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16. Abstract

The sealing and resealing of joints and cracks in PCC, HMA, and composite pavements is assumed to be an important component of pavement maintenance. Recently this practice has been challenged by research indicating that sealing may not be cost-effective. The Indiana Department of Transportation currently spends approximately four million dollars annually to perform joint/crack sealing. The primary objective of the research presented in this report is to investigate the cost-effectiveness of joint/crack sealing in relation to pavement performance. The results of a mail survey showed that most states, including Indiana, do not have quantitative justification for sealing policies, nor do they know the cost-effectiveness of the operations. Based on the experimental design for this research, nineteen test sites were selected in Indiana, each site having one sealed section and one unsealed section. Collected data including falling weight deflectometer measurements, pavement roughness, visual condition surveys, and core samples were used to evaluate the pavement performance between sealed and unsealed sections. A three-dimensional finite element pavement model was developed to evaluate the test location effect on the load transfer measurements. The temperature effect was evaluated by statistical analyses and a temperature correction factor for deflections on asphalt pavement is provided. A statistical model was developed to compare the pavement performance between sealed and unsealed sections for three pavement types, PCC, HMA and composite. The results indicated that there appears to be no significant differences between the performance of sealed and unsealed sections regardless of pavement type, drainage condition and road classification.

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INDOT Research

TECHNICAL *Summary*

Technology Transfer and Project Implementation Information

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Initial Study for Cost-Effectiveness of Joint/Crack Sealing

Introduction

The sealing and resealing of joints and cracks in concrete (PCC), asphalt (HMA) and composite pavements is assumed to be an important component of pavement maintenance and restoration and is one of the more commonly performed pavement maintenance activities. If performed effectively and in a timely manner, it is accepted that joint and crack sealing will help to reduce pavement deterioration and thereby prolong pavement life. One objective of the sealing is to reduce the amount of moisture that can infiltrate a pavement structure, thereby reducing moisture-related distresses. The second objective is to prevent the intrusion of incompressible materials into joints and cracks. It is believed that this will eliminate clogging thereby reducing harmful contraction and expansion pressure which may lead to further deterioration of joints and cracks. Therefore, pressure-related distresses, such as pumping and loss of support in PCC pavements and stripping in HMA pavements, are prevented.

In the past several years, the cost-effectiveness of crack and joint sealing has been questioned, at least in some applications. Additionally, studies that support a clear, quantitative defense that the practice is cost-effective appear to be few in number and limited in scope. Research conducted by the Wisconsin Department of Transportation (WDOT) on jointed concrete pavements over an extended period of time led that agency to discontinue joint sealing of PCC pavements. The agency claims to have saved 6 million dollars annually with no loss in pavement performance and with increased customer safety and convenience.

INDOT currently spends approximately 4 million dollars annually to accomplish crack and joint sealing. About one-half of this amount is allocated for sealing old pavements that are selected through a subjective process. There is no quantitative evidence to justify this expenditure. The sealing operations are conducted because industries assumed that the benefits of sealing weigh out the costs.

The primary objective of this research was to investigate the cost-effectiveness of joint/crack sealing in relation to pavement performance. This study focused on two specific questions:

1. Does joint/crack sealing improve the service life or serviceability performance of pavements (performance); and
2. If sealing does improve performance, is it cost-effective and in what situations?

These questions can only be effectively addressed through a rigorous review of the literature, a survey of practice, and finally the design and analysis of a field experiment. The potential outcomes will have immediate application to INDOT operations. It is hypothesized that the cost effectiveness of sealing is conditional and the final results of this study will identify those applications for which it is cost-effective. The results will then be formulated into a set of guidelines for implementation by maintenance and design personnel. The potential savings associated with this research could very well amount to a significant portion of the 4 million dollars now spent annually on joint and crack sealing by INDOT.

The study was divided into two phases. The first phase was a literature review and a synthesis of the current practice, intended to form a basis for determining whether or not further research was needed to determine the cost effectiveness of crack/joint sealing in Indiana. Based on the results of Phase I, Phase II involved an elaborate design of experiment, implementation of the design at in-service pavements, and monitoring the experimental sections.

Findings

The literature review considered over one hundred potential references and revealed that only eighteen specifically discussed cost-effectiveness of joint/crack sealing. Of these only four provided useful quantitative information related to the cost-effectiveness of joint/crack sealing. The statistical analysis of the practice survey results also showed that most of states, including Indiana, do not have quantitative justification for sealing policies nor do they know the cost-effectiveness of these operations. The literature review, as well as the survey of practice clearly indicated the need to develop and conduct a field study to answer the question of whether joint/crack sealing is cost effective in Indiana.

An experimental design for a field study was developed through a series of meetings with pavement technologists and a statistician. Nineteen test sites were selected under the principle that the chosen test sites must conform as closely as possible to the proposed experimental design. Both sealed and unsealed sections in each test site were rigorously maintained throughout the duration of the approximately two-year performance monitoring period. Pavement performance was monitored periodically during the field study. Performance response variables include International Roughness Index (IRI), Falling Weight Deflectometer (FWD), load transfer, individual pavement stress (condition survey), and physical and mechanical properties of pavement cores. A statistical model was developed to compare the pavement performance between sealed and unsealed sections for three pavement types, concrete, asphalt and composite. The results from two years of FWD measurements indicates that there appears to be no significant difference between the performance of sealed and unsealed sections, regardless of pavement type, drainage condition and road classification.

A three dimension finite element pavement model is developed to evaluate the effect that FWD test location has on load transfer measurements. This model consists of four dowel jointed concrete slabs supported by base and subgrade. Dowel bars are simulated using 3D mesh with one fixed end one lubricated end. To simply the simulation, it is assumed that all materials used in the model are elastic, and the FWD load is static. FWD tests are simulated at different locations to evaluate the effect test location has on the load transfer measurement. Based on both simulation and field measurements, it was concluded that FWD test location has no significant effect on FWD deflection.

Since pavement temperature may significantly affect the FWD deflection for both PCC and asphalt pavement, the effect of temperature is evaluated by statistical analyses based on a sample of FWD deflections collected at different temperatures at five research test sites. It is concluded that no temperature correction is recommended for FWD deflections and load transfer measurements for PCC pavement. A correction factor can be used to properly correct the FWD deflection within a certain temperature range for asphalt pavements. No correction is considered for load transfer across an asphalt crack.

Implementation

It is highly recommended that this research be extended for a ten years long period of time. Currently available researches indicate that there is no significant difference between the performance of sealed and unsealed sections, regardless of pavement type, drainage condition and road classification. However, it should be noted that only two years of data has been collected. No cost-effectiveness analysis for joint/crack sealing can be conducted with these limited pavement performance data and statistical analysis. The monitoring of the pavement test sites needs to be continued so that the long-term performance can be measured and additional conclusions can be drawn regarding the cost-effectiveness of joint/crack sealing.

It is also recommended that a more comprehensive study for temperature effect on FWD deflection be conducted. The temperature correction factor for FWD deflection on asphalt pavement is provided in this research. However, no pavement temperature gradient research is available in Indiana, and the statistical analyses of temperature correction factors are based on limited data collected from five test sites. Further research will be able to develop prediction models for pavement temperature in Indiana, and provide more reliable temperature correction factors for FWD measurements.

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FINAL REPORT

FHWA/IN/JTRP-2003/11

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CHAPTER 1 INTRODUCTION

1.1 Background

The sealing and resealing of joints and cracks in Portland cement concrete (PCC), hot-mix asphalt (HMA) and composite pavements is assumed to be an important component of pavement maintenance and restoration. If performed effectively and in a timely manner, it is accepted that joint and crack sealing will help to reduce pavement deterioration and thereby prolong pavement life. Joint and crack sealing is one of the more commonly performed pavement maintenance activities. One objective of the sealing is to reduce the amount of moisture that can infiltrate a pavement structure, thereby reducing moisture-related distresses, such as pumping, loss of support, faulting, and corner breaks. The second objective is to prevent the intrusion of incompressible materials into joints and cracks. These materials can interfere with normal expansion and contraction movements, thus creating compressive stresses in PCC slabs and increasing the potential for joint deterioration. If the compressive stresses exceed the compressive strength of the pavement, blowups or buckling may occur.

In HMA pavements, most believe that unsealed or poorly sealed cracks allow moisture and debris to enter the pavement structure contributing to asphalt stripping, secondary cracking, lipping (elevated transverse crack edges), and cupping (depressed transverse crack edges). In addition to the presence of excess water in the pavement base and subgrade, there tends to be reduced compressive and shear strength in the supporting materials immediately below and adjacent to the cracks. As a result, applied traffic loads in the vicinity of a crack create greater pavement deflections, additional cracking, cupping, and eventually potholes. Sealing operations on HMA pavements addresses various forms of cracking that may occur, such as thermal, reflection, block, and alligator cracking. However, crack sealing is believed to be most effective on transverse thermal and transverse reflection cracks; sealing individual alligator cracks is generally not believed to be cost effective.

1.2 Problem Statement

In the past several years, the practice of sealing and/or resealing joints and cracks has been questioned as less than cost-effective, at least in some applications. Additionally, studies that support a clear quantitative defense of crack sealing as cost-effective appear to be few in number and limited in scope. Research conducted by the Wisconsin Department of Transportation (WDOT) on jointed concrete pavements over an extended period of time has led that agency to discontinue joint sealing of concrete pavements. In 1990, WDOT implemented the “no-seal” policy on new concrete pavements and claims to have saved six million dollars annually with no loss in pavement performance and with increased customer safety and convenience.

The Indiana Department of Transportation currently spends approximately four million dollars annually to accomplish joint and crack sealing. About one-half of this is allocated for sealing old pavements that are selected through a subjective process. There is no quantitative evidence to justify this expenditure. The sealing operations are conducted because the agency assumes the benefits of sealing outweigh the costs.

1.3 Study Objectives

The primary objective of this research was to investigate the cost-effectiveness of joint and crack sealing in relation to pavement performance. The study focused on two specific questions:

1. Does joint and crack sealing in any way improve the service life or serviceability of pavement (performance); and
2. If sealing does improve performance, is it cost-effective and in what situations?

The questions can only be effectively addressed through a rigorous review of the literature, a survey of practice, and finally analysis of data collected from a well-designed field experiment. The results can then be formulated into a set of guidelines for implementation by maintenance and design personnel.

1.4 Scope of Study

The study was divided into two phases. The first phase was a literature review and synthesis of the state of the practice, intended to form a basis for determining whether or not further research was needed to determine the cost effectiveness of joint and crack sealing in Indiana. Based on the results of Phase I, Phase II involved the analysis of data collected from in-service pavements. The experimental design incorporated twelve cells with the consideration of three main factors, roadway classification, pavement type, and drainage. These are expected to have the greatest influence on pavement performance relative to joint and crack sealing effectiveness. Nineteen field sites (sections of in-service pavements) were selected for the experiment. Pavement performance was monitored periodically throughout the duration of the field study. Performance response variables include International Roughness Index (IRI), Falling Weight Deflectometer (FWD) measurements (deflections), load transfer, individual pavement stress (condition survey), and physical and mechanical properties of in-service pavement cores. Statistical analyses were performed to determine whether there were significant differences in pavement performance between sealed and unsealed sections based on the data. FWD simulation using finite element modeling techniques were employed to evaluate the impact of FWD test positions. The pavement temperature effect on the FWD deflection was evaluated based on statistical analyses.

1.5 Report Organization

This report contains ten chapters. Chapter one is the introduction of this research. A literature review on cost effectiveness of joint and crack sealing is provided in chapter two, while chapter three presents the survey of practice among agencies in the United States. A summary of findings from the literature review and survey of practice is discussed in chapter four. Chapter five introduces the preliminary and refined experimental designs for the study. The methodology for selection of test sites, test site physical descriptions, test sites treatment and maintenance, and the procedures for data collection are presented in chapter six. In chapter seven, the factorial and nested statistical model is introduced, and the statistical analysis results are discussed. The three-dimensional (3D) finite element model is developed in chapter eight to evaluate the effect of FWD test location on the load transfer measurements. The temperature impact on FWD deflections for concrete and HMA pavement is presented in chapter nine. Chapter ten summarizes the overall findings from this report and provides recommendations regarding joints and crack sealing practice in Indiana.

CHAPTER 2 LITERATURE REVIEW

A literature review was conducted on the subject of joint and crack sealing cost effectiveness. In an effort to obtain pertinent literature, the following databases were searched:

1. Transportation Research Information Services (TRIS);
2. Strategic Highway Research Program Reports (SHRP);
3. American Society of Civil Engineers (ASCE Journal of Transportation Engineering);
4. American Society of Testing and Materials (ASTM); and
5. Journal of the Association of Asphalt Paving Technologists (AAPT).

The initial searches revealed well over one hundred potential references. However, a review of the references revealed that the bulk of this literature focused on sealing materials and procedures rather than on the cost-effectiveness of sealing. Only eighteen specifically discussed cost-effectiveness and of these, only four provided useful quantitative data.

Recent research has focused on refinement of the materials and procedures rather than on the fundamental issue of whether or not sealing is cost-effective. This indicates that joint and crack sealing is generally assumed as to be cost-effective.

In addition to the literature search, individuals who are recognized experts on joint and crack sealing were contacted and asked to comment on the merits of the proposed research. This effort did not produce any additional references, but many of the individuals contacted reiterated the need for research on the cost-effectiveness issue.

The eighteen sources that contained some information on cost-effectiveness were separated into three categories: Non-supporters of sealing, supporters of sealing, and others. A synopsis of each of categories is presented in Sections 2.1, 2.2, and 2.3, respectively. Section 2.4 is a summary of the literature review based on the information presented in these sections.

2.1 Non- Supporters of Sealing

In 1997, Shober of the WDOT stated that there was significant information available on PCC pavement joint sealing by the early 1970's, but that most of it focused on joint and/or sealant performance (Shober, 1997). He further acknowledged that there was a definite lack of information available on overall pavement performance as influenced by joint sealing. For these reasons, he conducted a study to investigate the cost-effectiveness of joint sealing.

The WDOT initiated a study of PCC pavement performance as influenced by sealed and unsealed contraction joints at various spacings in 1974. Over 50 test sections were constructed from 1974 to 1988 incorporating both doweled and un-doweled PCC pavements with joints of various spacings placed on subgrades ranging from sand to silt to silty-clay and exposed to a range of traffic loadings. The performance of the five pavements that contained 51 test sections was summarized in Shober's 1997 report. The pavements ranged in age from eight to ten years at the time the performance data were collected. All five pavements had sealed sections and control sections that were not sealed. The seals in one pavement, USH51, were kept perfectly intact for at least 10 years. Any time a significant sealant failure was observed, it was corrected by resealing as quickly as possible. The seals on the other four pavements were not replaced if they failed.

Shober used four factors to evaluate pavement performance. They included:

1. Overall pavement distress;
2. Ride quality;
3. Encroachment on bridges; and
4. Material integrity.

The Pavement Distress Index (PDI) was used to characterize pavement distress. PDI is a combined pavement performance index that is a function of the severity and extent of several distresses obtained through visual condition surveys. Shober employed the International Roughness Index (IRI) to characterize pavement ride quality. Encroachment of the pavement on bridges was also evaluated by the observation of pavement expansion at bridges. The effect of joint sealing on material integrity was assessed by coring pavements at random locations. The cores were centered on pavement joints. The physical appearance of cores from both sealed and unsealed locations was used to determine if joint sealing had an effect on material integrity.

Statistical analyses were performed to compare the performance of sealed and unsealed test sections. The analyses indicated that joint sealing did not have a significant effect on pavement distress, ride quality, bridge encroachment, material integrity, and most importantly pavement life.

This study suggested that pavement performance was not positively influenced by joint sealing and that joint sealing may not be cost-effective for PCC pavement, at least within the state of Wisconsin. In some test sections there was improved (or at least equal) performance when joints were left unsealed. Several potential explanations were proposed for these findings. They included:

1. Stress concentrations;
2. Construction and maintenance; and
3. Funneling of water.

Even the sealed joints deteriorated some during the experiment thus became partially sealed and allowed incompressible material to enter. Extreme stress concentrations could have been generated when the pavements experienced expansion, which could have resulted in significant concentrated forces at the locations of the incompressible materials in the joints. The various operations involved in the resealing process itself often caused some joint spalling. Resealing could also cause bumps at the joint locations which would adversely affect ride quality. Wide joint sealant reservoirs could also cause tire noise and affect ride quality. Finally, the situation where partially sealed joints resulted in a water funneling effect existed in some sections. This could allow more water to enter a joint than would occur with a narrow, unsealed joint.

In concluding, Shober suggested that research on PCC joint sealant must remain focused on the customers needs. The customers' needs related to total pavement performance (distress, ride, life, and materials), convenience, and safety. The customers' performance needs were not positively influenced by joint sealing of the test sections considered in the research, and thus joint sealing was not cost-effective for PCC pavements.

Dunn, another WDOT engineer, developed a synthesis on the same topic in 1987 (Dunn, 1987). In the synthesis he reported that the majority of state highway agencies did seal and re-seal joints in rigid pavements, and that very few actual evaluations of the true effectiveness of sealing or resealing had been conducted. The basic objective of the synthesis was to summarize the available information relative to joint sealing of rigid pavements. The synthesis included the following statement by Stratton Hicks, Deputy State Highway Engineer, Wisconsin Highway Commission, "…we have some misgivings about the importance of sealing." The statement was made in a technical session at the annual Transportation Research Board (TRB) meeting in January 1967. Hicks provided the following five specific reasons for the misgivings:

1. The use of granular or stabilized bases tended to reduce the pumping problem;
2. Random observations of pavement performance indicated a lack of correlation between pavement durability and the maintenance of sealed joints;
3. There was only a slight chance of success in maintaining a truly effective seal over an extended period of time;
4. There are cost and traffic hazards associated with the periodic renewal of joint seals; and
5. The locations where there are concentrations of blow-ups and spalling are not apparently related with the locations of the joint seals in poor condition.

The synthesis was simply a review of the practice and did not attempt to incorporate specific research results. However, based on the information compiled by Dunn, he suggested that several factors needed to be considered and evaluated as part of the process of making a policy decision on sealing or not sealing joints in PCC pavements. He suggested that the following nine factors be considered:

1. Pavement slab design;
2. Base type;
3. Pavement subsurface drainage;
4. Concrete properties;
5. Sealant properties;
6. Maintenance commitment to continued resealing as needed;
7. Site-specific environment;
8. Traffic loading; and
9. Economics.

In concluding, Dunn summarized that although the majority of highway engineers believed in the purported benefits of sealing joints in rigid pavements, the only documented evidence available concerning the possible realization of longer or improved service attributed to sealing and resealing joints, were studies being conducted in Wisconsin. As described in the previous reference, the results of these studies have indicated that there was no statistical difference in the performance of PCC pavements regardless of whether joints were sealed or unsealed.

In 1990, Rutkowski reported on another PCC joint sealant study that was commissioned in Wisconsin in 1983 (Rutkowski, 1990). The original objective of the study was to compare the pavement performance of sealed and unsealed joints in PCC pavements. The research encompassed the analysis of seventeen projects, eight of which had test sections with unsealed traverse joints. Four pavement distress measures were considered for pavement performance comparisons; faulting, spalling, corner breaks and general cracking. The American Association of State Highway Transportation Officials (AASHTO) Present Serviceability Index (PSI) was also given consideration. Other distress types were initially considered, but had such low frequencies of occurrence that they were not discussed.

Pavement performance data were collected for the project pavements for the period from 1975 to 1989. A statistical analysis was conducted using the thirteen years of performance data for each test section. The results of the analysis indicated that sealed or partially sealed transverse joints in PCC pavements did not provide for significantly better distress ratings than unsealed joints with regard to faulting, spalling, corner break and general cracking. Additionally, for the observation period, the PSI of the pavements with unsealed transverse joints was similar to that of pavements with sealed or partially sealed transverse joints. Another finding of the study was that better pavement performance was not insured when an inspector continually monitored sealing operations.

A performance evaluation of drained pavement structures was conducted by Rutkowski, Shober and Schmeidlin of WDOT in 1998 (Rutkowski, Shober and Scheidlin, 1998). The research focused on drainage of pavement structures, but included provisions for assessment of cost-effectiveness of joint and crack sealing. The objectives of the study were to determine:

1. Which drainage features had the greatest impact on pavement serviceability;
2. Which drainage features were most effective in draining;
3. Which drainage features were the most cost-effective; and

4. Whether or not transverse joint sealing was effective.

Initially, five PCC surfaced projects were included in the study. During the course of the study, seven other projects were selected as secondary projects. Three of the secondary projects were PCC, three were HMA surfaced, and one project had both PCC and HMA test sections. Test sections and control sections were developed within each project site in 1987 or 1988. The test sections were used to compare various formats of positive drainage features. The control sections contained no positive drainage elements. The pavement performance was monitored annually for ten years.

Four measures were used to assess pavement performance as influenced by drainage, and for statistical performance analyses: PDI (the combined distress index), faulting, ride quality as indicated by IRI, and the physical properties of cores taken at transverse joints. Statistical "paired t-tests" were conducted at the 95 percent confidence level on control and test section PDI, faulting and IRI data. Investigation of joint sealing efficiency was one of the original study objectives. When the experimental designs were established, a redundant test section featuring sealed transverse joints was incorporated.

In the study, no statistical comparisons were performed on pavements with sealed and unsealed joints. However, when the data results of PDI, faulting and IRI were ranked, the effect of sealed transverse joints did not appear to have noticeable effects or benefits. The results supported the conclusions of Shober's earlier study (Shober, 1997) which stated that transverse joint sealing did not benefit pavement performance and was therefore not cost-effective.

2.2 Supporters of Sealing

Chong performed a cost-effectiveness analysis of the "rout and seal" technique when applied to flexible pavement for the Ontario Ministry of Transportation (OMT) in 1988 (Chong, 1989). The objectives of his research were to determine the:

1. Appropriate definitions and standards for rout and seal operational specifications;
2. Effectiveness of the treatment;
3. Extension of pavement service life due to the treatment;
4. Importance of timing of the treatment for cost-effectiveness; and
5. Consequences of deferred treatment.

