3,916	4,376	4,306	5,111	6,200	7,566	9,236
	4	13,448	3,932	4,338	4,380	5,133	6,226	7,598	9,274
	5	13,424	3,925	4,330	4,316	5,124	6,214	7,584	9,258
	Mean	13,454	3,953	4,363	4,349	5,104	6,200	7,601	9,331
	C .V. (%)	0.50	1.05	1.00	0.81	0.61	0.57	0.50	1.30
High	1	17,648	4,445	4,848	4,902	5,693	6,948	8,444	10,443
(15,500 - 20,000 lbf)	2	17,672	4,451	4,909	4,855	5,701	6,957	8,455	10,457
	3	17,584	4,429	4,884	4,778	5,672	6,923	8,413	10,405
	4	17,736	4,468	4,927	4,768	5,721	6,983	8,173	10,252
	5	17,512	4,456	4,811	4,810	5,649	6,894	8,379	10,362
	Mean	17,630	4,450	4,876	4,823	5,687	6,941	8,373	10,384
	C .V. (%)	0.49	0.32	0.96	1.16	0.49	0.49	1.38	0.79

TABLE 4.2. RESULTS OF ON-OFF REPEATABILITY TESTS AT CENTER OF LARGE SLAB

Load Range	Test Number	Load (lbf)	Sensor Load/Deflection Ratio (lbf/mil)						
			DF1	DF2	DF3	DF4	DF5	DF6	DF7
Low	1	10,080	4,271	4,603	4,561	5,390	6,720	7,814	9,600
(9,000 - 11,500 lbf)	2	10,976	4,573	5,105	5,153	5,998	6,775	8,509	10,453
	3	10,936	4,500	4,994	4,948	5,976	7,107	8,749	11,392
	4	10,400	4,280	4,749	4,705	5,810	6,582	8,320	9,905
	5	9,944	4,292	4,459	4,499	5,433	6,457	8,218	10,358
	Mean	10,467	4,343	4,782	4,774	5,722	6,727	8,322	10,342
	C .V. (%)	4.55	4.46	5.60	5.73	5.11	3.61	4.19	6.59
Medium	1	13,472	3,939	4,346	4,332	5,142	6,086	7,611	9,555
(13,000 - 14,500 lbf)	2	13,832	4,044	4,462	4,391	5,200	5,936	7,477	9,539
	3	13,696	4,004	4,362	4,404	4,999	6,087	7,567	9,192
	4	13,528	3,956	4,421	4,350	5,326	6,121	7,643	9,330
	5	13,264	3,878	4,279	4,321	5,063	6,001	7,667	9,148
	Mean	13,558	3,964	4,374	4,359	5,146	6,049	7,593	9,353
	C .V. (%)	1.60	1.60	1.61	0.84	2.46	1.27	0.99	2.03
High	1	18,072	4,507	5,020	4,965	5,830	7,229	8,647	10,446
(15,500 - 20,000 lbf)	2	18,152	4,619	5,099	5,042	6,011	6,824	8,365	10,492
	3	18,288	4,560	4,970	4,970	5,899	7,088	8,750	10,821
	4	18,016	4,538	4,949	4,896	5,664	7,093	8,620	10,660
	5	17,600	4,433	4,835	4,944	5,677	6,929	8,263	9,943
	Mean	18,026	4,531	4,975	4,963	5,816	7,033	8,529	10,473
	C .V. (%)	1.44	1.51	1.95	1.07	2.53	2.24	2.41	3.16

the FWD back and forth between tests, and not in the pavement response.

EFFECT OF TEMPERATURE ON FWD PAVEMENT RESPONSE MEASUREMENTS

From the preceding discussion in this chapter, it is apparent that environmental variations have a considerable influence on rigid pavements. The results of an investigation conducted to evaluate the direct effects of temperature on results obtained with the FWD are presented in this section. Of main interest were the result of daily temperature variation and seasonal effects on the response of rigid pavements. Figure 4.18 presents graphs of deflection measurements at the center of the large slab of the research pavement versus the daily temperature differential variation for typical days with clear and stable weather. The figure indicates that there is no appreciable influence of the daily temperature differential variation on pavement deflection at the center of the slab, except for a very minor increase in deflection at the higher temperature differentials. As can be seen from Figs 4.12, 4.13, and 4.14, in the warm Texas weather the pavement curls downward considerably in the afternoons. It is believed that this curling causes the slab to lose some

TABLE 4.3. RESULTS OF STATIONARY AND ON-OFF REPEATABILITY TESTS AT THE EDGE AND CORNER OF LARGE SLAB

FWD Location	Load Range	FWD Test Type	Load (lbf)	Sensor Load/Deflection Ratio (lbf/mil) DF1	DF2	DF3	DF4	DF5	DF6	DF7
Edge	Low	Stationary[2]	9,906	2,732	2,915	2,940	3,483	4,341	5,473	6,831
			(0.33)	(0.79)	(0.75)	(1.12)	(0.95)	(0.89)	(0.33)	(0.33)
		On-Off[3]	10,146	2,955	3,178	3,170	3,766	4,567	5,690	7,415
			(1.97)	(5.34)	(5.62)	(5.79)	(5.67)	(4.42)	(5.44)	(4.12)
	Medium	Stationary	13,302	2,611	2,815	2,830	3,351	4,199	5,237	6,592
			(0.29)	(0.37)	(0.54)	(0.29)	(0.29)	(0.65)	(0.81)	(0.81)
		On-Off	13,517	2,747	3,007	2,998	3,576	4,371	5,450	6,987
			(0.75)	(5.10)	(5.41)	(4.88)	(5.35)	(4.03)	(4.88)	(4.59)
	High	Stationary	17,899	2,917	3,154	3,173	3,772	4,71 3	5,849	7,452
			(0.36)	(0.75)	(0.40)	(0.48)	(1.14)	(0.44)	(0.36)	(0.55)
		On-Off	17,994	3,082	3,360	3,332	3,970	4,897	6,062	7,703
			(0.57)	(3.06)	(4.53)	(3.80)	(4.39)	(3.77)	(4.1 0)	(3.36)
Corner	Low	Stationary	9,512	816	1,018	2,666	1,355	1,867	2,599	3,622
			(0.62)	(0.35)	(0.82)	(0.61)	(0.90)	(1.04)	(1.04)	(0.98)
		On-Off	9,730	990	1,225	3,170	1,614	2,185	3,000	4,148
			(1.80)	(2.55)	(1.92)	(1.93)	(1.90)	(1.59)	(1 .13)	(1.48)
	Medium	Stationary	13,294	885	1,106	2,437	1,465	2,01 7	2,780	3,860
			(0.1 7)	(0.66)	(0.94)	(1 .48)	(1 .03)	(1.12)	(1.21)	(0.77)
		On-Off	13,507	1,042	1,289	2,832	1,700	2,310	3,144	4,320
			(0.38)	(3.29)	(2.91)	(2.70)	(3.17)	(2.58)	(2.47)	(2.19)
	High	Stationary	16,554	966	1,209	2,497	1,603	2,194	3,013	4,142
			(0.48)	(0.38)	(0.65)	(1.12)	(1.16)	(0-90)	(0.73)	(0.61)
		On-Off	16,771	1,131	1,381	2,859	1,836	2,499	3,389	4,604
			(0.72)	(3.34)	(2.54)	(3.16)	(2.95)	(2.08)	(1.43)	(1.73)

[1]Numbers in brackets are the coefficients of variation, percent, of five tests.
[2]FWD was parked at test location throughout these tests.
[3]FWD was moved from test location and then back to the same location in these tests.

contact with the foundation at the center, resulting in the slightly higher deflections at the higher temperature differentials. From Fig 4.19, it is also apparent that the effect of seasonal environmental changes on FWD deflection measurements at the center of the slab is minimal. There is, however, a distinct, albeit small, increase in deflection at the center of the slab between the winter and summer seasons. A possible explanation is that in the very warm summer months the slab curls downward more, both in the transverse and the longitudinal directions (Fig 4.13), than it does in the cooler winter months when there is also less moisture variation in the pavement (Fig 4.14). As a result, there is more loss of contact with the foundation at the slab center and, consequently, higher deflections in the summer. Another explanation is that a softening up of the asphalt-stabilized base of the pavement in the summer results in a less stiff foundation and therefore in higher deflections.

Figures 4.20 through 4.23 present typical results of FWD tests at the corners and edges of the large slab of the research pavement. In most cases there is a linear trend between the deflection measurements and the daily temperature differential variation. As is to be expected, the deflections at the corners and edges of the PCC pavement slab generally decrease as the temperature differential increases. There is little doubt that the downward curling of the pavement slab as the temperature differential increases (Figs 4.12, 4.13, and 4.14) causes the pavement to gain more foundation support at the corners and edges, resulting in an increase in stiffness and a decrease in deflection. When the data are grouped according to the season during which the results were taken, there is some indication that seasonal effects in some instances influence FWD deflections of the rigid pavement. From Figs 4.20 and 4.22 there is evidence that the deflections at the corner and edge of the slab with no voids, respectively, are higher in the summer (shaded) than in the winter (unshaded). In light of this, it is likely that the second explanation given for the cause of an increase in deflection at the center of the slab in the summer, a softening of the base material, is more probable. The increase in deflection in the summer was not the case, though, for deflections at the corner and edge of the slab with voids. There was no distinction between winter and summer measurements at these locations, as illustrated by Figs 4.21 and 4.23. It is believed that the presence of voids at these locations made the additional detrimental effect, a softening-up of the base material in the summer, minimal.

EFFECT OF VOIDS ON FWD MEASUREMENTS

One of the major factors responsible for rigid pavement distress is the development of voids underneath the pavement slabs and the resulting loss of foundation

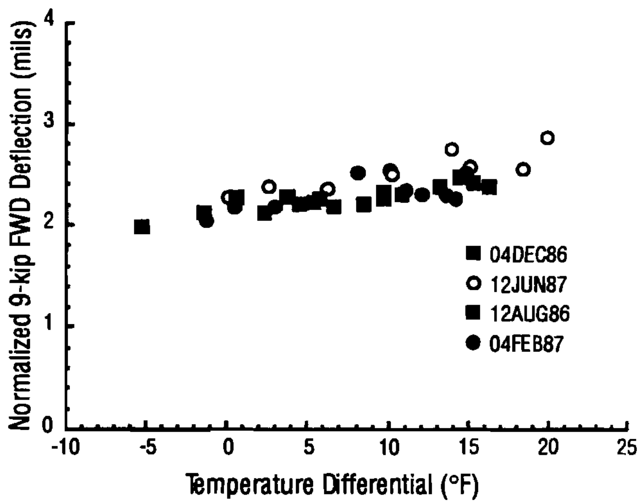


Fig 4.18. Effect of daily temperature variation on deflection at the center of pavement slab.

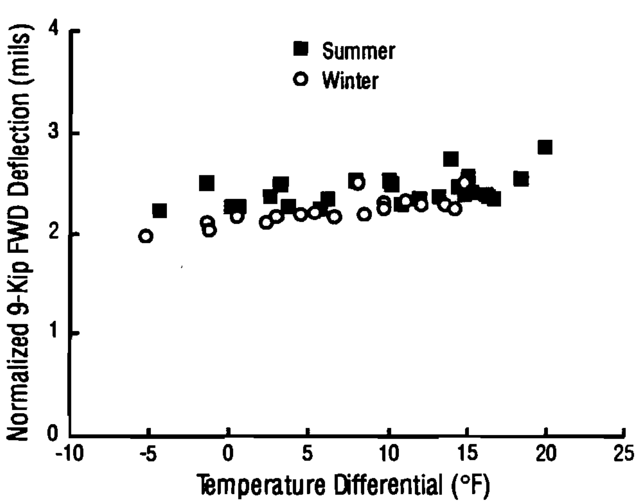


Fig 4.19. Effect of seasonal changes on deflection at the center of pavement slab.

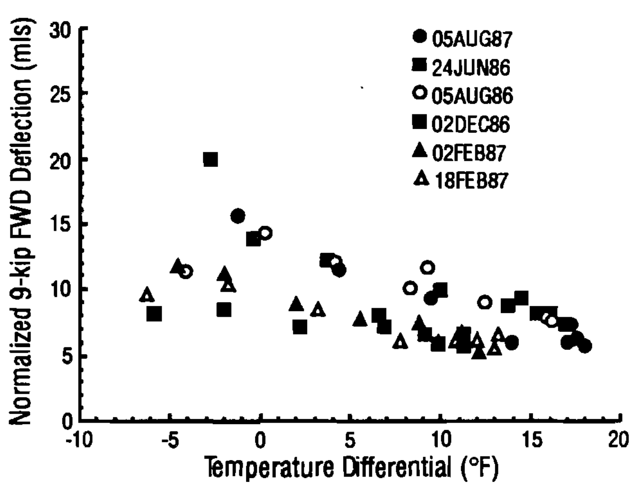


Fig 4.20. Effect of temperature differential variation on deflection at corner of slab with no void.

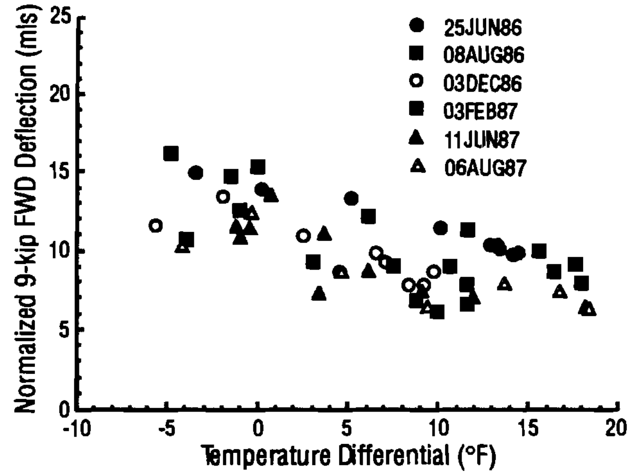


Fig 4.21. Effect of temperature differential variation on deflection at corner of slab with void.

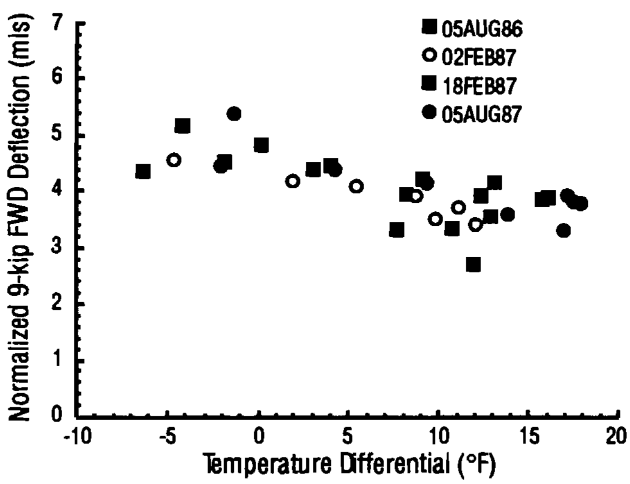


Fig 4.22. Effect of temperature differential variation on deflection at edge of slab with no void.

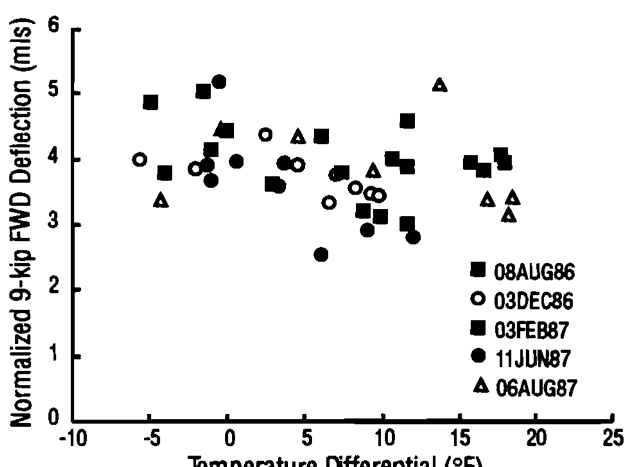


Fig 4.23. Effect of temperature differential variation on deflection at edge of slab with void.

support. Over the years, it has become evident that voids, even of the minutest depth, can contribute significantly to pavement distress as a result of this loss of support, which ultimately leads to the development of high stresses when the pavements are subjected to traffic loads. In fact, from the discussion above, it is clear that a loss of foundation support due to curling was responsible for most of the changes in the response of the research rigid pavement because of temperature variation. In any event, such high stresses lead to a reduction in fatigue life of the pavements, and consequently it is necessary to have methods for detecting the presence of voids that may exist under pavements so that adequate rehabilitation measures can be applied before the onset of severe pavement distress. A closer look is therefore taken in this section at the effect of a loss of foundation support, or voids, on FWD deflection measurements in an effort to determine if the FWD can be used in the detection of voids.

Figure 4.24 shows the influence of a void on pavement response in terms of the load/deflection ratio at the corners of the large slab of the research pavement for two cases typical of the results obtained. In the cold winter months, when the asphalt-stabilized base of the pavement was stiff, the effect of the void at the corner was evident, and the load/deflection ratio at the corner was higher at the corner with no void in comparison with the corner with a void. However, in the warm summer months there was no such clear distinction in the response of the pavement. Again it is suspected that a general softening-up of the asphalt-stabilized base material masked the effect of the void in the corner. This is backed by the higher stiffness recorded at both corners in the winter months in comparison with the summer months and is also illustrated by the example in Fig 4.24. Figures 4.25 and 4.26 illustrate further the influence of a void on the deflections at the corners of the large slab for the winter and summer seasons, respectively. From these results, it is clear that the FWD pavement response measurements on the research pavement were generally inadequate for detecting voids underneath the rigid pavement. Similar inconclusive results were obtained from comparisons between measurements at the edge of the pavement without a void and at the edge with a void.

SUMMARY

The influence of environmental effects on rigid pavements has been documented in this chapter. Temperature variations in PCC pavements have been shown to bring about temperature gradients which cause differential volumetric change and result in pavement curling. It was shown that, in Texas, the most curling occurs in the warm summer months, during which time large variations in temperature result in considerable pavement temperature differential variations. For the 10-inch-thick test pavement, temperature differential was found to range between -9.9 and 20.4 and was enough to cause maximum upward vertical displacements on the order of 30 to 40 mils and maximum downward vertical displacements on the order of 60 to 70 mils at the corners of the pavement. With such high displacements, the temperature differential was determined to be inversely related to FWD deflection measurements at the corners and edges of the pavement, with the pavement gaining more contact with the foundation as the temperature differential increased. A general softening of the asphalt-stabilized base of the pavement in the warm Texas summer months is suspected to be responsible for higher deflection measurements at all locations on the pavement obtained during the summer months in comparison with measurements during the winter months. From comparisons of FWD results for tests over locations with voids, there was some indication that FWD measurements can, to an extent, reveal the presence of voids underneath rigid pavements. However, for the thick research pavement on a stabilized base, the variation in pavement response due to a void was minimal and not very conclusive.

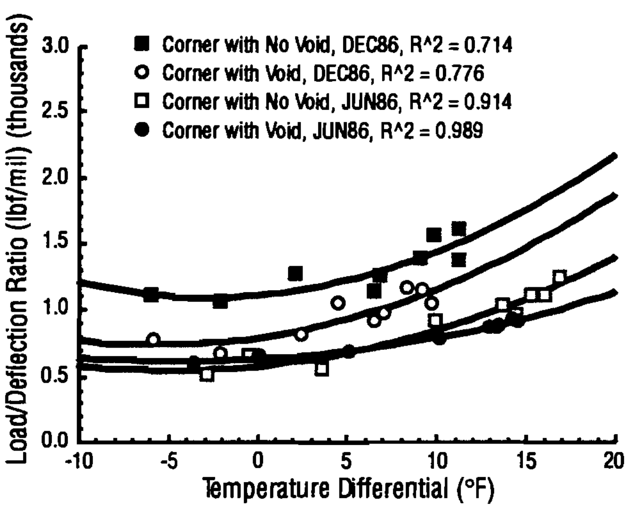


Fig 4.24. Typical effect of void on research pavement response.

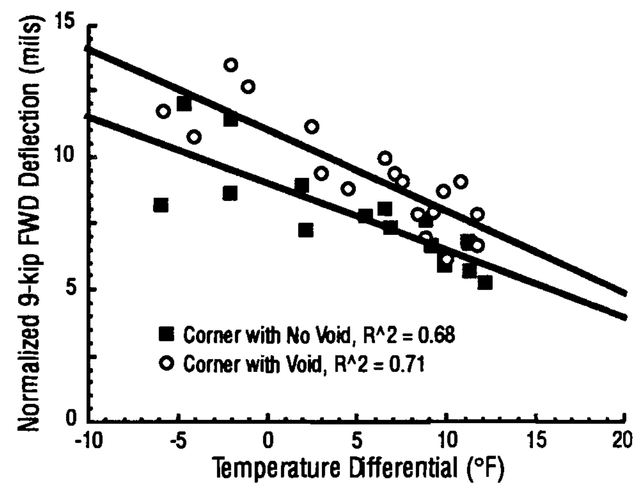


Fig 4.25. Effect of loss of support on deflection at corner of research pavement typical of the winter.

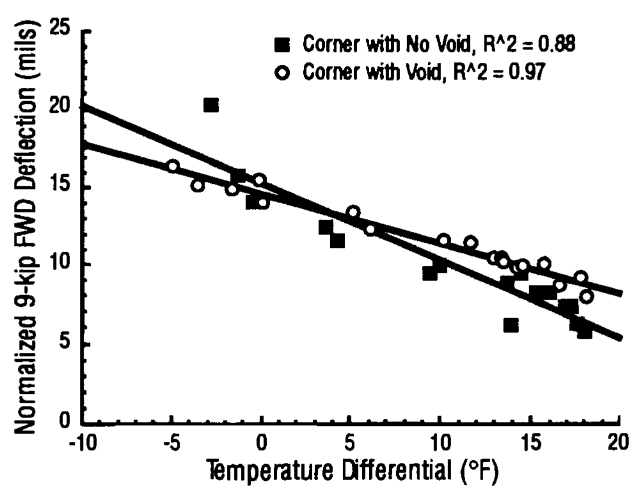


Fig 4.26. Effect of loss of support on deflection at corner of research pavement typical of the summer.

CHAPTER 5. ASSESSING LOAD TRANSFER EFFICIENCY ACROSS JOINT SYSTEMS IN RIGID PAVEMENTS WITH THE FWD

One of the main objectives of this study was to develop and verify procedures for assessing the efficiency of load transfer at the transverse and longitudinal joint systems which often exist in rigid pavements. As was pointed out earlier, such discontinuities represent a break in the structural continuity of the pavements and are the location of most of the distress in rigid pavements. Efficient load transfer across these discontinuities is therefore necessary to reduce the detrimental stress build-up at such locations when loads are applied in their vicinity. To ensure this occurrence, evaluation procedures are required to determine whether the joints and cracks in rigid pavements efficiently transfer load and whether stress levels are kept below acceptable limits during their design life. In this chapter the evaluation of such procedures for determining the efficiency of transverse joint systems is discussed. The effects of daily and seasonal environmental factors on the efficiency of joint systems and, consequently, on the load-carrying capacity of rigid pavements at such joints and cracks are also investigated.

LOAD TRANSFER EFFICIENCY AT JOINT SYSTEMS

The critical relationship of adequate load transfer across joints and cracks in rigid pavements to the performance of those pavements was discussed in some detail in Chapter 2. It was noted that poor load transfer at joints or cracks results in excessive stress and strain build-up which eventually lead to distress. Effective procedures are required to characterize the load transfer efficiency across the joints and cracks and to assist in performance evaluation of the joints and cracks. To this end, a number of load transfer determination procedures were evaluated, using the data collected in the laboratory research program described earlier, to determine their appropriateness for the effective characterization of the efficiency of joints and cracks in transferring applied loads.

CONCEPT OF LOAD TRANSFER EFFICIENCY

Most load transfer efficiency determination procedures are generally based on the following principle. Conceptually, for a particular pavement design, if the environmental, foundation support, and pavement material conditions are kept constant, a joint system will be considered fully efficient if it has the ability to transfer half of the load applied on one side of it across the joint such that the deflection or stress values are the same on the other side. Thus, methods based on a comparison of the deflection measurements or calculated stresses on either side are often adequate for the evaluation of the efficiency of joints and cracks. It should be noted that the joints and cracks in rigid pavements are by themselves not expected to provide any extra load-carrying capacity other than what would have been provided if no discontinuity existed. Evaluating load transfer efficiency then becomes a matter of determining whether the load transfer provided at a joint system is sufficient to give the appearance of no discontinuity in the pavement. With this in mind, and after an initial preliminary investigation of a number of methods founded on this principle, two methods based on FWD deflection measurements at a joint or crack were selected for use in this analysis.

PROCEDURES FOR LOAD TRANSFER EFFICIENCY DETERMINATION

Figure 5.1 illustrates the FWD load and deflection sensor arrangement used for load transfer efficiency evaluation at transverse joints and cracks and should be referenced in the following explanation of the load transfer efficiency procedures. The load transfer efficiency at a joint for each of the methods selected for this investigation is characterized by a joint deflection ratio (JDR) calculated as the average of an upstream deflection ratio (UDR) and a downstream deflection ratio (DDR). The difference between the methods is in the calculation of UDR and DDR.

In the first method, suggested by Ricci et al (Ref 3) and mentioned in Chapter 2, UDR and DDR are calculated as the ratio of the sensor 2 (S2) and sensor 3 (S3) deflections (Fig 5.1), with the largest always as the denominator, for the upstream and downstream load positions, respectively. This method is hereinafter referred to as procedure A. In the second method, UDR is calculated as the ratio of the sensor 2 (S2) deflection at the unloaded side of the joint over the sensor 1 (S1) deflection on the loaded side for the upstream load position. For the downstream load position, DDR is calculated as the ratio of the sensor 3 (S3) deflection on the unloaded side of the joint over the sensor 1 (S1) deflection on the loaded side of the joint. It is pointed out that the 11.81-inch-diameter loading plate and the 12-inch spacing between the deflection sensors of the FWD place sensors 1 and 2 in the upstream load position and sensors 1 and 3 in the downstream load position, approximately the same distance from the joint being evaluated. The second method is subsequently referred to as procedure B. These two procedures were used in the analysis of the data collected to determine an appropriate measure of load transfer efficiency at joints.

One of two other procedures also considered but dropped from consideration for different reasons is a procedure suggested by Westergaard (Ref 16) which defines the load transfer efficiency, LTE%, as follows:

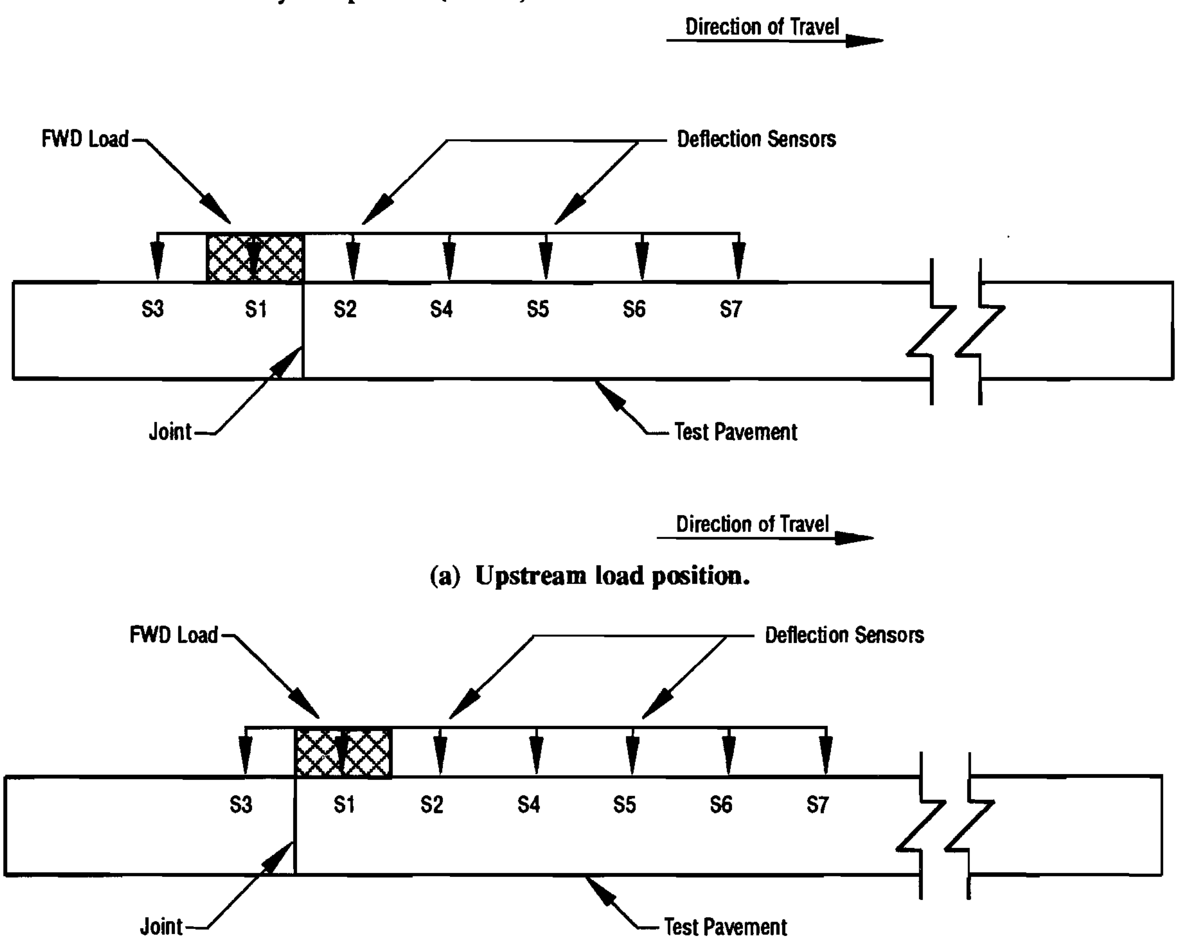
$$LTE\% = \left\{ 1 - \frac{d_{jl} - d_{ju}}{d_{zl} - d_{zu}} \right\} \times 100 \text{ percent} \tag{5.1}$$

where d_{jl} and d_{ju} are the deflections measured on the loaded and unloaded slab at the joint, and d_{zl} and d_{zu} are the corresponding deflections that would occur at the same locations if the joint had no capacity for load transfer. The difficulty in determining d_{zl} and d_{zu}—especially in the field, where there is always *some* load transfer, even when no load transfer devices are used at joints—made this procedure inadequate.