The experimental design employed in the research dictated that pavement sections selected for study represent three pavement age categories; less than 3 years, 4 to 6 years, and 7 to 9 years. Each age category had to have a minimum of two test sections. Each test section was divided into five subsections, each of which was 150 m in length. The treatments applied to the five subsections were as follows: the middle one was left unsealed and used as a control, two subsections had a rout size of 40x10 mm, and two subsections had a rout size of 19x19 mm. When the test sections were set up a single crew with standardized equipment was employed to minimize installation variability.

A total of 37 test subsections were established for the three different age categories. The distributions of test sections by pavement age and geographic regions are presented in Table 2-1.

Pavement condition surveys were conducted for each section using a standard survey form which incorporated total length of transverse cracks, total length of longitudinal cracks, transverse crack cupping/lipping, crack spalling, and crack opening sealant bond failure. Roughness measurements were also made with a Mays Meter. Monitoring of the rout and seal test sections and their corresponding control sections was conducted between January and March for a three-year period from 1987 to 1989.

Table 2-1 Distribution of Test Sections

Category		Number of Subsections
Age	1-3 years	10
	4-6 years	13
	7-9 years	14
Geographic Region	Northern	6
	Eastern	9
	Central	2
	Southwestern	20

In this study, the value of "crack factor" was used to assess crack development. Crack factor was defined as the total linear length of transverse and longitudinal cracks on the pavement surface in meters divided by the total surface area of the pavement section in square meters. For the combination of transverse and longitudinal cracking, the performance monitoring data showed that crack development initiated in year one of the pavement service life and increased steadily until year six. Crack development then became static until the eleventh year, when the increase became quite dramatic. For transverse cracking alone, cracks developed fully in the first year of the pavement service life and remained quite static until the eleventh year, when a sharp increase began to take place.

Crack deterioration was assessed based on evaluation of deformation in the form of lipping or cupping. The monitoring data showed that the performance of the rout and seal cracks remained static with time, whereas the cracks in the control sections showed significant increase in lipping/cupping deterioration after three winters.

The author concluded that rout and seal treatment of cracks did not appear to have significant influence on crack development since there was no discernable difference in crack development between the sealed test sections and the unsealed control sections. The criteria used to determine crack deterioration was the degree of deformation at the transverse crack, known as either lipping or cupping. After three winters of service, it was shown that the unsealed control sections indicated a marked increase in the severity of lipping/cupping distress.

The study suggested that the rout and seal treatment would either stop or retard the deformation commonly known as lipping/cupping, which is detrimental to pavement serviceability and, therefore pavement service life. It also suggested that maximum cost-effectiveness was achieved when the initial routed and seal treatment was performed between the third and the fifth year of the pavement service life. Finally it suggested that deferred maintenance, particularly on transverse cracks, was not an acceptable engineering or economical option.

In 1996, Ponniah and Kennepohl of the Ontario Ministry of Transportation conducted life-cycle cost analyses to determine the influence of crack sealing on pavement performance (Ponniah and Kennepohl, 1996). The objectives of that study were to develop an effective crack sealing procedure and to study the influence of crack sealing on pavement distress and performance. Specifically, the study targeted acquiring statistically based data that could be used for objective assessment of crack sealing benefits in extending pavement life. This was an extension of the previous study (Chong, 1989). The experimental design, test site selection, and data collection used in this research were the same as were employed in the previous study.

Crack maps were developed for each test section each winter to assess crack growth. Statistical analysis confirmed that the rout and sealed sections, in general, performed better than the control sections. However, the data at some test sites indicated that crack sealing had no effect on crack growth. Further investigation revealed that the crack sealing treatment was effective only for pavements in certain conditions. In general, it was more effective for pavements in relatively good condition and less effective for pavements in relatively poor condition. It was concluded that the performance of pavements with extensive cracking would not benefit from sealing.

On the basis of field data obtained during the seven-year monitoring period performance curves were developed for both rout and sealed and control sections. The performance curves indicated that the crack treatment could extend pavement life by at least two years, depending on the original condition of the pavement, the environment and the applied traffic volume. Further analysis confirmed that the observed difference in performance as measured by the Pavement Condition Index (PCI) was statistically significant.

For the life-cycle cost analyses (LCCA) conducted as part of this research, a mathematical model was developed and used to assess the loss in PCI due to both traffic and environment. The model predicts PCI at any given time in the pavement service life and is stated as:

$$PCI_t = PCI_i - LT - LE \qquad \text{Equation 2-1}$$

where,

PCI_t = PCI at any time t;

PCI_i = initial PCI immediately after construction or rehabilitation;

LT = loss due to traffic, expressed as a function of number of ESAL applications; and

LE = loss due to the environment, expressed as a function of time in years (t).

The model was calibrated using data collected over a twelve year period from several projects in Ontario with known performance histories. The calibrated model was used to estimate pavement service life after each of the major rehabilitations considered in the LCCA analysis period.

Because the calibrated model could be employed to estimate pavement service life, different alternative rehabilitation and maintenance strategies could be economically evaluated. The present-worth cost of alternative strategies incorporating user delay costs and salvage value for remaining pavement service life at the end of the analysis period, as well as effectiveness (defined as the area under the performance curve) were used to determine the most cost-effective strategies. Figure 2-1 illustrates a comparison of two alternatives. The PCI curves in the figure were predicted using the calibrated PCI prediction model. Termed as the "effectiveness", the areas under the PCI curves were used to economically evaluate the two alternative designs.

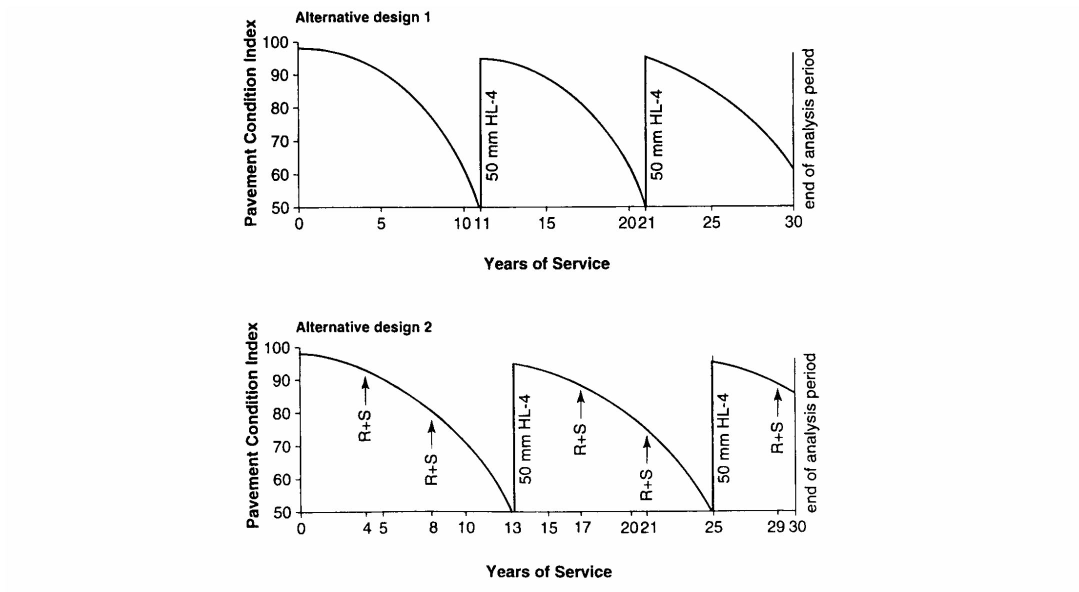

Figure 2-1 Two Alternative PCI Curves

The authors concluded that routing and sealing cracks could minimize secondary crack growth and increase service life by at least two years based on these types of analyses. The LCCA indicated that the rout and seal treatment was a cost-effective pavement maintenance procedure.

Rutkowski of WDOT conducted a study in 1998 with the objective of determining the effect of sealing and filling of cracks on HMA pavements on overall pavement performance (Rutkowski, 1998). He defined crack sealing as crack routing and sealing, and defined crack filling as sealing without routing. Three test projects, each with six or seven test sections, were included in the study. The projects consisted of different pavement structural sections. Pavement performance parameters used in analyses included Pavement Distress Index (PDI), a combined distress index and the AASHTO Present Serviceability Index (PSI). The statistical paired-t test was used to determine if there was a qualitative benefit to either PDI or PSI as a result of crack filling or sealing. Additionally, the WDOT Customer Service Index (CSI) was used to determine if there was a cost benefit associated with crack filling or sealing relative to PSI.

Pavement performance data was collected on one control (unsealed) section and multiple test sections at each project over a six-year period from 1987 to 1993. PSI was surveyed in both summer and winter. PDI was surveyed annually, usually in the summer. Statistical "paired t-tests" were performed at the 95 percent confidence level for both PSI and PDI to compare the data from the control section and test sections. The PSI was used as a tool to evaluate all pavement types and treatments for the purpose of assessing the quality of customer service.

The study concluded that crack filling and sealing in general rather than a specific sealant or filler provided the measured benefits. Crack filling and sealing appeared to have a beneficial effect on both HMA overlays on existing HMA pavement as well as for HMA overlays on PCC pavement. Rutkowski recommended that crack filling/sealing be considered as a means of benefiting ride quality (PSI) rather than to mitigate pavement distress (PDI), as crack filling/sealing could improve the ride quality of a pavement.

In this study, there were no useable comparison parameters to compare project PDI histories and determine whether the severity or extent of pavement distress influenced the need for crack filling/sealing. Additionally, due to limited data, no analysis was performed to determine the effects of base thickness and/or subgrade quality on crack development or pavement performance.

Eacher and Bennett of the Michigan Department of Transportation (MDOT) conducted a study on crack filling in 1998 (Eacher and Bennett, 1998). The purpose of the study was to have side by side comparisons of several different filler materials used for HMA pavements at a single location. The study primarily focused on the effect of filler materials on performance, but it incorporated a control (unsealed) section also. Twenty-one test sections involving nine materials with different additives were placed in May 1995. The different test sections were visually rated by several different groups. The properties considered in the rating included: bridging, abrasion, adhesion/cohesion loss, bleeding and tracking. They were rated on a scale of 1 to 5 with 5 being the best. Ratings were conducted one, three, seven, eleven, fifteen, and twenty-four months after the test sections were placed.

Based on the pavement condition of the test sections after two years, it was concluded that several of the materials could slow the deterioration of the cracks, since the sections sealed with these materials showed less crack deterioration than the untreated section. It was estimated that these materials could add 3-5 years to the life of the pavement. The study also showed that the performance of pavements sealed with different materials was significantly different.

In 1997, Morian and Epps concluded an evaluation of the Long Term Pavement Performance (LTPP) Special Pavement Studies-3 (SPS-3) and SPS-4 sites which included various maintenance treatments including crack sealing (Morian and Epps, 1997). The objectives of this study were to define the most effective timing for the application of various treatments and to evaluate the effectiveness of treatments in prolonging the life of the pavement. The project report presented an evaluation of the performance of LTPP SPS-3 (HMA pavement) and SPS-4 (PCC pavement) experimental sites based on field reviews after 5 years of service. The HMA pavement preventive maintenance treatments studied included; cracking sealing, slurry seals, chip seals and thin hot-mix asphalt overlay. The PCC pavement preventive maintenance treatments studied included joint and crack sealing and under sealing.

The field experiment was designed in 1987 by the Texas Transportation Institute (TTI) to evaluate the effectiveness of various preventive maintenance treatments. The main variables for HMA pavements were climate, subgrade type, traffic volume, and treatment type. A total of 96 test sites were considered for the HMA pavement preventive maintenance study. The main variables in the experimental design for the PCC pavements were climate, base type, pavement type and treatment type. A total of 24 test sites were considered for the PCC preventive maintenance study. The performance of each of the SPS-3 and SPS-4 sites was being evaluated under the LTPP program and by an Expert Task Group (ETG) for each LTPP region. The LTPP program determined the condition of the pavement before the preventive maintenance treatment was applied and at regular intervals after the treatment was applied. The evaluation tools used as part of the LTPP effort included the following:

1. Visual condition using the SHRP distress identification manual;
2. Photo log using the PASCO, USA device;
3. Deflection using the falling-weight deflectometer;
4. Ride quality using the K. J. Law-type profilometer;
5. Rut depth using the “dip stick” and PASCO data; and
6. Friction number as collected and submitted to LTPP by individual States.

Observed data for HMA crack seal treatments in terms of pre-treatment condition, climate region, and predicted performance life were evaluated. The treatment was observed in this study to have slowed the rate of pavement deterioration in several cases. The crack seal treatment was effective in the wet-freeze environmental zone. The wet-no freeze region also experienced good performance from the crack seal treatment using an overband technique, but the crack seal treatment did not perform well in the dry regions of the country.

Based on the limited number of PCC sites reviewed, the study found that unsealed joints in the control sections contained significantly more debris than sealed joint sections and unsealed joint sections had significantly more joint spalling than the sealed joint sections.

In this study, only five years of performance data were statistically analyzed, and the authors noted that more time might be required to obtain meaningful results from the PCC sections.

In 1986, Sharf and Sinha investigated the trade-off relations between two routine pavement maintenance activities used in Indiana, namely, patching (corrective maintenance) and sealing (preventive maintenance) (Sharaf and Sinha, 1986). In this study several cost models were developed. For model development purposes, the two highway systems (interstate and other) were subdivided by climatic zones (north and south) and pavement types (flexible, rigid and resurfaced). Data were collected and analyzed for a total of eight hundred twenty pavement sections. For each section, four major groups of information were summarized: traffic (AADT, percent trucks and ESAL), pavement characteristics (type, layer thickness and age), climatic zone (north and south), and pavement maintenance records (total production units, total man-hours, and type and quantities of materials). Pavement maintenance information was summarized for each highway by activity and fiscal year.

Three different prediction models; a total routine pavement maintenance cost model, a patching maintenance cost model, and a sealing maintenance model, were developed with the historical data routinely collected by INDOT. The cost savings in routine pavement maintenance in terms of direct fuel consumption could be assessed by one application of the models. The authors concluded that if more sealing is done prior to winter, less pavement repair is required in the spring and summer. Moreover, a direct cost savings of reduced fuel consumption could be achieved by increasing the level of sealing activity.

With the trade-off relationships between routine pavement maintenance activities, the savings in fuel used in pavement maintenance in Indiana were estimated. However, the different impact on pavement performance by different maintenance activities and the costs of different activities were not considered in the study.

In 1988, Chong and Phang stated that during the early 1970s, the OMT began to seal cracks using the rout and seal program to minimize the effects of cracking, particularly lipping and cupping of transverse joints, on pavement roughness (Chong and Phang, 1988). At the same time, the ministry sought to improve the rout and seal technique and identify sealant materials that provided better performance. A study was conducted to address these issues.

The authors stated that at that time the majority of HMA pavement mileage with untreated transverse cracks were developing either lipping or cupping deformations at the cracks. These deformations were costly to redress under the rehabilitation program, and simply resurfacing with HMA only perpetuated the cycle of reflective cracking and subsequent lipping or cupping. The authors further stated that not sealing cracks could result in:

1. Increased maintenance costs because deteriorated cracks were difficult and expensive to repair through corrective maintenance;
2. Increased user costs (vehicle repair and operation);
3. Increased rehabilitation costs, because deteriorated cracks demanded special treatment from the designer when pavement rehabilitation was scheduled; and
4. Loss of serviceability and, therefore, service life.

A study initiated by the Ottawa District Maintenance Office of the OMT in 1981 was summarized in this report. The study included several rout and seal sections as well as a control section. In 1985, an investigation was made on the deferred maintenance control unsealed section and one of the rout and seal study sections, which were adjacent to each other. The study concluded that the rout and seal treatment of transverse cracks effectively retarded internal and external pavement deterioration. It also suggested that the rout and seal treatment of transverse cracks effectively retarded the progression of cupping deformations. Finally, the comparison of the treatment sections with the control section indicated that the rout and seal treatment of transverse cracks could extend the serviceability of the pavement by at least four years.

In a 1998 TRB publication entitled “Joint Seal Practices in the United States – Observations and Considerations,” Morian and Stoffels summarized joint sealing practices for jointed PCC pavements that have developed throughout the country based on local experience (Morian and Stoffels, 1998). The authors stated that although the

LTPP SPS-4 sections (PVV pavements) had only been in service for five years, which was not long enough to truly see the benefits of the maintenance treatments on the pavement life, early findings indicated that joint-seal sections were performing better than unsealed sections.

It was further noted that a misconception of some agencies is the belief that the entrance of both water and incompressibles into joints could be reduced by the use of a single saw cut, rather than joint sealing, without constructing a sealant reservoir. Further the authors stated that, in the Shober study (Shober, 1997), insufficient performance history was provided to substantiate the conclusion that the performance of sealed and unsealed joint sections was indeed equivalent. That conclusion was drawn based on an analysis of test sections with less than ten years of performance history available for evaluation. However, numerous examples of PCC pavements with early failures, including material-related, load-transfer and slab-erosion problems, were available to confirm that 10 years is often too short a performance period to identify problems with PCC pavements. The authors concluded that no comprehensive field tests thoroughly evaluating joint sealing of PCC pavements in terms of pavement performance, in an appropriate manner over a significant period of time, existed.

2.3 Others

Ward of the INDOT reported on the evaluation of crack sealant performance on Indiana's pavements in 1993 (Ward, 1993). The objective of his study was to determine the most economical and effective sealing materials for routine transverse crack sealing applications in Indiana. He performed comparison tests of the typical sealing materials used in Indiana at that time (AE-90) with eleven other sealants. All sealants were applied on a typical HMA surfaced pavement, and their performance was observed over a three year period. Success Rate (SR) was the basis of comparison among and between different sealants, cleaning techniques, and application methods. The study stated that there were significant differences in the performance among sealant/treatment combinations, and routing appeared to improve the performance of most of the sealants. The study did not determine the cost-effectiveness relative to pavement performance.

Blais summarized the results of a cooperative value engineering study of crack and joint sealing undertaken by the Delaware, Georgia, Montana, Tennessee, and Utah Departments of Transportation under the sponsorship of the Federal Highway Administration (FHWA) (Blais, 1984). The objective of the study was to optimize the expenditures of maintenance resources through a study of crack and joint sealing materials and placement techniques. In this study, there was no evaluation of the cost benefits of sealing versus not sealing. However, the study members felt that sealing was necessary and believed that many referenced papers, as well as several other studies, had properly addressed the needs for sealing cracks and joints as a preventative maintenance activity. The study also stated that, before sealing, a crack analysis was necessary to determine if crack sealing was effective. The study suggested that climatic conditions could greatly affect material placement, but if pavement conditions were dry, a good bond could be formed regardless of the season.

The cost–effectiveness of crack sealing materials and techniques for HMA pavements was evaluated by Reed and David in 1999 (Freeman and Johnson, 1999). The objective of this research was to determine the most economical and effective materials and methods for sealing cracks in HMA pavements in the state of Montana. Four experimental test section sites were selected for study as part of the project. Two of those four sites included the control sections, where cracks were left unsealed. Eleven sealant materials and six sealing techniques were considered in the investigation.

Both transverse and longitudinal cracks were evaluated at all test sites over a two year period. During the evaluation, material failures and superficial sealant distress were measured and recorded. After two years of performance monitoring, the study found that routing transverse cracks improved the performance of sealants, but routing did not appear to be necessary for longitudinal cracks. The author stated that cracks in control sections were in good condition, but that no analysis or conclusions were made for the control section pavement performance.

The FHWA, *Techniques for Pavement Rehabilitation Reference Manual,* states that in HMA pavements, non-sealed or poorly sealed cracks allow moisture and debris to enter the pavement structures contributing to stripping, secondary cracking, cupping and lipping at transverse joints, and spalling (FHWA, 1998). The manual includes a section entitled, "Limitations and Effectiveness" in which it is stated, "In the past, the effectiveness of joint sealing has been questioned by some agencies. For example one agency contends that the purported benefits derived from joint sealing do not offset the costs of sealing and resealing operations. While this debate might never be completely resolved, those efforts should go a long way toward identifying whether sealing activities were effective and under what conditions they should be applied. Nonetheless, the overwhelming majority of States' experiences support the contention that sealing cracks and resealing joints was a meaningful rehabilitation activity."

CHAPTER 3 SURVEY OF PRACTICE

With the objective of obtaining a better knowledge of the current joint and crack sealing practices in the United States, a survey regarding joint and crack sealing was conducted by Galal and Ward of INDOT in June 1999. All 50 States were polled. The survey included eleven questions on joint and crack sealing, which are shown in Appendix A. The eleven concise questions resulted in quick responses from 42 of 50 States. California, Connecticut, Idaho, Kentucky, Massachusetts, Ohio, Tennessee and West Virginia did not respond. The eight states that did not respond to the survey were considered as the no-response group in the survey statistical analysis. A summary of the survey results is presented in Appendix B. The statistical analyses of the responses to each of the eleven questions are shown in the following sections

3.1 Question 1: Do you seal concrete pavements?

As shown in Table 3-1, almost three-fourths of the states surveyed seal PCC pavements. The responses of the states coincide with the common belief that joint sealing will extend pavement life and/or improve pavement performance. Only three respondents, Alaska, Hawaii, and Wisconsin, do not seal PCC pavements. However, only Wisconsin stated their reason for not sealing: because sealing is not cost-effective.

Table 3-1 Summary Responses to Question 1

Response	Number of States Responding	Percentage
Yes	36	72%
No	3	6%
N/A No concrete pavement	3	6%
No response	8	16%

3.2 Question 2a: How wide is your saw cut for joints on new concrete pavements (transverse)?

As shown in Table 3-2, 31 percent of responding states specify transverse joint widths less than or equal to 6.35 mm. An additional 31 percent employ 9.5 mm joint widths. 13 percent specify joint widths greater than 10 mm. Wide saw cut joints (>10 mm) are not commonly applied.

Table 3-2 Summary Responses to Question 2

Response	Number of States Responding	Percentage
3.18mm	10	18%
4.76mm	2	4%
6.35mm	5	9%
9.53mm	17	31%
10.00mm	2	4%
12.70mm	3	5%
20.00mm	2	4%
N/A No concrete pavement	5	9%
No response	9	16%

3.3 Question 2b: How wide is your saw cut for joints on new concrete pavements? (longitudinal)

As shown in Table 3-3, 8 percent of the states responding to the survey specify a longitudinal joint width of 6.35 mm. Two percent specify a 3.18 mm joint width. Unfortunately, 80 percent of the states did not respond to the question. Therefore, it is not possible to make any meaningful conclusions from the responses obtained.

Table 3-3 Summary Responses to Question 2b

Response	Number of States Responding	Percentage
6.35mm	4	8%
3.18mm	1	2%
N/A no concrete pavement	5	10%
no response	40	80%

3.4 Question 3: Do you reseal older concrete pavements

As Shown in Table 3-4, more than half of the responding states reseal older PCC pavements while 14 percent do not. This is significantly higher than the percentage of the states that do not seal new PCC pavements. It indicates that some states believe resealing older PCC pavements may not provide a benefit or be as cost-effective as sealing new ones.

Table 3-4 Summary Responses to Question 3

Response	Number of States Responding	Percentage
Yes	33	66%
No	7	14%
N/A	2	4%
No response	8	16%

3.5 Question 4: Do you reseal bituminous pavements?

Table 3-5 indicates that more than half of the states responding to the survey reseal older HMA pavements, but 14 percent do not. This suggests that some states do not believe that sealing HMA pavements is cost- effective.

Table 3-5 Summary Responses to Question 4

Response	Number of States Responding	Percentage
Yes	33	66%
Only occasional crack sealing	2	4%
No	7	14%
No response	8	16%

3.6 Question 5: How was the decision made to conduct joint or crack sealing?

As shown in Table 3-6, nearly half of states responded that their decisions to conduct joint and crack sealing are based on long standing policy or that they were unsure of the reasons for sealing. Only 17 percent of the states indicated that their decisions were based on research. Those basing their decisions on research include both supporters and non-supporters of sealing. The responses suggest that many states have not justified the benefit or cost-effectiveness of joint and crack sealing.

Table 3-6 Summary Responses to Question 5

Response	Number of States Responding	Percentage
a. Long standing policy	18	36%
b. research results	9	17%
c. unsure	7	13%
Others	9	17%
No response	9	17%

3.7 Question 6: Do you install subsurface drains on new pavements?

The data presented in Table 3-7 show that more than 60 percent of the states install subsurface drainage on new pavements, and more than 10 percent install subsurface drains occasionally or when necessary. These responses suggest that most states believe subsurface drainage is important to pavement performance. Further research may be needed to investigate the function and cost-effectiveness of installing drainage.

Table 3-7 Summary Responses to Question 6

Response	Number of States Responding	Percentage
Yes	31	62%
Occasionally	2	4%
When necessary	4	8%
No	5	10%
No response	8	16%

3.8 Question 7: Has your DOT studied the effect of joint and crack sealing with regard to the impact it has on the performance of your concrete, asphalt or composite pavements?

As shown in Table 3-8, over 60 percent of the responding states have not studied the effect of joint and crack sealing on the performance of their pavements. Only 20 percent of the states have studied the effect. These results coincide with the finding that most states base their joint and crack sealing decisions on long standing policy or are unsure of the reasons for sealing.

Table 3-8 Summary Responses to Question

Response	Number of States Responding	Percentage
Yes	10	20%
No	32	64%
No response	8	16%

3.9 Question 8a: Does your DOT plan on investigating the cost-effectiveness of joint/crack sealing in the near future?

Table 3-9 shows that the majority of the states who responded do not plan on investigating the cost-effectiveness of joint and crack sealing in the near future. This suggests that most states do not question the cost-effectiveness of joint and crack sealing. It is unclear from the survey whether or not the responding states are aware of the recent WDOT finding.

Table 3-9 Summary Responses to Question 8a

Response	Number of States Responding	Percentage
Yes	10	20%
Possibly	1	2%
No	31	62%
No response	8	16%

3.10 Question 8b: If your DOT is planning on investigating the cost of joint/crack sealing in the near future? How?

As indicated in Table 3-10, 60 percent of the states that responded to the survey do not plan on investigating the issue of cost effectiveness in the near future while over twenty percent are planning on research. This suggests that some state agencies believe it is necessary to justify the cost effectiveness of their current sealing policies.

Table 3-10 Summary Responses to Question 8b

Response	Number of States Responding	Percentage
a. in house research	10	20%
b. consultant	0	0%
c. university research	1	2%
No plan for investigation in near future	30	60%
No response	9	18%

3.11 Question 9: How do you define traffic level, in terms of ESALs and/or truck count/truck factor?

Table 3-11 shows that the vast majority of responding states define traffic level in terms of ESAL. It is commonly accepted as the standard traffic load definition for most states.

Table 3-11 Summary Responses to Question 9

Response	Number of States Responding	Percentage
ESALs	36	72%
truck count	1	2%
Both ESALs and truck count	3	6%
Modified AASHTO Procedures	1	2%
Unsure	1	2%
No response	8	16%

3.12 Question 10a: Do you have criteria defining thick and thin pavements?

Table 3-12 shows that more than half of states do have criteria defining thick and thin pavements. However, the criteria vary from one state to the next.

Table 3-12 Summary Responses to Question 10a

Response	Number of States Responding	Percentage
Yes	27	54%
No	15	30%
No response	8	16%

3.13 Question 11a: Do you have FWD criteria that define performing joints or cracks?

As shown in Table 3-13, most of the states that responded do not have criteria to define if joints and cracks are performing properly. The thirty percent that do have criteria use joint or load transfer efficiency to establish performance.

Table 3-13 Summary Responses to Question 11a

Response	Number of States Responding	Percentage
Yes	15	30%
No	27	54%
No response	8	16%

CHAPTER 4 CONCLUSIONS BASED ON LITERATURE REVIEW AND SURVEY OF PRACTICE

The objectives of sealing and resealing of joints and cracks in both PCC and HMA pavements are to reduce the amount of moisture infiltration, and to prevent the intrusion of incompressibles into the joints and cracks. It has been a widespread belief that this will extend pavement life and/or improve serviceability and is therefore cost effective. Since this belief was challenged by WDOT in the early 1950's, there has been growing pressure within highway agencies for further studies to determine if various construction and maintenance activities can be justified in terms of cost. A decision was made by INDOT to review all available literature and to contact individuals with regard to cost effectiveness issues as a first step, and then to determine whether it was necessary to conduct further research on the cost effectiveness of the joint and crack sealing on Indiana highways.

4.1 Summary of Literature Review and Practice Survey

Of over one hundred potential references reviewed in this study, only eighteen specifically discussed cost-effectiveness of joint and crack sealing, and of these only four provided useful quantitative information related to the cost-effectiveness of joint and crack sealing. Individuals who are recognized experts were also contacted and asked to comment on the merits of the proposed research. However, all these efforts revealed little quantitative evidence to prove the cost-effectiveness of joint and crack sealing. Furthermore, some discrepancies exist among different research results. For example, Shober (Shober, 1997) concluded from his study that total pavement performance was not positively affected by joint sealing, and joint sealing was not cost-effective for PCC pavements. This conclusion was also supported by two other research efforts conducted in Wisconsin. However, the LTPP SPS-4 test section data analyzed by Morian and Epps (Morian and Epps, 1997) showed that test sections with unsealed joints showed more joint deterioration than sections with sealed joints. In addition to discrepancies in the literature, some ambiguous statements regarding joint sealing were identified. For instance, Morian and Stoffel (Morian and Stoffel, 1998) stated that after Wisconsin, California and Arizona also adopted a "no seal" policy for rigid pavement joints. However, when those agencies were contacted by the authors of the current study via telephone, the use of a no-seal policy could not be verified. No publications regarding the issue from California or Arizona could be found. Morian and Stoffel also suggested that the relatively short span of available pavement performance associated with the Shober study was insufficient to support the conclusions.

With regard to crack sealing of HMA pavements, most available literature seems to support the idea that cracking sealing will retard the deterioration of cracks and therefore extend pavement service life. However, the cost effectiveness of crack sealing in terms of pavement performance is not substantiated by a preponderance of evidence. Additionally, the review of HMA pavement crack sealing performance on LTPP SPS projects suggests that it is only effective in specific climates. In addition, there are only superficial suggestions and comments on the cost effectiveness of crack sealing for HMA pavements. For example, the FHWA Techniques for Pavement Rehabilitation Manual (FHWA, 1998) suggests that crack sealing is most effective when conducted on pavements exhibiting little structural deterioration. However, HMA pavements displaying extensive alligator cracking or severe crack deterioration should not be treated by crack sealing. Chong (Chong, 1989) also concluded that deferred maintenance particularly on transverse cracks was not an acceptable engineering or economical option.

In the literature review, it was found that only two studies (Sharaf and Sinha, 1986; Ward, 1993) relative to joint and crack sealing have been carried out in the State of Indiana. In one study, Sinha demonstrated that when more crack sealing was performed in the Fall, less patching was required after the following Spring. In another study, Ward concluded that there were significant differences in the performance of sealant/treatment combinations, and routing appeared to improve the performance of most of the sealants. However, neither of these studies nor any other available research considered the overall pavement performance as influenced by sealing and the cost effectiveness for joint and crack sealing in Indiana. Instead the INDOT bases its joint and crack sealing programs on long standing policy; there is no available research to justify the policy. However, according to the survey of practice, this policy is consistent with that of 62 percent of the states surveyed.

The statistical results of the survey showed that 72 percent of the states seal PCC pavements. Sixty-six percent reseal both PCC and HMA pavements. However, only 17 percent of the states surveyed declared that joint and crack sealing policy decisions were based on research results and almost 50 percent declared that their decision was based on long standing policy or they were unsure of the reasons for sealing. The results illustrate that most states do not have quantitative justification for sealing policies nor do they know the cost-effectiveness of the operations.

4.2 Recommendations Based on Literature Review and Survey of Practice

Even though Wisconsin has adopted a no-seal policy based on research results, it is apparent that different climatic, subgrade, and drainage conditions may affect the performance of pavements with and without sealed joints and cracks. Furthermore, there are controversial and ambiguous research results in the literature regarding the cost-effectiveness of joint and cracking sealing of both HMA and PCC pavements. Therefore it would be inappropriate for INDOT or any other agencies to simply adopt a no-seal policy. However, without sound research to justify current sealing practices, they too can be questioned. INDOT currently spends approximately four million dollars annually to accomplish crack and joint sealing, but unfortunately, there is no quantitative evidence to justify this expenditure.

The following recommendations are therefore made, based on the Phase I results of this study:

1. Further research via field studies is strongly suggested to investigate the cost-effectiveness of joint and crack sealing in relation to pavement performance in Indiana;
2. Overall pavement performance, as influenced by sealing, and the cost effectiveness of joint and crack sealing should be the focus of the suggested research;

3. A two- to three-year field study incorporating pavements representing a large range in age should be considered;
4. Pavement type, thickness, base type, pavement drainage, site-specific environment, and traffic loading conditions should be considered as factors in the experimental design; and
5. Highway agencies and researchers should be contacted to request input on the experimental design.

CHAPTER 5 EXPERIMENTAL DESIGN

The literature review revealed that there is a lack of quantitative evidence to establish the cost-effectiveness of joint and crack sealing, and furthermore some discrepancies exist among research results. The statistical analysis of the survey of practice also showed that most states, including Indiana, do not have quantitative justification for sealing policies nor do they know the cost-effectiveness of the operations. Hence, a field study is needed to determine whether joint and crack sealing is cost-effective relative to pavement performance in Indiana. The design of a field experiment is presented in this section, the objective of which is to provide adequate evidence to answer the age old question of whether joint and crack sealing is cost effective.

The greatest challenge associated with the experimental design revolved around the fact that pavement lives typically range from ten to thirty or more years, yet a field experiment with a maximum duration of 20 to 24 months for field data collection was permitted. Therefore, the need to obtain pavement performance data representative of extended time periods needed to be obtained in a short two-year period. The short field data collection period allows for the collection of performance data through only two Spring and Fall seasons if the data collection is initiated in the Spring. The proposed experimental design overcomes this challenge through the selection of field sites that represent similar pavement types, but of differing ages. The reality is that even though pavement lives may range from ten to thirty or more years, joint and crack seals typically have much shorter lives. Therefore, pavement performance data over a large portion of typical joint and crack seal lives is actually what is needed. The methodology to capture this data is illustrated in Figure 5-1, with PSI used as an example measure of pavement performance.

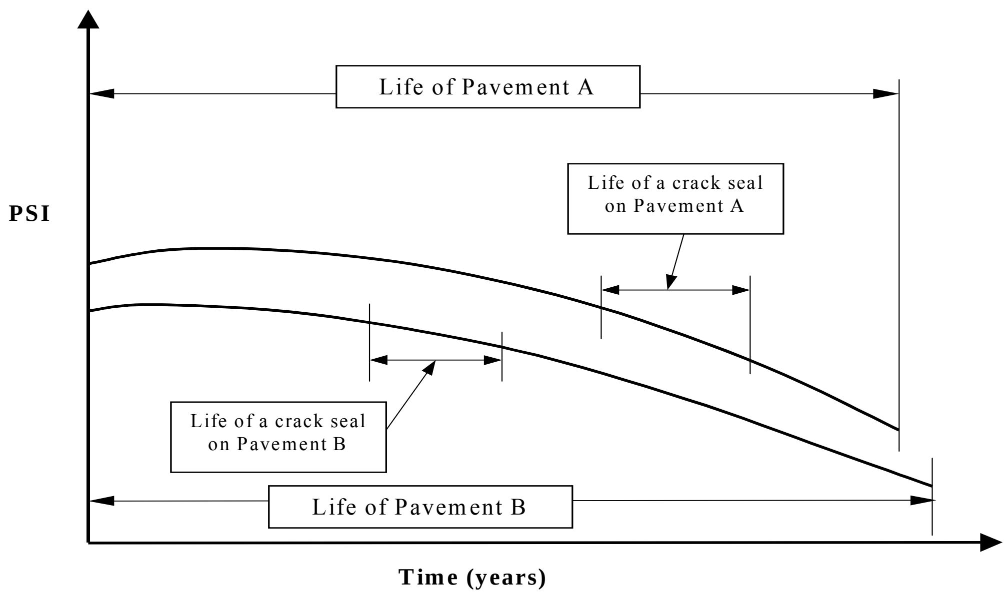

Figure 5-1 Pavement Life, Seal Life, Age Concepts of Experimental Design

Typical lives of a particular pavement type, for example a composite pavement (HMA overlay on PCC), are depicted in the figure. The lives of crack seals applied to a specific pavement type are also depicted (Pavement A and Pavement B crack seal lives). It is important to understand that the depicted crack seal lives would be associated with different pavement sections or physical locations. The obvious reason for this is that the allowable duration for field data collection must be limited to a maximum of two years. Using this technique, performance data may be obtained from different pavement sections with similar composition and pooled for statistical analysis. It must be noted that the performance data will be rigorously analyzed after the two-year performance monitoring period to ensure adequate data exists to meet the project objectives. Based on this analysis, a determination will be made as to the need to collect another year worth of performance data.

5.1 Preliminary Experimental Design

A preliminary experimental design, summarized in Table 5-1, was presented in the original project proposal. The design was developed through a series of meetings with pavement technologists and a statistician.

Table 5-1 Preliminary Experiment Design

Climate	Traffic	Pavement Thickness	New Pavement				Existing Pavement					
			PCC		HMA		PCC		HMA		Composite	
			Drainage				Drainage					
			Yes	No	Yes	No	Yes	No	Yes	No	Yes	No
North	High	Thick	**1**		**5**			**9**	**13**			**17**
	Med		**2**		**6**			**10**	**14**			**18**
	Low											
	High	Thin										
	Med		**3**		**7**			**11**	**15**			**19**
	Low		**4**		**8**			**12**	**16**			**20**
South	High	Thick										
	Med											
	Low											
	High	Thin										
	Med											
	Low											

The experimental design incorporated six factors, with levels of each factor ranging from two to three, which were expected to have the greatest impact on sealing effectiveness. The six factors were:

1. Climate (two levels-north and south);

2. traffic (three levels-high, medium, and low);
3. pavement thickness (two levels-thick and thin);
4. pavement age (two levels-new pavement and existing pavement);
5. pavement type (three levels-PCC, HMA, and composite); and
6. subsurface drainage (two levels- yes and no).

For the purposes of this project the following pavement type definitions are used:

1. PCC – a portland cement concrete pavement, often referred to as a rigid pavement;
2. HMA – a full depth HMA pavement, often referred to as a flexible pavement; and
3. Composite – a HMA layer resting on a PCC pavement.

Based on the fact that each cell in Table 5-1 would require an associated amount of fieldwork and data collection/analysis, several cells were eliminated to keep the research to a manageable level. The shaded and cross-hatched cells were acknowledged as potentially important, but were deemed non-essential based on the time and expense that would be associated with filling them. The two levels (north and south) for the climate factor were reduced to one level (north only). The reduction was made based on the fact that field locations had not yet been identified and the decision that climatic (precipitation, freeze-thaw cycles, etc.) data would simply be collected at each field location where ever they may be located within the state. The shaded cells were eliminated based on illogical combinations and historical INDOT practices. An example of an illogical combination would be a thin pavement exposed to high traffic because pavement thickness is determined as a function of expected traffic in the structural design process. An example of a historical INDOT practice might be that drainage is routinely incorporated into PCC pavement structures.

A refined experimental design was developed based on several factors. They included:

1. A critical review of the preliminary experimental design;
2. Recommendations of other researchers identified in the literature review process;
3. Suggestions of Study Advisory Committee (SAC) members;
4. Distribution of pavement types in Indiana; and
5. Time, physical data collection, and monetary constraints of the project.

5.2 Refined Experimental Design

Based on the critical review of the preliminary experimental design, the research team felt that pavement type, traffic, pavement thickness, and drainage had to be included as factors in the experimental design as they are expected to have the greatest influence on pavement performance related to joint and crack sealing effectiveness. The consideration of these factors is consistent with recommendations found in the previously discussed literature review and those extended by SAC members. SAC members with extensive field experience suggested that two very important factors specific to Indiana were drainage conditions and the inclusion of low volume facilities (e.g. State Routes). This lead to the formulation of the experimental design presented in Table 5-2.

Table 5-2 Refined Experiment Design

Roadway Classification	Pavement Type					
	PCC		HMA		Composite	
	Drainage		Drainage		Drainage	
	Yes	No	Yes	No	Yes	No
National	1	3	5	7	9	11
State	2	4	6	8	10	12

Note that the two factors, traffic and pavement thickness, were combined into the single factor roadway classification with two levels, national and state routes. There were two reasons for combining traffic and pavement thickness. First, because pavement thickness is established in the structural design process as a function of expected traffic, including both traffic and thickness would be redundant. The second reason was to provide the greatest statistical inference space with the smallest number of field projects. This is achieved by using data that represent the higher and lower limits for a main factor. In the case of the design presented in Table 5-2, the national routes represent the high traffic volume, thick pavements while the state routes represent the lower volume, thinner pavements. The refinements led to an experimental design (Table 5-2) with twelve cells. Within each cell, two test sites of different ages with two test sections per test site were planned.

It should be noted that one additional refinement was attempted. An attempt was made to determine the distribution of roadway miles in Indiana by pavement type (PCC, HMA, and composite) as well as the distribution of drainage conditions within each pavement type. The objective was to determine whether efforts should be focused within specific cells and/or if others should be deleted based on the percentage of the roadway network each cell represented in Indiana. Unfortunately this information was not readily available from INDOT. However, research underway at Purdue University, incorporates the development of a database with most of the required information. The database is in the developmental stage, so information extracted from it must ultimately be verified, but it does provide for an estimate of the distribution of pavement types within Indiana. Estimates of the percentage of each pavement type, extracted from the database, are presented in Table 5-3. Unfortunately, the distribution of drained versus undrained pavements is not available at this time. Table 5-3 shows that the combination of composite and HMA pavements represents approximately 84 percent of the Indiana network. This suggests that the field experiment should focus on these pavement types. However, the literature suggests that joint and crack sealing of PCC pavements may be the least cost effective. For these reasons it is recommended that the experimental design incorporate all three pavement types.

Table 5-3 Pavement Type and Drainage Distributions

Pavement Type	Percentage of Total Indiana Network	Percentage of Drained and Undrained by Pavement Type	
		Drained	Undrained
PCC	10		
HMA	25		
Composite	59		
Other(unknown)	6		

5.3 Planned Data Collection

Pavement performance was monitored periodically throughout the duration of the field study. Performance response variables included ride quality (IRI), seasonal pavement deflection (FWD), composite performance indices PSI, individual pavement distresses, and physical and mechanical properties of in-service pavement cores. These data were analyzed statistically to determine the effectiveness of joint and crack sealing, and coupled with remaining life predictions to evaluate the cost effectiveness of sealing. These analyses provide the basis for the formulation of a joint and crack sealing policy for INDOT.

CHAPTER 6 TEST SITE PREPARATION AND DATA COLLECTION

6.1 Test Site Selection

Field test sites were selected under the principle that candidate sites must conform as closely as possible to those in the proposed experimental design. During the first half of year 2000, the research team traveled extensively throughout Indiana to find and select field sites. All candidate sites were initially identified based on pavement type, drainage, and roadway classification. Approximately forty potential sites were selected according to the refined experimental design. The pavement types were identified by the pavement structure history records at INDOT or by direct pavement core information. Drainage conditions were identified by field inspection at each potential project. To minimize the climate impact on pavement performance, the selected sites were all in north or central Indiana, except for one site in southern Indiana. The pavement surfaces for all sites were in approximately equal condition. This helped to ensure that both sealed and unsealed sections were at the same baseline before the sealing treatment was applied.