The second procedure dropped from consideration was based on a method used by Sharpe et al (Ref 29) for transverse joint evaluation of rigid pavements from road rater deflections. In this procedure, the load transfer efficiency, LTE%, is defined as follows:

$$LTE\% = \frac{(dL_m - dU_m)}{(dL_j - dU_j)} \times 100 \tag{5.2}$$

where dL_j and dU_j are, respectively, the deflections of the loaded and unloaded slab at equal distances from a joint or crack, and dL_m and dU_m are the corresponding deflections at a midslab load position measured by the sensors used for dL_j and dU_j, respectively. Unreasonable load transfer efficiencies were obtained using this procedure. Although the load transfer efficiencies ranged between zero and 100 percent, most of the calculated values, even in instances where near-perfect load transfer efficiencies were expected, were less than 50 percent.



(a) Upstream load position.

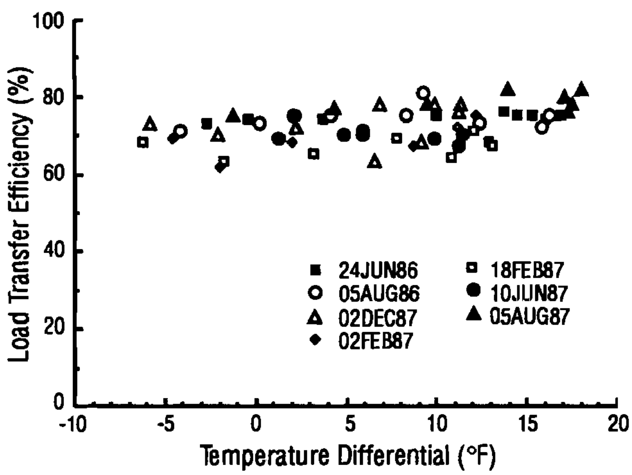
(b) Downstream load position.

Fig 5.1. FWD load plate and deflection sensor locations for joint load transfer efficiency evaluation.

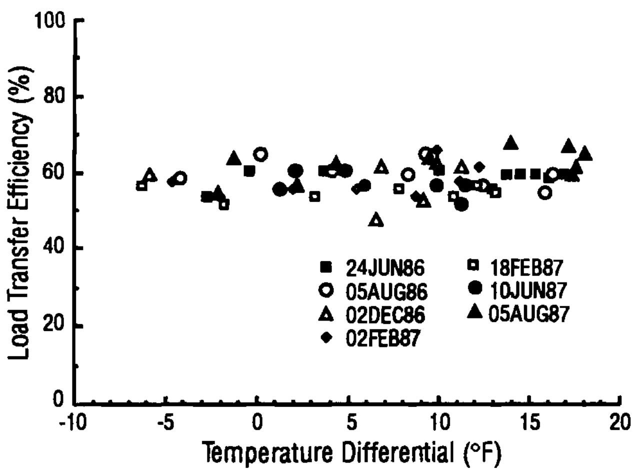
ANALYSIS OF LABORATORY LOAD TRANSFER EFFICIENCY STUDIES DATA

The first two procedures described in the preceding section were used in the analysis of the two major sets of data collected from the research pavement in the investigation on load transfer efficiency. As pointed out in Chapter 4, within the load range of operation of the FWD used in this study, there is less variability in measured FWD deflections for peak load levels of 9,000 lbs and over, with the variability decreasing somewhat with increasing peak load level. Not surprisingly, therefore, it was determined that the calculated load transfer efficiency ratios, especially for the last three drop heights corresponding to the average peak load levels of 9,000 lbs and over, were independent of the peak loads. As a result, for any particular test, UDR and DDR were calculated as the average of the three values determined separately for the FWD deflection results for the last three drop heights, and JDR was calculated as the mean of the averages.

As indicated in Chapter 3, two major types of tests were conducted on the research pavement in the laboratory test program for load transfer efficiency evaluation. The first group of tests consisted of FWD tests at the transverse joint of the pavement in its as-built condition, and the second group of tests was comprised of FWD tests at the joint for variable load transfer conditions obtained by pulling the two slabs apart. From the results obtained from these two tests, the deflection ratios characterizing load transfer efficiency at the transverse doweled joint of the research pavement at the void edge, midslab,

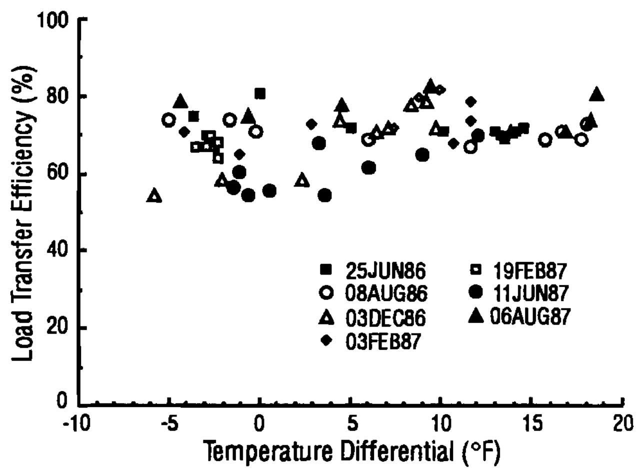


(a) Load transfer efficiency determined by procedure A.

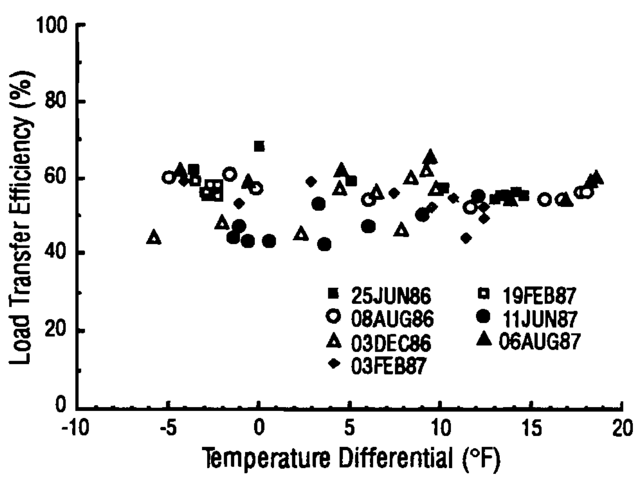


(b) Load transfer efficiency determined by procedure B.

Fig 5.2. Variation of load transfer efficiency at the transverse joint of research pavement at corner with no void.



(a) Load transfer efficiency determined by procedure A.



(b) Load transfer efficiency determined by procedure B.

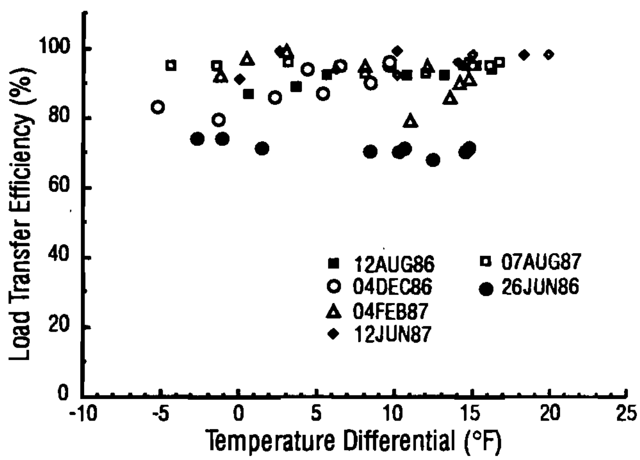
Fig 5.3. Variation of load transfer efficiency at the transverse joint of research pavement at corner with void.

and no-void edge test paths (Fig 3.4) were determined for various conditions using the statistical analysis computer software SAS. With the SAS software it was possible to directly read the data recorded on floppy diskettes by the FWD for data reduction. The results of the two groups of load transfer efficiency tests on the research pavement and their significance are discussed in the following sections.

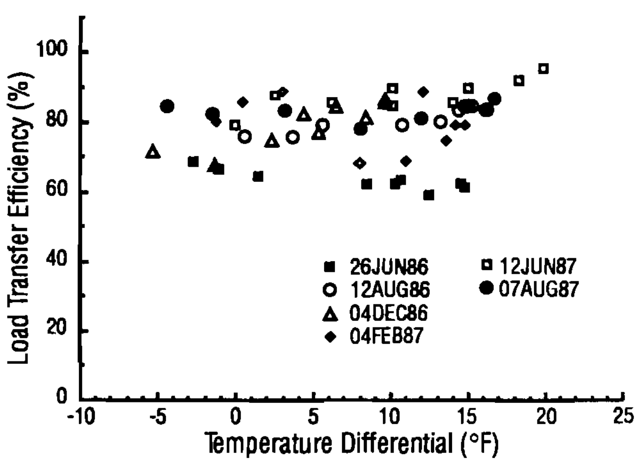
RESULTS OF CLOSED-JOINT STUDIES

The major objectives of the closed-joint studies were to determine how effectively the load transfer efficiency procedures quantified the load transfer condition at the doweled joint of this non-trafficked research JRCP and to investigate the effects of daily and seasonal changes on the load transfer efficiency. Under such controlled conditions, with no traffic on the pavement, the chance of the effects' of the environment being masked by other factors was considerably reduced. Figures 5.2, 5.3 and 5.4 illustrate the daily variation of load transfer efficiency at the joint of the research pavement with temperature differential for some typical days, at the corner with no void, at the corner with a void, and at the midspan of the joint, respectively. The load transfer efficiencies calculated by procedures A and B are shown in each case. It is evident from these graphs that there is no statistically significant trend in the load transfer efficiency at the doweled joint of the research pavement with the daily increase in temperature differential. In all cases the load transfer efficiency is virtually the same throughout the day. And, with regard to the different load transfer efficiency determination procedures, the only effect seems to be a general shift in the magnitude of the load transfer efficiency ratios, with procedure A giving higher values. A rough comparison of the load transfer efficiencies determined at the different times of the year seems also to indicate that there is no appreciable change in the load transfer efficiency at the joint from one season to the other. Figures 5.5, 5.6, and 5.7 further illustrate this characteristic of load transfer efficiency at the joint of the research pavement at the corner with no void, at the corner with a void, and at the midspan of the joint, respectively. It should be noted that the results presented so far are for load transfer efficiency conditions which would correspond to those at an ordinary tight joint or crack in the field with the provision of proper foundation support. It is evident that, under such conditions, adequate load transfer can be obtained throughout the year in a rigid pavement and that the influence of daily and seasonal environmental changes will be minimal.

In Figs 5.8, 5.9, and 5.10, typical load transfer efficiencies at the joint of the research pavement are presented together with the corresponding deflections measured on the large slab adjacent to the joint at the corner with no void, at the corner with a void, and at the midspan of the joint, respectively, to permit an investigation of the effect of load transfer efficiency on pavement response at the joint. The deflections presented are the normalized 9-kip deflections measured at the center of the load plate of the FWD. Except in a few instances, as was typical of the rest of the data collected, no correlation was found between the deflection at the joint of the research pavement and the load transfer efficiency, contrary to expectations. It is believed, again, that the conditions at the joint of the research pavement resulted in such good load transfer that the only influence on the deflection was the minimal effect of curling and warping of the pavement. In general, however, the results together show that there is an appreciable increase in load transfer at the midspan of the joint over the load transfer at the corners. Similar results were obtained in all the



(a) Load transfer efficiency determined by procedure A.



(b) Load transfer efficiency determined by procedure B.

Fig 5.4. Variation of load transfer efficiency at the transverse joint of research pavement at the midspan of the joint.

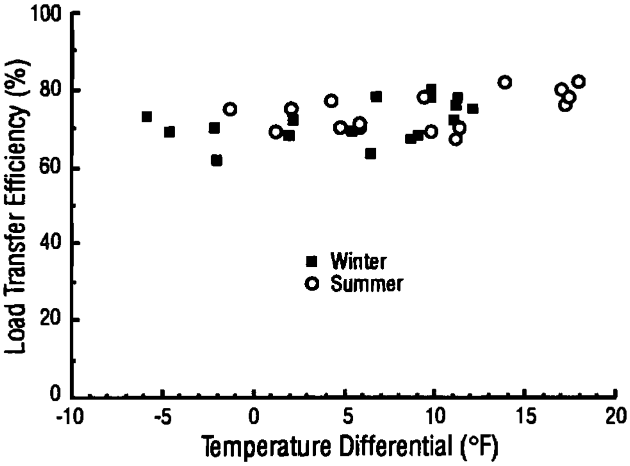


Fig 5.5. Effect of seasonal changes on joint load transfer efficiency at corner of research pavement with no void.

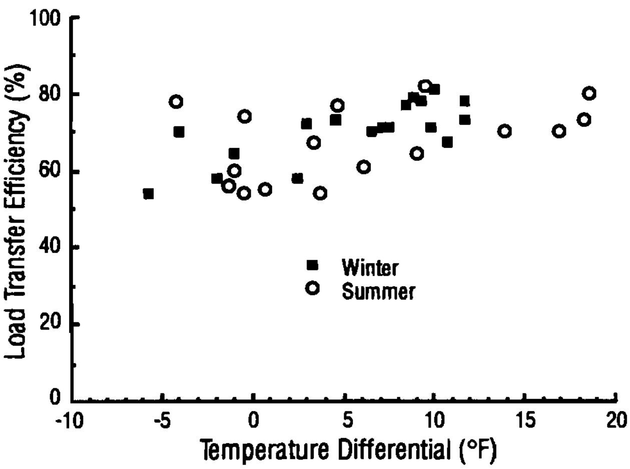


Fig 5.6. Effect of seasonal changes on joint load transfer efficiency at corner of research pavement with a void.

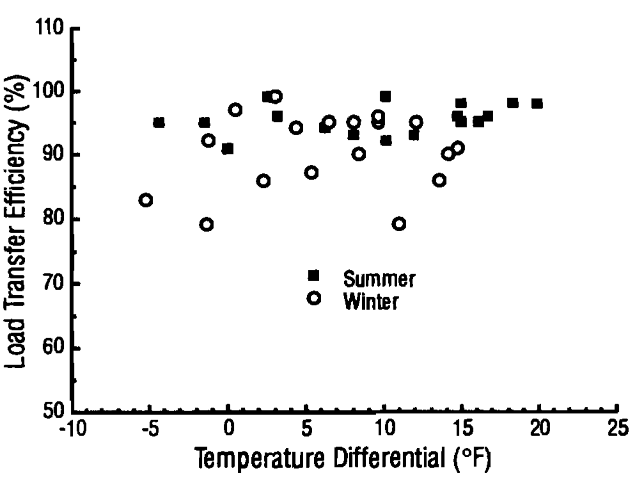


Fig 5.7. Effect of seasonal changes on joint load transfer efficiency at midspan of joint.

cases discussed above when procedure B was used in the determination of load transfer efficiency. For brevity, the results based on procedure B are not shown here.

RESULTS OF VARIABLE LOAD TRANSFER EFFICIENCY STUDIES

So far, the results presented for the FWD load transfer efficiency studies at the transverse joint of the research pavement indicate that, except for a few instances, the data are not very much in line with what has been observed in the field through a manifestation of certain pavement distresses associated with a loss in load transfer at joints and cracks. In general, minor changes were observed in the load transfer efficiency and the response of the pavement at the joint with changes in the environmental conditions. For example, the vertical pavement movements as high as 80 mils, recorded at both edges of

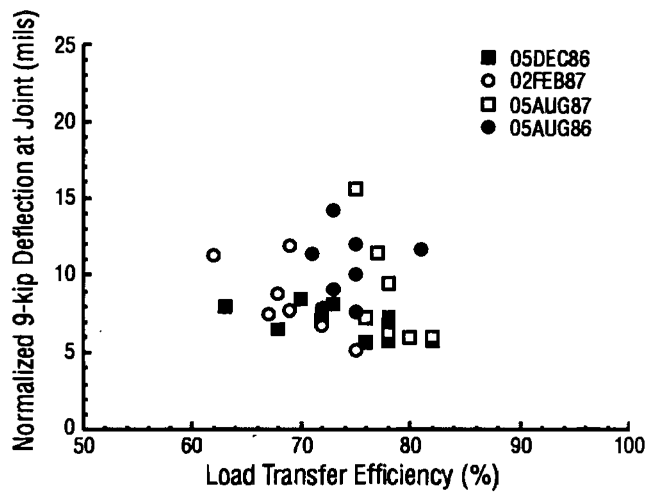


Fig 5.8. Relationship between load transfer efficiency and pavement response at joint of research pavement at corner with no void.

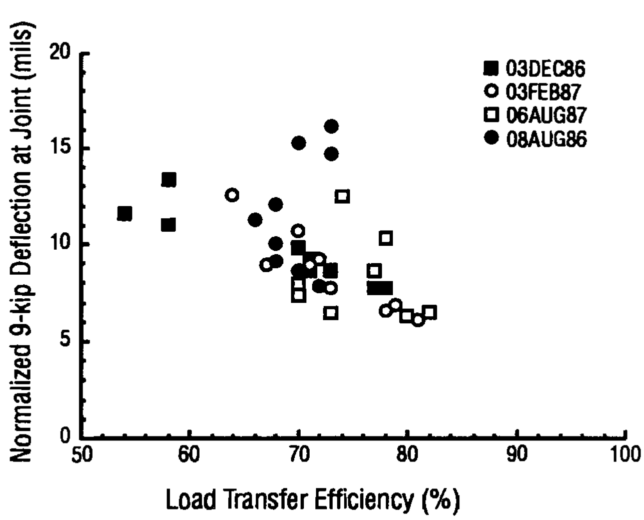


Fig 5.9. Relationship between load transfer efficiency and pavement response at joint of research pavement at corner with a void.

the research pavement in the curling tests, did not seem to have any noticeable effect on load transfer efficiency at the joint. As much as possible, this research pavement was designed as are the rigid pavements used in the field in Texas. It must be noted, however, that without any traffic loading, the pavement and the support provided by the foundation stayed almost intact throughout this study. By pulling apart the slabs of the pavement in constant increments from zero to 3/4 inch, however, the load transfer efficiency could be varied considerably. Results of such tests are presented in this section.

The variation of the load transfer efficiency at the joint with an increasing joint gap is illustrated in Fig 5.11 for various slab temperature differential ranges. Figure 5.11(a) shows the variation of the load transfer efficiency determined using procedure A, and Fig 5.11(b) is the corresponding graph for load transfer efficiency determined by procedure B. The two graphs represent the load transfer efficiency at the midspan of the transverse doweled joint. These two graphs illustrate the distinct change in load transfer efficiency at the doweled joint with increasing joint gap and show that deflection results from FWD tests can be effectively used to show this change. There is also strong evidence from the graphs of a change in load transfer efficiency at the joint with an increase in temperature differential. Clearly, for each joint gap, the efficiency of load transfer increases as the temperature differential increases. In this case, where the load transfer efficiency varies considerably, the effect of temperature curling and expansion of the pavement could be detected with the FWD. Again, as in the other tests discussed previously, both procedures A and B showed these trends in a similar manner. The load transfer efficiencies determined by procedure B were, however, of lower magnitude in all cases. The effect of a variation in load transfer efficiency, as a result of the increasing joint gap, directly on the response of the pavement in terms of FWD deflection at the midspan of the joint, is also shown in Fig 5.12. The relationship is shown for two slab temperature differential ranges to indicate also the minor but appreciable effect of the temperature differential on the deflection response of the pavement at the joint for a load transfer efficiency. Figure 5.13 shows a similar relationship between load transfer efficiency and pavement deflection for tests conducted at the joint of the research pavement on three consecutive days at the corner without a void, at the corner with a void, and at the midspan of the transverse joint, with the temperature differential ranging only between -0.6 and -5.3. In all cases the deflection at the joint decreased with an increase in load transfer efficiency. For the same load

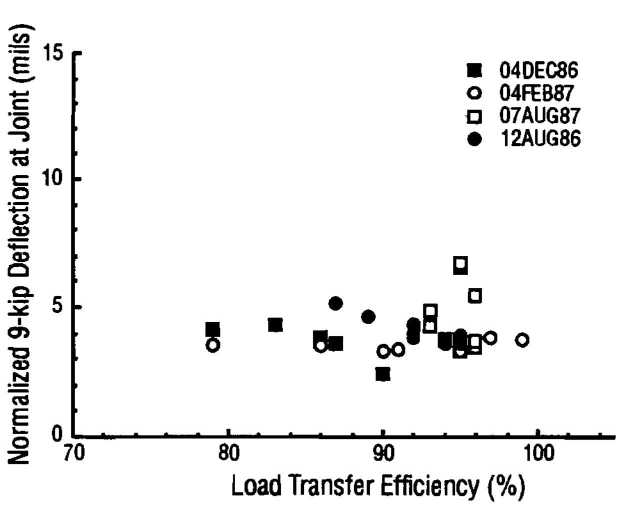
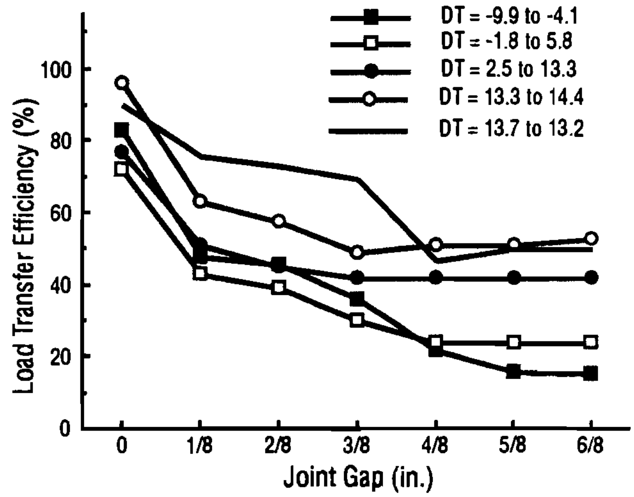
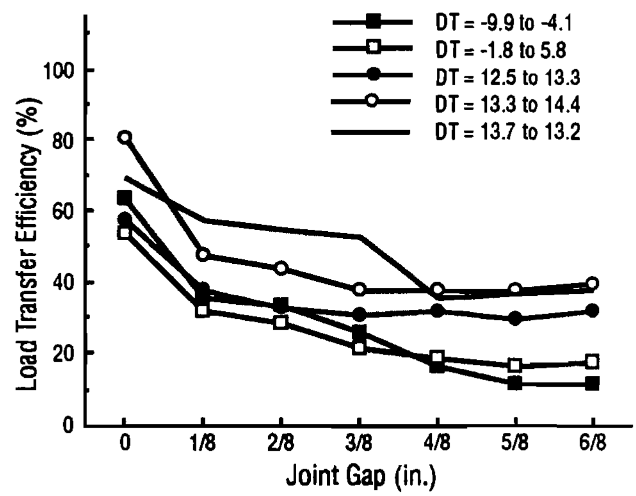


Fig 5.10. Relationship between load transfer efficiency and pavement response at joint of research pavement at midspan.



(a) Load transfer efficiency determined by procedure A.



(b) Load transfer efficiency determined by procedure B.

Fig 5.11. Typical effect of increasing joint gap and temperature differental on load transfer efficiency of transverse joint of research pavement.

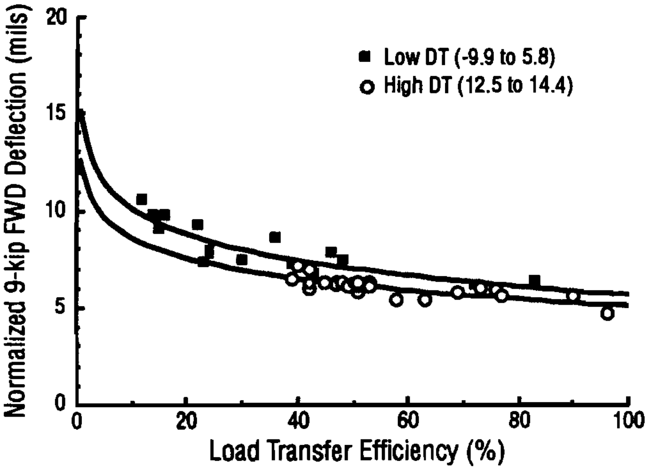


Fig 5.12. Influence of load transfer efficiency on pavement response at joint of research pavement.

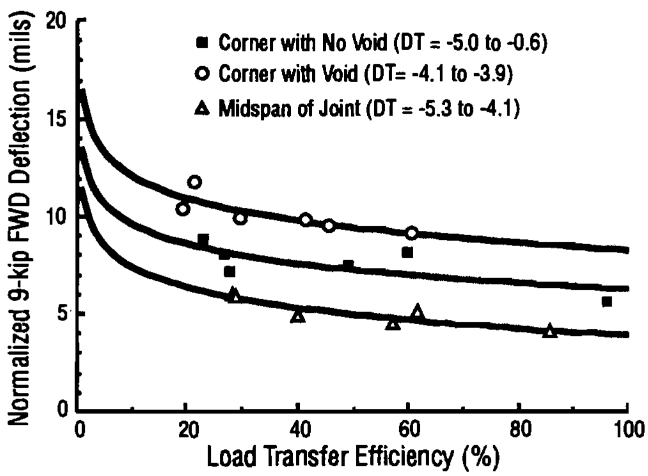


Fig 5.13. Influence of number of active dowels and loss of support on pavement response for equal joint load transfer efficiency.

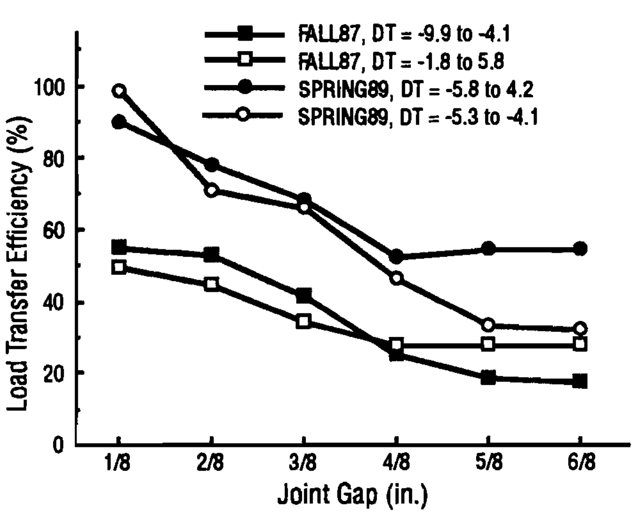


Fig 5.14. Effect of seasonal changes on load transfer efficiency of the joint.

transfer efficiency, higher pavement deflections were also observed at the corners than at the midspan location of the joint, and the deflections at the corner with a void were also higher than those at the corner with no void. A decrease in the number of active dowels at the corners and the effect of a loss of support, respectively, are believed to be responsible for these observations. This figure brings attention to the fact that, although a joint may be efficient in transferring load applied on one side to the other, resulting in equal slab deflections, proper pavement design is still necessary to keep actual pavement response values below acceptable levels. The effect of seasonal changes on load transfer efficiency is illustrated by Fig 5.14. The results, shown for the midspan of the joint, were true only for the load transfer efficiency at the lower temperature differentials, where pavement curling effects are minimal. At the higher positive temperature differentials, which are often associated with higher slab temperatures, slab expansion and larger curling effects masked these effects.

SUMMARY

From the results of the FWD tests on the research pavement for an increasing joint gap, it has been clearly shown that the device is capable of detecting the changes in load transfer efficiency at a joint system. It is also apparent that, to a large extent, any of the two procedures for determining the joint deflection ratios used to characterize load transfer efficiency can be used in the assessment of load transfer at a joint. In all cases the procedures showed similar trends, although the load transfer efficiency determined by procedure A was always of higher magnitude than the efficiency calculated from procedure B. Field evaluations are required to establish for each procedure the values of load transfer efficiency which correspond to certain standards. The data presented for FWD tests at the joint of the research pavement in its as-built condition did not, however, show the effects of factors such as rigid pavement curling, which are known to affect load transfer at a joint to a considerable extent. In general, relatively high load transfer efficiencies were obtained, which did not vary much with environmental variations. On the other hand, the effect of similar temperature differential variation was clearly discernible in tests carried out on the research pavement for increasing loss of load transfer efficiency. The effects of the change in load transfer efficiency on the response of the pavement at the joint were also documented.