6.1.1 Pavement Type

Pavement types are identified as HMA, PCC, or composite. The "1999 Pavement Surface Report," provided by the Pavement Management Unit at INDOT, was used for identification during the site search. This report is a compilation of pavement condition data for pavement surfaces in the INDOT highway system. It assisted in selecting test sites by providing pavement age, type, location and current condition. However, in most cases, the pavement type in the report is for the current surface at a given location, and there is very little history information available. For instance, the report classifies composite pavements as HMA if the surface is HMA, regardless of what may lie beneath. For this study, if the report classified the surface as HMA and there was insufficient pavement history to determine the pavement type, field cores were taken in order to clearly determine the pavement type. Figure 6-1 shows the coring operation in the field, while Figure 6-2 shows a composite pavement core sample.

Figure 6-1 Coring Operation

Figure 6-2 Core Sample of Composite Pavement

6.1.2 Drainage Condition

No records of drainage condition were avaiable for the INDOT highway system. Field inspection for each potential site was conducted to identify its drainage condition. Exposed drainage pipes along the edge of pavement subgrade in highway fill sections indicate pavement drainage. At least one drainage pipe usually appears every 152 m (500 ft), exiting to the roadside or to the median. If no drainage pipes were found at the test site, the test site was judged to have no drainage. Experienced engineers at various INDOT districts were aslo contacted to confirm the no-drainage condition for the sites before they were finally selected.

6.1.3 Road Classification

All potential sites were interstate, US, and state highways. Interstate and US highways are classified as part of the National Highway System (NHS), while state highways are classified as Non-National Highway System (Non-NHS). According to the refined experimental design, this factor is used as a surrogate for two factors: traffic and thickness. Since pavement thickness is a function of expected traffic, NHS classification represents high traffic conditions, while Non-NHS classification represents lower traffic conditions.

6.1.4 Site Search Result

Based on the refined experimental design, forty potential sites were found during the initial site search. In accordance with recommendations made during SAC meetings, the modifications shown in Table 6-1 were made.

Table 6-1 Refined Experimental Design

Roadway Classification	Pavement Type					
	PCC		HMA		Composite	
	Drain	No-drain	Drain	No-drain	Drain	No-drain
NHS	1a1& 1a2	3a	5a	7a	9a	11a
	1b	3b	5b	7b	9b	11b
Non-NHS	2a	4a	6a	8a	10a	12a
	2b	4b	6b	8b	10b	12b

Of the proposed 24 sites shown in Table 6-1, 3A and 3B were excluded from the research, because PCC, no-drainage NHS pavements are not typical in Indiana. All new NHS PCC pavements are required to have a drainage system as specified by the Indiana Highway Design Manual. Sites 6A and 6B were de-prioritized. After several months of searching, not a single potential site was found for these cells. This is most likely due to the fact that a high percentage of HMA surfaced pavements are composite pavements, and there are very few HMA pavements in non-NHS with drainage. No site was found for the 5A. All nineteen sites were finally chosen from among the original forty potential sites. Figure 6-3 shows the locations of the test sites. Table 6-2 gives the position, length, and pavement type for each site.

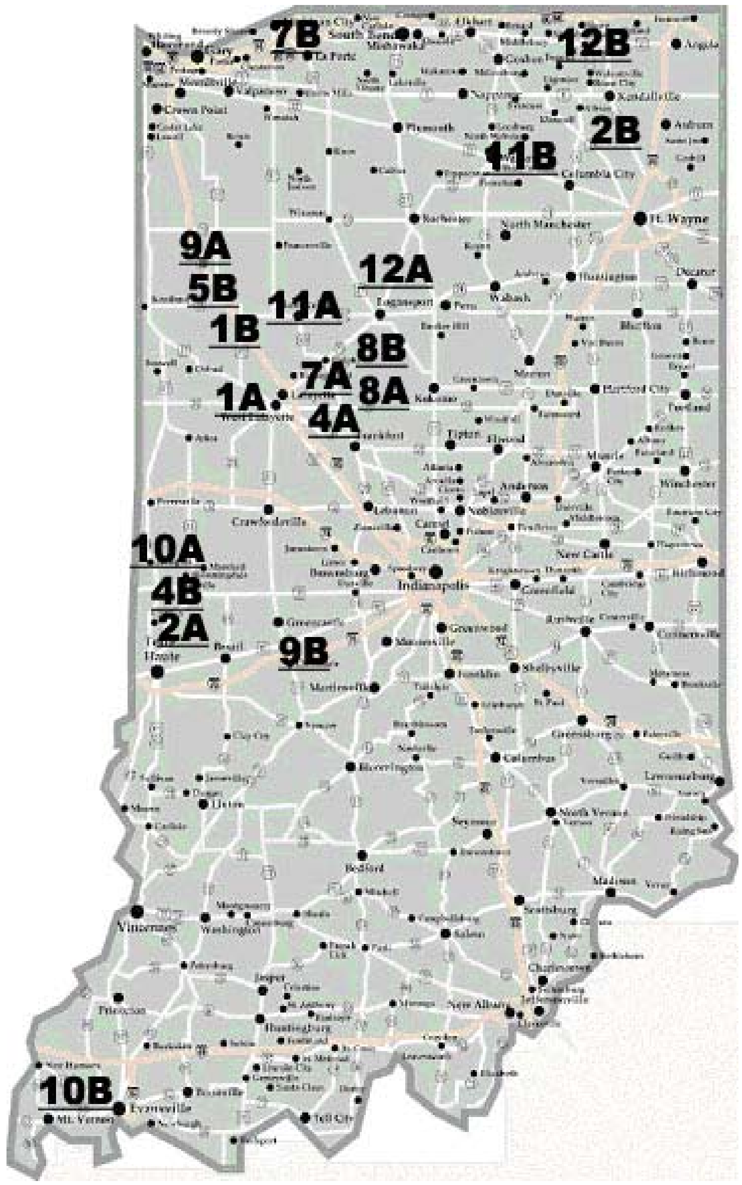

Figure 6-3 Location of the Final Nineteen Test Sites

Table 6-2 Test Sites List for Cost-Effectiveness of Joint/Crack Sealing

District.	Site No.	Road	Pavement	Drain	Direction	SITE Location	Length (Ft)				Note
							Sealed Section	Interval Section	Unsealed Section	Total Site	
CVI[1]	1A1	US231	PCC	Yes	NB	South River Bridge	144	103	162	409	
CVI	1A2	US231	PCC	Yes	NB	North River Bridge	160	284	158	602	
LAP[2]	1B	I-65	PCC	Yes	NB	MM217 + 930	165	403	165	733	
CVI	2A	SR63	PCC	Yes	NB	MM82 + 667	220	120	220	560	
FTW[3]	2B	SR3	PCC	Yes	SB	MM193 + 395	157	92	169	418	
	3A		PCC	No							Cancelled
	3B		PCC	No							Cancelled
CVI	4A	SR38	PCC	No	EB	MM4 + 67	160	149	160	469	
CVI	4B	SR63	PCC	No	SB	MM91 + 4255	199	200	220	619	
	5A		HMA	Yes							Not found
LAP	5B	I-65	HMA	Yes	NB	MM224 + 0	402	404	711	1517	
	6A		HMA	Yes							Low priority
	6B		HMA	Yes							Low priority
CVI	7A	US421	HMA	No	NB	MM126 + 4616	337	144	373	854	
LAP	7B	US35	HMA	No	NB	MM35 South of I80				0	
LAP	8A	SR18	HMA	No	EB	MM64 + 454	412	107	249	768	
LAP	8A	SR29	HMA	No	SB	MM29 + 919	434	244	735	1413	
LAP	9A	I-65	Composite	Yes	NB	MM232 + 68	590	234	403	1227	
CVI	9A	I-74	Composite	Yes	WB	MM31 + 1673	961	257	1005	2223	
CVI	10A	SR63	Composite	Yes	NB	MM93 + 7	283	159	200	642	
VIN[4]	10A	SR62	Composite	Yes	WB	MM3 + 0	1230	200	625	2055	
LAP	11A	US24	Composite	No	EB	MM32 + 7 (3902)	987	300	814	2101	
FTW	11A	US30	Composite	No	WB	MM103 - 217	518	169	635	1322	
LAP	12A	SR25	Composite	No	NB	MM78 + 2746	605	310	1253	2168	
FTW	12A	SR9	Composite	No	NB	MM175	492	713	482	1687	

[1]Crawfordsville District [2]Laporte District [3] Fort Wayne District [4]Vincennes District.

6.2 Site Preparation

To investigate the cost-effectiveness of joint and crack sealing in relation to pavement performance, each selected test site was divided into one sealed section and one unsealed section. The pavement performance for these two sections was statistically analyzed to evaluate the effect of treatment. In order to minimize other effects on pavement performance, the following criteria were followed when the sealed and unsealed sections of the test sites were selected:

1. The pavement surface for test sites should be in fairly good condition with no severe distresses except the transverse cracks. This should minimize the effect of pavement age, and provide an opportunity to evaluate the effect of sealing treatment in future research;
2. Each section should contain twelve transverse cracks and/or joints. Twelve was selected by the SAC members in an attempt to ensure enough data to complete the research;
3. The pavement conditions for sealed and unsealed sections should not be significantly different;
4. The lengths for sealed and unsealed sections need not necessarily be equal, but they should be similar;
5. Sealed and unsealed sections should have the same alignments. Straight and flat highway sections are ideal test site locations; and

6. No intersections or exits should be contained within the test sites. This acts to ensure equivalent traffic volumes for the sealed and unsealed sections.

In the refined experimental design, each section in the test site is approximately 61.0 m (200 ft) in length. This section length allows twelve or more transverse joints/cracks. In practice, however, the section length varies from 30.5 m (100 ft) to 152.5 m (500 ft), since twelve joints/cracks are included in each sealed or unsealed section. The transition zones between the sealed and unsealed sections are typically about 76 m (250 ft). Figure 6-4 shows the plan of a typical project.

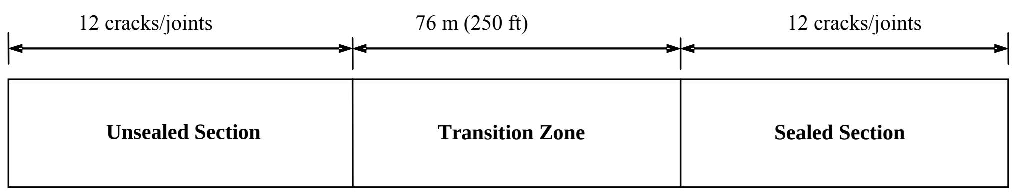

Figure 6-4 Typical Test Site Plan View

An extremely important component of the field experiment was maintaining the individual test sections. Once a test site was identified, the unsealed section, transition zone, and sealed section were marked on the pavement. For the sealed section, all joints/cracks were sealed using typical INDOT quality materials and practices. For the unsealed section, all transverse joint and crack sealants were removed. Both sealed and unsealed sections were then rigorously maintained throughout the duration of the performance monitoring period. Regular inspection of the sites was conducted every three months. When the inspection evaluation suggested that additional resealing or sealant removal was necessary, the maintenance was scheduled and completed the following autumn.

6.2.1 Sealed Section

All transverse and longitudinal joints/cracks in the sealed section were sealed using typical INDOT quality materials and practices. Sealing operations were conducted when air temperatures were moderately cool, around 7 to 18C (45 to 65F), usually in autumn. Hot-applied thermoplastic materials, such as crumb rubber and AE90S, were used for sealing materials. Joints and/or cracks in PCC pavements were cleaned before sealing. The HMA pavement cracks were typically routed and cleaned before being sealed. The standard reservoir-and-flush placement configuration (Figure 6-5) was implemented if crumb rubber or AE90S, an asphalt emulsion, were used for sealing HMA pavement cracks. Silicon caulking and Crack Stix are two alternatives under certain circumstances. However, since only AE90 was available in some districts, flush-fill (Figure 6-6) was used as an alternative sealant placement configuration. In this configuration, crack routing is not required and the AE90 can be poured directly over joints/cracks.

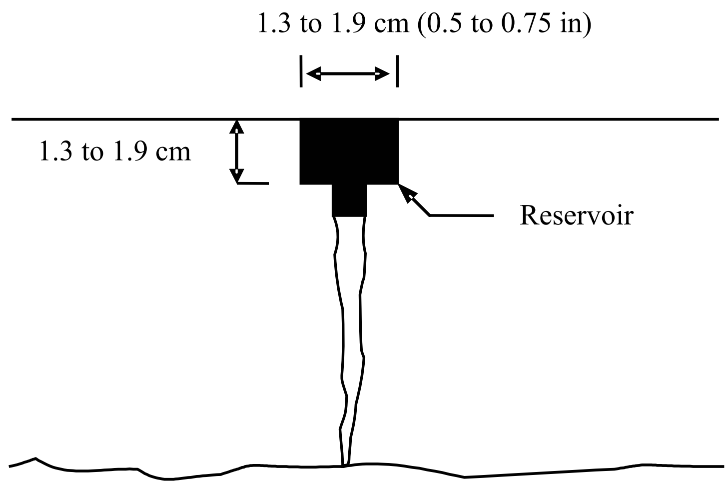

Figure 6-5 Standard Reservoir-and-Flush

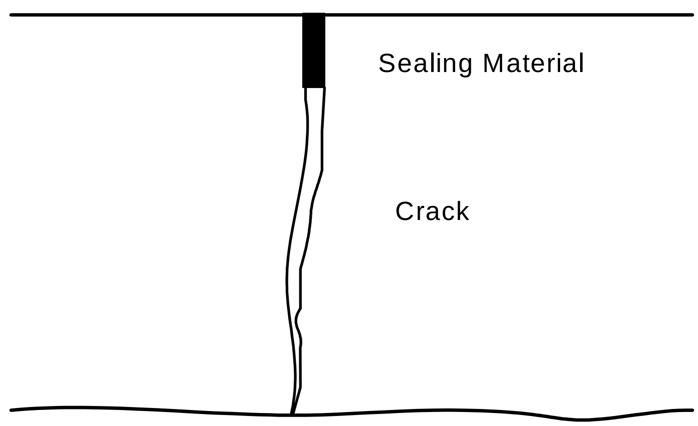

Figure 6-6 Flush-Fill

The following sealing procedures apply to the project (some districts may follow other standard procedures). If a standard reservoir and flush configuration is used, the HMA pavement cracks are routed 1.3 cm × 1.3 cm to 1.9 cm × 1.9 cm. After being cleaned with an air compressor, joints and/or cracks or routed cracks are sealed with crumb rubber, AE90S, or AE90, and the sealant is shaped using a squeegee. The pavement is kept closed to traffic for 20 minutes to cure the sealant.

The objective of routing a crack is to create a uniform, rectangular reservoir, centered as closely as possible over the crack, while inflicting as little damage as possible to the surrounding pavement. Because crack cutting can inflict additional damage to the pavement and is often the slowest activity in sealing operations, it is desirable to use a high production machine that follows cracks well and produces minimal spalls or fractures.

Cleaning and drying joints and cracks provides a clean, dry crack channel for the sealant material. This is perhaps the most important aspect of sealing and filling operations because a high percentage of treatment failures are adhesion failures that result from dirty and/or moist crack channels.

Hot-applied materials are usually heated with an asphalt melter, as shown in Figure 6-7. The sealant material must reach the designed operation temperature before the sealing operation starts. The operations for sealing on PCC and HMA pavements are shown in Figure 6-8 and Figure 6-9, respectively.

Figure 6-7 Crack Sealing Operation

Figure 6-8 Sealing an HMA Pavement Crack

Figure 6-9   Sealing a PCC Pavement Joint

Occasionally, scheduling conflicts arose with the sealing crews. This led to the use of two alternative sealing operations, silicon caulking and Crack Stix. These two operations are explained in detail as follows.

Silicon Caulking

Silicon sealants are widely used to seal and bond several kinds of materials, such as metals, plastic and concrete. Silicon caulking operations with a silicon gun (Figure 6-10) were performed only on PCC pavements. The advantage of this operation is that it is very efficient to reseal small areas of joints and cracks since limited traffic controls are required.

Figure 6-10 Silicon Caulking Operation

Crack Stix

Crack Stix, the material shown in Figure 6-11, is a direct heat rubberized joint and crack sealant. It does not need heating or mixing before being used. In practice, Crack Stix is much better than cold-pour or caulking type sealants. However, it is not efficient to seal when sealing large numbers of joints and cracks.

To apply Crack Stix, the joint or crack is first cleaned using a screwdriver and whisk broom. The Crack Stix is then cut to the appropriate length and pressed into the joint or crack with fingertip pressure until it is approximately 0.3 to 0.4 cm (0.13 to 0.17 in) below the actual pavement surface. A propane torch is then used to heat the material thus expanding it to completely fill the joint and crack. Traffic is kept off the material for 20 minutes after the application. A PCC crack sealed with Crack Stix is shown in Figure 6-12.

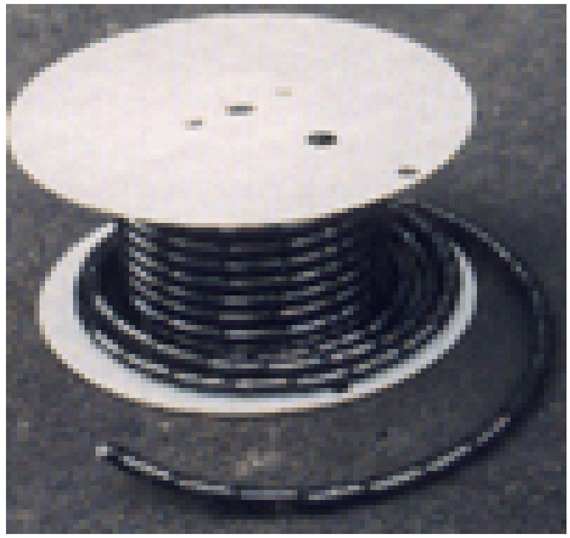

Figure 6-11   Crack Stix

Figure 6-12   PCC Crack Sealed by Stix

6.2.2 Unsealed Section

For the unsealed sections, the ideal candidates were sites where all the joints and cracks were already unsealed. However, most joints and cracks on INDOT highway surfaces are regularly sealed to prevent water penetration. For sealed sections that were selected to be unsealed sections, it was necessary to remove sealants from the twelve transverse joints and/or cracks. Three different methods were used to remove sealants; pulling out sealants, cutting out the sealant, and routing.

Pulling sealants out of joints and cracks by hand was accomplished using a hook tool designed for the purpose. The tool was very efficient and accomplished the task without damaging to the pavements. Figure 6-13 shows an example of the operation, while Figure 6-14 shows the joint condition after the sealant was removed. Based on field testing, this method was not deemed practical in removing sealant from HMA pavements. This is likely due to the fact that the sealant materials typically infiltrate into the cracks instead of staying at the pavement surface.

Removing sealant by cutting the pavement was a second method. A machine specially equipped for the task was used. Figure 6-15 shows the cutting operation of a sealed crack. While the machine was very efficient in removing sealant from HMA pavement cracks, the width of the resulting cut is only about 0.14 cm (0.06 in), and some sealant normally remains around the cracks (see Figure 6-16). These remaining sealant materials may clog the cracks when the pavement temperature becomes elevated.

Figure 6-13 Sealant Removal Operation

Figure 6-14 Joint Condition after Sealant Removal

Figure 6-15 Cut-off Machine

Figure 6-16 Crack after Cut

A router was also used to open cracks for unsealed sections. Figure 6-17 shows the routing operation, while Figure 6-18 shows a routed crack in an unsealed section. The width of the opened cracks is about 1.3 cm (0.5 in), which is wider than the original cracks.

Figure 6-17   Crack Routing for Unsealed Section

Figure 6-18   Routed Crack in Unsealed Section

6.3 Data Collection

FWD measurements were taken on both the sealed and unsealed sections. The FWD data taken across joints and cracks was used as a measure of the load transfer; FWD data was also taken between the joints and cracks. The International Roughness Index (IRI) values were determined based on pavement roughness measurements of both the sealed and unsealed sections. Visual condition surveys (distress surveys) were conducted to assess the severity and extent of individual distresses such as faulting and cracking. Pavement cores were collected near joints and cracks in order to investigate both the physical and mechanical properties of the pavement.

6.3.1 FWD Deflection and Load Transfer

Nondestructive deflection testing has been used to evaluate pavement structures and rehabilitation processes for many years. The pavement deflection measured under a particular load can be used as a direct indicator of the pavement structural capacity. The FWD used for this project was the Dynatest Model 8000, shown in Figure 6-19.

FWD measurements were conducted annually at each of the test sites from the end of May through early October, when subgrade conditions were relatively dry. In addition to the structural capacity evaluation, the load transfer across joints and cracks was also used to evaluate pavement performance. Figure 6-20 illustrates the FWD testing geometry and the spacing of the deflection sensors during measurements. The deflection at the first sensor (D_0) was used to measure the pavement deflections between joints and cracks, while both the first (D_0) and second (D_1) sensors were used for load transfer across the joints and cracks. The normalized deflection ratio D_1/D_0 was used as a measure of the load transfer.

Because pavement serviceability deteriorates over the life of a pavement it is expected that the pavement deflection and load transfer capability also deteriorate with increased pavement life as illustrated by the thin line in Figure 6-20. It is hypothesized that sealing joints and cracks retards the deterioration of the pavement. Consequently, the pavement deflection and load transfer should be greater for a sealed pavement as a function of time, as illustrated by the thick line in Figure 6-21.

Figure 6-19 Dynatest Model 8000

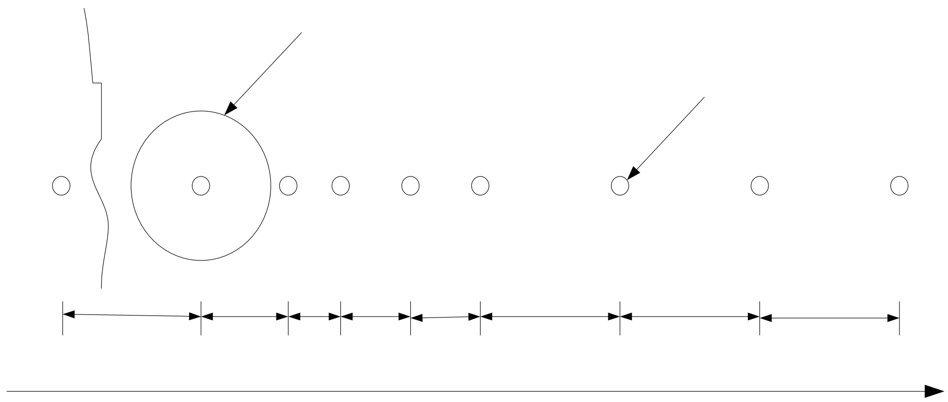

Figure 6-20   FWD Sensor Spacing and Geometry of Test

Crack/Joint

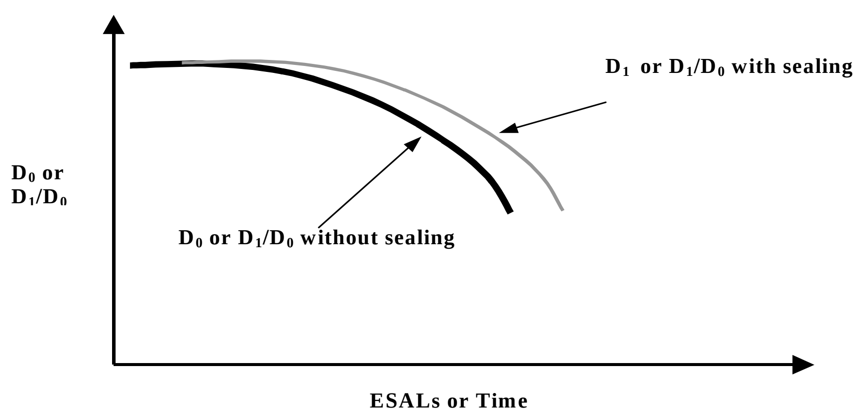

Figure 6-21   FWD Deflection versus Time

D1

From a statistical standpoint, if the difference between the thin and thick lines or between their slopes is significant, then the sealing of joints and cracks has an effect on performance. On the other hand, if the difference is not significant, then the sealing of joints and cracks does not have an effect on performance. The difference between the two lines results in a quantitative measure of the cost effectiveness of joint and crack sealing. The pavement deflection/load transfer provides quantification of the cost associated with sealing the joints and cracks in different pavement types. Consequently, recommendations can be made as to the cost effectiveness of sealing on these pavement types. A seal/no-seal policy can then be recommended based on the quantitative analysis.

Since the load transfer might be significantly different at different test locations, particularly for HMA pavements, even at the same crack, it was decided to test the same spot as closely as possible for each joint and crack in different data collection seasons to minimize data collection errors. In practice, each test spot was first selected and marked with nails, and painted as the reference point for the FWD, as shown in Figure 6-22. For all subsequent tests, the marked point was matched as close as possible to the FWD. The FWD data at each joint and crack were used to develop the load transfer ratio; the deflections between joints and cracks were used to evaluate pavement performance directly. Figure 6-23 shows the FWD deflection test spots across a joint or crack and between the joints and cracks.

Figure 6-22   Mark for FWD Sensor Arm

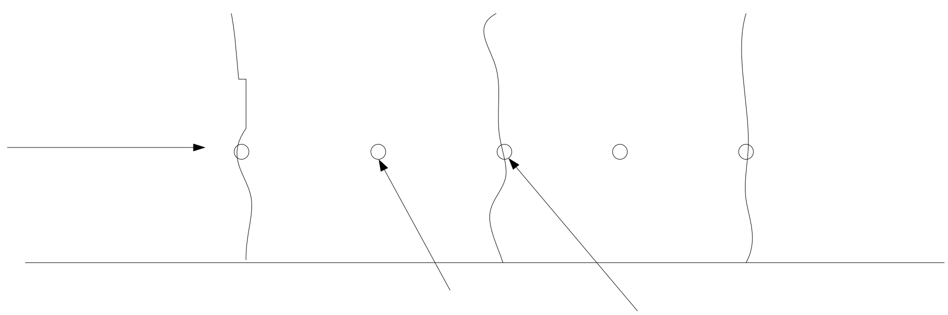

Figure 6-23   Plain View of FWD Data Collection Spots

After test sites were selected, the test spots both at the joints or cracks and between the joints or cracks were selected at the middle of the travel lanes, and each test spot was marked with paint and nails to define a reference point as shown in Figure 6-24, and Figure 6-25. As part of the FWD collection procedure, the site name, test time, temperature and FWD file name are required (Table 6-3). For each field test, the FWD sensor arm must be spotted as closely as possible to the marked spot. The deviation of distances between the actual test and marked spots should be less than 5 cm (2 in). Each deviation value was recorded on the FWD test sheet. For each joint and crack, care was taken to make sure the tested joint and crack was located in midway between sensors D_0 and D_1 before testing (Figure 6-26).

Figure 6-24 Marked Test Spot

Figure 6-25 Marked Test Spots

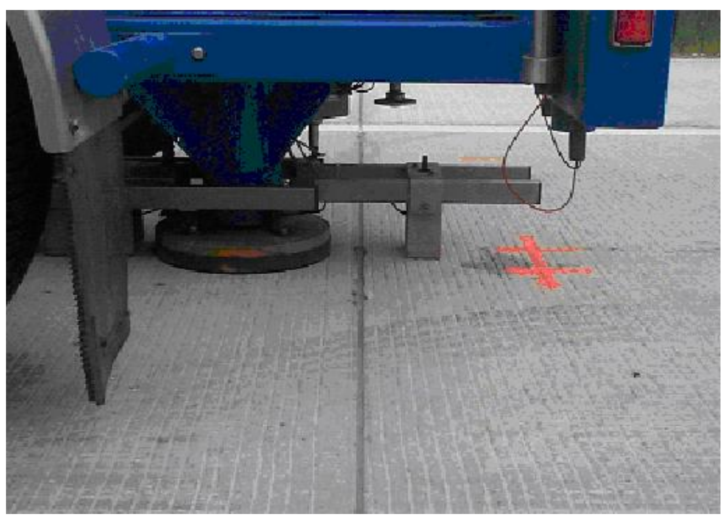

Figure 6-26 Joint/Crack Test between Two Sensors

Table 6-3 Example of FWD Test Form

FWD Test Form

Location: SR 62 Proj #: 10B Section: SEALED

Date: 8-8-00 Time: 9:30 Attendants: Fang, Danny

FileName: SR62_10

No	Test1	Test2	Test3	Position Y	Position X
				200	
12	3 R	0	0	1230	5
	1 R	0	0	1210	5
11	1 R	1 R	0	1179	5
	2 R	2 R	1 R	1149	5
10	0	2 R	2 R	1119	5
	0	0	1 R	1072	5
9	2 R	2 L	1 L	1045	5
	0	0	0	990	5
8	2 R	0	0	795	5
	2 R	1 R	2 R	752	5
7	2 R	2 R	2 R	722	5
	0	1 R	2 R	670	5
6	1 R	0	0	528	5
	2 R	1 R	2 R	442	5
5	1 R	1 R	1 R	424	5
	1 R	2 R	2 R	406	5
4	1 R	0	1 R	346	5
	1 R	0	0	304	5
3	1 R	1 R	1 R	220	5
	0	1 R	0	158	5
2	0	0	0	127	5
	1 R	1 R	1 R	77	5
1	2 R	1 R	1 R	45	5
	2 R	1 L	0	0	5

Notes
Dash Line indicates the test position for middle slab; Solid line indicates the test position for joints/cracks;
Y is the test distance from joint 1. X is the distance from white line near the shoulder.
“R/L” indicate the test distance deviation was right/left direction, and the first number is the dista nce in inch.

Initially each test location was tested three times with the FWD. At the end of the year 2000, after analyzing the data collected from several different sites, it was shown that one test at each FWD location provides sufficient accuracy for the research (see Chapter 7). It was therefore decided to test only once at each FWD test site annually.

6.3.2 Pavement Roughness

Pavement roughness can be described by the magnitude of longitudinal profile irregularities and their distribution over the measurement distance. The American Society of Testing and Materials (ASTM), ASTM E 867 (1998) defines roughness as, "…the deviations of a pavement surface from a true planar surface with characteristic dimensions that affect vehicle dynamics, ride quality, dynamic loads, and drainage, for example, longitudinal profile, transverse profile and cross slope." There are several possible sources of pavement roughness (Yoder and Hampton, 1958):

1. Construction techniques that allow deviations from the design profile;
2. Repeated loads, particularly in channelized areas, that cause pavement distortions by plastic deformation in one or more of the pavement components;
3. Frost heave and volume changes due to shrinkage and swell of the subgrade; and
4. Non-uniform initial compaction.

Pavement roughness is measured by engineers for several different purposes (Hudson 1981). It has been used to measure and control pavement construction quality, to locate abnormal changes in the highway, such as drainage, subsurface problems, or extreme construction deficiencies, to help establishing a statewide basis for allocating road maintenance resources, and to evaluate pavement serviceability-performance life histories for evaluation of alternate designs.

The most well-known profile-based mechanical system simulation index is the International Roughness Index (IRI). IRI is a scale of roughness based on the response of a generic motor vehicle to the roughness of a pavement surface. It is obtained by simulating the response of a Response-Type Road Roughness Measuring (RTRRM) system as it travels the road profile. The response properties of an automobile are simulated by a relatively simple dynamic model commonly known as the quarter-car model. As shown in Figure 6-27 (Sayers and Karamihas, 1998), the parameters of the quarter-car model include the sprung mass of the vehicle body, the suspension spring and the damper constants, the unsprung mass of the suspension, tire, and wheel, and the spring constant of the tire. Pavement surface profiles provide input to the car, which flex the tire as the spring constant, stroke the suspension, and cause the sprung and unsprung masses to vibrate in the vertical direction. This simulated suspension motion response is accumulated and divided by the distance traveled to give an index with units of m/km (in/mile) (Shahin, 1994)

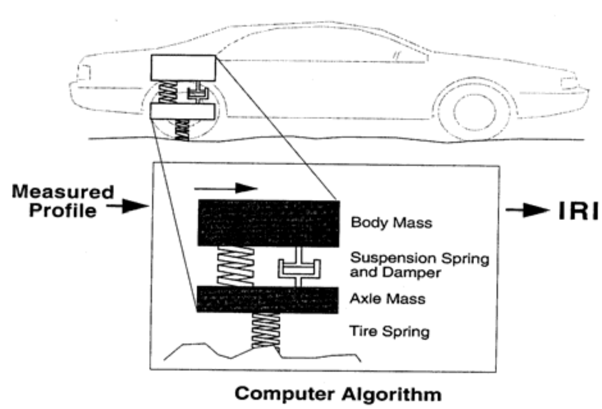

Figure 6-27 Schematic of Computer Algorithm Used to Compute IRI

In 1989 the FHWA adopted IRI as the standard for measuring smoothness and has required all states to report pavement smoothness in terms of IRI units for interstate highways as well as paved rural and urban freeways (Queiroz and Hudson, 1984). In this study, IRI measurements were chosen as one index to evaluate the difference in ride quality between sealed and unsealed sections. This IRI data for this study was first collected in 2001 and annually thereafter.

6.3.3 Condition Survey

The Long-Term Pavement Performance (LTPP) Distress Identification Manual (SHRP, 1993) was used to conduct condition surveys throughout the duration of the study. In 1987, the Strategic Highway Research Program (SHRP) began the LTPP program. The purpose of the program is to collect data on pavement condition, climate, and traffic volumes and loads for more than a thousand pavement test sections over a 20 year period. This information should provide engineers with the ability to design better, longer-lasting pavements. The distress manual was developed to provide a consistent, uniform basis for collecting distress data for the LTPP program.

The distress manual is divided into three sections, each focusing on a particular type of pavement: 1) HMA surfaced pavement, 2) Jointed PCC, and 3) Continuously reinforced PCC. In this study, only the fist two sections were used, since none of the selected test sites are continuously reinforced PCC pavements. The various types of distresses and units of measurement identified by the LTPP Distress Identification

Manual are summarized in Table 6-4 for HMA surfaced pavements (including HMA and composite pavements) and in Table 6-5 for PCC surfaced pavements.

Table 6-4 Distress Types and Measurements Units for HMA Pavements

Distress categories	Distress	Unit of Measurement	Defined Severity Levels?
Cracking	Fatigue Cracking	M^2	Yes
	Block Cracking	M^2	Yes
	Edge Cracking	M	Yes
	Longitudinal Cracking	M	Yes
	Reflection Racking	Number, M	Yes
	Transverse Cracking	Number, M	Yes
Patching & Pothole	Patch/ Deterioration	Number, M^2	Yes
	Potholes	Number, M^2	Yes
Surface Deformation	Rutting	Millimeters	No
	Shoving	Number, M^2	No
Surface Defects	Bleeding	M^2	Yes
	Polish Aggregate	M^2	No
	Raveling	M^2	Yes
Miscellaneous Distresses	Lane-to-shoulder Drop off	Millimeters	No
	Water Bleeding & Pumping	Number, M	No

Table 6-5 Distress Types and Measurement Unit for PCC Pavements

Distress categories	Distress	Unit of Measurement	Defined Severity Levels?
Cracking	Corner Breaks	Number	Yes
	Durability Cracking	Number, M^2	Yes
	Longitudinal Cracking	M	Yes
	Transverse Cracking	Number, M	Yes
Joint Deficiencies	Transverse Joint Seal Damage	Number	No
	Longitudinal Joint Seal Damage	Number, M	Yes
	Spalling of Longitudinal Joints	M	Yes
	Spalling of Longitudinal Joints	Number, M	No
Surface Defects	Map Cracking	Number, M^2	No
	Scaling	Number, M^2	No
	Polished Aggregate	M^2	No
	Popouts	Number, M^2	No
Miscellaneous Distresses	Blowups	Number, M	No
	Faulting of Transverse Joints and cracks	Millimeters	No
	Lane-to-Shoulder Dropoff	Millimeters	No
	Lane-to-Shoulder Separation	Millimeters	No
	Patch/Patch Deterioration	Number, M^2	Yes
	Water Bleeding and Pumping	Number, M	No

Surveying the condition of test sites followed the instructions provided by the LTPP Distress Identification Manual. The following equipment was used for performing field distress surveys.

1. Distress Identification Manual. This manual provides a description of all distress types, measurement methods, and detailed instructions for surveys.
2. Extra blank data sheets and maps. A map sheet contains one 25×13 cm (10×5 in) map that represents a 10×4 m (30 ×15 ft) area of the test section.
3. Clipboard, pencils, measurement tape, and measurement wheel.
4. A straight edge, 1.2 m (4 ft) long. This edge is used to record the maximum rut depth in millimeters for HMA surfaced pavements.
5. Digital camera. Pictures were taken of each transverse joint and crack during the first condition survey in 1999. These pictures were kept as historic documents and will be reviewed in the future to visually compare joint and crack conditions; and
6. Video camera. Videos were taken for each sealed and unsealed section during the first condition survey in 1999. These videos were kept as historic documents and will be reviewed in the future to visually compare pavement conditions.

The most important part of the condition survey is the distress mapping, which shows the exact location of each existing distress type in the section. The distress types and their severities are also identified on the distress map. An example from the SR38 map is shown in Figure 6-28.

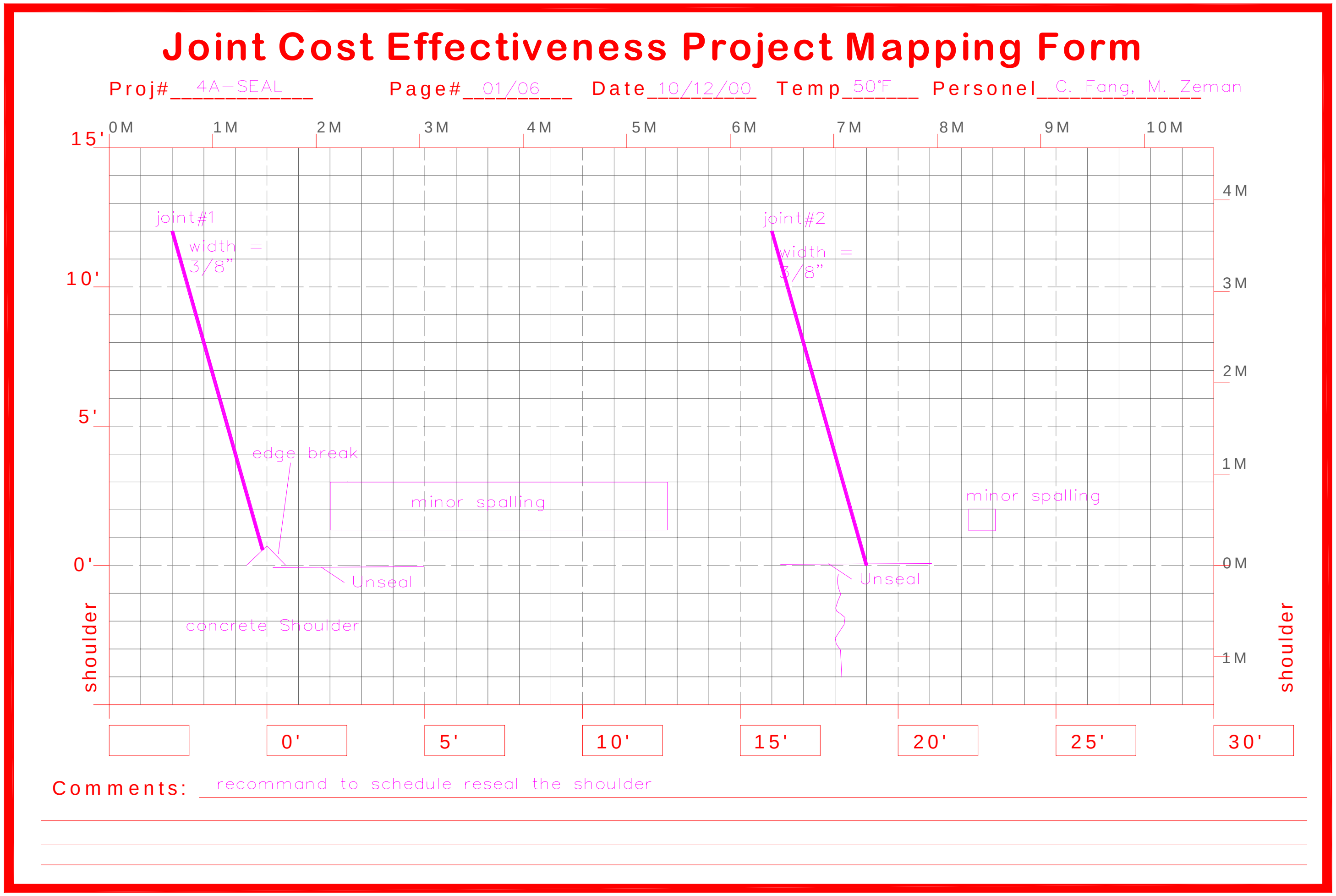

Figure 6-28 Survey Map of SR38

Condition surveys to monitor overall pavement conditions during the experimental period were performed twice during the research. The first survey was conducted in 1999 when the study began. The second survey was conducted in 2002 at the conclusion of the study.

6.3.4 Pavement Core Samples

Core samples were collected near the joints and cracks to investigate both physical and mechanical properties of the pavement. Most HMA pavement test sites were sampled at the beginning of the research. Usually the core sample was taken from the center of the travel lane. The typical core diameter was 30.5 cm (12 in), and the depth depended on the pavement structure and the core bit length. Usually, the core operation stopped when the subgrade was reached or when the core bit reached the maximum length. Within the limits of the core bit length, the thickness of each layer, material of each layer and the subgrade materials for each test site were determined from the core sample. Core samples from six test sites are listed in Appendix C.

CHAPTER 7 DATA ANALYSIS

In order to quantitatively measure benefits, the pavement performance was monitored periodically. Since pavement conditions and material properties do not vary significantly over short time periods, condition surveys and core samples are usually used to investigate the long-term pavement performance. Due to the relatively short term nature of the research, only FWD deflection, load transfer, and IRI statistical analyses are presented here.

Using FWD and IRI data collected during the project, statistical analyses were performed to determine whether significant differences could be found in pavement performance between sealed and unsealed sections. Since these statistical analyses involved the study of the effects of several factors, such as pavement type, drainage condition, and road classifications, factorial experimental designs were conducted to investigate all possible combinations of the factor levels in each complete trial or replication. This method provides statistical results that can be used to determine the effectiveness of sealing in general for all pavement types, for specific pavement types regardless of roadway classification, for specific pavement types and roadway classifications, and for specific pavement types, roadway classifications and drainage conditions. Applying this technique to the performance measures collected during the study can help to determine if sealing is effective as well as if it is effective for individual pavement types exposed to different loading and drainage conditions.

7.1 Temperature Correction for FWD Deflection

The FWD deflections for both HMA and PCC pavements are affected by pavement temperature. Since HMA changes consistency with temperature, pavement temperature is a major environmental factor affecting the pavement surface deflection. When a PCC pavement is subjected to a temperature gradient through its depth, the slabs tend to warp. This can cause significant deflection differences under any given load. As a result of pavement sensitivity to temperature changes, the deflections under the FWD loading system must be standardized to an arbitrary temperature. In general, measured deflections were adjusted to a reference pavement temperature (usually 20 or 25C). As the derived factor from FWD deflections, the load transfer may be also dependent on the test temperature as well. However, only a temperature correction for D_0 is found in the literature. To address the temperature issue during the measurement of load transfer, field FWD measurements were collected at different temperatures on five different test sites, and statistical models were developed based on the data. These models are used to predict the FWD measurements at one reference pavement temperature. The details for the temperature correction models are presented in Chapter 9.