CHAPTER 6. FIELD TEST PROGRAM AND DATA COLLECTION

In addition to the laboratory tests discussed in the previous chapters, field tests were conducted on in-service rigid pavements to further clarify and verify the findings made in the laboratory study, as well as for the purpose of additional investigations on rigid pavements that could not be conducted on the research pavement at BRC because of certain constraints. The studies were conducted on test sections selected from representative pavements consisting of both JRCP and CRCP. The tests on the JRCP were conducted for transverse joint and crack and for longitudinal joint evaluations, and those on the CRCP for transverse crack and longitudinal joint evaluations. The tests for longitudinal joint evaluation on the CRCP also included tests to study the beneficial effects of PCC shoulders attached to existing rigid pavements. In all, there were six pavement sections selected for the field tests. A complete description of each test section and the details of the field test program and data collected on each section are given in this chapter.

TEST PROGRAM

The scope of this study, as far as the field test program was concerned, was to conduct tests on in-service pavements to allow an evaluation of the load transfer efficiency determination procedures developed with data collected under controlled laboratory conditions. In addition, efforts were to be made to develop procedures for evaluating the beneficial effects of PCC shoulders attached to existing rigid pavements, on the basis of the results obtained from the laboratory study. To accomplish this objective, three JRCP and three CRCP test sections on some of the major highways in Texas were selected for testing with the FWD. Since lane closures are necessary during such testing, however, the test sections could be chosen only on highway sections already earmarked for closure by SDHPT for some major maintenance or rehabilitation work. Although this introduced some limitations, such as the inability to select sections at locations spread throughout the state, every effort was made to minimize the effects of such limitations on the results of the study. In all cases, except for longitudinal centerline and shoulder joint testing, the FWD tests were conducted in the outermost lane which, as the most travelled lane on all pavements, requires the most attention in the structural evaluation of pavements. The FWD load and sensor arrangements shown in Fig 2.3(a) were used for transverse joint and crack testing, and the configuration in Fig 2.3(b) was used for longitudinal joint testing. In the tests, FWD measurements at any particular location included load and deflection measurements for a drop of the 440-pound-weight set-up from each of four preset heights to give peak loads ranging from 5,000 to 20,000 pounds. Attempts were also made to collect supplementary data in the form of ambient temperature, solar radiation, pavement surface temperature, and an estimate of the slab temperature at various depths, during testing.

FIELD MEASUREMENTS ON JRCP TEST SECTIONS

The three JRCP test sections selected for FWD testing in the field program included one section on an Interstate highway and two sections on a U. S. highway in southeastern Texas. The test section on the Interstate system was on IH10, an important east-west link between most of the the major cities along the southern border of the United States, with an average daily traffic (ADT) of over 10,000. A section of this highway near Beaumont was selected for study. The two other test sections were a section near Columbus and another section near Beaumont, both on US90, which in most areas runs parallel to IH10 and is moderately travelled, with between 3,000 and 10,000 ADT. Following a description of each test section, a detailed account is given of the tests conducted on that section.

BEAUMONT JRCP TEST SECTION, IH10

This JRCP test section is on the eastbound lanes of IH10 between Houston and Beaumont in SDHPT District 20. Specifically, a 10-mile section of this Interstate approximately 20 miles southwest of Beaumont and between the intersections with State Highway (SH) 61 and Farm Road (FM) 1406 (Fig 6.1) in Chambers County was chosen for FWD testing. The section is made up of two 12-foot-wide travel lanes and a 10-foot flexible base shoulder. The original JRCP structure of the travel lanes was composed of a 12-inch-thick concrete layer and a 4-1/2-inch-thick flexible base consisting of a mixture of sand and shells on a 9-inch-thick lime-stabilized subbase, all over a compacted subgrade (Fig 6.1). This rigid pavement with a short 20-foot transverse joint spacing had been overlaid with 3 inches of asphalt concrete, but at the time of testing most of the rigid pavement joints had reflected through this overlay.

Within the 10-mile section, seven subsections, each approximately 500 feet long, were selected for testing. A visual condition survey was conducted of each subsection to permit its subjective classification into the three groups (good, fair, and poor), based on the observance of pavement distress, especially at the joints. The locations of these subsections and their conditions are also given in Table 6.1. Testing on this JRCP test section included FWD deflection measurements at the transverse joints in each of the subsections for load transfer efficiency evaluation of the joints. The number of joints tested in each section is also given in Table 6.1. Testing was conducted in the outer wheel path of the outer lane, at the upstream (U), downstream (D), and midspan (M) locations of each transverse joint, as shown in Fig 6.2, using the load and sensor arrangement illustrated in Fig 3.5. The ambient temperature made up the only environmental data collected on this test section and was obtained for all the FWD deflection measurements.

COLUMBUS JRCP TEST SECTION, US90

The second JRCP test section on which FWD measurements were conducted is a 400-foot section of US90 near Columbus (Fig 6.3). This section of US90, approximately 1,000 feet east of the intersection with Eagle Road in Columbus, is a 22-foot-wide two-lane two-way frontage road which runs parallel to the westbound lanes of IH10. This JRCP with unpaved gravel shoulders is comprised of a concrete layer with thickened edges, placed directly on the subgrade. The concrete layer is 6 inches

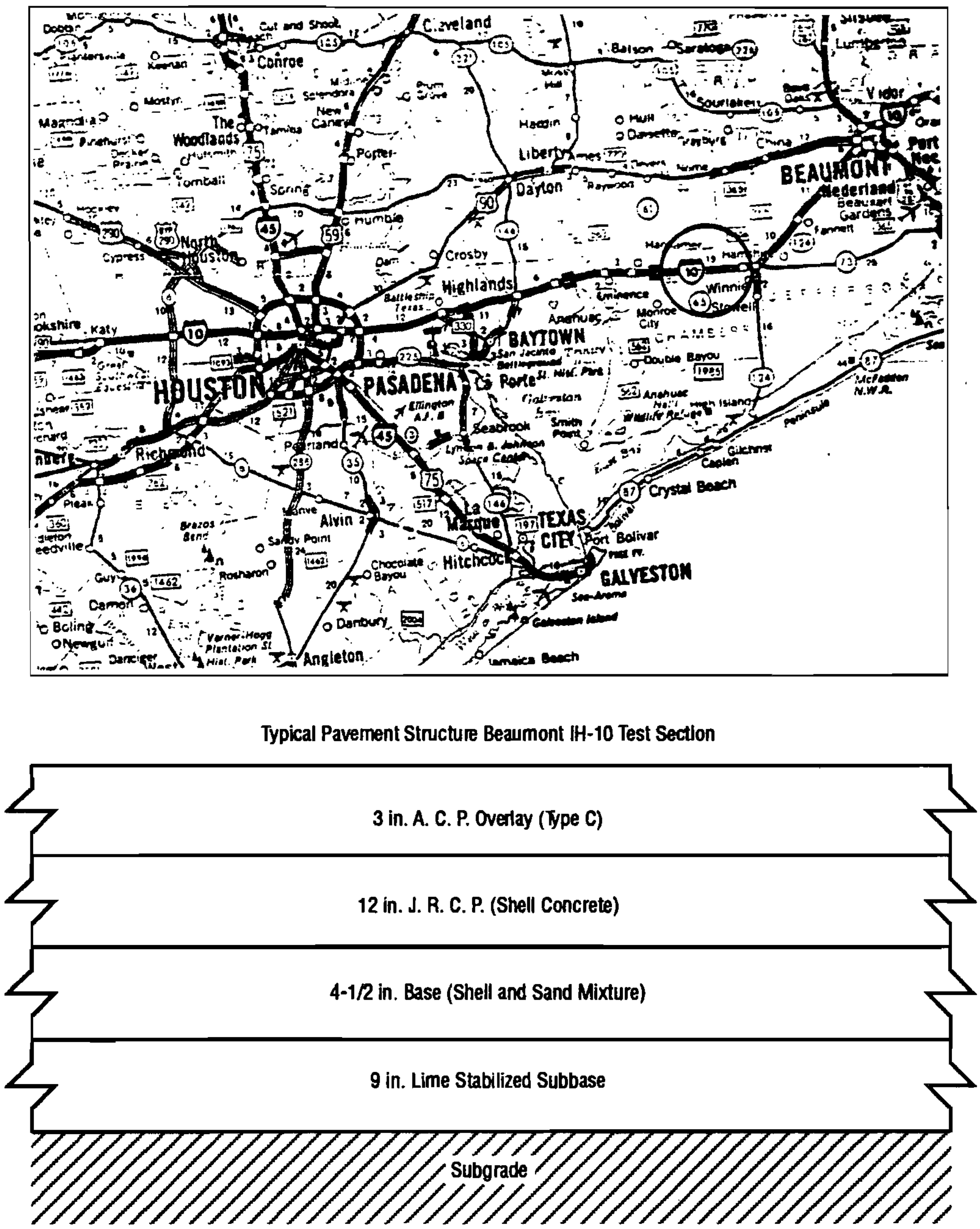


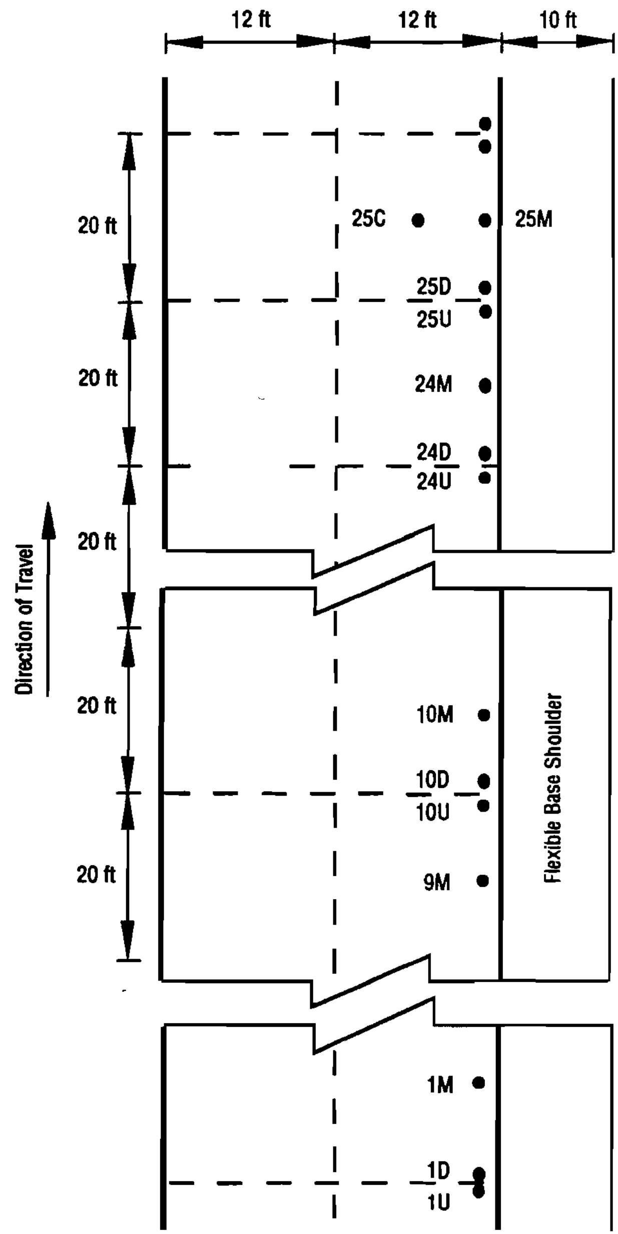
Fig 6.1. Test site location and pavement structure of JRCP test section on IH10 in Beaumont.

thick in the middle, 14 feet and tapering, starting 4 feet from each edge, to 8 inches at the edges, as shown in Fig 6.3. A longitudinal saw-cut joint divides the roadway into two equal 11-foot lanes. This JRCP with 20-foot joint spacing had also been overlaid with a 1-1/2-inch-thick asphaltic concrete layer, through which most of the joints had reflected at the time of testing.

The tests on this section were the first of the field tests conducted with the FWD load and sensor arrangement shown in Fig 2.3(b), used for the evaluation of longitudinal centerline and shoulder joints. Figure 6.4 illustrates how this configuration may be used to take FWD deflection measurements for shoulder joint evaluation. Testing on this JRCP test section included FWD deflection measurements along the two test paths adjacent to the longitudinal centerline joint and in the outer wheel path of the eastbound lane, as illustrated in Fig 6.5. Measurements were taken at twenty selected transverse joints at load positions upstream, downstream, and at midspan between two consecutive transverse joints. The FWD arrangement for longitudinal joint testing (Fig 6.4) was used for testing along the centerline joint, and the ordinary arrangement was used for testing in the wheel path for load transfer efficiency evaluation of the transverse joints. The ambient temperature was also recorded during testing at each location.

BEAUMONT JRCP TEST SECTION, US90

Another portion of US90 was the last JRCP field test section on which FWD tests were conducted for the study. The westbound section of US90 between IH10 and FM364, east of Beaumont Municipal Airport in Jefferson County, was the third JRCP field test section selected for testing. Figure 6.6 shows the specific location of this section, which is a medium-trafficked alternate roadway linking Beaumont and Houston. The pavement structure of the section is comprised of a uniform 10-inch-thick concrete layer, a 4-inch-thick cement-stabilized base on a 6-inch-thick lime-treated subbase, all over a compacted subgrade (Fig 6.6). A saw-cut longitudinal joint equally divides the 25-foot-wide one-way highway. The



Note: U = upstream loading;
D = downstream loading;
M = midspan loading

Fig 6.2. FWD test locations on JRCP subsection in Beaumont, IH10.

TABLE 6.1. TEST SITE LOCATION AND PAVEMENT STRUCTURE OF JRCP TEST SECTION ON IH10 IN BEAUMONT

Subsection	Number of Joints	Condition	Beginning of Section
1	25	Poor	+MP 813 +2,000 ft
2	24	Good	MP 814 +1,000 ft
3	24	Good	MP 814 +2,500 ft
4	11	Fair	MP 816 +2,500 ft
5	15	Poor	MP 817
6	24	Fair	MP 818
7	25	Good	MP 822

+MP = Mile Post

transverse joints of the original JRCP were spaced at 61-1/2-foot intervals and were doweled at an average spacing of 12 inches with No. 11 steel bars, 22 inches long and coated with oil-asphalt for smoothness. At the time of testing, transverse cracks had formed between the joints at intervals of between 25 and 30 feet. The unpaved shoulders of the highway were comprised of backfills in some sections and a cemented mixture of sand and shell in others.

Three subsections of the JRCP were chosen for testing, as illustrated in Fig 6.7. In each of these subsections, FWD load and deflection measurements were taken along four test paths made up of (1) the outer wheel path of the outer lane, (2) a test path in the middle of the outer lane, and (3) two test paths adjacent to, and one on each side of, the grooved longitudinal joint. Tests in these test paths were similar and consisted of FWD measurements upstream and downstream of the transverse joints and cracks and at midspan between each pair of joints and cracks. In two days of testing, FWD load and deflection measurements were taken in the outer wheel path and the middle of the outer lane the first day, and in the two test paths adjacent to the grooved longitudinal joint on the second day. The FWD load and sensor arrangement

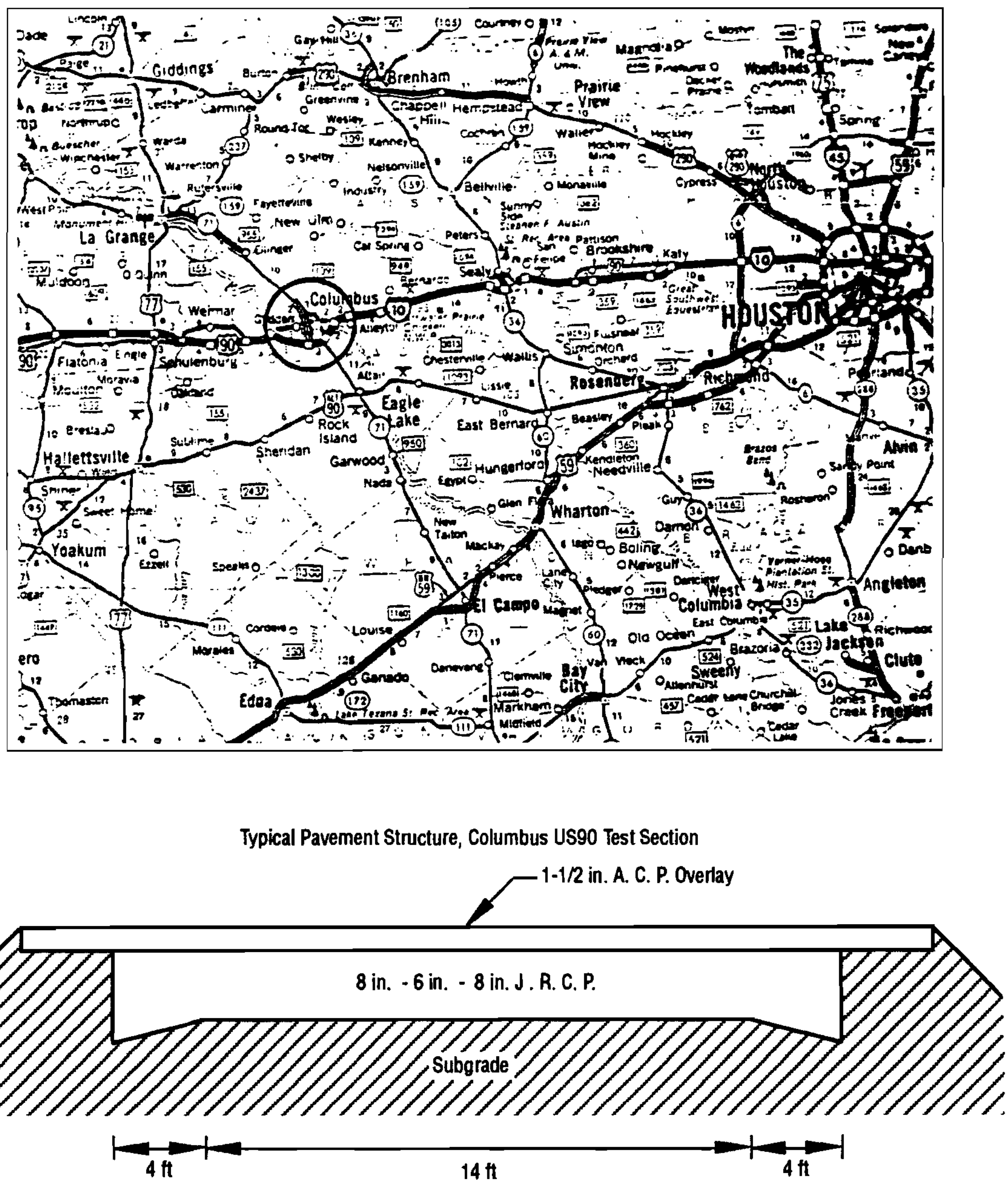


Fig 6.3. Test site location and pavement structure of JRCP section on US90 in Columbus.

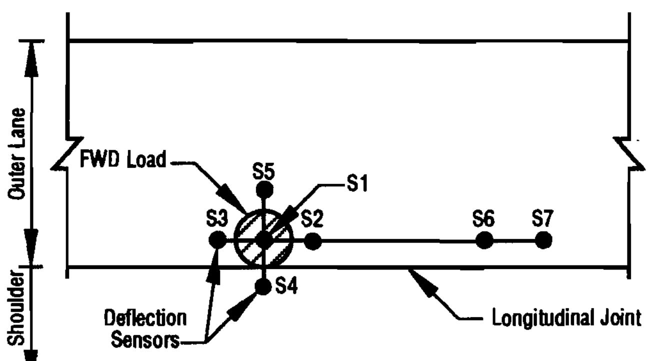
shown in Fig 2.3(a) was used for the tests on the first day, and the arrangement in Fig 2.3(b) was used for the tests on the second day along the longitudinal joint.

To monitor slab temperature variation in the pavement, thermistors were placed in 1-1/2-inch-diameter holes drilled into the pavement to depths of 1, 5, and 9 inches from the top of the 10-inch-thick concrete layer and grouted with cement paste. Prior to testing, the thermistors had been calibrated in the laboratory and a calibration curve of temperature versus resistance obtained. Using an ohmmeter, the resistances of the thermistors were monitored and used to estimate the temperature of the pavement slab from the curve. In addition, slab surface temperature measurements using the hand held infrared pyrometer (Omegascope, Model OS-2000A) and solar radiation measurements using the microprocessor controlled solar monitor (Licor, Model LI-1776) used in the laboratory study, as well as ambient temperature measurements, were taken throughout the tests.

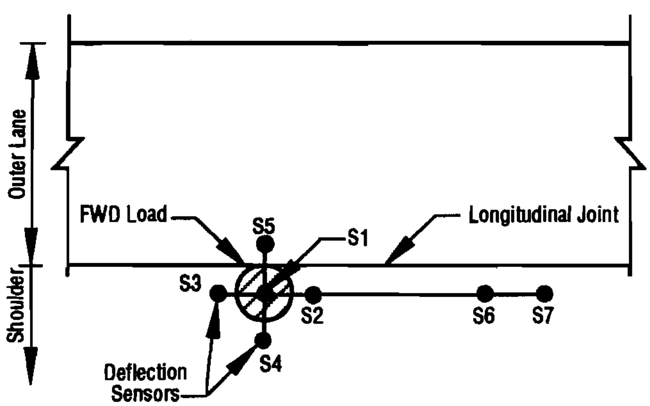
FIELD MEASUREMENTS ON CRCP TEST SECTIONS

Three CRCP test sections, one on a U. S. highway and two on Interstate highways, were chosen for study. These were a section of US59 near Victoria, moderately travelled with between 3,000 and 10,000 ADT; a section on the westbound lanes of IH20 near Weatherford; and a section near Flatonia on the westbound lanes of IH10. The CRCPs were all retrofitted with tied PCC shoulders to replace original flexible base shoulders and thus presented the opportunity to study the effect of PCC shoulders and other improvements on the rigid pavements.

In the tests on these sections, a hand-held digital thermometer (Fluke, Model 51) was used to measure the ambient temperature, and a hand-held solar meter (Dodge Products, Model 776) was used to measure solar radiation on the pavements. The portable infrared pyrometer (Omegascope, Model OS-2000A) used in the laboratory

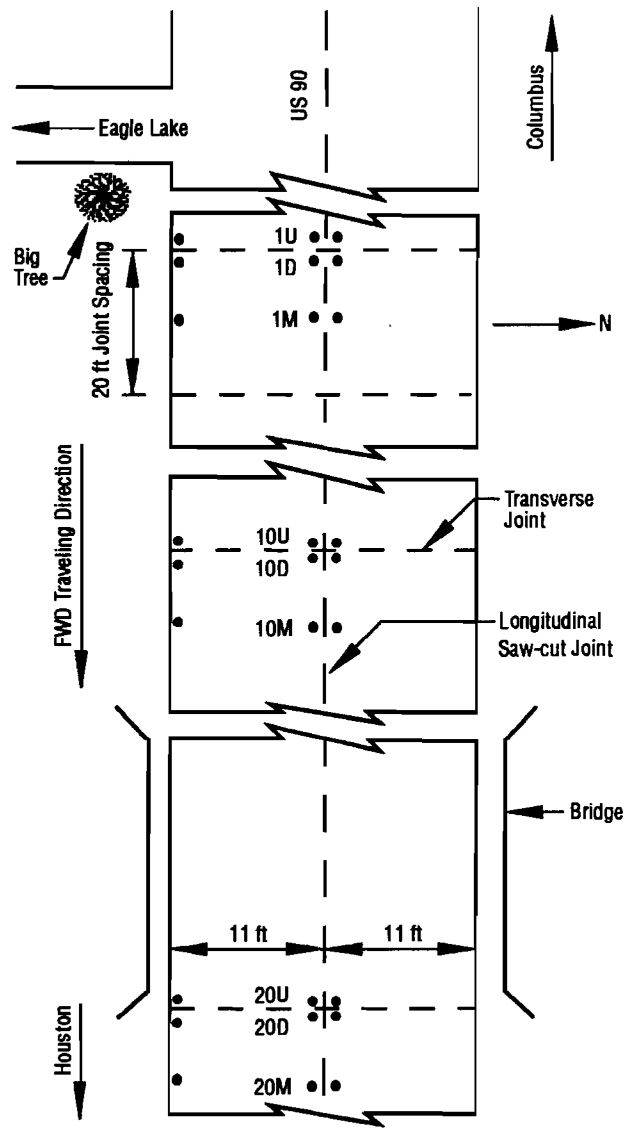


(a) Main lane pavement loading.



(b) Shoulder loading.

Fig 6.4. Arrangement of FWD load and sensors at a longitudinal shoulder joint for load transfer evaluation.



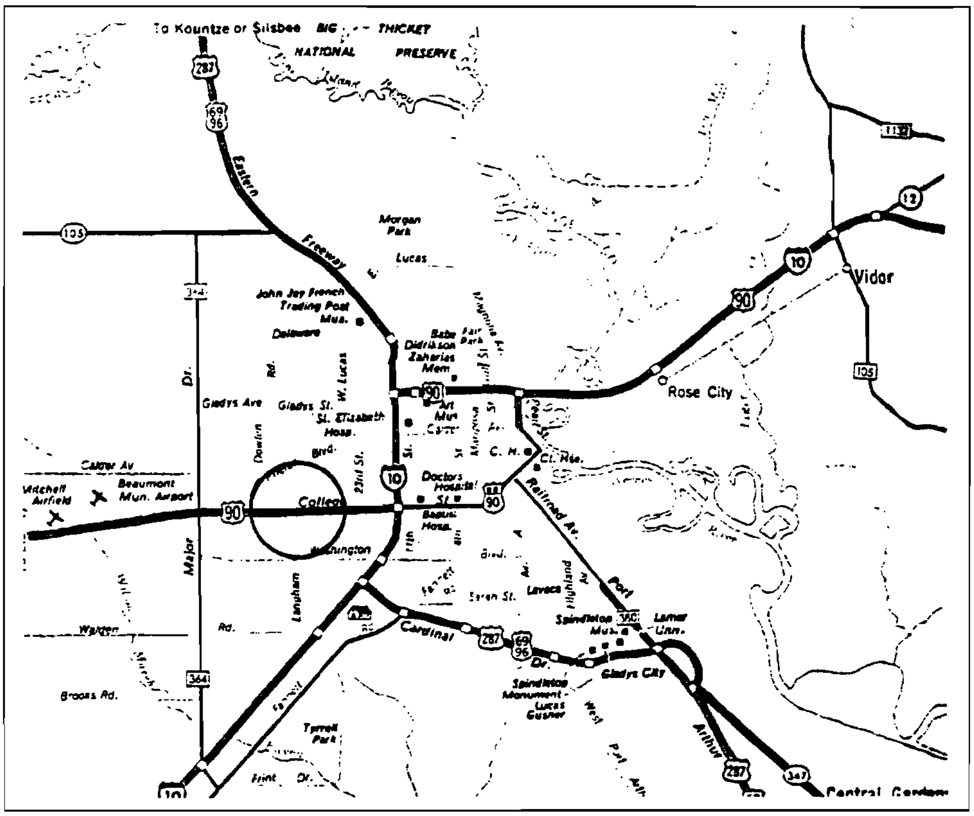
Note: U = upstream loading; D = downstream loading;
M = midspan loading

Fig 6.5. FWD test locations on JRCP section on US90 in Columbus.

tests was also used to take pavement surface temperature measurements. To estimate the temperature within the pavements, a 14-by-14-by-10-inch portable concrete block, with thermocouple wires embedded 1 inch from the top, at mid-depth, and 1 inch from the bottom, was used. This block was buried, at least a day before FWD testing, at the side of the test road and was kept buried for the duration of the tests. The thermocouple wires (type J, 20-gage, double-insulated wires), marked to distinguish between the wires from the different depths, were plugged into the Model 51 Fluke digital thermometer for measurement of the temperatures within the block at selected time intervals.

VICTORIA CRCP TEST SECTION, US59

This CRCP field test site is located some 12 miles northeast of Victoria (Fig 6.8) on a four-lane divided highway portion of US59 between Houston and Victoria. A 4,000-foot section of the two 12-foot-wide southbound lanes of this roadway, with a 10-foot flexible base shoulder, approximately 5,000 feet from the interchange at Inez, was selected for testing. The section is composed of an 8-inch-thick continuously reinforced concrete layer on 6 inches of cement-stabilized subbase over a clay subgrade (Fig 6.8). The flexible shoulder was replaced with an 8-inch-thick continuously reinforced PCC



Typical Pavement Structure Beaumont US90 Test Section

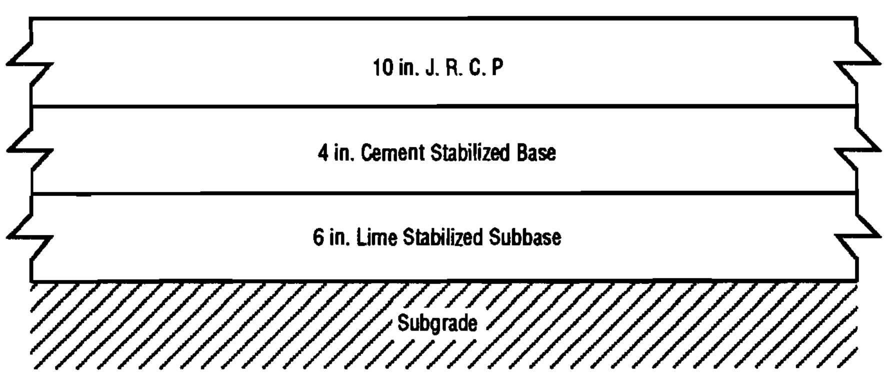


Fig 6.6. Test site location and pavement structure of JRCP section on US90 in Beaumont.

shoulder tied to the main pavement with No. 5 deformed steel tie bars, 25 inches long, at 48-inch spacings. In between the tie bars, No. 8 steel bars, 6 inches long, at 12-inch spacings, were used to dowel the shoulder joint. To achieve this, holes were drilled into the side of the existing pavement after the removal of the flexible shoulder and half lengths of the tie bars and dowels grouted in before the concrete shoulder was placed. (This method of tying PCC shoulders was used on all the CRCP.)