7.2 Test Repetition

At the beginning of the data collection, it was decided to conduct three repetitions of FWD measurements for each sealed and unsealed section during each data collection season. In the field test, it was very difficult to set the FWD load plate at the exact test spot position. Usually, between the actual testing position and the marked test spot, there was a 30-150mm test position deviation. Fearing that this might cause a significant variation in the FWD results, three FWD repetitions were used to statistically minimize the data collection errors caused by test position deviations. However, this proved to be very inefficient, and after the first year of data collection, it was thought that one repetition might be sufficient. First, the variation caused by the distance deviation is most likely smaller than the FWD system error. It might also be smaller than the temperature correction error. Secondly, since different loads are dropped at each test location, three repetitions were already being done.

In order to determine whether three repetitions were needed at each FWD test location, a statistical model was designed with three factors, repetition, crack and load to show the significance of repetition in FWD measurements. This model is shown in Equation 7-1.

$$y_{ijk} = u + \tau_i + \beta_j + \gamma_k + (\tau\beta)_{ij} + (\tau\gamma)_{ik} + (\beta\gamma)_{jk} + + \varepsilon_{ijk} \qquad \begin{cases} i = 1,2,3 \ \mathrm{Re}\textit{petition} \\ j = 1,2,\ldots 12 \ \textit{Crack} \\ k = 1,2,3 \quad \textit{Load} \end{cases} \qquad \text{7-1}$$

where,

μ is the mean value of load transfer ratio;

τ_i is the effect of the ith Repetition;

β_j is the effect of jth Crack;

τ_k is the effect of kth Load;

$(\tau\beta)_{ij}$ is the Repetition × Crack interaction;

$(\tau\gamma)_{ik}$ is the Repetition × Load interaction;

$(\beta\gamma)_{jk}$ is the Crack × Load interaction; and

ε_{ijk} is the error term.

The FWD data collected on Project 9A (I-65) in 2000 was selected as the example input data. The analysis of variance for the load transfer is shown in Table 7-1.

Table 7-1 Analysis of Variance for the Load Transfer Ratio on Project 9A

Source	DF	Type I SS	Mean Square	F value	Pr > F
Repetition	2	0.01560237	0.00780118	69.40	<0.0001
Crack	11	0.30772432	0.02797403	248.87	<0.0001
Load	2	0.00288937	0.00144468	12.85	<0.0001
Repetition*Crack	22	0.01419016	0.00064501	5.74	<0.0001
Repetition*Load	4	0.00053383	0.00013346	1.19	0.3297
Crack*Load	22	0.00238451	0.00010839	0.96	0.5225

Table 7-1 shows that the P-values for repetition, crack, load and repetition×crack are all less than 0.0001, indicating that all three factors are significant, and there is a correlation between repetition and crack. However, the mean square of repetition and load are much smaller than of crack.

In order to see how significant the repetition factor is, the relative residual is calculated for load transfer ratio and FWD deflection. Both relative repetition errors for ratio and deflection are approximate 2 percent. This error is relatively small when compared to the combined FWD system and temperature correction error, which is believed to be approximately 10 percent. It was therefore decided to conduct only one repetition of FWD testing during all following data collection seasons.

7.3 Factorial and Nested Designs

The research involved the study of the effects of several factors, such as pavement type, drainage condition, and road classifications. The most efficient design for this type of experiments is a factorial design, in which all possible combinations of the factor levels are investigated in each replication. Factorial designs have several advantages. First, they are more efficient than one-factor-at-a-time experiments. Second, a factorial design is necessary when interactions may be present in order to avoid misleading conclusions. Third, factorial designs allow the effects of a factor to be estimated at several levels of the other factors, yielding conclusions that are valid over a range of experimental conditions.

The linear statistical model with both factorial and nested factors was developed according to the experiment design. In addition to pavement (Pave), drainage (Drain) and classification (Class), there are another two factorial factors: treatment (Treat) and time (Time). Project (Proj) is nested in Pave, Drain and Class in the factorial and nested design, since each project is always unique for each combination of those three factors.

The three factor levels of Pave are PCC pavement, HMA pavement, and composite pavement. Drain has two factor levels, drained and undrained. The factor levels of Class are National Highway and Non-National Highway. The two Treat factor levels are sealed and unsealed, while two Time factor has two factor levels, 2001 and 2002. The Treat is the fixed factor and Proj is the random factor. The linear analysis model for this design is shown in Equation 7-2.

$$
\begin{aligned}
y_{ijklmtn} &= \mu + \tau_i + \beta_j + \gamma_k + (\tau\beta)_{ij} + (\tau\gamma)_{ik} + (\beta\gamma)_{jk} + (\tau\beta\gamma)_{ijk} + \phi_{l(ijk)} \\
&+ \theta_m + (\theta\tau)_{mi} + (\theta\beta)_{mj} + (\theta\gamma)_{mk} + (\theta\tau\beta)_{mij} + (\theta\tau\gamma)_{mik} + (\theta\beta\gamma)_{mjk} + (\theta\tau\beta\gamma)_{mijk} \\
&+ (\theta\phi)_{ml(ijk)} + \delta_t + (\delta\tau)_{ti} + (\delta\beta)_{tj} + (\delta\gamma)_{tk} + (\delta\tau\beta)_{tij} + (\delta\tau\gamma)_{tik} + (\delta\beta\gamma)_{tjk} + (\delta\tau\beta\gamma)_{tijk} \\
&+ (\delta\phi)_{tl(ijk)} + (\theta\delta)_{mt} + (\theta\,\delta\tau)_{mti} + (\theta\delta\beta)_{mtj} + (\theta\delta\gamma)_{mtk} + (\theta\delta\tau\beta)_{mtij} + (\theta\delta\tau\gamma)_{mtik} \\
&+ (\theta\delta\beta\gamma)_{mtjk} + (\theta\delta\tau\beta\gamma)_{mtijk} + (\theta\delta\phi)_{tl(ijk)} + \varepsilon_{(ijklmt)n}
\end{aligned}
$$

$$
\begin{cases}
i = 1,2,3 & Pave\ (pavment\ \ type) \\
j = 1,2 & Drain\ (drainage\ \ condition\ ) \\
k = 1,2 & Class\ (road\ classifica\ tion) \\
l = 1,2 & \mathrm{Pr}\, oj\ (project\ , \mathrm{site\,A\ and\ B}) \\
m = 1,2 & Treat\ (sealed\ \ or\ unsealed\ ) \\
t = 1,2 & Time\,(year\ \ 2000\ \ and\ \ 2001) \\
n = 1,2,\cdots,12 & \mathrm{Re}\ petition
\end{cases}
\qquad (7\text{-}2)
$$

where,

μ is the mean value;

τ_i is the effect of the ith Pave;

β_j is the effect of jth Drain;

γ_k is the effect of kth Class;

$(\tau\beta)_{ij}$ is the Pave × Drain interaction;

$(\tau\gamma)_{ik}$ is the Pave × Class interaction;

$(\beta\gamma)_{jk}$ is the Class × Drain interaction;

$(\tau\beta\gamma)_{ijk}$ is the Pave × Drain× Class interaction;

$\phi_{l(ijk)}$ is the effect of lth Proj nested in the ith Pave × jth Drain × kth Class interaction;

θ_m is the effect of the mth Treat;

$(\theta\tau)_{mi}$ is the Treat × Pave interaction;

$(\theta\beta)_{mj}$ is the Treat × Drain interaction;

$(\theta\gamma)_{mk}$ is the Treat × Class interaction;

$(\theta\tau\beta)_{mij}$ is the Treat × Pave × Drain interaction;

$(\theta\tau\gamma)_{mik}$ is the Treat × Pave × Class interaction;

$(\theta\beta\gamma)_{mjk}$ is the Treat × Class × Drain interaction;

$(\theta\tau\beta\gamma)_{mijk}$ is the Treat × Pave × Drain× Class interaction;

$(\theta\phi)_{ml(ijk)}$ is the Treat × Proj interaction, and Proj nested in Pave × Drain× Class;

δ_t is the effect of the tth Time;

$(\delta\tau)_{ti}$ is the Time × Pave interaction;

$(\delta\beta)_{tj}$ is the Time × Drain interaction;

$(\delta\gamma)_{tk}$ is the Time × Class interaction;

$(\delta\tau\beta)_{tij}$ is the Time × Pave × Drain interaction;

$(\delta\tau\gamma)_{tik}$ is the Time × Pave × Class interaction;

$(\delta\beta\gamma)_{tjk}$ is the Time × Class × Drain interaction;

$(\delta\tau\beta\gamma)_{tijk}$ is the Time × Pave × Drain× Class interaction;

$(\delta\phi)_{tl(ijk)}$ is the Time × Proj interaction, and Proj nested in Pave × Drain× Class;

$\varepsilon_{(ijklmt)n}$ is the error term.

This model was applied to the data from all nineteen test sites including all three pavement types. For one particular pavement type (PCC, HMA, or composite), the model, as shown in Equation 7-3, was derived from Equation 7-2 by removing the factor Pave and all its interactions.

$$
\begin{aligned}
y_{ijklmn} &= \tau_i + \beta_j + \gamma_k + (\tau\beta)_{ij} + (\tau\gamma)_{ik} + (\beta\gamma)_{jk} + (\tau\beta\gamma)_{ijk} + \phi_{l(ijk)} \\
&\quad + \theta_m + \theta\tau_{mi} + (\theta\beta)_{mj} + (\theta\gamma)_{mk} + (\theta\tau\beta)_{mij} + (\theta\tau\gamma)_{mik} + (\theta\beta\gamma)_{mjk} + (\theta\tau\beta\gamma)_{mijk} \\
&\quad + (\theta\phi)_{ml(ijk)} + \varepsilon_{(ijklm)n}
\end{aligned}
$$

$$
\begin{cases}
i = 1,2,3 & \textit{pave} \\
j = 1,2 & \textit{drain} \\
k = 1,2 & \textit{class} \\
l = 1,2 & \textit{project} \\
m = 1,2 & \textit{treat} \\
n = 1,2,3,4,5,6,7,8,9,10,11,12 & \textit{repetition}
\end{cases}
\qquad 7\text{-}3
$$

where, all factors are the same as in Equation 7-2.

7.4 FWD Deflection Comparison

With the linear statistical models shown in Equations 7-2 and 7-3, the statistical analyses for the FWD deflections at the crack or joint and at mid-slab, and the load transfer at each joint or crack were conducted with the two years of data. All analyses were conducted at a 95 percent confidence level ($\alpha = 0.05$). Tables 7-2 and 7-3 show the results of FWD deflection and load transfer for all pavements, respectively. Tables 7-4 and 7-5 show the results for PCC pavements, while Tables 7-6 and 7-7 show the results for HMA pavements. Tables 7-8 and 7-9 contain the results for composite pavements.

The data in Table 7-2 indicates that when all pavement types are listed together, pavement type appears to be the only significant factor. Figure 7-1 shows the FWD deflection for each pavement type. The interaction of treatment and pavement type also appears to be significant. Figure 7-2 and 7-3 shows the FWD deflection for each pavement type at two different treatments respectively. For load transfer measurements on all pavement types, Table 7-3 shows that there are no significant factors. However, two interactions, Treat × Drain and Treat × Class, are significant.

For deflection and load transfer measurements on PCC pavements, Table 7-4 and Table 7-5 show that none of the main factors or interactions is significant. The effect of pavement type on load transfer is shown in Figure 7-4. Note that since the 3A and 3B cells were not filled, the interaction of Treat × Class cannot be tested.

Table 7-6 and Table 7-7 again show that for deflections and load transfer measurements on HMA pavements, none of the main factors or interactions are significant. For this group of pavements, since cells 5A, 6A and 6B were not filled the main factor Drain and all its interactions could not be tested.

The deflections and load transfer measurements on composite pavements also show no main factors or interactions to be significant (Table 7-8 and Table 7-9).

Table 7-2 Analysis of Mid-Joint/Crack Deflections for All Pavement Types

Source	DF	SS (III)	Mean Square	F	Pr > F	Error Term
Pave	2	62284.8	31142.4	12.36	0.0026	Proj( )
Drain	1	6673.1	6673.1	2.65	0.1380	
Class	1	7258.8	7258.8	2.88	0.1238	
Pave*Drain	2	2755.5	1377.8	0.55	0.5968	
Pave*Class	2	7418.1	3709.0	1.47	0.2797	
Drain*Class	1	8.4	8.4	0.00	0.9552	
Pave*Drain*Class	0	0.0	.	.	.	
Proj(Pav*Dra*Cla)	9	22670.6	2518.9			
Trt	1	150.8	150.8	0.42	0.5349	Trt*Proj( )
Trt*Pave	2	3641.9	1820.9	5.03	0.0342	
Trt*Drain	1	38.9	38.9	0.11	0.7503	
Trt*Class	1	217.6	217.6	0.60	0.4582	
Trt*Pave*Drain	2	389.8	194.9	0.54	0.6015	
Trt*Pave*Class	2	752.9	376.5	1.04	0.3926	
Trt*Drain*Class	1	930.3	930.3	2.57	0.1435	
Trt*Pave*Drain*Class	0	0.0	.		.	
Trt*Proj(Pav*Dra*Cla)	9	3260.3	362.3			
Time	1	3147.3	3147.3	3.32	0.1018	Trt*Proj( )
Time*Pave	2	4757.1	2378.5	2.51	0.1362	
Time*Drain	1	0.6	0.6	0.00	0.9806	
Time*Class	1	763.1	763.1	0.80	0.3931	
Time*Pave*Drain	2	37.7	18.8	0.02	0.9804	
Time*Pave*Class	2	2702.1	1351.1	1.42	0.2901	
Time*Drain*Class	1	2.1	2.1	0.00	0.9634	
Time*Pave*Drain*Class	0	0.0	.		.	
Time*Proj(Pav*Dra*Cla)	9	8535.9	948.4			
Trt*Time	1	228.0	228.0	1.94	0.1972	Trt*Proj( )
Trt*Time*Pave	2	506.8	253.4	2.16	0.1719	
Trt*Time*Drain	1	1.6	1.6	0.01	0.9075	
Trt*Time*Class	1	204.4	204.4	1.74	0.2199	
Trt*Time*Pave*Drain	2	80.3	40.2	0.34	0.7194	
Trt*Time*Pave*Class	2	190.8	95.4	0.81	0.4743	
Trt*Time*Drain*Class	1	255.1	255.1	2.17	0.1748	
Trt*Time*Pave*Drain*Class	0	0.0	.		.	
Trt*Time*Proj(Pav*Dra*Cla)	9	1058.2	117.6			
Error	836	530959.	635.1			
Total	911	8869475				

Table 7-3 Analysis of Joint/Crack Load Transfer for All Pavement Types

Source	DF	SS (III)	Mean Square	F	Pr > F	Error Term
Pave	2	0.0798	0.0399	4.01	0.0569	Proj( )
Drain	1	0.0002	0.0002	0.03	0.8680	
Class	1	0.0079	0.0079	0.69	0.4278	
Pave*Drain	2	0.0084	0.0042	0.42	0.6673	
Pave*Class	2	0.0194	0.0097	0.97	0.4144	
Drain*Class	1	0.0159	0.0159	1.60	0.2381	
Pave*Drain*Class	0	0.0	.	.	.	
Proj(Pav*Dra*Cla)	9	0.0896	0.0099			
Trt	1	0.0000	0.0000	0.04	0.8546	Trt*Proj( )
Trt*Pave	2	0.0004	0.0002	0.30	0.7484	
Trt*Drain	1	0.0038	0.0038	5.89	0.0381	
Trt*Class	1	0.0051	0.0051	7.84	0.0207	
Trt*Pave*Drain	2	0.0005	0.0002	0.37	0.6981	
Trt*Pave*Class	2	0.0036	0.0018	2.79	0.1142	
Trt*Drain*Class	1	0.0003	0.0003	0.53	0.4871	
Trt*Pave*Drain*Class	0	0.0	.	.	.	
Trt*Proj(Pav*Dra*Cla)	9	0.0059	0.0007			
Time	1	0.0006	0.0006	0.10	0.7537	Trt*Proj( )
Time*Pave	2	0.0002	0.0001	0.02	0.9803	
Time*Drain	1	0.0007	0.0007	0.11	0.7487	
Time*Class	1	0.0033	0.0033	0.54	0.4792	
Time*Pave*Drain	2	0.0027	0.0013	0.22	0.8072	
Time*Pave*Class	2	0.0022	0.0011	0.18	0.8359	
Time*Drain*Class	1	0.0137	0.0137	2.23	0.1698	
Time*Pave*Drain*Class	0	0.0	.		.	
Time*Proj(Pav*Dra*Cla)	9	0.0553	0.0061			
Trt*Time	1	0.0002	0.0002	0.13	0.7264	Trt*Proj( )
Trt*Time*Pave	2	0.0011	0.0006	0.35	0.7123	
Trt*Time*Drain	1	0.0028	0.0028	1.78	0.2151	
Trt*Time*Class	1	0.0042	0.0042	2.62	0.1397	
Trt*Time*Pave*Drain	2	0.0004	0.0002	0.13	0.8782	
Trt*Time*Pave*Class	2	0.0008	0.0004	0.25	0.7856	
Trt*Time*Drain*Class	1	0.0001	0.0001	0.06	0.8080	
Trt*Time*Pave*Drain*Class	0	0.0	.		.	
Trt*Time*Proj(Pav*Dra*Cla)	9	0.01444	0.0016			
Error	836	4.04189	0.0048			
Total	911	20.4208				

Table 7-4 Analysis of Mid-Joint/Crack Deflections for PCC Pavements

Source	DF	SS	Mean Square	F value	Pr > F	Error Term
Drain	1	53.1	53.1	1.69	0.2840	Proj(Dra*Cla)
Class	1	0.1	0.1	0.00	0.9461	
Drain*Class	0	0.0	.	.	.	
Proj(Dra*Cla)	3	93.9	31.3			
Trt	1	55.0	55.0	1.07	0.3779	Trt*Proj(Dra*Cla)
Trt*Drain	1	53.7	53.7	1.04	0.3826	
Trt*Class	1	35.4	35.4	0.69	0.4686	
Trt*Drain*Class	0	0	.	.	.	
Trt*Proj(Dra*Cla)	3	154.8	51.6			
Time	1	85.6	85.6	1.10	0.3717	Trt*Proj(Dra*Cla)
Time*Drain	1	138.7	138.7	1.78	0.2745	
Time*Class	1	0.24	0.24	0	0.9592	
Time*Drain*Class	0	0.00	.	.	.	
Time*Proj(Dra*Cla)	3	233.9	77.9			
Trt*Time	1	136.5	136.5	0.99	0.3926	Trt*Proj(Dra*Cla)
Trt**Time*Drain	1	0.7	0.7	0.01	0.9457	
Trt*Time*Class	1	127.6	127.6	1.13	0.3994	
Trt*Time*Drain*Class	0	0.00	.	.	.	
Trt*Time*Proj(Dra*Cla)	3	412.7	137.6			
Error	264	16641.42	63.04			
Total	287	112277.7				

Table 7-5 Analysis of Joint/Crack Load Transfer for PCC Pavements

Source	DF	SS	Mean Square	F value	Pr > F	Error Term
Drain	1	0.0405	0.0405	2.14	0.2395	Proj(Dra*Cla)
Class	1	0.0044	0.0044	0.23	0.6615	
Drain*Class	0	0.0000	.	.	.	
Proj(Dra*Cla)	3	0.0566	0.0189			
Trt	1	0.0002	0.0002	0.31	0.6142	Trt*Proj(Dra*Cla)
Trt*Drain	1	0.0000	0.0000	0.00	0.9782	
Trt*Class	1	0.0003	0.0003	0.54	0.5163	
Trt*Drain*Class	0	0.0000	.	.	.	
Trt*Proj(Dra*Cla)	3	0.0015	0.0005			
Time	1	0.0007	0.0007	0.58	0.5028	Trt*Proj(Dra*Cla)
Time*Drain	1	0.0003	0.0003	0.29	0.6262	
Time*Class	1	0.0004	0.0004	0.38	0.5819	
Time*Drain*Class	0	0.00	.	.	.	
Time*Proj(Dra*Cla)	3	0.0034	0.0011			
Trt*Time	1	0.0000	0.0000	0.00	0.9907	Trt*Proj(Dra*Cla)
Trt**Time*Drain	1	0.0004	0.0004	0.79	0.4394	
Trt*Time*Class	1	0.0014	0.0014	2.43	0.2172	
Trt*Time*Drain*Class	0	0.00	.	.	.	
Trt*Time*Proj(Dra*Cla)	3	0.0017	0.0006			
Error	264	1.4088	0.0053			
Total	287	7.2638				

Table 7-6 Analysis of Mid-Joint/Crack Deflections for HMA Pavements

Source	DF	SS	Mean Square	F value	Pr > F	Error Term
Drain	0	.	.	.	.	Proj(Dra*Cla)
Class	1	8095.1	8095.1	0.88	0.4466	
Drain*Class	0	0.0	.	.	.	
Proj(Dra*Cla)	2	18334.7	9167.4			
Trt	1	71.1	71.1	0.08	0.8073	Trt*Proj(Dra*Cla)
Trt*Drain	1	0.0	.	.	.	
Trt*Class	1	1600.8	1600.8	1.74	0.3181	
Trt*Drain*Class	0	0.0	.	.	.	
Trt*Proj(Dra*Cla)	2	1841.8	920.8			
Time	1	8023.8	8023.8	3.78	0.1912	Trt*Proj(Dra*Cla)
Time*Drain	1	0.0	.	.	.	
Time*Class	1	1.5	1.5	0.00	0.9809	
Time*Drain*Class	0	0.0	.	.	.	
Time*Proj(Dra*Cla)	2	4241.3	2120.6			
Trt*Time	1	33.6	33.6	1.67	0.3257	Trt*Proj(Dra*Cla)
Trt**Time*Drain	1	0.0	.	.	.	
Trt*Time*Class	1	0.1	0.1	0.00	0.9641	
Trt*Time*Drain*Class	0	0.0	.	.	.	
Trt*Time*Proj(Dra*Cla)	2	40.3	20.2			
Error	220	347669.7	1580.3			
Total	239	3151983.6				

Table 7-7 Analysis of Joint/Crack Load Transfer for HMA Pavements

Source	DF	SS	Mean Square	F value	Pr > F	Error Term
Drain	0	.	.	.	.	Proj(Dra*Cla)
Class	1	0.0027	0.0027	0.34	0.6190	
Drain*Class	0	0.0000	.	.	.	
Proj(Dra*Cla)	2	0.0159	0.0008			
Trt	1	0.0000	0.0000	0.08	0.8033	Trt*Proj(Dra*Cla)
Trt*Drain	1	0.0000	.	.	.	
Trt*Class	1	0.0000	0.0000	0.09	0.7902	
Trt*Drain*Class	0	0.0000	.	.	.	
Trt*Proj(Dra*Cla)	2	0.0001	0.0000			
Time	1	0.0015	0.0015	1.21	0.3859	Trt*Proj(Dra*Cla)
Time*Drain	1	0.0000	.	.	.	
Time*Class	1	0.0000	0.0000	0.00	0.9893	
Time*Drain*Class	0	0.0000	.	.	.	
Time*Proj(Dra*Cla)	2	0.0025	0.0012			
Trt*Time	1	0.0000	0.0000	0.06	0.8306	Trt*Proj(Dra*Cla)
Trt**Time*Drain	1	0.0000	.	.	.	
Trt*Time*Class	1	0.0006	0.0006	4.30	0.1739	
Trt*Time*Drain*Class	0	0.0	.	.	.	
Trt*Time*Proj(Dra*Cla)	2	0.0003	0.0001			
Error	220	0.2763	0.0012			
Total	239	1.1012				

Table 7-8 Analysis of Mid-Joint/Crack Deflections for Composite Pavements

Source	DF	SS	Mean Square	F value	Pr > F	Error Term
Drain	1	2202.2	2202.2	2.08	0.2230	Proj(Dra*Cla)
Class	1	164.3	164.3	0.15	0.7139	
Drain*Class	1	3.2	3.2	0.00	0.9587	
Proj(Dra*Cla)	4	4241.9	1060.5			
Trt	1	1676.7	1676.7	5.31	0.0826	Trt*Proj(Dra*Cla)
Trt*Drain	1	500.5	500.5	1.58	0.2766	
Trt*Class	1	486.0	486.0	1.54	0.2827	
Trt*Drain*Class	1	333.1	333.1	1.05	0.3626	
Trt*Proj(Dra*Cla)	4	1263.9	315.9			
Time	1	2217.6	2217.6	2.18	0.2135	Trt*Proj(Dra*Cla)
Time*Drain	1	1102.9	1102.9	1.09	0.3561	
Time*Class	1	419.2	419.2	0.41	0.5555	
Time*Drain*Class	1	139.7	139.7	0.14	0.7295	
Time*Proj(Dra*Cla)	4	4060.7	1015.2			
Trt*Time	1	893.2	893.2	5.90	0.0720	Trt*Proj(Dra*Cla)
Trt**Time*Drain	1	31.5	31.5	0.21	0.6717	
Trt*Time*Class	1	133.5	133.5	0.88	0.4007	
Trt*Time*Drain*Class	1	80.5	80.5	0.53	0.5061	
Trt*Time*Proj(Dra*Cla)	4	605.2	605.2			
Error	352	166648.	473.4			
Total	383	830135.				

Table 7-9 Analysis of Joint/Crack Load Transfer for Composite Pavements

Source	DF	SS	Mean Square	F value	Pr > F	Error Term
Drain	1	0.0100	0.0010	2.35	0.1997	
Class	1	0.0040	0.0040	0.95	0.3860	Proj(Dra*Cla)
Drain*Class	1	0.0001	0.0001	0.03	0.8740	
Proj(Dra*Cla)	4	0.0171	0.0043			
Trt	1	0.0000	0.0000	0.02	0.8999	
Trt*Drain	1	0.0034	0.0034	3.19	0.1487	
Trt*Class	1	0.0002	0.0002	0.23	0.6569	Trt*Proj(Dra*Cla)
Trt*Drain*Class	1	0.0000	0.0000	0.02	0.8837	
Trt*Proj(Dra*Cla)	4	0.0042	0.0011			
Time	1	0.0112	0.0112	0.91	0.3944	
Time*Drain	1	0.0292	0.0292	2.36	0.1990	
Time*Class	1	0.0010	0.0010	0.09	0.7849	Trt*Proj(Dra*Cla)
Time*Drain*Class	1	0.0000	0.0000	0.00	0.8919	
Time*Proj(Dra*Cla)	4	0.0494	0.0124			
Trt*Time	1	0.0002	0.0001	0.05	0.8342	
Trt**Time*Drain	1	0.0119	0.0119	3.84	0.1217	
Trt*Time*Class	1	0.0020	0.0020	0.63	0.4707	
Trt*Time*Drain*Class	1	0.0012	0.0012	0.40	0.5626	Trt*Proj(Dra*Cla)
Trt*Time*Proj(Dra*Cla)	4	0.0124	0.0003			
Error	352	2.3568	0.0067			
Total	383	4.0539				

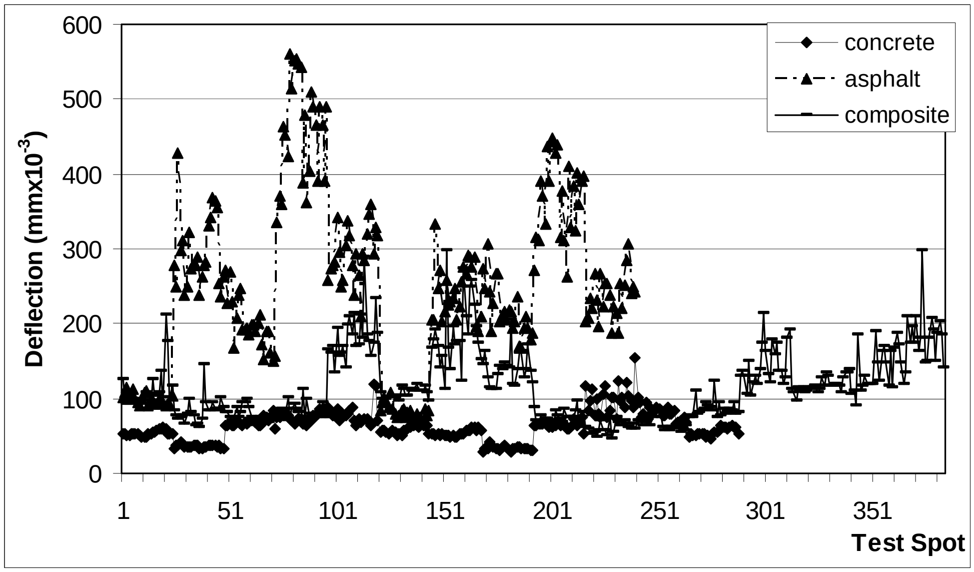

Figure 7-1 Pavement Type with FWD Deflection

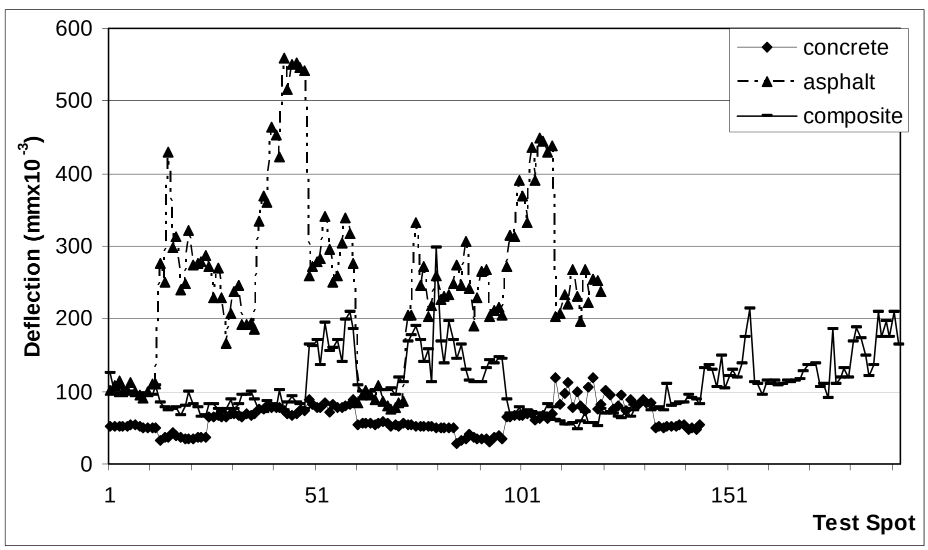

Figure 7-2 Pavement Type with FWD Deflection at Sealed Section

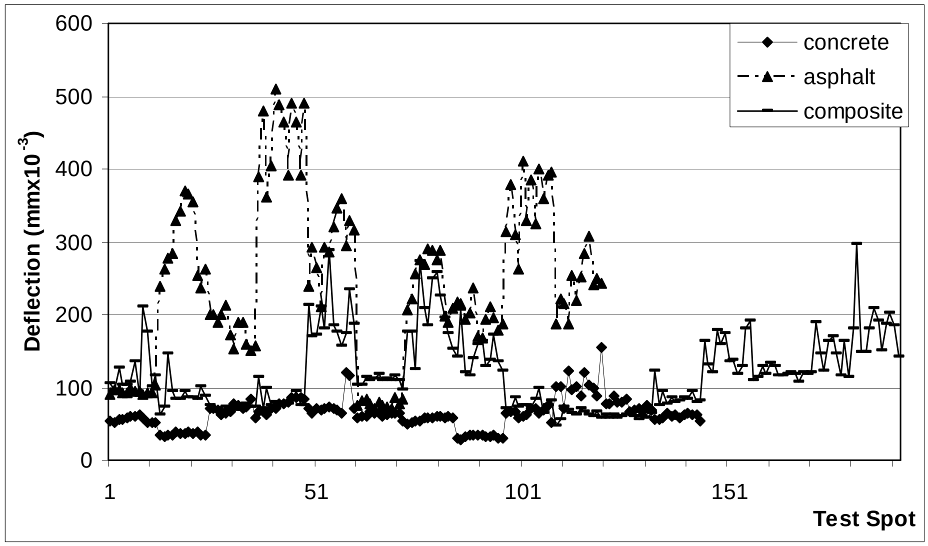

Figure 7-3 Pavement Type with FWD Deflection at Unsealed Section

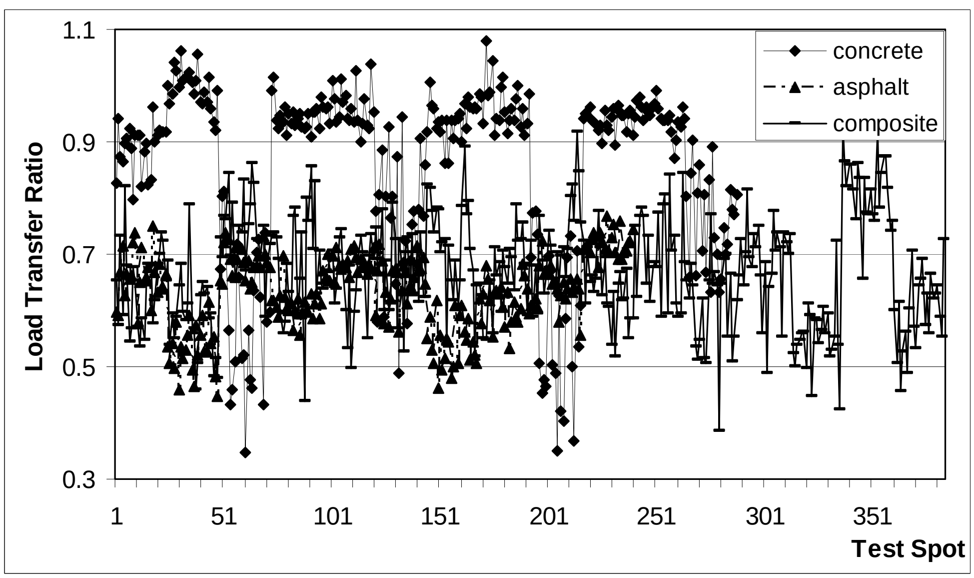

Figure 7-4 Pavement Type with Load Transfer

One section of I94, in the east bound lane from mile marker (MM) 35.0 to 39.6, was left unsealed when the pavement was constructed in 1997. During the field observation conducted by engineers, no obvious pavement performance differences were found between this section and the nearby sealed section. FWD data were collected from the two sections in September 2002, and a simple statistical t-test was done to compare the sealed and unsealed sections. The results are shown in Table 7-10. Note that the t-test results indicate that there is significant difference in load transfer, but no significant difference for FWD deflection.

Table 7-10 T-Test for Sealed and Unsealed on I94

Source		Mean	No. of Observations	variance	T-stat	Pr > F
Load Transfer	Sealed	0.909	12	0.0008	6.0193	<0.001
	Unsealed	0.843	12	0.0006		
FWD Deflection	Sealed	36.25	12	3.2954	-0.8845	0.3874
	Unsealed	37.08	12	7.3561		

Given the statistical analysis presented in this section, the following conclusions appear to be warranted:

1. There are no significant differences in mid-slab and crack or joint deflections and joint or crack load transfer between the sealed and unsealed sections during the two years of data collection. The results also illustrate that the data should be grouped by pavement type to conduct the statistical analyses, due to the significance of pavement type. Although the pavement type is not a significant factor for the load transfer (the p-value is relatively small, 0.0569), the load transfer results are grouped by pavement type for future statistical data analysis;

2. Since for each pavement type, time and sealed or unsealed condition are insignificant factors for both mid-slab and joint or crack deflections and joint and crack load transfer, there seems to be no significant differences between the sealed and unsealed sections during the two year period;
3. Drainage and road classification do not appear to affect the effectiveness of joint and crack sealing for PCC and composite pavements; and
4. No conclusion can be made about the drainage factor for HMA pavements since these test sites are missing from the experiment. However, road classification appears to be as an insignificant factor for HMA pavements.
5. Since the first two years of data shown no significant differences between sealed and unsealed sections, the data from the experimental sites can serve as a baseline for future data comparisons.

7.5 Roughness (IRI)

The IRI testing was conducted in the autumn of 2001. The statistical analysis model developed for the FWD deflection analysis is also suitable for the IRI data.

7.5.1 Test Repetition

At the beginning of IRI data collection, three repetitions were conducted on test sites 1A1, 1A2, and 7A. These data were used to determine whether the repetition was necessary for the remainder of the project sites. Table7-11 shows the original three repetitions of collected IRI data.

Table 7-11 Three Repetitons IRI Data

Project	Repeat	IRI(in/mile)	
		Sealed Section	Unsealed Section
1A1	1	79.57	83.07
	2	79.37	76.52
	3	84.68	82.55
1A2	1	82.41	71.72
	2	83.95	76.69
	3	78.83	74.77
7A	1	99.87	83.39
	2	103.69	82.40
	3	100.40	80.90

The IRI data can be described by the linear statistical model, shown in Equation 7-3. Table 7-12 shows the statistical analysis results for the data.

$$y_{ijk} = \mu + \tau_i + \beta_j + \gamma_k + (\tau\beta)_{ij} + (\tau\gamma)_{ik} + (\beta\gamma)_{jk} + \varepsilon_{ijk}$$

$$\begin{cases} i = 1,2 & Treat(Sealed\ and\ unsealed) \\ j = 1,2,3 & Pave(Pavement\ Type) \\ k = 1,2,3 & \mathrm{Re}\,petiton \end{cases}$$

7-3

where,

y_{ijk} is the IRI observation value;

μ is the mean value;

τ_i is the effect of the ith Treat;

β_j is the effect of jth Pave;

γ_k is the effect of kth Repetition;

$(\tau\beta)_{ij}$ is the Treat × Pave interaction;

$(\tau\gamma)_{ik}$ is the Treat × Repetition interaction;

$(\beta\gamma)_{jk}$ is the Pave × Repetition interaction; and

ε_{ijk} is the error term.

Table 7-12 Statistical Analysis of IRI Repetition Data Collection

Source	DF	Type I SS	Mean Square	F value	Pr > F
Treat	1	362.34	362.34	39.38	0.0008
Road	2	626.84	313.42	34.06	0.0005
Treat*road	2	265.40	132.70	14.42	0.0051
Repeat	1	0.36	0.36	0.04	0.8482
Treat*Repeat	1	0.34	0.34	0.04	0.8359
Repeat*road	2	5.93	2.96	0.32	0.7363
Treat*repeat*road	2	20.85	10.4	1.13	0.3824
Error	6	55.21	9.20		
Total	17	1337.3			

As shown in Table 7-12, with a confidence interval of 95 percent, the p-value for repetition and all of its interactions range from 0.38 to 0.84. This indicates that Repeat is not a significant factor for IRI data collection. After presenting this statistical analysis to the SAC members, it was decided that only one repetition would be conducted for IRI data on the remaining sixteen test sites. However, during further data collection, it was noticed that each IRI data collection only took 10 to 20 minutes once the test vehicle and traffic control were in place. Therefore three repetition tests for IRI data collection were conducted anyway. The IRI data is shown in Table 7-13. Note that no data were collected for test sites 7B, 10B, and 12B due to scheduling conflicts.

7.5.2 Baseline Check

The first year of IRI data can be used to check whether both sealed and unsealed sections are at the same baseline when the treatment was initially applied. The linear statistical model designed for FWD data analysis can be applied to the IRI data. Table 7-14 shows the statistical analysis of IRI for all pavement types. Tables 7-15, 7-16, and 7-17 show the statistical analyses of the IRI data for PCC, HMA, and composite pavements respectively. Again, a confidence level of 95 percent ($\alpha = 0.05$) was chosen for the analyses.

As can be seen from the tables, some main factors and interaction terms could not be tested due to zero degrees of freedom. This is because data for several tests were not available during the data analysis.

Table 7-13 IRI Data

Project No.	Sealed Section				Unsealed section IRI(in/mi)				Length(ft)		Average IRI (in/mi)	
	Position (ft)		IRI(in/mi)		Position (ft)		IRI(in/mi)					
	start	End	Left	Right	Start	End	left	right	unsealed	sealed	unsealed	sealed
1A1	0	144	79.57	101.5	247	411.76	83.07	89.04	144.0	164.8	90.54	86.06
1A2	0	160	82.41	73.07	444	610.57	71.72	58.51	160.0	166.6	77.74	65.12
1b	0	165	65.48	74.3	568	724.37	54.42	62.25	165.0	156.4	69.89	58.34
2a	0	220	129.77	128.03	340	554.33	139.63	160.34	220.0	214.3	128.90	149.99
2b	0	157	74.71	83.19	249	414.37	75.31	82.74	157.0	165.4	78.95	79.03
4a	0	260	112.45	137.33	309	461.46	147.32	129.56	260.0	152.5	124.89	138.44
4b	0	199	113.97	139.36	399	610.57	90.71	116	199.0	211.6	126.67	103.36
5b	0	402	44.5	49.59	1612	1645.2	63.76	83.27	402.0	33.2	47.05	73.52
7a	0	337	99.87	90.09	481	812.01	83.39	101.41	337.0	331.0	94.98	92.40
7b	0											
8a	0	485	74.77	132.36	577	916.65	101.37	159.6	485.0	339.7	103.57	130.49
8b	0	434	53.54	77.38	678	1356	64.33	72.25	434.0	678.0	65.46	68.29
9a	0	590	51.19	76.16	824	1204.41	48.32	73.45	590.0	380.4	63.68	60.89
9b	0	961	30.83	35.41	1218	2174.94	34.86	46.79	961.0	956.9	33.12	40.83
10a	0	283	68.36	73.17	442	694.29	46.96	58.28	283.0	252.3	70.77	52.62
10b	0											
11a	0	987	49.05	53.97	1287	2072.92	57.28	65.47	987.0	785.9	51.51	61.38
11b	0	518	78.72	73.17	687	1309.05	68.56	81.04	518.0	622.1	75.95	74.80
12a	0	530	53.56	65.36	1680	2436.54	74.58	72.73	530.0	756.5	59.46	73.66
12b	0											

Table 7-14 Statistical Variance Analysis for All Pavement IRI

Source	DF	SS (III)	Mean Square	F Value	Pr > F	Error Term
Pave	2	6.79	3.39	0.04	0.9647	proj(pav*dra*clas)
Drain	1	273.59	273.59	2.91	0.1389	proj(pav*dra*clas)
Class	1	26.91	26.91	0.29	0.61	proj(pav*dra*clas)
Pave*drain	0	.	.	.	.	proj(pav*dra*clas)
Pave*class	0	.	.	.	.	proj(pav*dra*clas)
Drain*class	0	.	.	.	.	proj(pav*dra*clas)
Pave*drain*class	0	.	.	.	.	proj(pav*dra*clas)
Proj(pav*dra*cla)	6	563.93	93.98			
Trt	1	381.33	381.33	3.49	0.1111	trt*proj(pav*drai*class)
Trt*pave	2	3.76	1.88	0.02	0.9830	trt*proj(pav*drai*class)
Trt*drain	1	45.14	45.14	0.41	0.5443	trt*proj(pav*drai*class)
Trt*class	1	141.91	141.91	1.30	0.2980	trt*proj(pav*drai*class)
Trt*pave*drain	0	.	.	.	.	trt*proj(pav*drai*class)
Trt*pave*class	0	.	.	.	.	trt*proj(pav*drai*class)
Trt*drain*class	0	.	.	.	.	trt*proj(pav*drai*class)
Trt*pave*drain*class	0	.	.	.	.	trt*proj(pav*drai*class)
Trt*Proj(pav*dra*clas)	6	656.02	109.33			
Error	2	301.16	150.58			
Total	33	28132.54				

Table 7-15 Statistical Variance Analysis for PCC Pavement IRI

Source	DF	SS	Mean Square	F value	Pr > F	Error Term
Drain	1	82.07	82.07	0.98	0.3941	Proj(drain*class)
Class	1	26.91	26.91	0.32	0.6091	Proj(drain*class)
Drain*Class	0	0	.	.	.	Proj(drain*class)
proj(drain*class)	3	249.97	83.32			
Trt	1	234.79	234.79	1.55	0.3009	Trt*Proj(drain*class)
Trt*Drain	1	3.46	3.46	0.02	0.8893	Trt*Proj(drain*class)
Trt*Class	1	141.91	141.91	0.94	0.4039	Trt*Proj(drain*class)
Trt*Drain*Class	0	0	.	.	.	Trt*Proj(drain*class)
Trt*Proj(drain*class)	2	3	453.11	151.0		
Error	2	301.1618	150.58			
Total	13	11640.54				

Table 7-16 Statistical Variance Analysis for HMA Pavement IRI

Source	DF	SS(III)	Mean Square	F value	Pr > F	Error Term
Drain	0	0.00	.	.	.	Proj(drain*class)
Class	1	218.15	218.15	0.45	0.5718	Proj(drain*class)
Drain*class	0	0.00	.	.	.	Proj(drain*class)
Proj(drain*class)	2	917.46	485.69			
Trt	1	105.06	105.06	1.43	0.3540	Trt*Proj(drain*class)
Trt*Drain	0	0.00	.	.	.	Trt*Proj(drain*class)
Trt*Class	1	157.69	157.69	2.15	0.2802	Trt*Proj(drain*class)
Trt*Drain*Class	0	0.00	.	.	.	Trt*Proj(drain*class)
Trt*Proj(drain*class)	2	146.68	73.34			
Error	0	0	.			
Total	9	16214.93				

Table 7-17 Statistical Variance Analysis for Composite Pavement IRI

Source	DF	SS(III)	Mean Square	F value	Pr > F	Error Term
Drain	1	355.91	355.91	1.10	0.3527	Proj(drain*class)
Class	1	239.15	239.15	0.74	0.4377	Proj(drain*class)
Drain*Class	1	1.96	1.96	0.01	0.9415	Proj(drain*class)
Proj(Drain*Class)	4	1289.87	322.47			
Trt	1	0.0026	0.0026	0.00	0.9944	Trt*Proj(drain*class)
Trt*Drain	1	244.35	244.35	5.13	0.0863	Trt*Proj(drain*class)
Trt*Class	1	6.00	6.00	0.13	0.7406	Trt*Proj(drain*class)
Trt*Drain*Class	1	23.28	23.28	0.49	0.5231	Trt*Proj(drain*class)
Trt*Proj(drain*class)	4	190.62	47.66			
Error	0	0	.			
Total	15	12768.5				

Although additional data needs to be gathered, some preliminary conclusions can be made. No significant factors can be found for the IRI data analysis for the first year of data. It can be seen that the IRI values for sealed and unsealed sections are at the same baseline for each individual pavement type. The effect of sealing on IRI value should be studied further as additional data become available in future years.