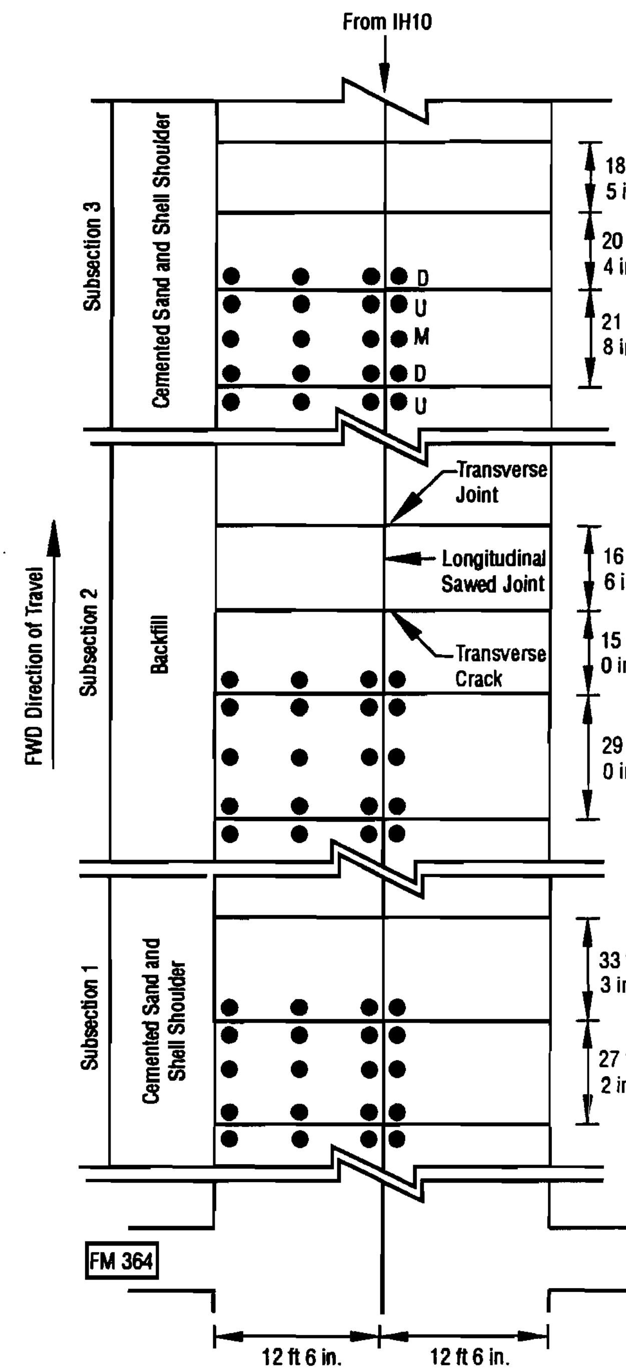
On this section, measurements were taken on the CRCP for the three cases of the pavement with a flexible shoulder, without a shoulder, and with a PCC shoulder. Figure 6.9 shows the layout of the test section, test paths, and test locations at which FWD measurements were taken. The test paths were made up of (1) two test paths adjacent to, and one on each side of, the longitudinal centerline joint; (2) a test path in the middle of the outer lane; (3) a test path in the outer wheel path of the outer lane; and (4) two test paths adjacent to, and one on each side of, the longitudinal shoulder joint. During each phase of testing, data were collected twice along each test path, once in the morning and once in the afternoon, to permit comparisons to show the effect of environmental variations.

In the first phase of testing on the CRCP with a flexible shoulder, FWD load and deflection measurements were taken along three of the test paths shown in Fig 6.9. The load and sensor arrangement shown in Fig 2.3(a) was used to take measurements (1) in the middle of the outer lane at 400-foot intervals; (2) in the outer wheel path of the outer lane at 200-foot intervals; (3) in the outer wheel path of the outer lane at ten selected cracks, at load positions upstream, downstream, and at midspan between consecutive cracks; and (4) at the edge of the outer lane at 200-foot intervals. The FWD arrangement shown in Fig 2.3(b) was also used for load and deflection measurements at the edge of the outer lane at 200-foot intervals.

In the second phase of testing on the pavement without a shoulder, FWD load and deflection measurements using the load and sensor arrangement in Fig 2.3(a) were taken (1) in the middle of the outer lane at 400-foot intervals; (2) in the outer wheel path of the outer lane at 200-foot intervals; and (3) in the outer wheel path of the outer lane at the ten selected cracks, for load positions upstream, downstream, and at midspan of the cracks.

The third phase of testing was conducted shortly after curing of the newly-constructed rigid shoulder, and included six sets of FWD load and deflection measurements along four test paths. The first five measurements were a repeat of the tests conducted in the first phase of testing; the sixth consisted of load and deflection measurements at 200-foot intervals along the shoulder joint, with loading on the rigid shoulder using the load and sensor arrangement for longitudinal joint evaluation. Another set of FWD load and deflection measurements was also taken at a later date on this CRCP section along the longitudinal joint separating the two lanes for an evaluation of centerline joints. Measurements were taken in the two test paths parallel to and one on either side of the longitudinal joint (Fig 6.9).

Thus, in all, six sets of data were collected. Four consisted of measurements using both kinds of load and



Note: U = upstream loading; D = downstream loading; M = midspan loading

Fig 6.7. FWD test locations on JRCP section on US90 in Beaumont.

sensor arrangement in the two test paths at 200-foot intervals. With the load and sensor arrangement for transverse joint evaluation, FWD load and deflection measurements were also taken at nine selected transverse cracks in both test paths, at upstream, downstream, and at midspan load positions at each crack for the other two data sets. Supplementary data in the form of ambient temperature, solar radiation, pavement surface temperature, and an estimate of the pavement slab temperature at various depths were collected during all testing.

WEATHERFORD CRCP TEST SECTION, IH20

The CRCP section selected on IH20 for joint and crack evaluation studies is a 9,500-foot section of the westbound lanes of IH20 approximately 2 miles southeast of Weatherford, ending at the State Highway (SH) 171 exit ramp (Fig 6.10). This was part of a pavement fitted with a PCC shoulder to replace an existing flexible base shoulder in a state highway improvement (Federal-Aid) project. The original highway, made up of two 12-foot-wide lanes and a 10-foot-wide flexible shoulder, was composed of an 8-inch-thick reinforced concrete layer on a 4-inch-thick asphalt-stabilized base placed in two equal lifts, over 6 inches of lime-treated subbase placed on a

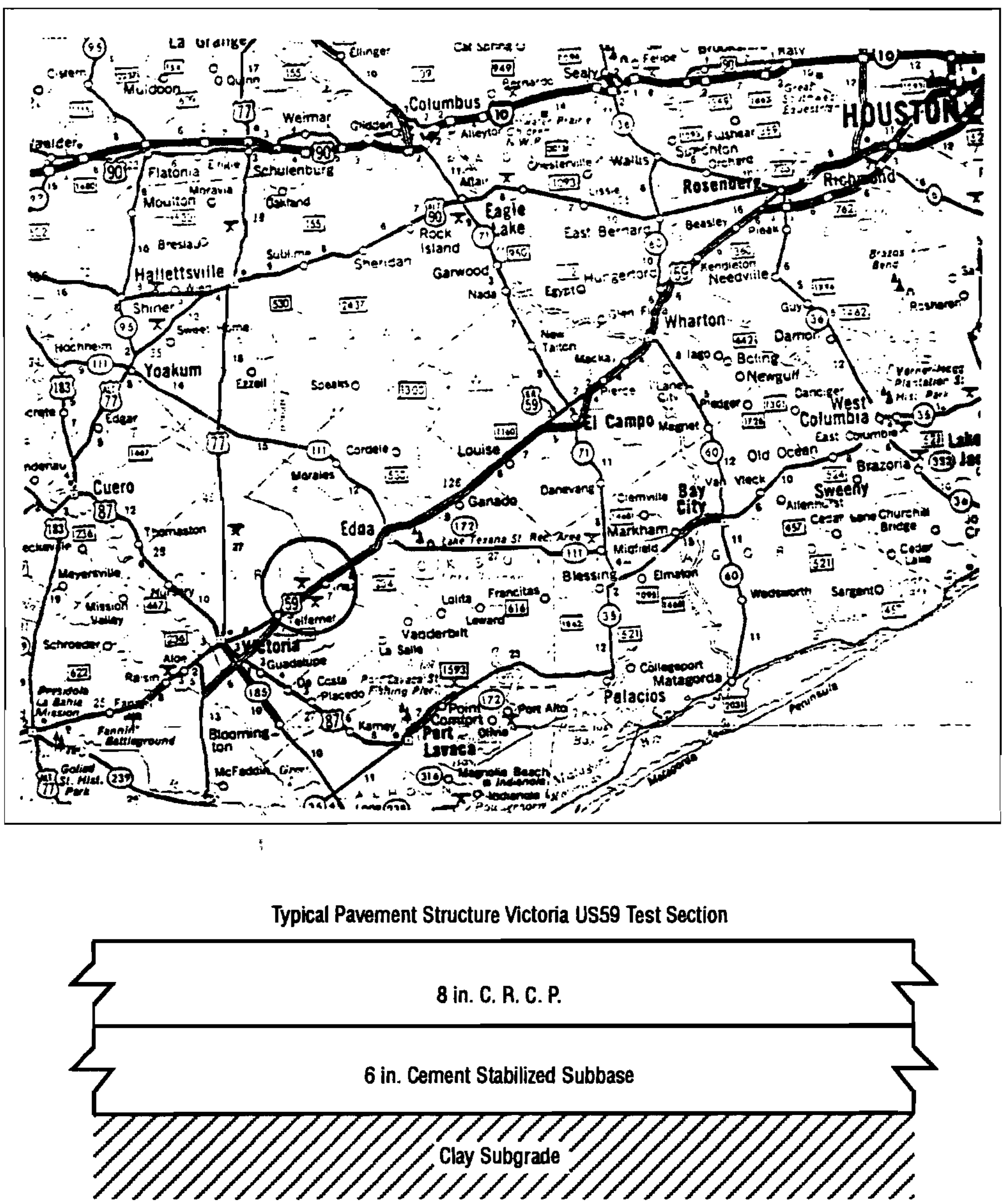


Fig 6.8. Test site location and pavement structure of CRCP section on US59 in Victoria.

subgrade, as illustrated in Fig 6.10. The flexible shoulder was partially removed and replaced with a continuously reinforced PCC shoulder approximately 8 inches thick over a remaining underlayer of flexible base about 4 inches thick. Three different types of rigid shoulder joints were used, providing an opportunity to evaluate these specific types of shoulder joints. The three construction joint types used are illustrated in Fig 6.11. Of particular interest are the construction joint types B and C, called "inverted-tee" joints. Tie bars were also used in section C. Figure 6.12 shows the general layout of the test site and the location of the joint types, as well as the test paths along which testing was conducted on the CRCP test section. The test section is divided into subsections corresponding to the type of construction joint used.

Testing on this test section was comprised of tests on the pavement with the flexible shoulder and tests after construction of the PCC shoulder. In the first phase of testing, FWD load and deflection measurements were taken in each of the three subsections: (1) in the middle of the outer lane at 200-foot intervals for the first 1,000 feet; (2) in the outer wheel path of the outer lane at six selected cracks, for upstream, downstream, and midspan load positions at each crack; and (3) at the edge of the outer lane at ten selected locations. These test paths are illustrated in Fig 6.12. In the second phase of testing, the

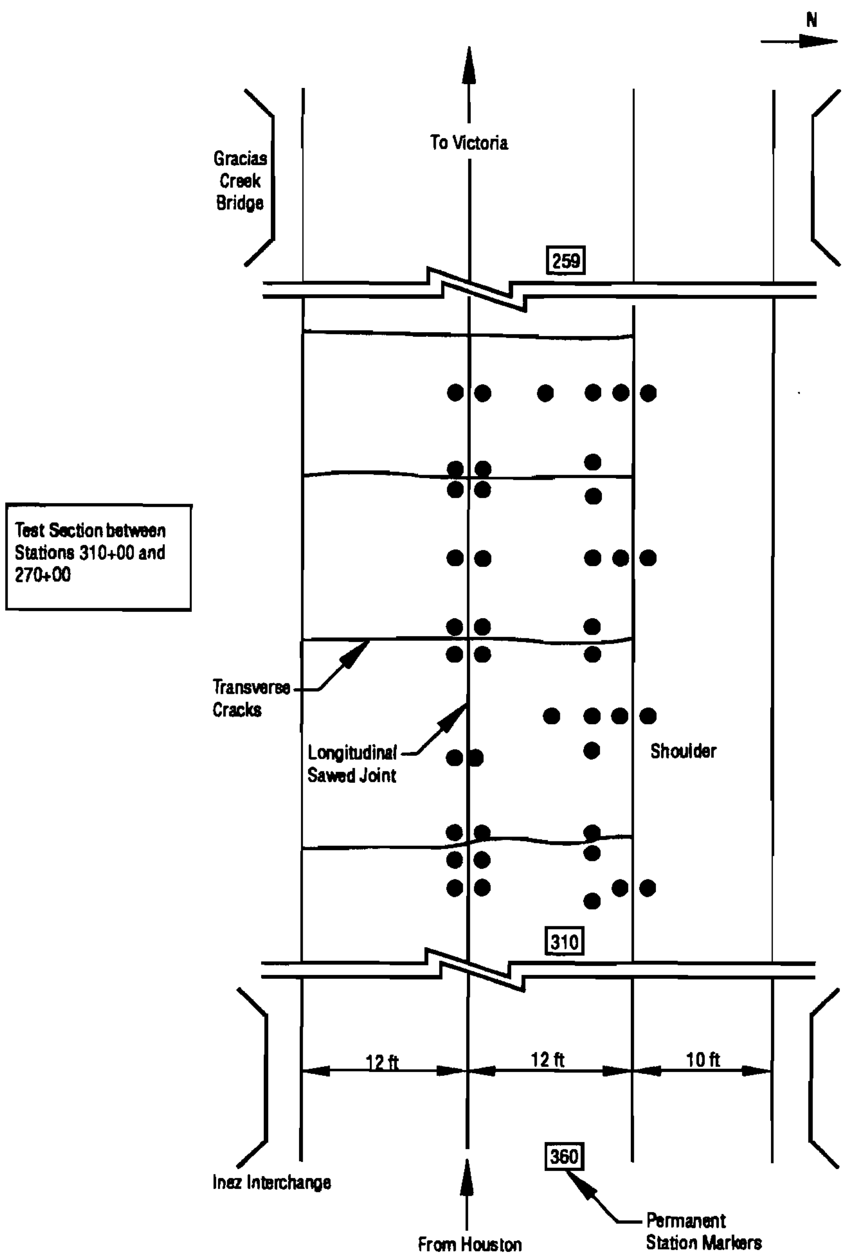


Fig 6.9. FWD test locations on CRCP section on US59 in Victoria.

tests were repeated at the same locations with the rigid shoulder in place. In addition, a fourth group of FWD load and deflection measurements was taken along the longitudinal shoulder joint at the ten selected locations in each subsection with loading on the shoulder (Fig 6.4). Ambient temperature, solar radiation, pavement surface temperature, and estimated slab temperature data were collected, using the equipment and instruments previously described, during the FWD testing in both phases.

FLATONIA CRCP TEST SECTION, IH10

A 1,400-foot section of IH10 near Flatonia was the third CRCP test section selected for FWD testing. This section on the westbound portion of the Interstate, between US77 at Schulenburg and FM609 at Flatonia in Fayette County (Fig 6.13), was made up of a 24-foot-wide two-lane pavement with a 10-foot-wide flexible lime-treated base shoulder. A grooved longitudinal joint separates the two lanes. The pavement, composed of an 8-inch-thick reinforced concrete layer on a 6-inch-thick cement-stabilized subbase over a compacted subgrade, had its flexible base shoulder replaced with a reinforced PCC shoulder 8 inches thick. As in the two previous cases, this section provided the opportunity to take FWD measurements on the CRCP for different shoulder conditions. On this test section, which was part of an 11-mile

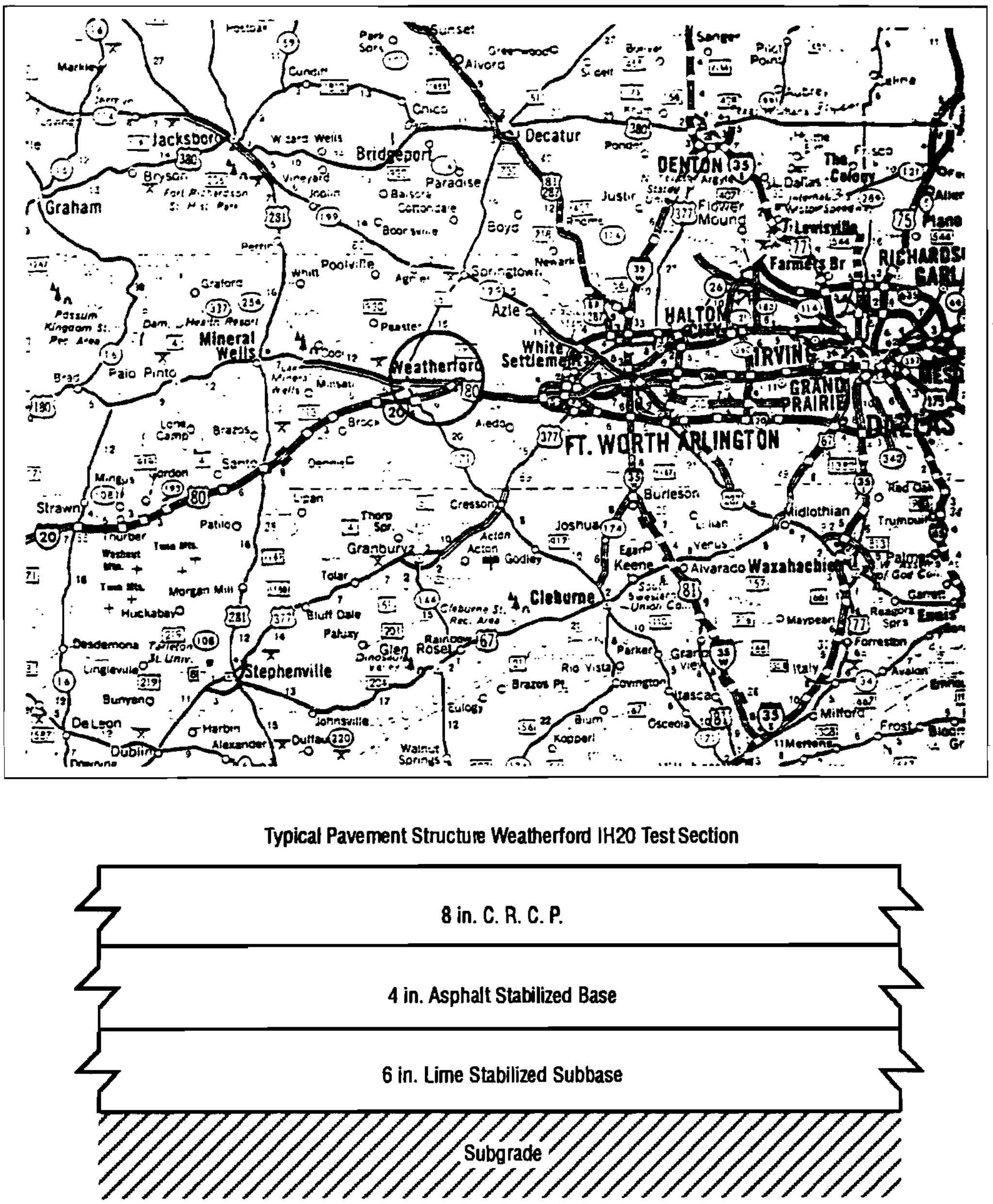


Fig 6.10. Test site location and pavement structure of CRCP section on IH20 in Weatherford.

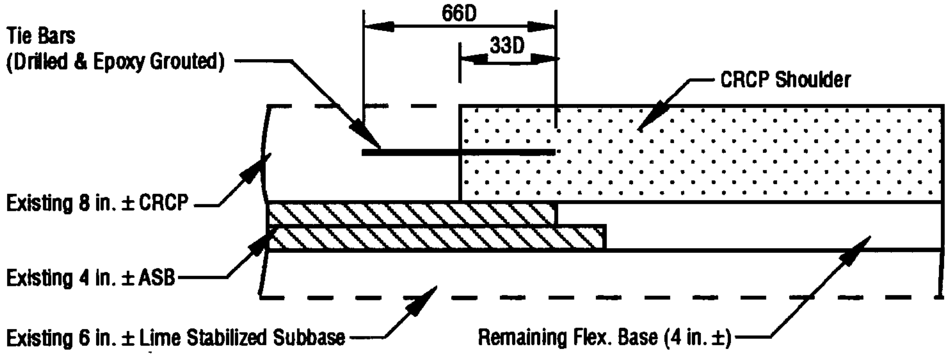
highway improvement project, different tie bar sizes, lengths, and spacings were also used to tie the rigid shoulder to the existing pavement. This provided yet another opportunity for an evaluation of the effects of such factors on longitudinal shoulder joints. As illustrated in Table 6.2, the test section was divided into fourteen 100-foot subsections with different combinations of tie bar size, length, and spacing.

Two phases of testing were conducted on each of these subsections. The first phase of testing, conducted on the existing pavement with a flexible shoulder, consisted of FWD load and deflection measurements along three of the test paths shown in Fig 6.14. The measurements were taken in the test paths (1) in the middle of the outer lane of the pavement; (2) in the outer wheel path of the outer lane at two selected cracks, for load positions upstream, downstream, and at midspan between consecutive cracks; and (3) at the edge of, and with loading on, the outer lane. Following construction of the rigid shoulder, the second phase of testing was conducted and included a repetition of all the tests conducted in phase one. In addition, FWD load and deflection measurements were taken on the test path along the shoulder joint at the midspan load positions between the cracks, with loading on the shoulder (Fig 6.4). Ambient temperature, direct solar radiation, pavement surface temperature, and estimated slab temperature data were obtained using the equipment used on the other CRCP sections.

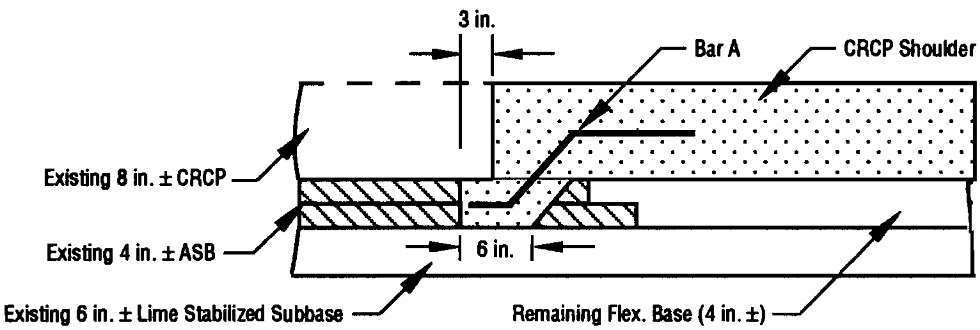
TABLE 6.2. TIE BAR SIZES, LENGTHS, AND SPACINGS FOR JRCP TEST SECTION ON IH10 IN FLATONIA

Subsection	Bar Size	Bar Length (in.)	Bar Spacing (in.)
1[a]	#5	25	24
2	#4	15	24
3	#5	15	24
4	#4	25	24
5	#4	15	36
6	#5	15	36
7	#4	25	36
8	#5	25	36
9	#4	15	48
10	#5	15	48
11	#4	25	48
12	#5	25	48
13[b]	#4	30	30
14[a]	#5	25	24

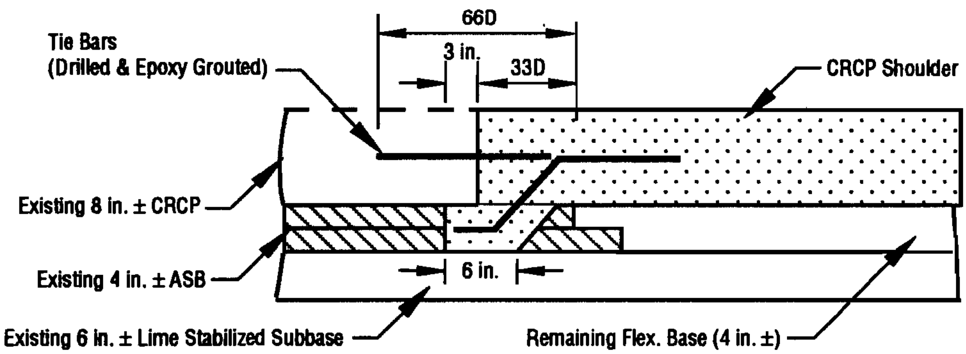
[a] Standard design for shoulder joint.
[b] Same design as center line joint of existing pavement.



(a) Joint detail A.

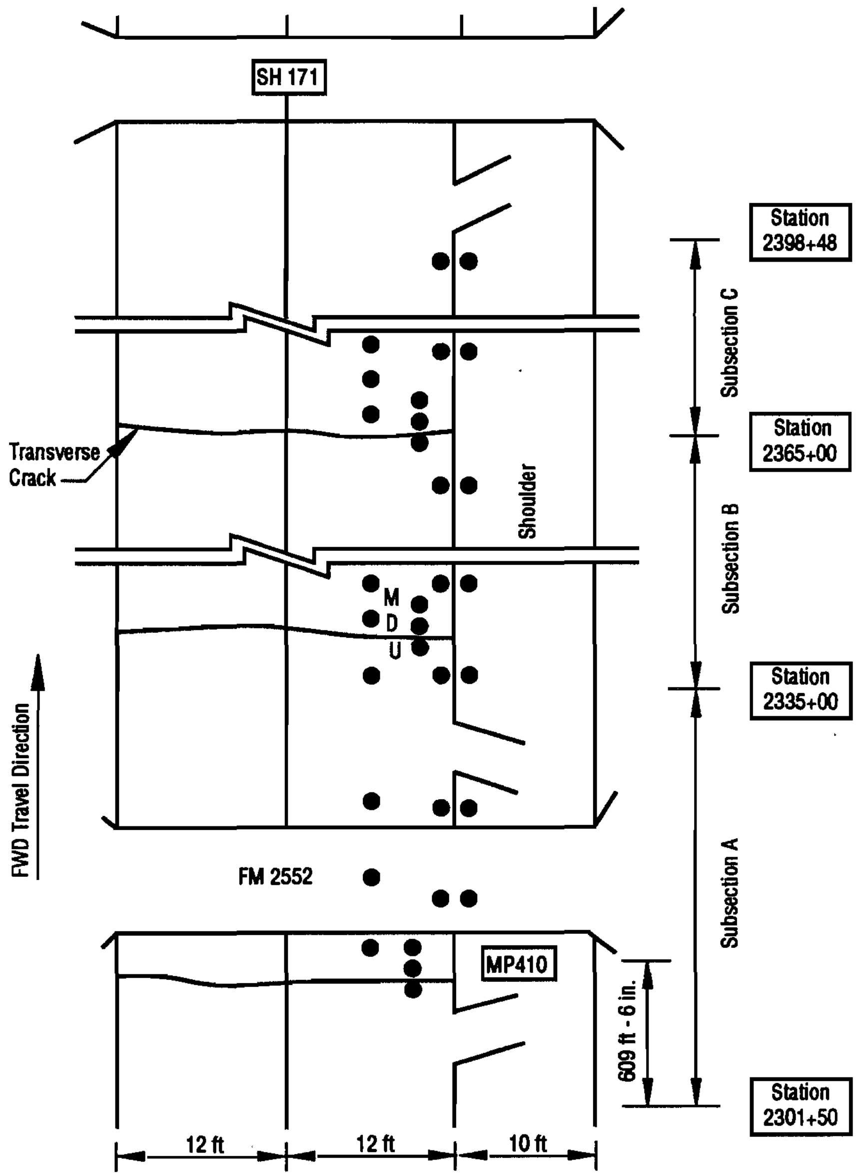


(b) Joint detail B.



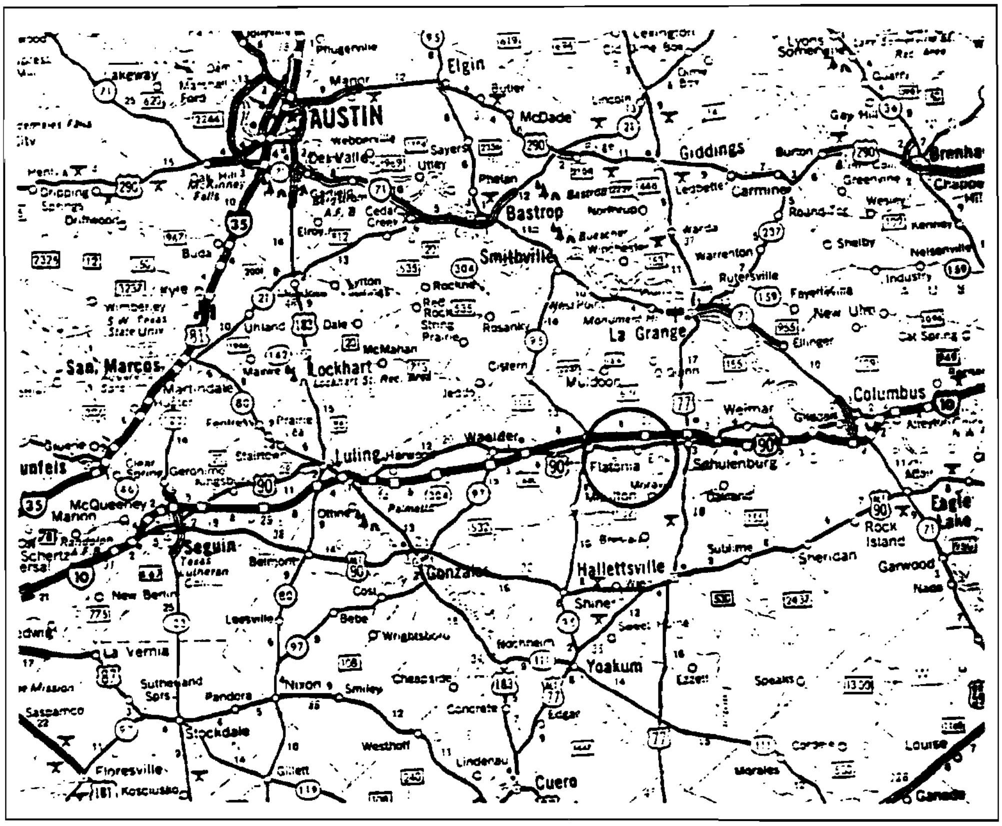
(c) Joint detail C.

Fig 6.11. Weatherford CRCP longitudinal shoulder joint details.



Note: U = upstream loading; D = downstream loading; M = midspan loading

Fig 6.12. FWD test locations on CRCP section on IH20 in Weatherford.



Typical Pavement Structure Flatonia IH10 Test Section

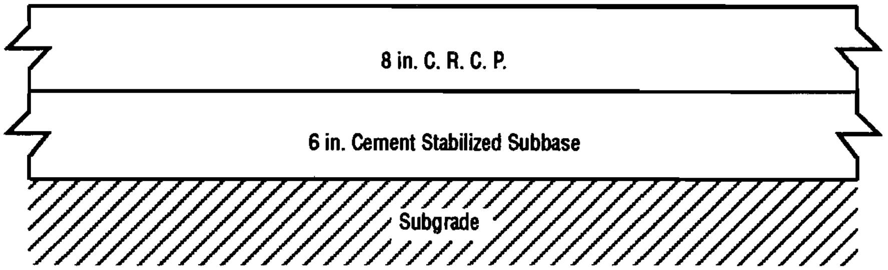
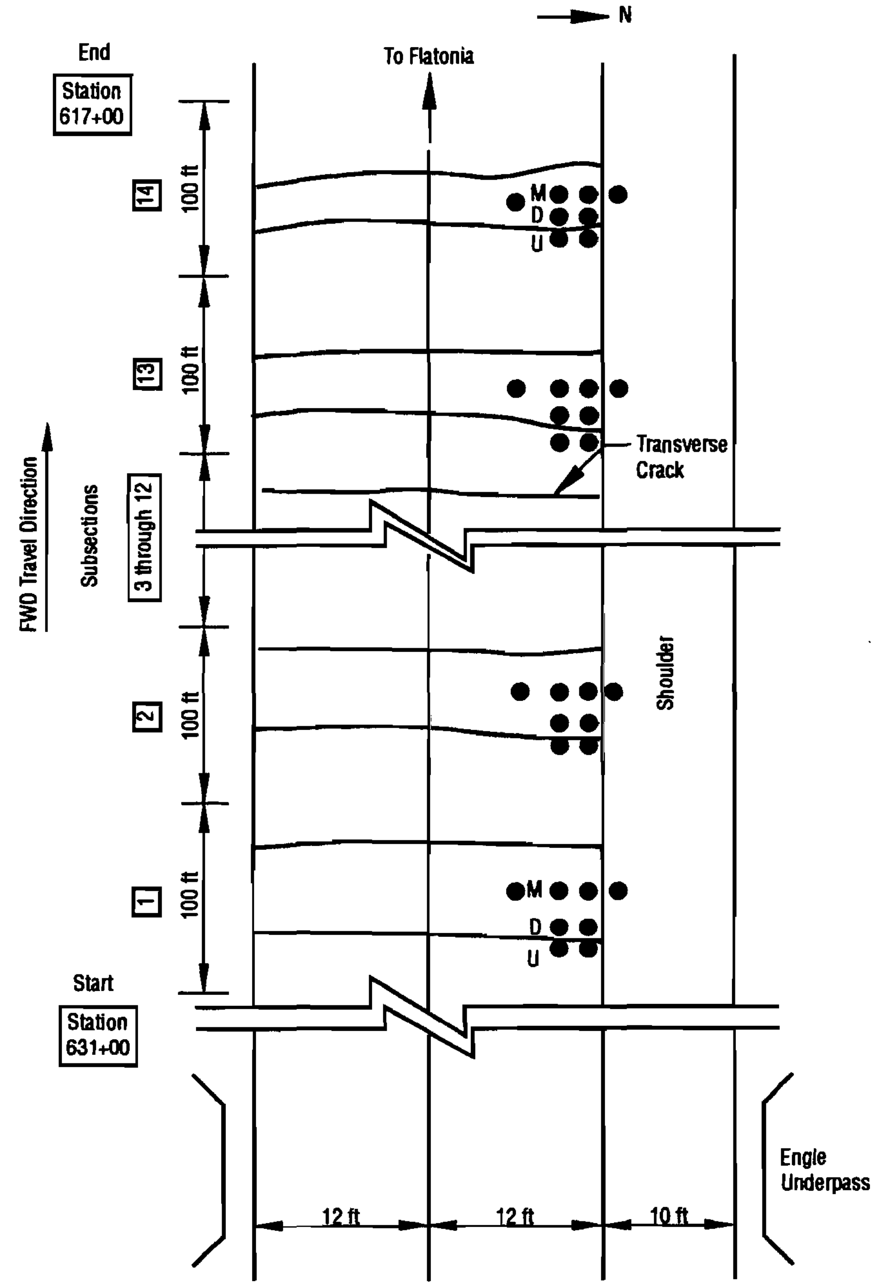


Fig 6.13. Test site location and pavement structure of CRCP section on IH10 in Flatonia.



Note: U = upstream loading; D = downstream loading; M = midspan loading

Fig 6.14. FWD test locations on CRCP section on IH10 in Flatonia.

CHAPTER 7. ANALYSIS OF FIELD DATA

The major objectives of the field tests included evaluation of the procedures developed in the laboratory study for assessing load transfer efficiency across transverse joints and cracks in rigid pavements and the adaptation of these procedures for evaluation of longitudinal joint efficiency in PCC pavements. On the three JRCP test sections, data were collected for the evaluation of load transfer efficiency across transverse joints and cracks in the pavements, and, on two of them, for an evaluation of the efficiency of longitudinal saw-cut joints. Data from the three CRCP test sections also consisted of data for evaluation of load transfer efficiency across cracks, and in one case data for the evaluation of longitudinal joint efficiency. In addition, the three CRCP test sections were used in an evaluation of the beneficial effects of PCC shoulders attached to existing CRCP pavements as well as in an evaluation of the effectiveness of a number of methods used by SDHPT to tie the shoulders to the rigid pavements. In the latter case, some factors which affect the effectiveness of the method of tying were also investigated. In this chapter the results of the analyses of the field data collected are presented. The results are presented separately for each test section and, in each case, outline the subject of the tests carried out on that particular section. A discussion of these results and their implications is presented later, together with a discussion of the other findings of this study, in Chapter 8.

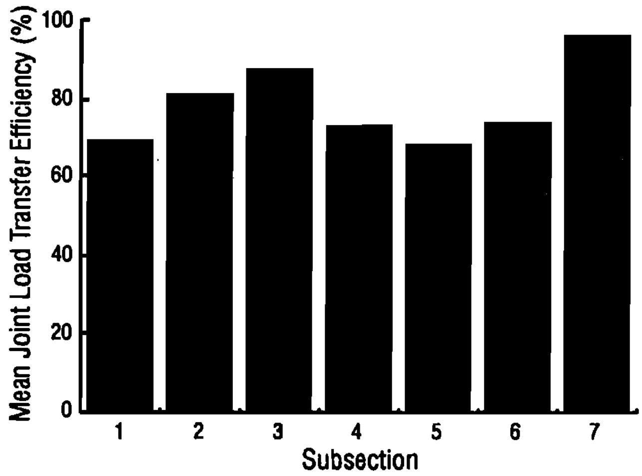
METHODS OF ANALYSIS

The methods for calculating load transfer efficiency across the transverse joint of the research pavement facility at Balcones Research Center (presented in Chapter 5) were the same as those used in assessing load transfer efficiency at the transverse joints and cracks in the field. Load transfer efficiency for the transverse joint and cracks was determined in all cases by both procedures A and B described earlier. An adaptation of the procedures was used in the evaluation of the efficiency of longitudinal joints. Referring to Fig 6.4, the joint efficiency of a longitudinal shoulder joint, for example, was characterized by a joint deflection ratio (JDR) determined from a main lane loading deflection ratio (MDR) and a shoulder loading deflection ratio (SDR). JDR based on procedure A is calculated as the average of MDR and SDR, both of which are determined as the ratio of the sensors 4 and 5 deflections with the higher deflection always the denominator. On the other hand, for procedure B, MDR is calculated as the deflection of sensor 4 (S4) on the unloaded side of the joint divided by the deflection of sensor 1 (S1) on the loaded side of the joint, and SDR as the deflection of sensor 5 (S5) on the unloaded side of the joint over the deflection of the loaded side measured by sensor 1 (S1). JDR is also in this instance calculated as the average of MDR and SDR. Longitudinal centerline and lane-dividing joints were evaluated similarly with the lanes left and right of the joints in the direction of travel of the FWD essentially corresponding to the outer mainline and shoulder lanes in the above example, respectively.

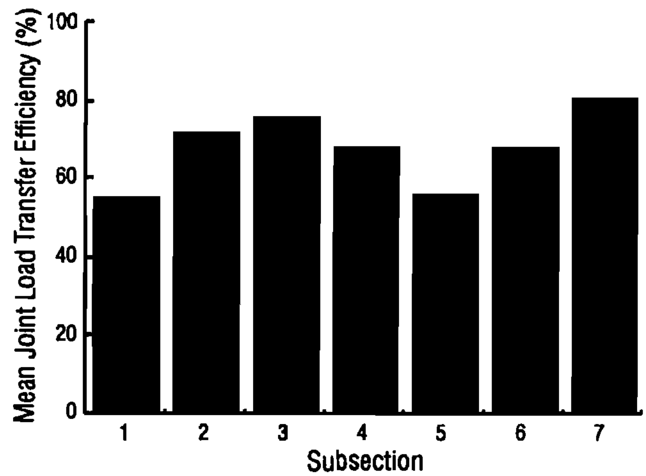
As in the laboratory study, the load transfer efficiency at any particular joint was calculated as the mean of the three load transfer efficiencies determined from the deflection measurements corresponding to the last three drop heights of the FWD. Likewise, deflection results presented are the mean of the 9-kip normalized deflection measurements under the FWD load plate, for the last three FWD drop heights at any particular location. The statistical software Statistical Analysis System (SAS) was used for the data reduction required and the calculation of the values of UDR, DDR, and JDR for transverse joints and cracks and of MDR, SDR, and JDR for longitudinal joints. SAS was also the software of choice for the necessary statistical analyses for this study.

BEAUMONT JRCP TEST SECTION, IH10

The tests on this JRCP were conducted to determine how effectively the FWD could be used to characterize load transfer efficiency across transverse joints in PCC pavements in the field. The section was divided into seven subsections of almost equal length classified as good, fair, or poor on the basis of a visual condition survey of observed pavement distress, particularly at the joints in the pavement. The results of FWD tests for load transfer efficiency evaluation at the transverse joints in the wheel path of the outer lane of this two-lane roadway were then compared to these subjective classifications. Some pertinent statistics on the load transfer efficiency of the joints in each of the seven subsections are shown in Table 7.1. The statistics for load transfer efficiency determined by procedures A and B are shown in Table 7.1(a) and Table 7.1(b), respectively. An examination of the results indicates that the statistics on load transfer efficiency of the transverse joints are in agreement with the results of the visual condition survey. The good sections generally have higher mean load transfer efficiencies in association with lower coefficients of variation in comparison with the fair and poor sections. Minimum load transfer efficiencies for the good sections are also higher than the corresponding values for the fair and poor sections. Figures 7.1(a) and 7.1(b) illustrate the differences in the mean load transfer efficiency of the seven sections determined by procedures A and B, respectively. Figure 7.1(a) clearly shows the difference between the good sections on one hand and the fair and



(a) Load transfer efficiency determined by procedure A.



(b) Load transfer efficiency determined by procedure B.

Fig 7.1. Comparison of mean load transfer efficiencies of visual condition-rated subsections (Beaumont JRCP, IH10).

TABLE 7.1. LOAD TRANSFER EFFICIENCY STATISTICS FOR DETERMINING PAVEMENT SUBSECTION CONDITIONS (BEAUMONT JRCP, IH10)

(A) STATISTICS ON LOAD TRANSFER EFFICIENCY DETERMINED BY PROCEDURE A

Subsection	Visual Condition Rating	Statistics on Joint Load Transfer Efficiency				
		Mean	S. D.[1]	C. V.[2] (%)	Maximum	Minimum
1	Poor	69	26	38	99	20
2	Good	81	19	24	99	41
3	Good	87	17	21	100	45
4	Fair	73	28	39	99	35
5	Poor	68	18	27	99	33
6	Fair	74	29	40	99	23
7	Good	96	5	5	99	77

[1] S.D. = Standard Deviation
[2] C.V. = Coefficient of Variation

(B) STATISTICS ON LOAD TRANSFER EFFICIENCY DETERMINED BY PROCEDURE B

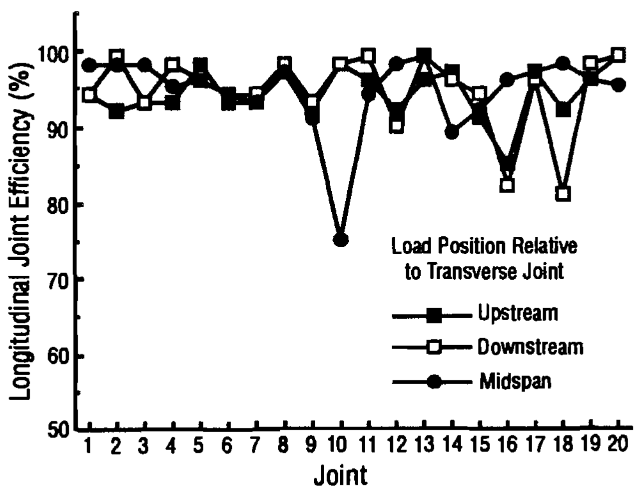
Subsection	Visual Condition Rating	Statistics on Joint Load Transfer Efficiency				
		Mean	S. D.[1]	C. V.[2] (%)	Maximum	Minimum
1	Poor	54	22	40	85	16
2	Good	71	20	28	97	33
3	Good	75	18	24	92	36
4	Fair	67	24	36	89	30
5	Poor	55	17	31	92	27
6	Fair	67	29	44	95	19
7	Good	80	10	13	89	43

[1] S.D. = Standard Deviation
[2] C.V. = Coefficient of Variation

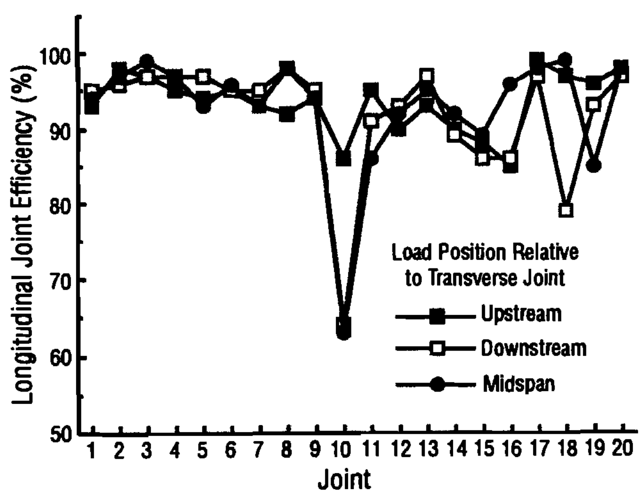
poor sections on the other, but there is no appreciable distinction between the fair and poor sections. Figure 7.1(b), which is based on load transfer efficiencies determined by procedure B, however, shows that the mean load transfer efficiencies and their associated measures of dispersion fall into three distinct groups very much in agreement with the results of the visual condition survey. The mean load transfer efficiencies of the good, fair, and poor sections range from 70 to 80 percent, 60 to 70 percent, and 50 to 60 percent, respectively. From the results of the tests on this pavement test section, there is, therefore, the indication that the potential exists for using load transfer efficiency as an effective evaluation parameter in the pavement evaluation process to detect relatively poor sections needing more attention. Since load transfer efficiency addresses a specific aspect of the condition of a rigid pavement, such use of the parameter in an overall pavement evaluation program will be a definite advantage in arriving at specific maintenance or rehabilitation remedies.

COLUMBUS JRCP TEST SECTION, US90

The tests on this JRCP test section (Fig 6.5) represented the first time the FWD was used for assessing the efficiency of a longitudinal joint using the sensor arrangement shown in Fig 6.4, with FWD deflection measurements also made for the evaluation of load transfer efficiency at twenty transverse joints. The original configuration of the FWD was used in addition to take deflection measurements for the evaluation of load transfer efficiency across the twenty transverse joints in the outer wheel path of the eastbound lane of this two-way,

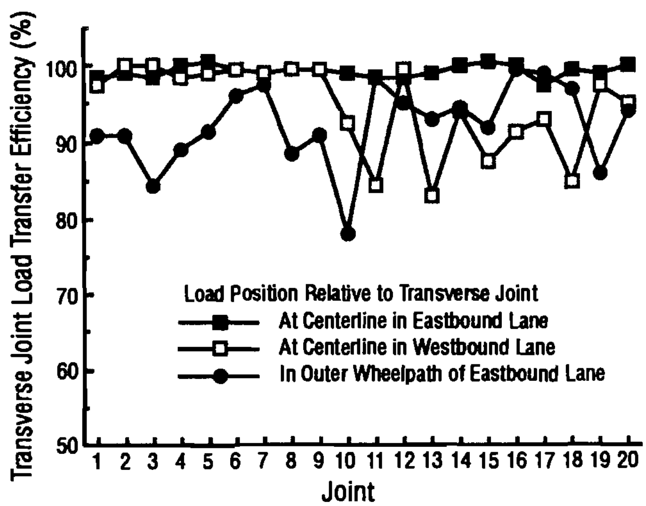


(a) Procedure A.

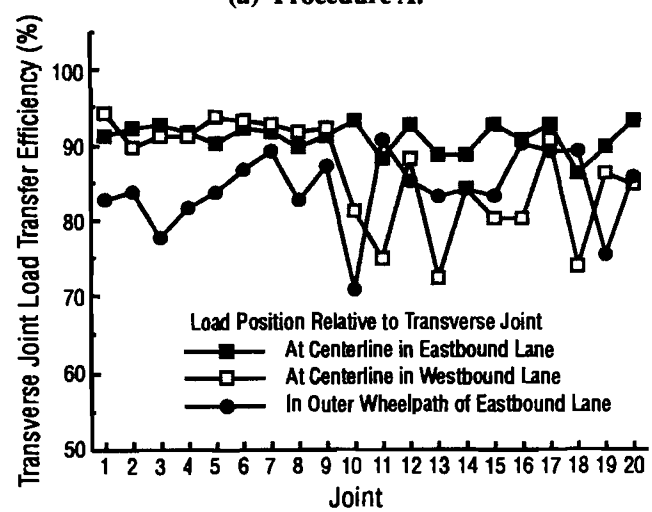


(b) Procedure B.

Fig 7.2. Example of joint efficiency evaluation along a longitudinal joint using the FWD (Columbus JRCP, US90).



(a) Procedure A.



(b) Procedure B.

Fig 7.3. Example of comparison of load transfer efficiency across transverse joints at different locations (Columbus JRCP, US90).

two-lane highway. The profile of the joint efficiency along the centerline joint using the modified FWD sensor arrangement is presented in Fig 7.2. The joint efficiencies with the FWD load located upstream, downstream, and at midspan of the transverse joints are shown. Both procedures A and B adequately show the variation in joint efficiency at the longitudinal joint of this rigid pavement in the field, and the results are in agreement. On the average, the longitudinal joint efficiency of the pavement from the first transverse joint up to the tenth is relatively higher than the efficiency from the tenth transverse joint to the end of the section, and there is more variability between the joint efficiencies at the three load locations in the latter section. It is clear from these results that such a profile of the longitudinal joint efficiency can be used to divide the pavement into stratified sections needing different levels of attention. In this example, as far as the longitudinal joint is concerned and with respect to the efficiency of the joint, it is evident that the proper decision—to look further at the section from the location of the tenth transverse joint to the end of the roadway—can be made on the basis of the results obtained.

Similar profiles of the load transfer efficiency at different locations across the transverse joints in the pavement section are shown in Fig 7.3. It is interesting to note that the transverse joint load transfer efficiency along the centerline, as shown in Fig 7.3, reinforces the results of the longitudinal joint efficiency evaluation depicted in Fig 7.2. It is clear from both figures that at the area around the centerline joint, load transfer efficiency in the orthogonal directions does indeed decline from the tenth joint onwards, and a decision to pay more attention to the end portion of the test section would have been justifiable. Figure 7.3 also shows that load transfer efficiency across the transverse joints is lower in the wheel path than in the test paths along the centerline joint up until the tenth joint, after which the load transfer efficiency in the test path in the westbound lane deteriorates significantly. There is no doubt that the ability to come to such conclusions, especially on a specific matter such as the efficiency of joints in a rigid pavement to transfer load, is a definite advantage in any pavement evaluation process.

BEAUMONT JRCP TEST SECTION, US90

The tests on this field section (Fig 6.7) were the first in which attempts were made to determine the influence of such factors as pavement temperature and temperature differential on load transfer efficiency across joints and cracks in rigid pavements. The tests included FWD measurements at adjacent transverse joints and cracks in the outer wheel path of the outer lane, and along the saw-cut longitudinal joint of this JRCP. Figure 7.4 illustrates the variation in deflection at transverse joints and cracks in the outer wheel path of three subsections in the test section with pavement temperature differential. There is a

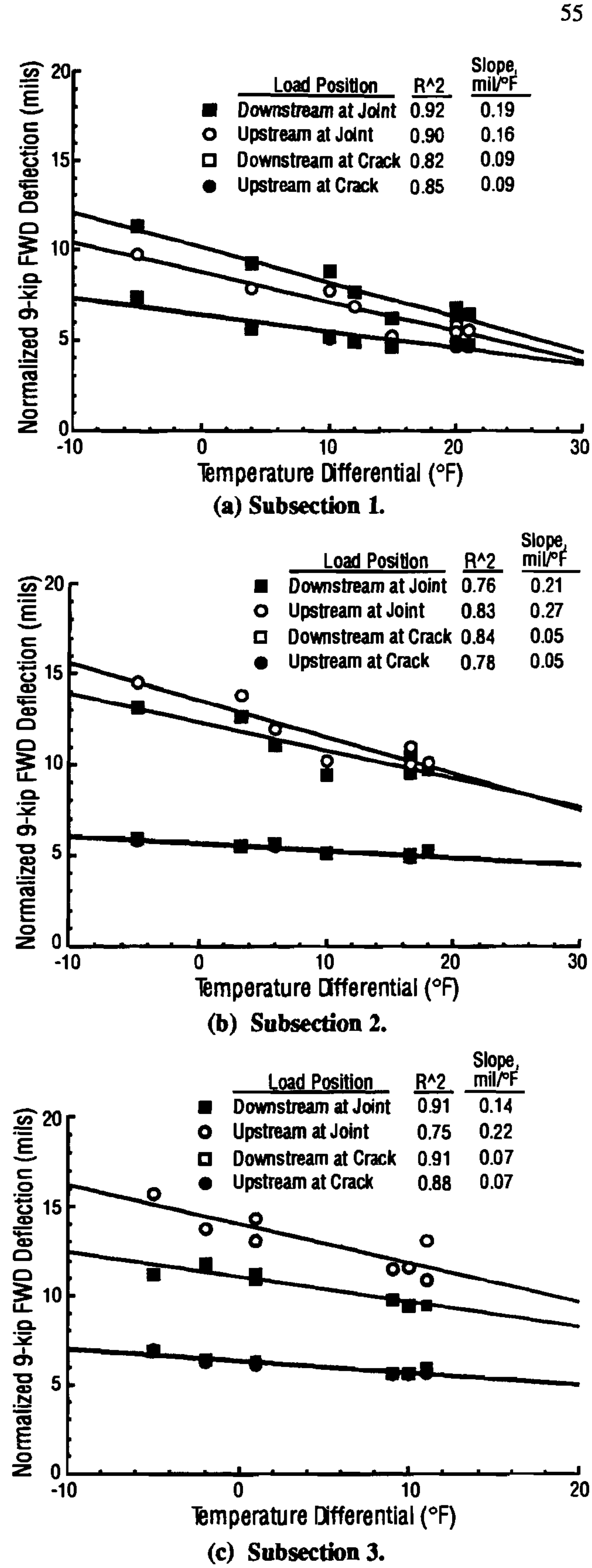
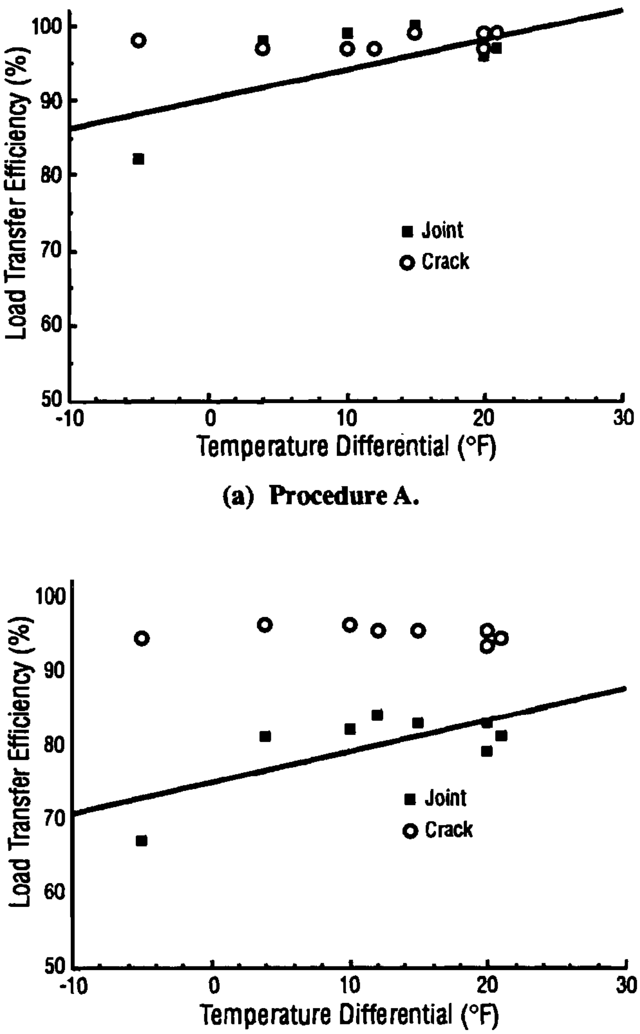


Fig 7.4. Effect of temperature differential on pavement deflection in outer wheel path (Beaumont JRCP, US90).

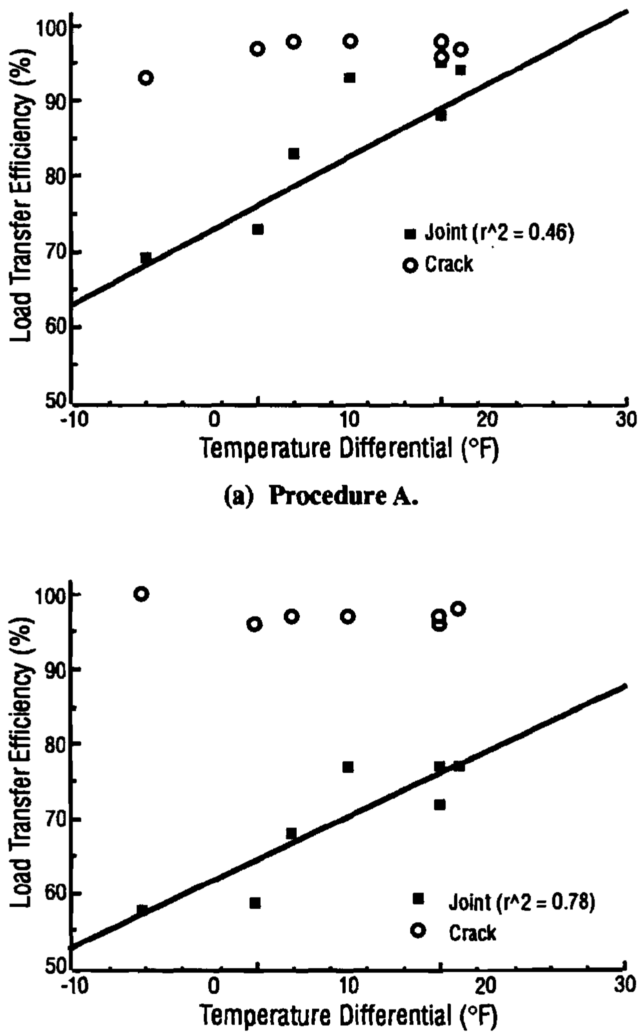
high negative correlation between pavement deflection and temperature differential. As shown, R^2 values between 76 and 92 percent were obtained for linear regression model fits to the data obtained from this section. An increasing temperature differential essentially corresponds to the pavement's going from a relatively curled-up position at negative temperature differentials to a curled-down position when the temperature differential is positive. The increasing pavement contact with the foundation owing to the increasing temperature differential results in a decrease in pavement deflection. The pavement deflections at the transverse joints are also higher than the deflections at the cracks, and the effect of the temperature differential change is more pronounced at the joints than at the cracks as shown by the slopes of the linear regression lines. In the three cases, the linear regression lines of deflection against temperature differential at the upstream and downstream locations of the transverse cracks virtually overlap. This is not the case for the corresponding linear regression models between deflection and temperature differential at the transverse joints, and there are some appreciable differences in the upstream and downstream deflections at the same temperature differentials, as shown in the figures. In all cases, there is evidence that the deflection at the joints approaches the deflection at the cracks with increasing temperature differential. Thermal expansion, and a tightening-up of the joints and increased foundation support at the joints because of the downwards curl of the pavement, are some of the factors that, by themselves or in combination, may be responsible for this decrease in deflection at the joints. These results are similar to earlier



(a) Procedure A.

(b) Procedure B.

Fig 7.5. Effect of temperature differential on load transfer efficiency at a joint and crack in outer wheel path of subsection 1, Beaumont JRCP section, US90.



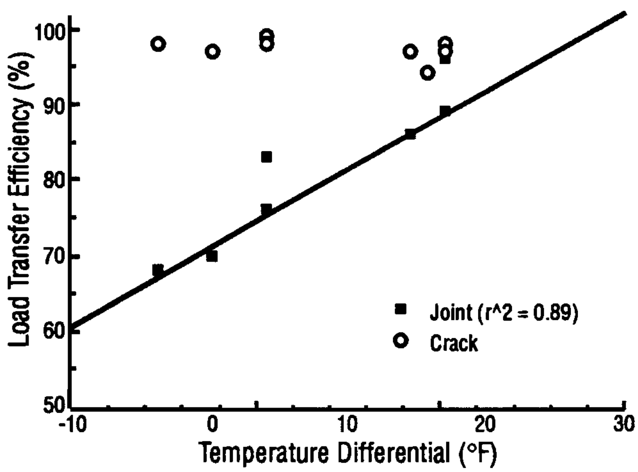
(a) Procedure A.

(b) Procedure B.

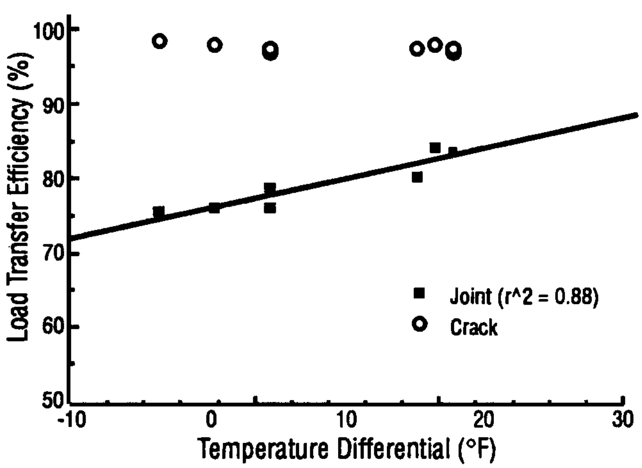
Fig 7.6. Effect of temperature differential on load transfer efficiency at a joint and crack in outer wheel path of subsection 2, Beaumont JRCP section, US90.

results obtained by McCullough and Treybig (Ref 30) and by Uddin et al (Ref 31) on deflections on rigid pavements. It is evident that such a relationship between deflection and temperature differential will have an impact on the load transfer efficiency at joints and cracks in rigid pavements.

Examples of the relationship between load transfer efficiency across selected joints and cracks in the three subsections and temperature differential are given in Figs 7.5, 7.6, and 7.7. Best-fit regression lines are shown for load transfer efficiency at the joints where applicable. The load transfer efficiencies determined by procedures A and B are shown in (a) and (b), respectively, in each case. An examination of the graphs gives mixed results. Except for subsection 1, there is a strong correlation between load transfer efficiency at the joints and temperature differential, with load transfer efficiency increasing with temperature differential. Problems associated with the measurement of pavement temperature for temperature differential determination in subsection 1 are believed to be responsible for the anomaly in the data shown in Fig 7.5. For subsections 2 and 3, a high coefficient of determination (R^2) is obtained, and the variation in load transfer efficiency at the joints because of temperature differential variation can be explained by a linear regression equation. No such relationship is evident between load transfer efficiency and temperature differential at the cracks in the same sections. Best-fit linear equations to the data for load transfer efficiency at the cracks in the three subsections give coefficients of determination less than 35 percent, and in all cases there

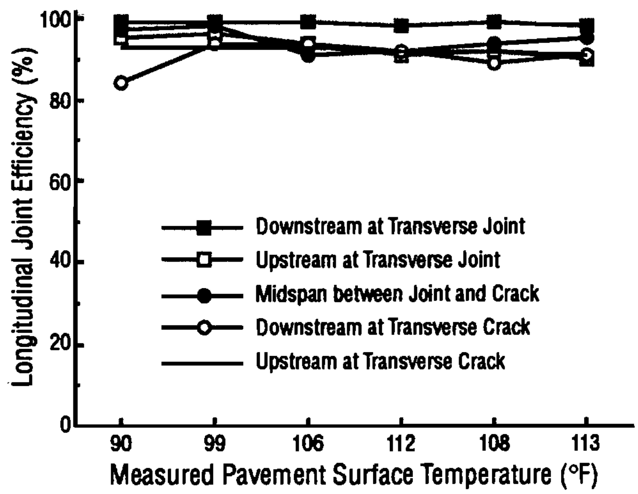


(a) Procedure A.

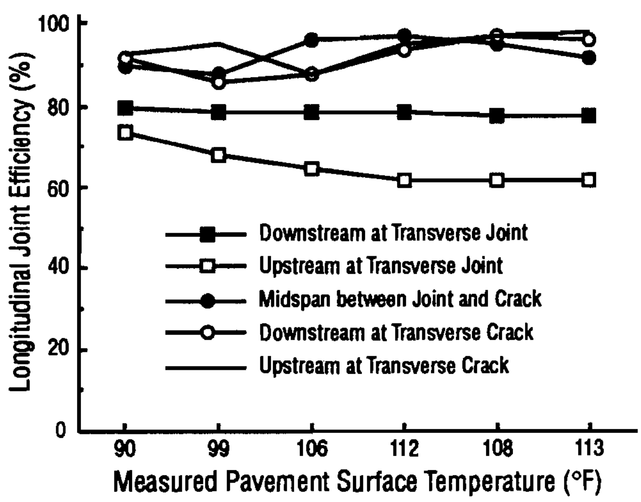


(b) Procedure B.

Fig 7.7. Effect of temperature differential on load transfer efficiency at a joint and crack in outer wheel path of subsection 3, Beaumont JRCP section, US90.



(a) Procedure A.



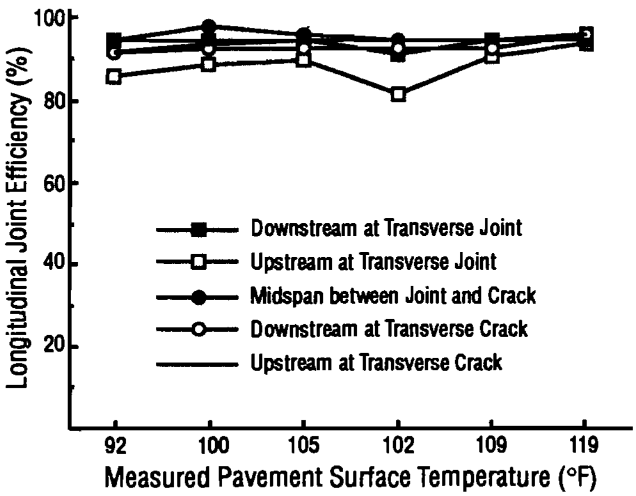
Note: Pavement surface temperatures are in chronological order.

(b) Procedure B.

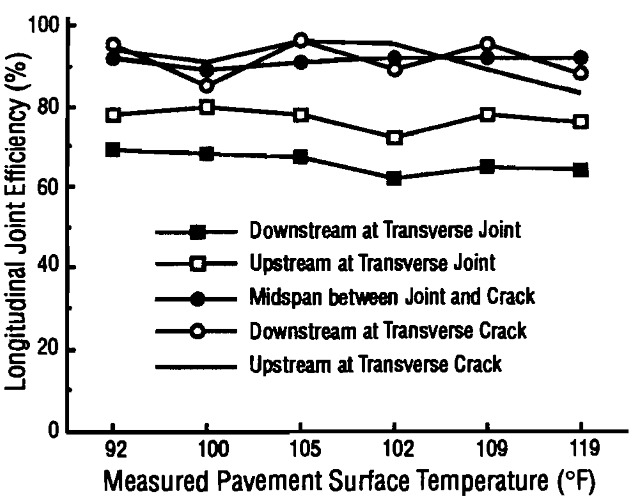
Fig 7.8. Typical center line longitudinal joint efficiency at different pavement temperatures in subsection 1 (Beaumont CRCP, US90).

is virtually no change in load transfer efficiency at the cracks with variation in temperature differential. Load transfer efficiency at the cracks is also higher than load transfer efficiency at the joints, as is to be expected. At the lower temperature differentials, this difference in load transfer efficiency at the cracks—in comparison with that at the joints—was as much as 50 percent in some cases. As was the case with deflection at the joints and cracks, the load transfer efficiency at the joints approaches that at the cracks with increasing temperature differential. In this instance also, thermal expansion, and a tightening-up of the joints and increased foundation support at the joints owing to the downwards curl of the pavement, are some of the factors responsible for this increase in load transfer at the joints. For the rigid pavement, the variation of such factors at the cracks is relatively minimal and, consequently, the load transfer efficiency stays almost the same.

Figures 7.8 through 7.10 show typical results of the longitudinal centerline joint efficiency evaluation for this test section using the FWD. The results obtained with procedures A and B are shown in parts (a) and (b) of each figure, respectively. Because of equipment malfunction, the temperature at the bottom of the pavement could not be determined for calculation of pavement temperature differential. Consequently, the graphs show the joint efficiency at locations relative to adjacent transverse joints and cracks along the longitudinal centerline joint of the three subsections, and the corresponding pavement surface temperature measurements, presented in chronological order. Not much distinction is observed between the joint efficiency at the various locations along



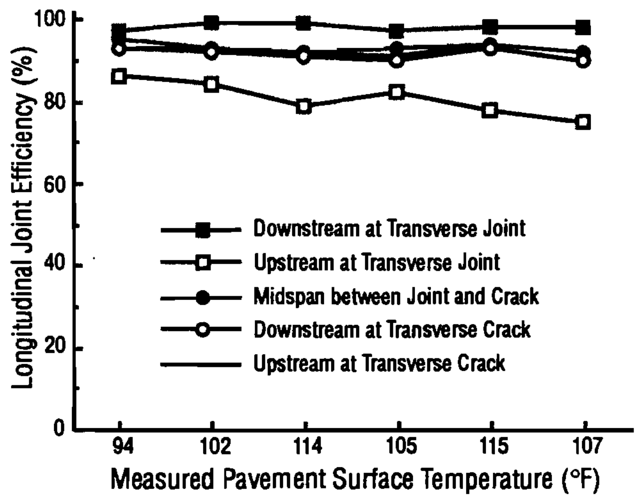
(a) Procedure A.



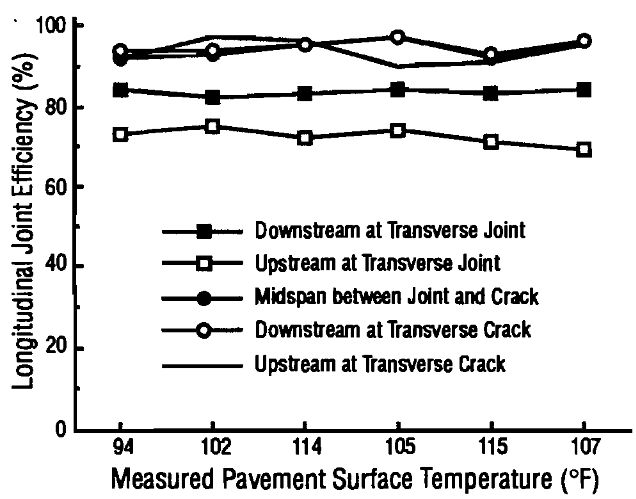
Note: Pavement surface temperatures are in chronological order.

(b) Procedure B.

Fig 7.9. Typical center line longitudinal joint efficiency for different pavement temperatures in subsection 2 (Beaumont CRCP US90).



(a) Procedure A.



Note: Pavement surface temperatures are in chronological order.

(b) Procedure B.

Fig 7.10. Typical center line longitudinal joint efficiency at different pavement temperatures in subsection 3 (Beaumont CRCP, US90).

the longitudinal joint determined by procedure A. The joint efficiency calculated by procedure B, however, shows a systematic variation. In all cases, the longitudinal joint efficiency determined by procedure B is considerably lower, at the transverse joint, than at the adjacent transverse crack and at midspan between the joint and crack. The longitudinal joint efficiency at the crack and midspan locations are virtually equal. In each case there is an appreciable difference between the joint efficiency determined upstream and downstream of the considerably more open transverse joints. On this pavement section surface temperature did not seem to have much influence on the longitudinal joint efficiency for the range of temperature observed.

VICTORIA CRCP TEST SECTION, US59

On this test pavement section retrofitted with a PCC shoulder to replace a flexible base shoulder, a study was conducted to determine the beneficial effects of a rigid shoulder. Specifically, the effects of the rigid shoulder on load transfer efficiency across the transverse cracks in the pavement, and on pavement deflections in the locale of the longitudinal shoulder joint, were investigated. Tests on this pavement included FWD load and deflection measurements at various locations (1) with the flexible shoulder in place; (2) after the removal of the flexible shoulder but prior to construction of the PCC shoulder; and (3) after curing of the rigid shoulder constructed. A summary of the mean load transfer efficiency across ten selected cracks in a 4,000-foot section of this CRCP, determined by procedures A and B, is presented in Table 7.2 for the three shoulder conditions. In Table 7.3, results of pavement deflection measurements at the ten selected transverse cracks are also given. The mean normalized 9-kip deflection measurements in each case, at the center of the FWD load plate at the upstream, downstream, and midspan locations of the selected cracks, in the outer wheel path of the outer lane, for the three shoulder conditions encountered, are given. Also given in, both Tables 7.2 and 7.3, are estimates of the pavement temperature differential from the buried temperature block, described earlier for each group of measurements, corresponding to the algebraic difference between the top and bottom temperature measurements from the block. An example of the typical temperature and temperature differential measurements from the block are shown in Fig 7.11 for testing on the pavement with a flexible shoulder. Figure 7.12 shows a comparison of the estimates of the temperature differential of the pavement for the two important phases of testing on this CRCP with a flexible and with a rigid shoulder. The second group of tests on the pavement with no shoulder of any type—an unacceptable alternative— were conducted for observational purposes only, and the results obtained are not included in subsequent analyses.

TABLE 7.2. SUMMARY OF FWD DEFLECTION MEASUREMENTS AT SELECTED TRANSVERSE CRACKS IN OUTER WHEEL PATH OF VICTORIA CRCP TEST SECTION FOR THREE SHOULDER CONDITIONS

Shoulder Condition	DT[1]	Load Transfer Efficiency Procedure	Mean Load Transfer Efficiency at Selected Crack (%)									
			1	2	3	4	5	6	7	8	9	10
Pavement with a flexible shoulder	6.0 - 11.50	A[2]	86	93	98	96	96	94	95	96	97	95
		B[3]	72	90	93	94	94	94	91	91	91	91
	19.25 - 22.50	A	88	93	98	96	97	96	93	94	98	97
		B	97	95	94	94	92	93	93	93	93	93
Pavement with a shoulder removed	4.25 - 5.50	A	86	93	97	95	98	96	96	95	91	*
		B	83	91	93	91	90	90	88	91	92	*
	3.00 - 2.50	A	86	94	98	96	99	97	98	95	98	*
		B	89	89	93	90	91	88	89	91	91	*
Pavement with a rigid shoulder	3.25 - 11.25	A	84	93	99	93	99	95	94	95	96	99
		B	83	87	92	89	92	88	92	90	89	90
	15.75 - 13.75	A	91	97	94	95	90	98	*	88	91	98
		B	90	90	93	91	92	90	*	91	89	91

[1]DT = Temperature Differential Range (°F)
[2]Load Transfer Efficiency Determined by Procedure A
[3]Load Transfer Efficiency Determined by Procedure B
*Missing Data

Analysis of variance (ANOVA) techniques were used to determine the statistically significant effect of the type of shoulder (flexible or rigid) on load transfer efficiency across the transverse cracks in the outer wheel path of the CRCP as determined by the procedures described previously. Analysis of variance was also conducted to determine the effect of the shoulder type on pavement deflection upstream, downstream, and at midspan of the cracks. These deflections were considered as separate dependent or response variables, and analysis of variance was conducted on them one at a time. The concepts of analysis of variance pertinent to this study can be found in statistics textbooks such as Refs 32, 33, and 34. As shown by the earlier analyses and the results presented previously, temperature differential in most instances has a considerable time effect on both load transfer efficiency and pavement deflection. Consequently, the ANOVA on the mean values of the dependent variables was conducted with blocking by temperature differential to increase the accuracy of the analysis. The values were classified under a low and a high temperature differential level corresponding to FWD tests conducted in the morning and afternoon periods, respectively, with ten replicates for the ten cracks in the section.

The results of the analysis of variance for load transfer efficiency across the transverse cracks, and for the upstream, downstream and midspan deflections, are presented in Table 7.4. From the results it is evident that the effect of type of shoulder on load transfer efficiency across the transverse cracks in the outer wheel path determined by procedures A and B is not significant even at the 10 percent level. On the other hand, the type of shoulder clearly has a significant effect on pavement deflection in the outer wheel path at lower than the 5 percent level. A comparison of means revealed an average 16 percent decrease in pavement deflections upstream and downstream of the cracks and an average 11 percent decrease at midspan with replacement of the flexible shoulder of this CRCP section with a rigid shoulder. Thus, although the measures of load transfer efficiency across the cracks in the outer wheel path do not show this trend in the example, pavement deflection measurements clearly indicate an increase in the load-carrying capacity of the CRCP with the addition of the PCC shoulder.

TABLE 7.3. LOAD TRANSFER EFFICIENCY, PERCENT, AT SELECTED CRACKS IN THE OUTER WHEEL PATH OF VICTORIA CRCP FOR THREE SHOULDER CONDITIONS

Shoulder Condition	DT[1]	Load Transfer Efficiency Procedure	Mean Normalized 9 - Kip Deflection at Selected Crack (mils)									
			1	2	3	4	5	6	7	8	9	10
Pavement with a flexible shoulder	6.0 - 11.50	U[2]	5.91	3.46	3.67	3.52	3.75	3.78	4.06	4.47	3.65	4.64
		D[3]	10.29	3.75	3.48	3.70	3.76	3.75	3.92	4.53	3.44	4.80
		M[4]	3.44	3.31	3.47	3.38	4.17	3.58	3.71	4.82	3.22	4.44
	19.25 - 22.50	U	5.97	4.35	4.36	5.21	4.65	4.60	5.23	5.00	3.98	5.16
		D	6.07	4.76	4.26	5.27	4.91	4.84	5.02	5.11	3.99	5.17
		M	3.90	4.56	4.09	4.86	4.59	4.79	4.60	5.44	3.93	5.04
Pavement with a shoulder removed	4.25 - 5.50	U	5.73	3.77	3.43	3.31	3.78	3.31	3.67	4.22	3.19	*
		D	9.60	3.89	3.25	3.53	3.64	3.26	3.61	4.38	3.27	*
		M	3.50	3.39	3.42	3.18	4.74	3.08	3.47	4.58	3.00	*
	3.00 - 2.50	U	5.97	3.23	3.58	3.37	3.66	3.44	3.50	4.26	3.21	*
		D	8.02	3.51	3.36	3.66	3.70	3.41	3.60	4.53	3.24	*
		M	3.54	3.08	3.42	3.31	4.72	3.22	3.47	4.70	2.99	*
Pavement with a rigid shoulder	3.25 - 11.25	U	5.14	3.10	3.45	3.17	3.55	3.50	4.05	4.22	3.42	3.90
		D	7.33	2.98	3.26	3.43	3.35	3.65	4.14	4.54	3.42	3.90
		M	3.39	3.22	3.39	3.48	4.05	3.73	3.64	4.75	3.33	4.44
	15.75 - 13.75	U	4.93	3.02	3.70	3.48	3.64	3.66	*	4.30	3.51	3.91
		D	5.48	2.99	3.59	3.61	3.64	3.78	*	4.47	3.44	3.94
		M	3.34	3.16	3.68	3.45	3.99	3.64	*	4.58	3.31	4.19

[1]DT = Temperature differential range (°F)
[2]Upstream deflection
[3]Downstream deflection
[4]Midspan deflection
*Missing data

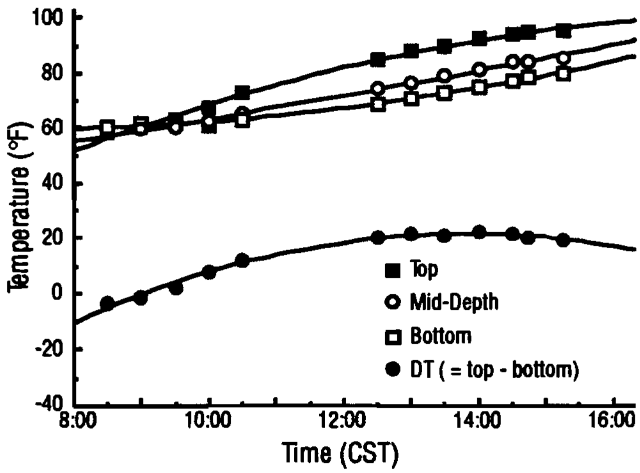


Fig 7.11. Example of temperature block results used to estimate pavement temperature and temperature differential — Victoria CRCP, April 15, 1987.

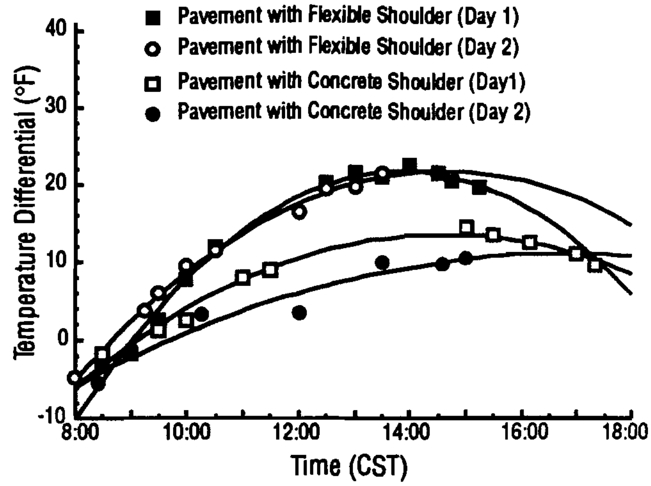


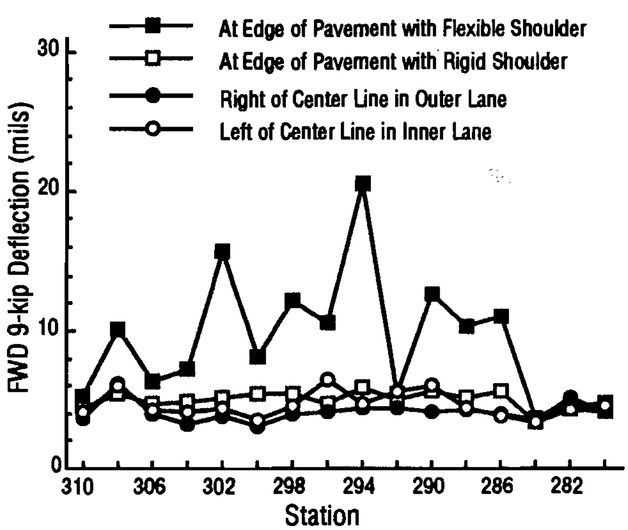
Fig 7.12. Temperature block temperature differential variation for various test days on Victoria CRCP.

TABLE 7.4. ANOVA RESULTS ON EFFECT OF TYPE OF SHOULDER ON LOAD TRANSFER EFFICIENCY AND PAVEMENT DEFLECTION IN OUTER WHEEL PATH OF VICTORIA CRCP SECTION (US59)

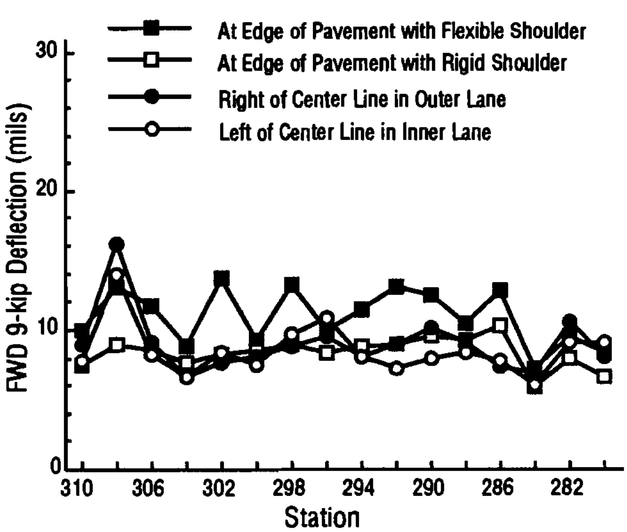
Dependent Variable	Source of Variation	DF[1]	Sum of Squares	F value	PR > F[2]
Load transfer efficiency at cracks determined by procedure A	Replications	9	293.99	4.73	0.0009
	Shoulder	1	4.37	0.63	0.4335
	DT[3]	1	1.44	0.21	0.6521
	Shoulder*DT	1	6.40	0.93	0.3445
	Error	26	179.54		
Load transfer efficiency at cracks determined by procedure B	Replications	9	155.65	1.37	0.4116
	Shoulder	1	35.41	2.80	0.2526
	DT	1	69.27	5.48	0.1063
	Shoulder*DT	1	8.80	0.70	0.0272
	Error	26	328.77		
Upstream deflection at cracks	Replications	9	12.38	22.80	0.0001
	Shoulder	1	4.59	76.11	0.0001
	DT	1	1.76	29.13	0.0001
	Shoulder*DT	1	1.15	19.11	0.0002
	Error	26	1.57		
Downstream deflection at cracks	Replications	9	43.33	7.98	0.0001
	Shoulder	1	6.18	10.24	0.0035
	DT	1	0.16	0.27	0.6073
	Shoulder*DT	1	0.74	1.22	0.2788
	Error	26	15.69		
Midspan deflection between cracks	Replications	9	7.98	50.44	0.0001
	Shoulder	1	1.91	24.61	0.0001
	DT	1	1.60	53.05	0.0001
	Shoulder*DT	1	1.82	44.48	0.0001
	Error	26	0.94		

[1] DF = Degree of freedom of source of variation.
[2] PR > F is the significance probability value associated with the F value.
[3] DT = Temperature differential estimate of pavement.

Figure 7.13, which compares pavement edge deflections at sixteen locations for the two types of shoulders considered to deflections along the centerline joint of the CRCP, further illustrates this point. Results of ANOVA show the effect of the type of shoulder on the pavement edge deflections is significant at the 5 percent level, and a comparison of means revealed an average 38 percent reduction in pavement edge deflections when the flexible shoulder was replaced with the rigid shoulder. As is apparent from Fig 7.13, the pavement edge deflections are brought down to the level of deflections measured under similar conditions along the centerline joint with the replacement of the flexible shoulder with a PCC shoulder.



(a) Low temperature differential — AM period.



(b) High temperature differential — PM period.

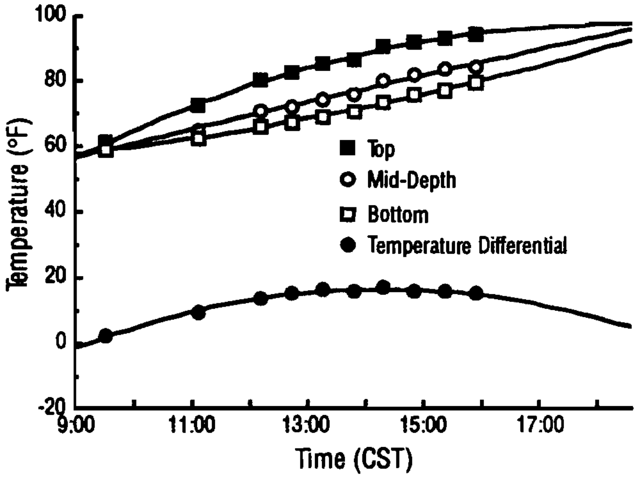
Fig 7.13. Influence of rigid shoulder on Victoria CRCP edge deflections.

WEATHERFORD CRCP TEST SECTION, IH20

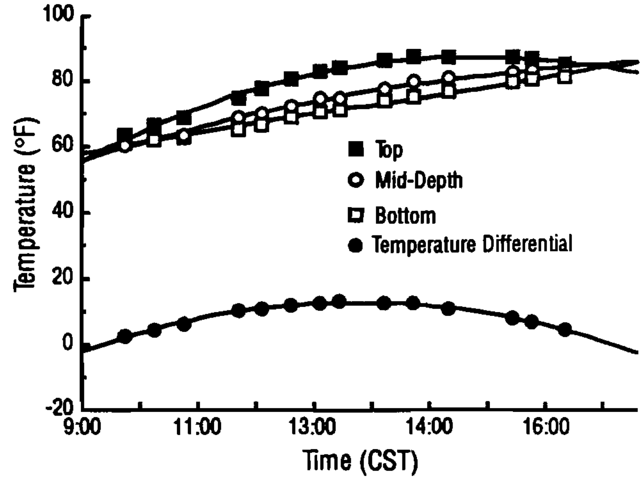
The Weatherford CRCP section was another test section whose flexible shoulder was replaced with a rigid shoulder. On this section, however, the major objective was to compare the relative effectiveness of three different methods used to tie the newly-constructed rigid shoulder to the existing main pavement. These methods, shown in Fig 6.11, were an ordinary tied joint; an inverted-tee joint; and a combination of the inverted-tee and tied joint, respectively denoted as joint type 'A,' joint type 'B,' and joint type 'C.' The pavement response and load transfer efficiency across cracks for the two shoulder conditions were first compared in three adjacent subsections, one each with the longitudinal joint types A, B, and C. With significant results, analysis of variance and multiple comparison of means techniques could then be used to determine the relative effectiveness of the joint types.

Table 7.5 is a summary of the load transfer efficiencies determined by the two procedures introduced, across six transverse cracks selected in the three subsections. The mean load transfer efficiencies determined in the outer wheel path of the outer lane, for the pavement with the two shoulder types, are given. Table 7.6 is a similar table which summarizes the mean normalized 9-kip FWD deflection measurements, also in the outer wheel path, of the outer lane upstream, downstream, and at midspan of the six selected transverse cracks for the two shoulder types. The results of measurements from a temperature block buried beside the pavement section for estimation of the pavement temperatures are illustrated in Fig 7.14. A comparison of pavement temperature differential estimates during the two phases is also shown in Fig 7.15. The data presented in Tables 7.5 and 7.6 are from FWD measurements in the morning period with the pavement temperature differential estimate below the 10-degree Fahrenheit mark in this figure.

The results from this CRCP section were similar to the results obtained on the Victoria CRCP. A preliminary paired t-test comparison of the deflection measurements upstream, downstream and at midspan of the six selected cracks in each subsection, showed there was no significant difference, even at the 10 percent level, between measurements at the same temperature differential level in the outer wheel path of the pavement for the two shoulder types. Similar results were also obtained for load transfer efficiency across the cracks in the outer wheel path determined by both procedures A and B. As a result, no significant difference could be determined between the three joint types with respect to load transfer efficiency and pavement deflection in the outer wheel path of the CRCP. Figure 7.16, however, illustrates the influence of the rigid shoulder on pavement deflection at the edge of the CRCP measured at locations less than a foot from the longitudinal shoulder joint of the pavement



(a) Pavement with flexible shoulder.



(b) Pavement with rigid shoulder.

Fig 7.14. Temperature block measurements used to estimate pavement temperature during testing on Weatherford CRCP section.

TABLE 7.5. LOAD TRANSFER EFFICIENCY ACROSS TRANSVERSE CRACKS IN OUTER WHEELPATH OF WEATHERFORD CRCP SECTION FOR TWO SHOULDER CONDITIONS

Shoulder Condition	Section	Load Transfer Efficiency Procedure	Mean Load Transfer Efficiency at Crack (%)					
			1	2	3	4	5	6
Pavement with a flexible shoulder	A[1]	A[2]	97	97	98	98	*	*
		B[3]	92	92	92	90	*	*
	B[1]	A	95	97	97	97	97	96
		B	93	91	91	91	90	91
	C[1]	A	96	97	98	96	98	96
		B	94	94	94	95	93	94
Pavement with a rigid shoulder	A	A	97	95	96	96	96	96
		B	91	93	91	90	92	91
	B	A	96	91	94	96	95	93
		B	94	96	90	92	91	92
	C	A	94	98	97	96	95	96
		B	95	94	95	95	95	92

[1] Corresponds to subsection joint detail (Fig 6.11)
[2] Load transfer efficiency determined by procedure A
[3] Load transfer efficiency determined by procedure B
* Missing data

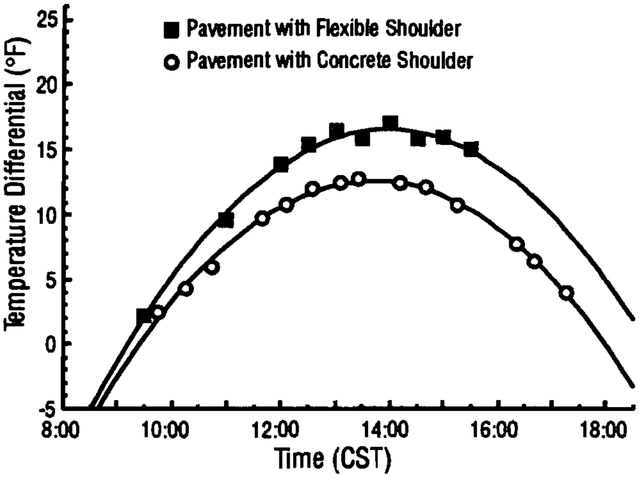


Fig 7.15. Comparison of temperature differential for the two phases of testing on Weatherford CRCP section.

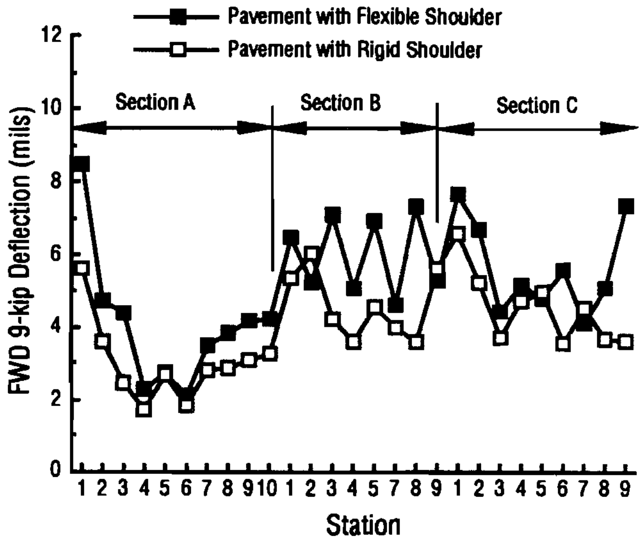


Fig 7.16. Influence of a rigid shoulder on Weatherford CRCP edge deflections.

TABLE 7.6. SUMMARY OF FWD DEFLECTION MEASUREMENTS AT SELECTED TRANSVERSE CRACKS IN OUTER WHEEL PATH OF WEATHERFORD CRCP SECTION FOR TWO SHOULDER CONDITIONS

Shoulder Condition	Subsection	Location at Crack	Mean Normalized 9-kip Deflection at Crack (mils)					
			1	2	3	4	5	6
Pavement with a flexible shoulder	A[1]	U[2]	3.30	3.23	3.08	2.78	4.59	*
		D[3]	3.17	3.19	3.15	2.79	*	*
		M[4]	3.10	3.05	3.07	2.89	*	*
	B[1]	U	3.82	3.41	3.19	3.37	3.44	4.23
		D	3.73	3.41	3.18	3.52	3.26	4.35
		M	3.54	3.22	3.08	3.50	3.15	4.37
	C[1]	U	5.34	5.42	5.26	5.54	5.38	5.58
		D	5.40	5.55	5.22	5.72	5.41	5.84
		M	5.34	5.37	5.14	5.66	5.32	5.61
Pavement with a flexible shoulder	A	U	3.17	3.23	3.08	2.68	2.93	3.04
		D	3.29	3.24	3.01	2.64	2.99	3.13
		M	3.02	3.10	2.91	2.63	2.92	2.88
	B	U	4.69	5.92	3.64	3.84	2.96	3.72
		D	4.37	5.52	3.32	3.84	2.89	3.67
		M	4.53	4.34	3.28	3.71	2.76	3.56
	C	U	6.67	5.92	5.87	6.56	5.12	5.36
		D	6.87	5.72	5.94	6.68	5.21	5.34
		M	6.64	5.42	5.84	6.75	5.01	5.15

[1] Corresponds to subsection joint detail (Fig 6.11)
[2] Upstream deflection
[3] Downstream deflection
[4] Midspan deflection
* Missing data

for the same temperature differential level. In paired t-test comparisons the effect of the type of shoulder on the deflection measurements at the edge of the pavement in each section were all found to be significant at the 5 percent level. From a multiple comparison of means, the average reductions in pavement edge deflection due to the replacement of the flexible shoulder with a rigid shoulder were determined to be 26, 28, and 20 percent, respectively, for section A with a tied shoulder joint, section B with an inverted-tee shoulder joint, and section C with a tied inverted-tee joint. With the similar reduction levels, however, no statistically significant differences were discernible between the three joint types with respect to their relative effectiveness. Analysis of variance failed to show any significance between the reduction levels in the three sections and pointed to a similarity between the effectiveness of three joint types. Since the inverted-tee joint types B and C cost more to build than the ordinary tied joint type A, there is, therefore, the indication that the tied joint is the best choice of the three types used on this CRCP on a cost basis.

FLATONIA CRCP TEST SECTION, IH10

On this last CRCP test section in Flatonia, a study was conducted to determine the effect of tie bar dimensions on rigid pavement response at joints and cracks. Specifically, the influence of tie bar diameter, length, and spacing on pavement response, especially at the longitudinal joint between a concrete shoulder and the existing rigid pavement to which it is attached in replacement of a flexible shoulder, were examined. The effects of these parameters on load transfer efficiency across transverse cracks in this section were also investigated, as was done in the case of the previous test sections. In this instance, however, the load transfer efficiency across the transverse cracks between the outer wheel path of the outer lane and the edge of the pavement was also investigated. As described in Chapter 6, on this CRCP section, twelve 100-foot subsections were selected for a 2 x 2 x 3 factorial design. Tie bar sizes #4 and #5 (4/8- and 5/8-inch diameters); lengths of 15 and 25 inches; and spacings of

Tie Bar Size	Tie Bar Length (in.)	Tie Bar Spacing (in.): 24	36	48
#4	15	2[1]	5	9
	25	4	7	11
#5	15	3	6	10
	25	1[2]	8	12

[1] Section numbers
[2] Standard shoulder joint design,. Also used for section 14.
Note: #4 tie bars, 30 inches long, at 30-inch spacing, similar to center line design, used to tie shoulder in section 13.

Fig 7.17. Factorial arrangement of sections on Flatonia CRCP.

24, 36, and 48 inches were chosen for study. The factorial arrangement of the sections and the dimensions of the tie bars used at their longitudinal shoulder joints are shown in Fig 7.17. A thirteenth 100-foot section with #4 tie bars, 30 inches long, and at 30-inch spacing, used at the shoulder joint, a design similar to that used at the centerline joint of this CRCP, was added to the factorial; and a fourteenth section, 100 feet long at the end of the test section with a standard joint design similar to that of section one, was included for observational purposes.

The load transfer efficiencies determined by procedures A and B across the transverse cracks in the outer wheel path of the Flatonia CRCP section for the two shoulder conditions are given in Tables 7.7(a) and (b), respectively. The average reductions in pavement deflection upstream, downstream, and at midspan of the two cracks in each subsection, in the outer wheel path of the outer lane of this CRCP, are presented in Tables 7.7(a), (b) and (c), respectively. The reductions are the differences in measured pavement deflection for similar weather conditions resulting from a replacement of the existing flexible shoulder with a PCC shoulder. The temperature differential estimates for the pavement during the periods of testing on the CRCP are shown in Fig 7.18.

These measurements, from the two selected cracks in each of the 100-foot sections representing two replicates, were used in statistical analysis to determine the significance of a number of factors on the dependent variables load transfer efficiency and pavement deflection. The load transfer efficiency across the cracks and the pavement deflections upstream, downstream, and at midspan of the cracks were the specific dependent variables investigated. A preliminary analysis of variance was conducted to determine, independently for each section, the significance of the PCC shoulder attached on the five dependent variables with blocking by temperature differential. This

TABLE 7.7(A). SUMMARY OF LOAD TRANSFER EFFICIENCY, PERCENT, AT CRACKS IN OUTER WHEEL PATH OF FLATONIA CRCP SECTION FOR TWO SHOULDER CONDITIONS — PROCEDURE A

Tie Bar Size	Tie Bar Length (in.)	Tie Bar Spacing (in.) 24		36		48	
#4	15	99[1]	99[5]	97	98	96	97
		100[2]	100	98	99	98	99
		96[3]	98	94	98	92	96
		99[4]	99	98	99	98	97
	25	97	99	96	98	96	95
		98	98	96	97	98	99
		99	98	97	99	94	98
		99	99	96	99	97	97
#5	15	98	97	98	98	95	94
		97	98	98	98	97	98
		99	96	99	100	96	88
		99	97	98	99	97	93
	25	96	99	98	99	97	96
		97	99	99	98	95	99
		98	97	98	99	94	99
		98	99	99	99	95	99

1 Load transfer efficiency at first crack in the morning for pavement with flexible shoulder.
2 Load transfer efficiency at first crack in the afternoon for pavement with flexible shoulder.
3 Load transfer efficiency at first crack in the morning for pavement with concrete shoulder.
4 Load transfer efficiency at first crack in the afternoon for pavement with concrete shoulder.
5 Replicate measurements at second crack in subsection.

TABLE 7.7(B). SUMMARY OF LOAD TRANSFER EFFICIENCY, PERCENT, AT CRACKS IN OUTER WHEEL PATH OF FLATONIA CRCP SECTION FOR TWO SHOULDER CONDITIONS — PROCEDURE B

Tie Bar Size	Tie Bar Length (in.)	Tie Bar Spacing (in.) 24		36		48	
#4	15	91[1]	92[5]	89	90	87	87
		90[2]	90	90	90	88	86
		95[3]	94	94	92	90	90
		93[4]	93	92	93	90	90
	25	91	92	91	92	87	86
		91	91	91	90	81	87
		93	93	93	93	91	90
		93	93	93	93	90	89
#5	15	91	91	90	91	87	87
		91	92	90	90	88	89
		93	93	93	92	89	94
		94	94	93	93	89	94
	25	91	91	98	88	89	87
		90	89	99	88	88	88
		86	92	98	89	89	90
		93	92	99	89	89	89

1 Load transfer efficiency at first crack in the morning for pavement with flexible shoulder.
2 Load transfer efficiency at first crack in the afternoon for pavement with flexible shoulder.
3 Load transfer efficiency at first crack in the morning for pavement with concrete shoulder.
4 Load transfer efficiency at first crack in the afternoon for pavement with concrete shoulder.
5 Replicate measurements at second crack in subsection.

preceded an analysis of variance to establish the combination of tie bar dimensions with the most favorable results on the dependent variables. An assumption of similar pavement design and even construction practices throughout the section was made.

The results of the initial analysis of variance in all cases indicated that the type of shoulder had a significant effect at the 5 percent level on pavement deflections upstream, downstream, and at midspan of the cracks in each of the sections in the outer wheel path. From a multiple comparison of means, the reduction in pavement deflection in the individual sections with the addition of the rigid shoulder was found to range from 28 percent to as high as 45 percent in the outer wheel path of the outer lane of the pavement. As in the previous cases, no significant effect of the type of shoulder was found for the load transfer efficiency determined by procedures A and B across the transverse cracks in the outer wheel path of each section. No further analysis was therefore conducted

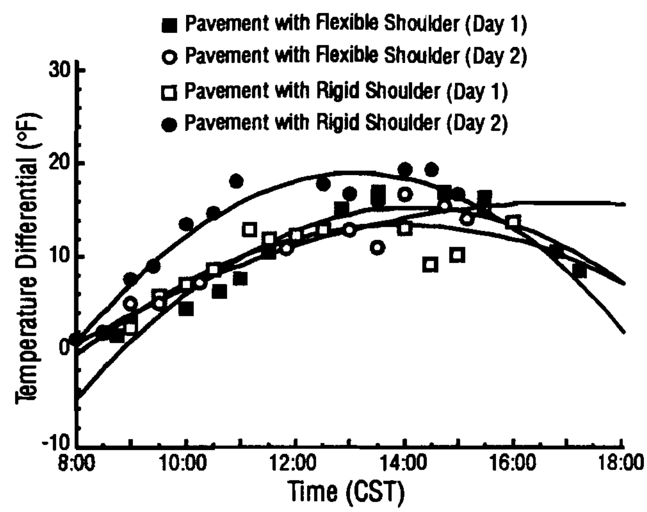


Fig 7.18. Comparison of temperature differential estimates during testing on Flatonia CRCP section.

TABLE 7.8(A). AVERAGE REDUCTION IN PAVEMENT DEFLECTION, MILS, UPSTREAM OF TRANSVERSE CRACKS IN OUTER WHEEL PATH OF FLATONIA CRCP SECTION

Tie Bar Size	Tie Bar Length (in.)	Tie Bar Spacing (in.) 24	36	48
#4	15	0.95[1]	0.85	1.04
		2.36[2]	1.86	1.50
		1.56[3]	2.66	0.67
		2.46[4]	2.89	2.01
	25	1.79	1.91	1.28
		2.12	2.20	2.27
		1.97	2.35	0.64
		2.56	3.12	2.08
#5	15	1.72	1.28	1.46
		2.50	1.80	1.60
		1.40	1.43	0.20
		2.76	2.05	1.23
	25	1.37	1.91	2.45
		3.13	1.98	1.46
		1.66	1.71	1.33
		2.32	2.54	1.61

[1] Deflection reduction at first crack in AM (morning) period.
[2] Deflection reduction at first crack in PM (afternoon) period.
[3] Deflection reduction at second crack in AM period.
[4] Deflection reduction at second crack in PM period.

TABLE 7.8(B). AVERAGE REDUCTION IN PAVEMENT DEFLECTION, MILS, DOWNSTREAM OF TRANSVERSE CRACKS IN OUTER WHEEL PATH OF FLATONIA CRCP SECTION

Tie Bar Size	Tie Bar Length (in.)	Tie Bar Spacing (in.) 24	36	48
#4	15	1.01[1]	1.07	0.91
		2.36[2]	1.98	1.64
		1.44[3]	2.59	0.75
		2.29[4]	2.97	2.13
	25	1.94	2.18	1.15
		2.17	2.49	2.58
		2.02	2.25	0.66
		2.68	3.21	2.12
#5	15	1.81	1.40	1.47
		2.59	1.86	1.59
		1.78	1.49	0.72
		3.04	2.02	1.33
	25	1.24	1.72	2.45
		2.65	1.96	1.46
		1.72	1.72	1.33
		2.45	2.31	1.61

[1] Deflection reduction at first crack in AM (morning) period.
[2] Deflection reduction at first crack in PM (afternoon) period.
[3] Deflection reduction at second crack in AM period.
[4] Deflection reduction at second crack in PM period.

on the effect of the independent factors on load transfer efficiency.

Results of the analysis of variance on the effect of the factors tie bar size (2 levels), tie bar length (2 levels) and tie bar spacing (3 levels) on the average reduction in pavement deflection in the outer wheel path of the outer lane of the CRCP are given in Table 7.9. The results show that tie bar spacing (24, 36, and 48 inches) is the most important of the factors, and has a significant effect at the 5 percent level on pavement deflection reduction upstream, downstream, and at midspan of the cracks. In fact, away from the discontinuities at the midspan of the cracks, neither the tie bar size (#4 and #5) nor the tie bar length (15 and 25 inches) has any significant effect on pavement reduction. Of these two, only tie bar length has a significant effect at the 5 percent level on pavement deflection reduction at the cracks. This observation points to a similarity in the influence of the tie bar sizes used. The 1/8-inch difference between their diameters apparently did not have any significant effect on pavement deflection in the cases investigated.

A multiple comparison of means indicated that the tie bar spacings of 24 and 36 inches both resulted in an average reduction in pavement deflection of about 2.0 mils in the outer wheel path. There was no significant difference between them at the 5 percent level. The mean

TABLE 7.8(C). AVERAGE REDUCTION IN PAVEMENT DEFLECTION, MILS, MIDSPAN OF TRANSVERSE CRACKS IN OUTER WHEEL PATH OF FLATONIA CRCP SECTION

Tie Bar Size	Tie Bar Length (in.)	Tie Bar Spacing (in.) 24	36	48
#4	15	1.23[1]	1.36	0.92
		1.93[2]	2.24	1.58
		1.14[3]	2.47	0.51
		2.20[4]	2.70	1.90
	25	1.92	2.13	1.79
		2.34	2.42	2.06
		1.90	2.20	0.69
		2.59	3.20	1.64
#5	15	1.82	1.36	0.93
		2.69	1.86	1.01
		2.03	1.45	0.45
		3.29	2.09	1.32
	25	1.20	1.82	3.09
		1.93	2.26	1.52
		1.14	1.04	0.34
		2.20	1.43	1.61

[1] Deflection reduction at first crack in AM (morning) period.
[2] Deflection reduction at first crack in PM (afternoon) period.
[3] Deflection reduction at second crack in AM period.
[4] Deflection reduction at second crack in PM period.

pavement deflection reduction due to the 48-inch spacing was on the order of 1.5 mils and was significantly different from the reduction due to the 24- and 36-inch spacings at the 5 percent level. At the cracks, the longer 25-inch tie bars also resulted in an average reduction in deflection of approximately 2.0 mils which, according to the results obtained from the statistical analysis, was significantly different from the 1.6-mil mean deflection reduction offered by the 15-inch-long tie bars. Thus, on the basis of the analysis, as far as pavement response in the outer wheel path of the CRCP is concerned, #4 or #5 tie bars, 25 inches long, and at a 24- or a 36-inch spacing have a beneficial effect and reduce pavement deflections.

A similar analysis was conducted on load transfer efficiency across the cracks and deflection at the edge of the pavement. A summary of the load transfer efficiency determined by procedures A and B across the transverse cracks at the pavement edge for the two shoulder conditions is given in Tables 7.10(a) and (b), respectively. In this instance also, an analysis of variance showed the PCC shoulder had no immediate significant effect on load transfer efficiency across the cracks. Tables 7.11(a), (b), and (c) show the average reduction in pavement deflection upstream, downstream, and at midspan of the cracks at the edge of the CRCP. An initial analysis of variance to determine the effect of the PCC shoulder on pavement edge deflections upstream, downstream, and at midspan of the cracks was significant at the 5 percent level in each subsection. A multiple comparison of means indicated that the addition of the rigid shoulder resulted in average pavement edge deflection reduction ranging from 25 percent to 50 percent. In nine out of the fourteen sections the immediate reduction in pavement deflection at the edge was over 40 percent, representing a remarkable reduction.

An analysis of variance was also conducted on the effect of the main factors tie bar size (#4 and #5), tie bar length (15 and 25 inches), and tie bar spacing (24, 36, and 48 inches) on pavement deflection at the edge of the CRCP. The results of the analysis is given in Table 7.12. All the main factors, except length of tie bar in relation to deflection reduction upstream of the cracks (PR > F = 6.21), were significant at the 5 percent level. A multiple comparison of means indicated that the average reductions in pavement edge deflection because of tie bar spacings of 24 and 36 inches were not significantly different at the 5 percent level. On the average, the difference between their effects was approximately 0.03 mils at the upstream, downstream, and midspan locations of the cracks. The effect of a tie bar spacing of 48 inches was, however, different from the effect of the other two tie bar spacings at the 5 percent significance level. The pavement deflection reduction levels achieved with tie bars spaced at 24 and 36 inches were, on the average, 2.4 and 2.1 mils, respectively, more than that achieved with a spacing of 48 inches. These findings with respect to tie bar spacing at the edge of the CRCP are in agreement with the results obtained previously for pavement response in the outer wheel path. Contrary to the findings on the effect of tie bar length on deflection reduction in the outer wheel path, a multiple comparison of means for the significance of the reduction in deflections downstream and at midspan of the cracks indicated that the shorter 15-inch tie bars were more effective in reducing deflection at the pavement edge than the 25-inch-long bars. The average difference between the reduction levels due to the tie bar lengths was on the order of 0.4 mils. Similarly, the #4 tie bars were found to reduce pavement deflections approximately 0.7 mils more than the #5 tie bars.

To facilitate a comparison of the centerline joint design used in section 13 to that of the other 12 sections in the factorial shown in Fig 7.17, analyses of variance were conducted to determine the effect of the section treatments on reduction in pavement deflection at the edge of the CRCP and in the outer wheel path of the outer lane. Section 14, with a standard design similar to the that of section 1, was also included in this analysis of variance for comparative purposes. Table 7.13 shows the results of the analysis of variance on the effect of the section joint designs or treatments on deflection at the edge of the CRCP. Clearly, the effect of the treatments is significant (PR > F = 0.0001). With this result, a multiple comparison of means was then performed to compare the various joint designs on the basis of the their effectiveness in reducing pavement reduction at the edge of the Flatonia CRCP. The rankings of the section treatments based on

TABLE 7.9. ANOVA RESULTS ON EFFECT OF TIE BAR DIMENSIONS ON PAVEMENT DEFLECTION REDUCTION IN OUTER WHEEL PATH OF FLATONIA CRCP

Dependent Variable	Source of Variation	DF[1]	Sum of Squares	F value	PR > F[2]
Deflection reduction upstream of cracks	Replications	1	0.248	1.18	0.2850
	DT[3]	1	6.214	29.57	0.0001 #
	Barsize, B	1	0.029	0.14	0.7148
	Length, L	1	1.323	6.30	0.0170 #
	Spacing, S	2	3.608	8.59	0.0010 #
	B*L	1	0.053	0.25	0.6198
	B*S	2	0.710	1.69	0.1996
	L*S	2	0.225	0.54	0.5903
	B*L*S	2	0.358	0.85	0.4360
	Error	34	7.144		
Deflection reduction downstream of cracks	Replications	1	0.254	1.62	0.2118
	DT	1	6.534	41.70	0.0001 #
	Barsize, B	1	0.123	0.78	0.3818
	Length, L	1	0.793	5.06	0.0311 #
	Spacing, S	2	3.349	10.68	0.0003 #
	B*L	1	0.115	0.73	0.3975
	B*S	2	1.142	3.64	0.0369 #
	L*S	2	0.228	0.73	0.4903
	B*L*S	2	0.496	1.58	0.2200
	Error	34	5.328		
Deflection reduction midspan between cracks	Replications	1	0.044	0.19	0.6671
	DT	1	4.839	20.51	0.0001 #
	Barsize, B	1	0.760	3.22	0.0816
	Length, L	1	0.203	0.86	0.3604
	Spacing, S	2	4.919	10.42	0.0003 #
	B*L	1	0.433	1.84	0.1843
	B*S	2	1.114	2.36	0.1097
	L*S	2	1.159	2.46	0.1008
	B*L*S	2	1.254	2.66	0.0847
	Error	34	8.022		

1 DF = Degree of freedom of source of variation.
2 PR > F is the significance probability value associated with the F value.
3 DT = Temperature differential estimate of pavement.
Factors and interactions significant at the 5 percent level.

the relative effectiveness of the shoulder joint designs are given in Table 7.14. The rankings agree with the result of the analysis of variance on the effect of tie bar size, length, and spacing on the reduction in deflection at the edge of the pavement. Sections with the smaller tie bar spacings rank higher, and so do sections tied with #4 bars. For a given tie bar size and spacing, the sections with 15-inch-long tie bars rank higher than those with the longer 25-inch tie bars.

TABLE 7.10(A). SUMMARY OF LOAD TRANSFER EFFICIENCY, PERCENT, AT CRACKS AT THE EDGE OF THE FLATONIA CRCP SECTION FOR TWO SHOULDER CONDITIONS — PROCEDURE A

Tie Bar Size	Tie Bar Length (in.)	Tie Bar Spacing (in.) 24		36		48	
#4	15	99[1]	99[5]	98	99	97	98
		98[2]	99	98	98	97	98
		95[3]	98	97	99	94	87
		98[4]	97	98	99	96	97
	25	96	99	98	*	97	95
		96	99	97	*	98	95
		99	99	95	*	97	99
		99	99	96	*	98	98
#5	15	99	97	98	98	97	93
		98	99	98	97	96	94
		99	96	98	99	95	99
		97	99	98	98	95	89
	25	99	99	98	98	99	95
		97	99	98	98	92	97
		97	99	100	96	86	92
		96	98	99	99	86	99

[1] Load transfer efficiency at first crack in the morning for pavement with flexible shoulder.
[2] Load transfer efficiency at first crack in the afternoon for pavement with flexible shoulder.
[3] Load transfer efficiency at first crack in the morning for pavement with concrete shoulder.
[4] Load transfer efficiency at first crack in the afternoon for pavement with concrete shoulder.
[5] Replicate measurements at second crack in subsection.

TABLE 7.10(B). SUMMARY OF LOAD TRANSFER EFFICIENCY, PERCENT, AT CRACKS AT THE EDGE OF THE FLATONIA CRCP SECTION FOR TWO SHOULDER CONDITIONS — PROCEDURE B

Tie Bar Size	Tie Bar Length (in.)	Tie Bar Spacing (in.) 24		36		48	
#4	15	89[1]	90[5]	90	89	86	86
		90[2]	90	90	88	86	86
		93[3]	93	92	92	89	92
		93[4]	93	91	93	90	89
	25	89	91	90	*	86	84
		89	91	90	*	85	84
		93	93	93	*	88	88
		93	93	92	*	89	88
#5	15	90	91	90	89	86	85
		90	91	89	90	84	84
		94	93	92	93	88	88
		93	94	92	92	86	91
	25	89	89	88	87	85	86
		89	89	87	87	85	85
		91	92	88	90	86	88
		91	94	88	89	86	87

[1] Load transfer efficiency at first crack in the morning for pavement with flexible shoulder.
[2] Load transfer efficiency at first crack in the afternoon for pavement with flexible shoulder.
[3] Load transfer efficiency at first crack in the morning for pavement with concrete shoulder.
[4] Load transfer efficiency at first crack in the afternoon for pavement with concrete shoulder.
[5] Replicate measurements at second crack in subsection.

TABLE 7.11(A). AVERAGE REDUCTION IN PAVEMENT DEFLECTION, MILS, UPSTREAM OF TRANSVERSE CRACKS AT THE EDGE OF THE FLATONIA CRCP SECTION

Tie Bar Size	Tie Bar Length (in.)	Tie Bar Spacing (in.) 24	36	48
#4	15	3.96 [1] 4.03 [2] 4.41 [3] 4.69 [4]	3.22 3.82 5.63 5.08	2.11 2.08 2.19 1.76
	25	3.64 4.09 4.86 4.58	3.49 3.73 3.92 *	1.04 1.54 1.28 0.99
#5	15	3.48 3.51 3.20 3.78	2.52 3.23 3.67 4.86	1.34 1.45 0.33 0.36
	25	2.72 3.35 3.80 3.38	2.32 2.93 3.19 2.38	2.05 1.18 1.96 1.96

[1] Deflection reduction at first crack in AM (morning) period.
[2] Deflection reduction at first crack in PM (afternoon) period.
[3] Deflection reduction at second crack in AM period.
[4] Deflection reduction at second crack in PM period.

TABLE 7.11(B). AVERAGE REDUCTION IN PAVEMENT DEFLECTION, MILS, DOWNSTREAM OF TRANSVERSE CRACKS AT THE EDGE OF THE FLATONIA CRCP SECTION

Tie Bar Size	Tie Bar Length (in.)	Tie Bar Spacing (in.) 24	36	48
#4	15	3.44 [1] 4.11 [2] 4.50 [3] 4.80 [4]	3.66 4.06 4.58 4.54	2.02 2.30 2.09 1.61
	25	4.01 4.52 4.58 4.54	3.48 3.87 3.77 *	1.02 1.54 1.43 1.22
#5	15	3.41 3.48 3.49 3.92	2.48 3.07 3.59 5.11	1.24 1.43 0.68 0.63
	25	2.63 3.20 3.85 3.63	2.26 2.75 3.09 2.23	2.05 1.18 1.96 1.96

[1] Deflection reduction at first crack in AM (morning) period.
[2] Deflection reduction at first crack in PM (afternoon) period.
[3] Deflection reduction at second crack in AM period.
[4] Deflection reduction at second crack in PM period.

TABLE 7.11(C). AVERAGE REDUCTION IN PAVEMENT DEFLECTION, MILS, MIDSPAN OF TRANSVERSE CRACKS AT THE EDGE OF THE FLATONIA CRCP SECTION

Tie Bar Size	Tie Bar Length (in.)	Tie Bar Spacing (in.) 24	36	48
#4	15	3.66 [1] 3.86 [2] 4.48 [3] 4.32 [4]	3.36 3.96 5.21 5.00	2.15 2.30 1.93 1.50
	25	4.04 4.84 4.30 4.50	3.35 3.73 3.89 *	1.21 1.79 1.55 1.46
#5	15	3.18 3.47 3.94 4.29	2.97 3.23 3.53 5.21	0.85 1.08 0.81 0.91
	25	2.57 3.01 3.74 3.79	2.69 2.92 2.03 1.48	1.99 2.06 0.96 1.16

[1] Deflection reduction at first crack in AM (morning) period.
[2] Deflection reduction at first crack in PM (afternoon) period.
[3] Deflection reduction at second crack in AM period.
[4] Deflection reduction at second crack in PM period.

TABLE 7.12. ANOVA RESULTS ON EFFECT OF TIE BAR DIMENSIONS ON PAVEMENT DEFLECTION REDUCTION AT THE EDGE OF THE FLATONIA CRCP

Dependent Variable	Source of Variation	DF[1]	Sum of Squares	F value	PR > F[2]
	Replications	1	1.415	4.77	0.0362
	DT[3]	1	0.057	0.19	0.6646
	Barsize, B	1	5.664	19.10	0.0001 #
	Length, L	1	1.106	3.73	0.0621
Deflection reduction	Spacing, S	2	53.174	89.65	0.0001 #
upstream of cracks	B*L	1	0.658	2.22	0.1460
	B*S	2	1.457	2.46	0.1013
	L*S	2	1.782	3.00	0.0633
	B*L*S	2	1.827	3.08	0.0594
	Error	33	9.787		
	Replications	1	1.647	7.02	0.0123
	DT	1	0.320	1.36	0.2512
	Barsize, B	1	6.281	26.77	0.0001 #
	Length, L	1	1.474	6.28	0.0173 #
Deflection reduction	Spacing, S	2	53.99	115.08	0.0001 #
downstream of cracks	B*L	1	0.170	0.73	0.4004
	B*S	2	1.767	3.77	0.0337 #
	L*S	2	2.220	4.73	0.0156 #
	B*L*S	2	1.404	2.99	0.0640
	Error	33	7.742		
	Replications	1	0.432	1.45	0.2376
	DT	1	0.448	1.50	0.2295
	Barsize, B	1	6.375	21.35	0.0001 #
	Length, L	1	1.863	6.24	0.0177 #
Deflection reduction	Spacing, S	2	52.939	88.66	0.0001 #
midspan between cracks	B*L	1	0.000	0.00	1.0000
	B*S	2	0.987	1.65	0.2070
	L*S	2	3.629	6.08	0.0057 #
	B*L*S	2	1.846	3.09	0.0588
	Error	33	9.852		

1 DF = Degree of freedom of source of variation.
2 PR > F is the significance probability value associated with the F value.
3 DT = Temperature differential estimate of pavement.
Factors and interactions significant at the 5 percent level.

TABLE 7.13. ANOVA RESULTS ON EFFECT OF SECTION TREATMENT COMBINATIONS ON PAVEMENT DEFLECTION REDUCTION AT THE EDGE OF FLATONIA CRCP

Dependent Variable	Source of Variation	DF[1]	Sum of Squares	F value	PR > F[2]
Deflection reduction upstream of cracks	Replications	1	1.338	3.87	0.0598
	DT[3]	1	0.112	0.32	0.5746
	Treatments, T	13	85.618	19.06	0.0001
	T*DT	13	1.398	0.31	0.9844
	Error	26	8.985		
Deflection reduction downstream of cracks	Replications	1	1.558	5.56	0.0262
	DT[3]	1	0.302	1.08	0.3086
	Treatments, T	13	88.454	24.28	0.0001
	T*DT	13	1.479	0.41	0.9542
	Error	26	7.286		
Deflection reduction midspan between cracks	Replications	1	0.143	0.41	0.5287
	DT[3]	1	0.379	1.08	0.3085
	Treatments, T	13	92.411	20.21	0.0001
	T*DT	13	1.452	0.32	0.9830
	Error	26	9.142		

[1] DF = Degree of freedom of source of variation.
[2] PR > F is the significance probability value associated with the F value.
[3] DT = Temperature differential estimate of pavement.

TABLE 7.14. RANKING OF THE JOINT DESIGNS OF FLATONIA CRCP SECTION BASED ON THEIR RELATIVE EFFECTIVENESS IN REDUCING PAVEMENT EDGE DEFLECTIONS

Tie Bar Size	Tie Bar Length (in.)	Tie Bar Spacing (in.) 24	36	48
#4	15	2[a] b, c, d 3, 3, 3	5 1, 1, 2	9 9, 9, 9
	25	4 2, 2, 1	7 4, 4, 6	11 12, 12, 11
#5	15	3 6, 5, 5	6 5, 6, 4	10 14, 13, 14
	25	1 7, 7, 7	8 8, 8, 8	12 10, 10, 10

[a] Section numbers.
[b] Ranking with upstream deflection as dependent variable.
[c] Ranking with downstream deflection as dependent variable.
[d] Ranking with midspan deflection as dependent variable.

Note: Using the same nomenclature in table, section 13 ranked 13, 14, and 13; and section 14 ranked 11, 11, and 12.

CHAPTER 8. DISCUSSION OF STUDY RESULTS

The major topics addressed in this study included (1) use of the FWD for assessing the load transfer efficiency at joints and cracks in rigid pavements; (2) the benefits of rigid shoulders attached to existing rigid pavements in replacement of flexible type shoulders; and (3) an evaluation of a number of longitudinal shoulder joint types, including some of the factors influencing their effectiveness. This chapter presents the central findings of the study.

ASSESSING LOAD TRANSFER EFFICIENCY IN PAVEMENTS

In both the laboratory tests on the research pavement facility and on field test sections, a procedure, developed for assessing load transfer efficiency across joints and cracks in PCC pavements using the FWD, was evaluated. Results from the study indicate that the procedure can be used to measure load transfer efficiency at a joint or crack in a rigid pavement and that it compares favorably with other methods. The procedure adequately showed changes in load transfer efficiency of joints resulting from increased pressure owing to thermal expansion. The results obtained indicate that environmental factors have a significant effect on load transfer efficiency for open joint conditions. For any particular joint gap, increases in percent load transfer efficiency on the order of 20 to 40 percent were observed for daily temperature differential variation from -10 to +15°F on a research pavement facility. Evidence was also obtained showing that at a particular joint opening, seasonal variations resulted in changes in load transfer efficiency. In tests with the joint closed (to simulate conditions at a crack or a tight joint), daily and seasonal environmental changes observed at the site did not significantly affect load transfer efficiency.

The results obtained from the JRCP and CRCP field sections were in agreement with the laboratory findings. On JRCP sections the FWD was successfully used to characterize load transfer efficiency across transverse joints. The results obtained indicate that the FWD can be effectively used to determine the condition of joints in pavements for evaluation purposes, and that such use compares favorably with visual condition surveys. Since visual surveys to determine the condition of joints take a much longer time than evaluations using the FWD, larger-scale evaluations of such joints in JRCP can be conducted with the FWD. The FWD was also successfully used to detect the change in load transfer efficiency at transverse joints in pavements in service because of temperature differential variation. On the sections tested, a direct relationship between load transfer efficiency at the transverse joints and temperature differential was determined. No such relationship was detected at transverse cracks in either JRCP or CRCP sections tested. As on the laboratory research pavement, load transfer efficiency was a direct function of the gap at a joint or crack. On the relatively good sections with most cracks still held tight by reinforcement, load transfer efficiency across the cracks was for the most part constant, even though the actual pavement load-carrying capacity as indicated by the deflection of the pavement was reduced.

Results from the field tests indicate that longitudinal joint efficiency profiles along pavements determined from FWD measurements can be used to stratify a pavement section into like subsections and to detect the location of sections with reduced and low efficiencies. Such profiles can be obtained for longitudinal shoulder joints as well as for center line joints between adjacent travel lanes. Profile histories of the longitudinal joint efficiency can be used in pavement evaluation programs to show joint efficiencies below required levels and to reflect the need for maintenance or rehabilitative measures for the upkeep of the pavements. Similar methods can also be used for transverse joints and cracks in rigid pavements to determine when and where maintenance and rehabilitation are needed to restore load transfer across such discontinuities.

BENEFITS OF RIGID SHOULDERS

The deflection measurements on in-service rigid pavement sections show the significant beneficial effects obtained when rigid shoulders are attached to existing PCC pavements. In the three cases tested, rigid shoulders were found to decrease pavement deflection in the outer wheel path of the outer lane and at the edge of pavements when they were constructed to replace flexible base type shoulders. On the CRCP sections tested, pavement deflections in the outer wheel path of the outer lane of the pavements were significantly decreased—on the order of 10 to 45 percent—when a rigid shoulder was added. Edge deflections were reduced by as much as 40 to 50 percent. Pavement deflections at the edge were in effect brought down to levels comparable to those of deflections measured at the same locations along the longitudinal center line joints of the pavements. According to Taute (Ref 35), pavement life in terms of the maximum number of applications of equivalent 18-kip single axle loads (ESAL) required to cause fatigue failure can be defined by equations which show that, for a given flexural strength of pavement quality concrete, the number of ESAL applications to failure is inversely proportional to the critical tensile stress in the bottom fibers of the concrete layer of a pavement. Clearly, for the pavements in question, the reduction in deflection of the existing pavement will reduce the critical tensile stress at the bottom of the pavements resulting from future load applications. Cumulatively, this reduction in the critical tensile stress will result in an increase in the life of the pavements. The up-to-50-percent immediate reduction in deflection due to the addition of PCC shoulders to the pavements will, theoretically, have a significant and beneficial effect on the pavements by extending their lives in terms of the number of ESAL applications to failure. Ultimately, the extended lives of the pavements must be proved by observation of field performance; therefore, these pavements should be observed over the next ten to twenty years.

EVALUATION OF SHOULDER JOINT TYPES

Deflection measurements were made for use in an evaluation of certain shoulder types and the effect of tie bar size, length, and spacing on the effectiveness of joint designs used on in-service pavements. Comparisons were made between the ordinary tied joint, the inverted-tee joint, and the tied inverted-tee joint used at shoulder joints. In statistical analysis of deflection measurements in the outer wheel path and at the edge of a pavement, which border the critical area of a pavement as far as load-carrying capacity is concerned, no significant differences were found between these three types of shoulder joints tested. The effects of the different combinations of No. 4 and No. 5 tie bar size, 15 or 25 inches long, and at 24, 36, or 48-inch spacings were also investigated. Tie bar spacings of 24 and 36 inches were found to be clearly more effective than spacings of 48 inches, but the difference in the effectiveness between a 24 and a 36-inch spacing was not statistically significant. No significant difference could be determined between the two tie bar sizes used. Although the effect of tie bar length on pavement deflection reduction was significant, the more effective length could not be determined from the analysis. A comparison of the sections indicated that, if the pavement design and construction practices are equal, the 15-inch-long tie bars are just as effective as the 25-inch-long tie bars.

CHAPTER 9. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

SUMMARY

In the evaluation of rigid pavements, the assessment of load transfer efficiency of joints and cracks is important, since the performance of this type of pavement is dependent on structural continuity and the resulting load-carrying capacity of the pavement. In this study, procedures for evaluating load transfer efficiency across joints and cracks using the Falling Weight Deflectometer (FWD) were developed and verified. A number of preliminary objectives were addressed, including (1) an evaluation of the repeatability of the FWD procedures; (2) documentation of the effects of daily and seasonal environmental variations on the curling and warping of rigid pavements; and (3) an evaluation of the effects of environmental variations and of voids on FWD deflections. Conclusions concerning these subjects are presented based on tests on a research pavement facility.

The other objective of this study was to apply the procedures to assess the load transfer efficiency of transverse joints and cracks in pavements in service. The procedures were shown to be adequate for field evaluation of load transfer efficiency of transverse joints and cracks to determine rehabilitation and maintenance needs. The procedures can be used in evaluating the efficiency of longitudinal joints, especially rigid pavement shoulder joints. In an extension of the study, the beneficial effects of concrete shoulders attached to existing rigid pavements were evaluated and documented. A comparison was also made to determine the most effective type of joint for use at a shoulder joint of a rigid pavement: a tied joint, an inverted-tee joint, or a tied inverted-tee joint. Last, the influence of tie bar size, length, and spacing on the effectiveness of longitudinal shoulder joints was determined.

CONCLUSIONS

The major conclusions based on the findings of this study are presented in the following sections.

(1) The FWD is a repeatable and reliable nondestructive testing device for rigid pavement evaluation. The repeatability for measurements is quite good with a coefficient of variation of not more than 6 percent. The variability in pavement response recorded with the FWD, however, increases with decreasing applied load. It is recommended that FWD deflection data be collected for load levels of 9,000 lbs or larger.

(2) The following important conclusions have been reached concerning the effects of environmental variations on curling and warping and on pavement response:

(a) Temperature gradients from top to bottom of rigid pavements in Texas can range from -15 to +25°F and can lead to the curling of pavements.

(b) Vertical movements from one extreme to the other, measured at the unrestrained slab corners of a 10-inch-thick research JRCP and due to such variations in temperature gradient, were on the order of 100 mils.

(c) The maximum upward vertical movement at unrestrained slab corners occurs in the early morning hours between 3:00 and 7:00 AM, and downward vertical movements reach a maximum in the afternoon between 1:00 and 5:00 PM in Texas.

(c) The exact times of the occurrence of the maximum values vary and depend on the prevailing weather conditions. The maximum values were concurrent with the maximum positive and negative temperature differentials in the pavements.

(d) The extreme vertical movements at the slab corners occur in the warmer summer months when temperature fluctuations are at a maximum.

(e) A downward corner movement of 70 mils and an upward corner movement of 38 mils in July 1986, relative to the slab profile at zero temperature differential, were the maximum recorded on the 10-inch-thick research pavement. Slab movements were not always symmetrical.

(f) At specific locations on the research pavement, and for the same temperature differential, movements due to seasonal environmental variations were on the order of 20 mils.

(g) Results of FWD deflection tests on the research pavement indicate an inverse relationship between temperature differential and pavement deflection at the edge and along the dowelled transverse joint.

(h) As the temperature differential increases, there is more contact with the foundation, as well as a tightening-up of the joint. On typical days, the coefficient of variation—in deflection measurements at a specific location on the research pavement as a result of temperature differential variation—was between 8 and 15 percent.

(i) In general, higher deflections were recorded along the edge and doweled transverse joint of the research pavement in the summer than in the winter. A softening of the asphalt-stabilized base of the pavement in the summer is believed to be the cause.

(j) The effect of temperature differential on pavement response in the field was not found to be statistically significant.

(3) For the detection of voids underneath rigid pavements, it is concluded that:

(a) For pavement thicknesses of 8 to 10 inches on stabilized bases, the variation in pavement response due to the presence of a void is low.

(b) Current FWD methods do an insufficient job of detecting voids beneath rigid pavements 8 or more inches thick.

(4) Evaluation of the load transfer efficiency determination procedure developed indicates a favorable comparison with other methods. The following important conclusions were reached:

(a) The load transfer efficiency determination procedure developed is suitable for the evaluation of joints and cracks in PCC pavements in the field, and FWD deflection measurements at cracks and joints taken as part of pavement evaluations can be used to determine the efficiency and performance of joints and cracks.

(b) Joint efficiency increases with an increase in vertical temperature differential in pavements. No change was noted across cracks in rigid pavements.

(c) Slab temperature data for the development of joint load transfer efficiency versus temperature differential correction curves for pavements evaluated should be part of any evaluation procedure which involves the evaluation of joints.

(d) The load transfer efficiency determination procedure developed is also useful for the evaluation of longitudinal joint efficiency.

There are many factors which influence the performance of rigid shoulders, including adequate base support, adequate drainage, and extensive heavy truck traffic. For the parameters involved in these data, the following conclusions are made.

(5) Field evaluations document that the attachment of a rigid shoulder to an existing rigid pavement in replacement of a flexible shoulder can reduce pavement edge deflections by up to 50 percent.

(6) A field comparison of inverted-tee shoulder joints, ordinary tied shoulder joints, and tied inverted-tee joints indicates that an inverted-tee joint is no more effective in reducing pavement edge deflections than an ordinary tied joint.

(7) A field investigation of the effect of tie bar dimensions on the effectiveness of a tied shoulder joint gave the following results:

(a) Increasing tie bar size from No. 4 bars to No. 5 bars had no significant effect on pavement edge deflections.

(b) Increasing tie bar length from 15 to 25 inches had no significant effect on pavement edge deflections.

(c) Tie bar spacings of 24 and 36 inches were not significantly different with respect to their influence on reduction in pavement edge deflection. Both were significantly better at reducing pavement edge deflections than a spacing of 48 inches.

NOTE: Conclusion (7) is based on short-term data. Recommendation (4), below, proposes continued monitoring of these shoulders to obtain long-term results.

RECOMMENDATIONS

Based on the findings of this study, the following recommendations are made:

(1) FWD deflection measurements at cracks and joints should be incorporated into rigid pavement evaluations to determine the efficiency and performance of joints and cracks. The load transfer efficiency of the joints and cracks in new pavements should be determined to establish baseline values for future evaluations to determine maintenance or rehabilitation needs.

(2) Studies should be conducted to determine more precisely the relationship between pavement temperature differential and load transfer efficiency at joints and cracks vis-a-vis rigid pavement response. Investigations into better and more precise pavement temperature measuring devices and techniques are necessary to ensure any appreciable success in this area.

(3) When rigid pavements are evaluated for rehabilitation, the replacement of flexible shoulders with rigid shoulders should be considered as a means of improving performance and extending the service life of the pavement.

(4) The tied-shoulder test section near Flatonia on IH-10 should continue to be monitored to evaluate the performance of the different shoulder-joint designs over time.

(5) From the findings of the study it is recommended that use of the inverted tee-type of shoulder joint be suspended in favor of use of tied joints.

REFERENCES

1. Torres, Francisco, and B. Frank McCullough, "Void Detection and Grouting Process," Research Report 249-3, Center for Transportation Research, The University of Texas at Austin, April 1983.
2. Uddin, Waheed, A. H. Meyer, W. Ronald Hudson, and K. H. Stokoe II, "A Structural Evaluation Methodology for Pavements Based on Dynamic Deflections," Research Report 387-1, Center for Transportation Research, The University of Texas at Austin, August 1985.
3. Ricci, Eduardo A., A. H. Meyer, W. R. Hudson, and K. H. Stokoe II, "The Falling Weight Deflectometer for Nondestructive Evaluation of Rigid Pavements," Research Report 387-3F, Center for Transportation Research, The University of Texas at Austin, November 1985.
4. American Association of State Highway and Transportation Officials, *AASHTO Guide for Design of Pavement Structures*, Washington, D. C., 1986.
5. Highway Research Board, *Design, Construction, and Maintenance of PCC Pavement Joints*, NCHRP Synthesis of Highway Practice 19, Washington, D. C., 1973.
6. Yoder, E. J., and M. W. Witczak, *Principles of Pavement Design*, Second Edition, John Wiley and Sons, Inc., New York, 1975.
7. Teller, L. W., and Earl C. Sutherland, "The Structural Design of Concrete Pavements: Part 4 – The Study of the Structural Action of Several Types of Transverse and Longitudinal Joint Designs," *Public Roads*, Vol 17, Nos. 7 and 8, September and October 1936, pp 143-171 and pp 175-192.
8. Colley, B. E., and H. A. Humphrey, "Aggregate Interlock at Joints in Concrete Pavements," *Highway Research Record No. 189*, Highway Research Board, Washington, D. C., 1967, pp 1-18.
9. Teller, Leslie W., and Harry D. Cashell, "Performance of Dowelled Joints Under Repetitive Loading," *HRB Bulletin 217*, Highway Research Board, Washington, D. C., 1958, pp 8-43.
10. Niu, H. P., and Gerald Pickett, "The Effect of Degree of Continuity Across a Void or Crack on Performance of Concrete Pavements," *Highway Research Record No. 191*, Highway Research Board, Washington, D. C., 1969, pp 186-193.
11. Ball, C. G., and L. D. Childs, *Tests of Joints for Concrete Pavements*, Research and Development Bulletin RD026.01P, Portland Cement Association, Skokie, Illinois, 1975.
12. Westergaard, H. M., "Spacing of Dowels," *Proceedings*, 8th Annual Meeting, Highway Research Board, Washington, D. C., 1928, pp 154-158.
13. Bradbury, R. D., "Design of Joints in Concrete Pavements," *Proceedings*, 12th Annual Meeting, Highway Research Board, Washington, D. C., December 1932, pp 105-137.
14. Friberg, Bengt F., "Design of Dowels in Transverse Joints of Concrete Pavements," *Proceedings*, American Society of Civil Engineers, November 1938, pp 1809-1828.
15. Friberg, Bengt, "Load and Deflection Characteristics of Dowels in Transverse Joints of Concrete Pavements," *Proceedings*, 18th Annual Meeting, Highway Research Board, Washington, D. C., November 1938, pp 140-154.
16. Westergaard, H. M., "New Formulas for Stresses in Concrete Pavements of Airfields," *Proceedings*, American Society of Civil Engineers, Vol 73, No. 1, January 1947.
17. Finney, Edwin A., Better Concrete Pavement Serviceability, ACI Monograph No. 7, Detroit, Michigan, 1973.
18. Al-Rashid, Nasser I., Clyde E. Lee, and William P. Dawkins, "A Theoretical and Experimental Study of Dynamic Highway Loading," Research Report 108-1F, Center for Highway Research, The University of Texas at Austin, May 1972.
19. Highway Research Board, *The AASHO Road Test: Report 5 – Pavement Research*, Special Report 61E, Washington, D. C., 1962.
20. Bohn, A., P. Ullidtz, R. Stustad, and A. Sørenson, "Danish Experiments with the French Falling Weight Deflectometer," *Proceedings*, Third International Conference on the Structural Design of Asphalt Pavements, Vol 1, pp 1119-1128, September 1972.
21. Morales-Valentin, G. E., A. H. Meyer, and W. R. Hudson, "Temperature Differential Effect on Falling Weight Deflectometer Deflections Used for Structural Evaluation of Rigid Pavements," Research Report 460-1, Center for Transportation Research, The University of Texas at Austin, February 1987.

22. Shahin, M. Y., "Use of the Falling Weight Deflectometer for the Nondestructive Testing of Jointed Concrete Airfield Pavements," *Proceedings*, Third International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, April 1985, pp 549-556.

23. Foxworth, P. T., and M. I. Darter, "Preliminary Concepts for FWD Testing and Evaluation of Rigid Pavements," *Transportation Research Record No. 1070*, Transportation Research Board, Washington, D. C., 1986, pp 63-76.

24. White, R., W. Ronald Hudson, Alvin H. Meyer, and K. H. Stokoe II, "Design and Construction of a Rigid Pavement Research Facility," Research Report 355-1, Center for Transportation Research, The University of Texas at Austin, September 1984.

25. Meyer, A. H., W. R. Hudson, and K. H. Stokoe II, "Instrumented Rigid Pavement for Multipurpose Research," Research Report 355-2F, Center for Transportation Research, The University of Texas at Austin, November 1986.

26. State Department of Highways and Public Transportation, *Texas State Department of Highways and Public Transportation Standard Specifications for Construction of Highways, Streets, and Bridges*, Texas, September 1982.

27. Westergaard, H. M., "Analysis of Stresses in Concrete Pavements Due to Variations of Temperature," *Proceedings*, 16th Annual Meeting, Highway Research Board, Washington, D. C., December 1926, pp 201-215.

28. Janssen, Donald J., "Moisture in Portland Cement Concrete," *Transportation Research Record No. 1121*, Transportation Research Board, Washington, D. C., 1987, pp 40-44.

29. Sharpe, G. W., M. Anderson, R. C. Deen, and H. F. Southgate, "Nondestructive Evaluation of Rigid Pavements Using Road Rater Deflections," *Proceedings*, Third International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, 1985.

30. McCullough, B. F., and Harvey J. Treybig, "Determining the Relationship of Variables in Deflection of Continuously Reinforced Concrete Pavement," Technical Report No. 46-4, Texas Highway Department, Highway Design Division, Austin, Texas, August 1965.

31. Uddin, Waheed, Soheil Nazarian, W. Ronald Hudson, Alvin H. Meyer, and Kenneth H. Stokoe II, "Investigations into Dynaflect Deflections in Relation to Location/Temperature Parameters and Insitu Material Characterization of Rigid Pavements," Research Report 256-5, Center for Transportation Research, The University of Texas at Austin, December 1983.

32. Neter, John, William Wasserman, and Michael H. Kutner, *Applied Linear Statistical Models – Regression, Analysis of Variance and Experimental Designs*, 2nd ed., Irwin, Inc., Homewood, Illinois, 1985.

33. Anderson, Virgil L., and Robert A. McLean, *Design of Experiments – A Realistic Approach*, Marcel Dekker, Inc., New York, New York, 1974.

34. Snedecor, George W., and William G. Cochran, *Statistical Methods*, 7th ed., The Iowa State University Press, Ames, Iowa, 1982.

35. Taute, Arthur, B. F. McCullough, and W. R. Hudson, "Improvements to Material Characterization and Fatigue Life Prediction Methods of the Texas Rigid Pavement Overlay Design Procedure," Research Report 249-1, Center for Transportation Research, The University of Texas at Austin, March 1981.

36. Willett, T. O., "Paved Shoulders," FHWA Technical Advisory T 5040.29, Washington, D. C., February 2, 1990.