7.6 Condition Survey Data

The condition survey data is to be reduced to one index for use in the statistical analyses when additional data become available. Since the LTPP manual was designed to investigate long-term pavement performance, the survey result obtained for this study do not provide enough information to identify the differences in pavement performance between sealed and unsealed sections due to the short-term duration of the research. However, as additional data is collected, the survey data could be used in the future.

7.7 Pavement Core Sample

The physical properties of the cores extracted at or near the joints and cracks were evaluated by visual inspection and documented with photographs. These cores help provide insights into the condition of sealants, bonds, and depth of sealant penetration. However, no physical testing was performed on the cores.

7.8 Conclusions

A statistical model was developed to compare the pavement performance between sealed and unsealed sections for three pavement types, PCC, HMA, and composite. The analysis results for one year of IRI data indicate that the IRI values for sealed and unsealed sections are at the same baseline for each individual pavement type. The results from two years of FWD measurements indicate that there are no significant differences between the performance of sealed and unsealed sections regardless of pavement type, drainage condition and road classification. However, it should be noted that only 2 years of data has been collected to date. No cost-effectiveness analysis for joint and crack sealing can be conducted with the limited pavement performance data and statistical analysis results. It is strongly suggested that the data collection from the project sites be extended for a period of 10 to 15 years so that the long-term performance can be measured and additional conclusions drawn regarding the cost-effectiveness of joint and crack sealing.

CHAPTER 8 EFFECT OF FWD POSITION DURING TESTING

8.1 Introduction

Joints are placed in PCC pavements to control cracking and provide ample space for slab movement. In some PCC pavements, dowel bars are used to transfer the load across the joint. The load transfer efficiency of PCC pavements is usually estimated by the joint efficiency, which is defined as the ratio of the deflections of the unloaded to the loaded slab across a joint. One method of measuring joint efficiency is by using the FWD and placing a deflection sensor on either side of the joint. However, the different positions, in both the longitudinal and traverse directions, of the FWD load plate may cause significantly different deflection results for estimating load transfer.

In this study, a three-dimensional finite element method (FEM) model was developed to simulate the FWD test across joints. The appropriateness of the boundary conditions for the model was first evaluated. Field FWD tests were conducted to verify the simulation results.

8.2 Finite Element Model for Load Transfer at PCC Pavement Joint

The finite element method is a numerical technique of solving engineering problems for which exact analytical solutions are hard or impossible to obtain due to complicated geometries, loadings and material properties. The analytical solutions generally require setting up ordinary partial differential equations that are often unsolvable. Hence it is necessary to use the finite element method, a numerical method, to obtain acceptable solutions. The finite element problem formulation is a system of simultaneous algebraic equations. This method yields approximate values at discrete points in the continuum.

Finite element analysis has a number of advantages (Hua, 1998). These advantages include the abilities to model irregularly shaped or complex model geometry configurations, various types of loading, and materials, an unlimited number and kinds of boundary conditions and other special features, like multi-point constraints, individually analyze dynamic, thermal, acoustic and other special effects, or any of their combinations, and nonlinear behavior with large deformations and/or nonlinear material properties.

In this study, a general-purpose finite element program, ABAQUS, was used to perform numerical computations. ABAQUS for civil engineering applications has been successfully implemented in the past at Purdue University (Zaghloul, 1994; Hua, 1998; and Fang, 2001). The use of ABAQUS allows for the modeling of features such as structural discontinuities, transverse joints and cracks, surface contact such as aggregate interlock at the joint and/or crack as well as contact between dowel bar and concrete, and different boundary conditions.

8.2.1 2D vs. 3D Pavement Model

A two-dimensional analysis has been successfully used to model pavement rutting as a plane strain problem (Hua, 2000; Fang, 2001). Hua found no significant difference between 2D and 3D models for the rutting problem. Plane strain elements were selected by Fang to reduce computation time without a significant loss in accuracy. However, for this study, a 3D pavement model was selected. There are several reasons for its use. Firstly, the dowel bars at the transverse joint are at discrete positions and the plane strain elements cannot represent this three-dimensional structure. Secondly, since the PCC pavement joints are not axi-symmetric, but rather they cross the pavement in the transverse direction, it would not be appropriate to select axi-symmetric elements. Thirdly, in this study, different lateral boundary conditions were simulated to evaluate their impact on the FWD simulation results. It was necessary to use a 3D model to make the lateral boundary conditions possible.

8.2.2 Contact and Interaction Modeling in ABAQUS

Transverse joints, longitudinal joints, and dowel bar contact were modeled in this study. Each involves surface contact and interaction. There are two methods for surface contact and interaction modeling in ABAQUS. One uses surfaces while the second uses contact elements (ABAQUS 2001).

Surface-Based Contact Simulation

Most contact problems are modeled by using surface-based contact, such as contact between two deformable bodies, contact between a rigid and a deformable body, small-sliding or finite-sliding interaction between rigid surfaces, etc. ABAQUS specifies surfaces that interact and then defines mechanical surface interaction models that govern the behavior of the surfaces when they are in contact.

Several standard mechanical surface interaction models available in ABAQUS include friction, finite sliding, softened contact, and user-defined. The friction model is most commonly used when surfaces transmit shear as well as normal forces across their interface.

Contact Element Simulation

ABAQUS also offers a variety of contact elements that can be used when the contact and interaction between two bodies cannot be simulated with the surface-based contact approach. These elements include gap contact elements, tube-to-tube contact elements, slide line contact elements, and rigid surface elements.

GAP elements are commonly used in pavement FEM analysis (Uddin, 1995). GAP elements are defined by specifying the two nodes forming the gap and providing geometric data defining the initial state and, if necessary, the direction of the gap. One of the GAP elements, the element GUAUNI, models contact between two nodes when the contact direction is fixed in space and is commonly used in pavement crack simulation.

The GAP elements allow two continuous surfaces to be in contact, or not in contact, through contact pressure and friction between the contacting surfaces. The GAP element controls the interaction between the contact surfaces in such a way that these surfaces do not penetrate each other under any contact pressure.

8.2.3 Model Geometry

A typical pavement section of US231 in Indiana was used to configure the 3D FEM model for the study. The cross section and plan view of this highway are shown in Figures 8-1 and 8-2, respectively. The four layer pavement structure consists of 279 mm (11 in) of concrete, 254 mm (10 in) of subbase, 609 mm (24 in) of special subgrade treatment, and the natural subgrade. The PCC pavement slabs are 6.09m (20 ft) long and 3.66 m (12 ft) wide, PCC bicycle lane slabs are 6.09 m (20 ft) long and 1.83 m (6 ft) wide. The soil shoulders are 1.83 m (6 ft) wide. Dowel bars are used to transfer the loads across joints. Tie bars are used across the longitudinal joints to hold adjoining slabs together, thus maintaining aggregate interlock.

Figure 8-3 shows the cross section geometry of the 3D FEM model in which the depths of all layers are the same as the pavement structure, except that the subgrade depth is assumed as 2,540 mm (100 in) to the top of the bedrock. The transverse width of the model is 12.2 m (40 ft), consisting of one travel lane slab, one passing lane slab, the concrete bicycle lane and the shoulder. Figure 8-4 shows a 3D view of the meshed model in ABAQUS.

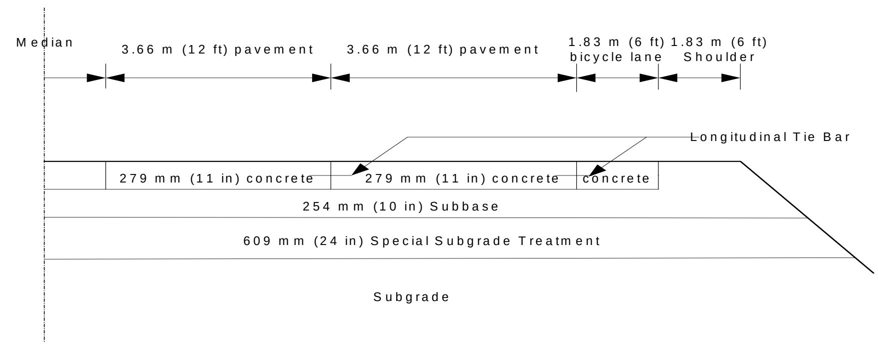

Figure 8-1 Typical Cross Section of US231

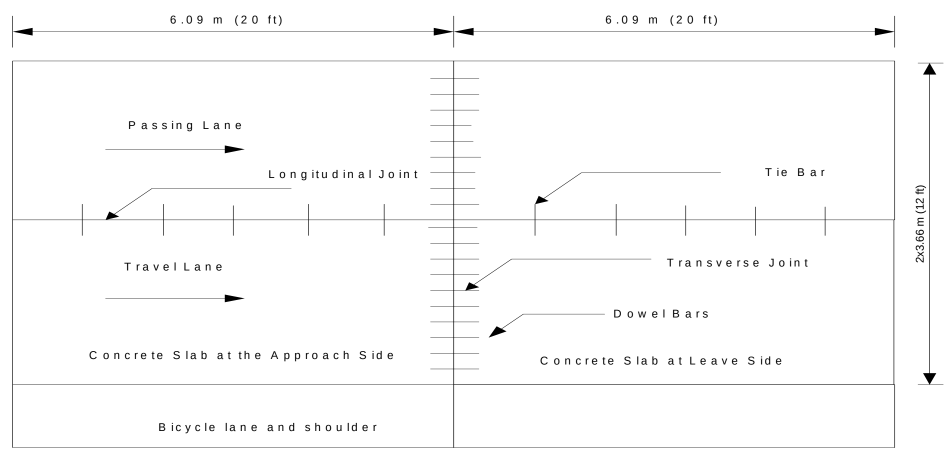

Figure 8-2 Typical Plan View of US231

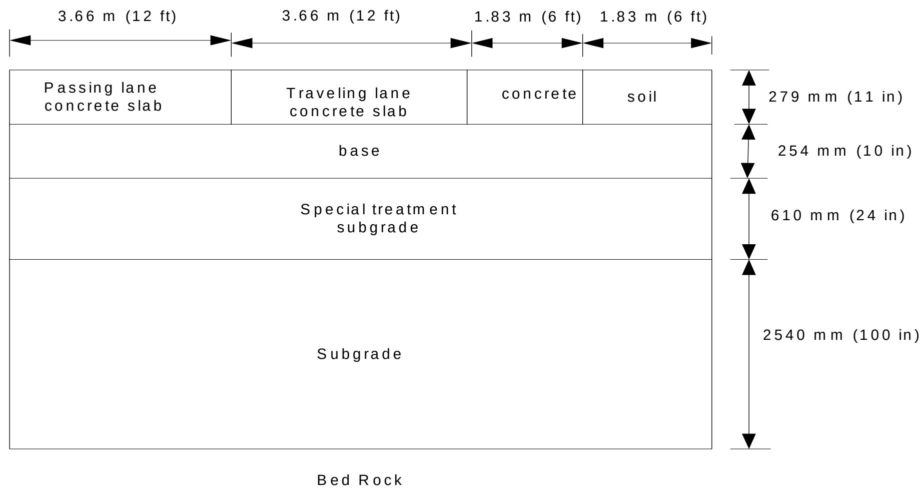

Figure 8-3 Cross Section of the Model

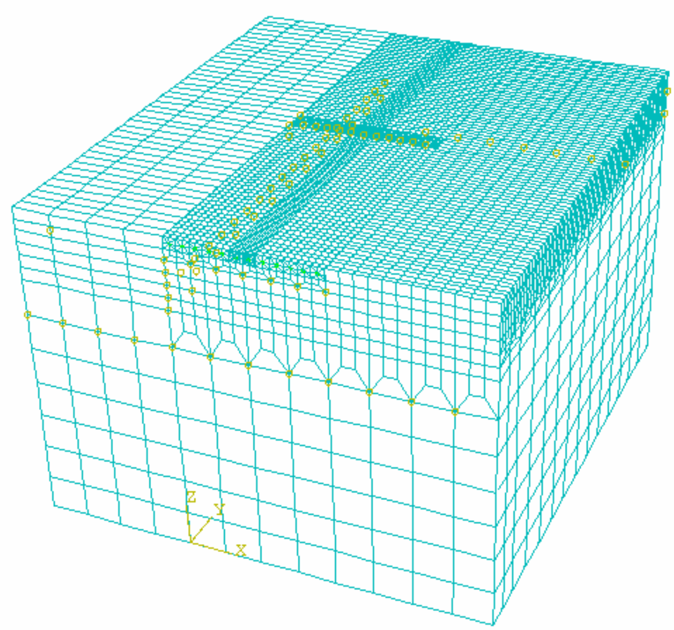

Figure 8-4 3D view of the Meshed Model

8.2.4 Joint Modeling

Both longitudinal and transverse joints are included in the model. Tie bars are typically used across longitudinal joints to hold adjoining slabs together. Since the simulated FWD test load locations are in the middle of the slabs, and are thus relatively far from the longitudinal joints, the effect of tie bars should be insignificant on the simulated FWD deflection. To simplify the model, the longitudinal joint was simulated using only contact interface elements. Dowel bars, as the load transfer devices, are placed across the transverse joints. The simulation of transverse joints was done by modeling both dowel bars and aggregate interlock.

Dowel Bar

As shown in Figure 8-5, dowel bars were placed across the joint as a positive load transfer device. They were located mid-height in the slab, and spaced 310 mm (12 in) apart, as shown in Figure 8-6. Each dowel bar is 457 mm (18 in) in length and 31.8 mm (1.25 in) in diameter. The fixed end of the dowel bar is grouted to the concrete while the free end is lubricated to permit sliding within the slab. Previous researchers (Hua 1998, Ruddin 1995) used beam elements to simulate the dowel bars, but it is difficult to model the mechanism of dowel contact at the free end. In this study, the dowel bar was modeled using a 3D mesh. The contact interface with a frictional interaction model was used to simulate the free end contact with a slab. The fixed end is tied into the slab in the simulation. A Young's modulus of 206,700 MPa (30,000 kpsi) and a Poisson's ratio of

0.3 are used to characterize the dowel bar. The mesh for slab and bars is shown in Figure 8-7, and the 3D mesh of one dowel bar is shown in Figure 8-8.

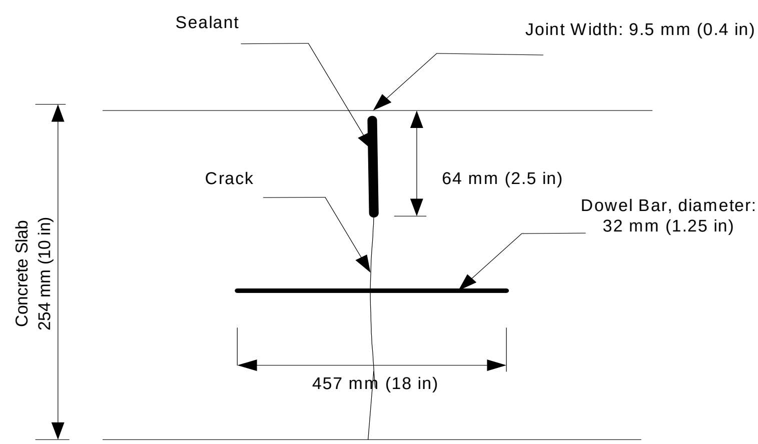

Figure 8-5 Dowel Bar Placement

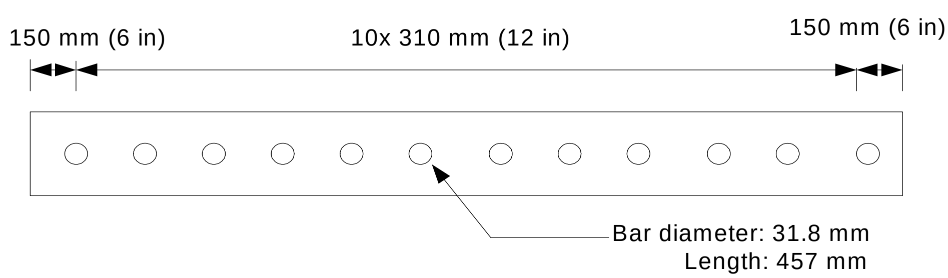

Figure 8-6 Dowel Bar Location

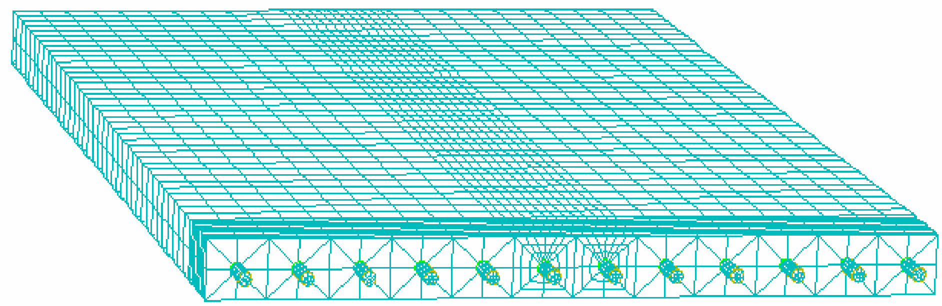

Figure 8-7 3D View of the Concrete Slab and Dowel Bar Mesh

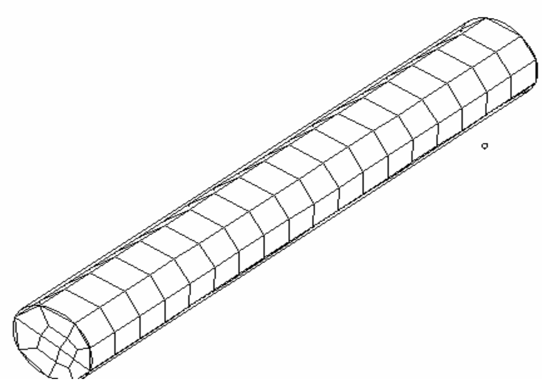

Figure 8-8 Dowel Bar Mesh

In order to test the accuracy of simulation results with the 3D bar models, the simulation results from three 3D bar models were compared with the analytical beam solution. These three models are the FEM beam element, the 3D bar mesh using 8-node elements and the 3D bar mesh using 20-node elements. These three models are configured in ABAQUS to simulate the beam shown in Figure 8-9. Note that the beam is fixed at one end and a point load is applied at the other. The beam length is 229 mm (9 in). The beam has a circular cross-section with a radius of 16 mm (0.625 in). The load, P, is 4.4 kN (1000 lb). According to the analytical solution, the displacement is

$$v(x) = \frac{Px^2}{6EI}(3l - x) \qquad 8\text{-}1$$

where,

P = load (N),

E = Young's modulus (Pa),

I = area moment of inertia (m^4),

l = total length (m), and

x = displacement location at the x-axis (m).

The simulation and analytical results are shown in Figure 8-10. The figure shows that the ABAQUS beam element provides the same result as does the analytical solution. The 3D bar mesh using 20-node elements provides an accurate simulation, while the 3D bar mesh using 8-node elements yields results having about 20 percent error. Since the dowel bars are the most important element for transferring loading, the 20-node elements were used to mesh the 3D bar.

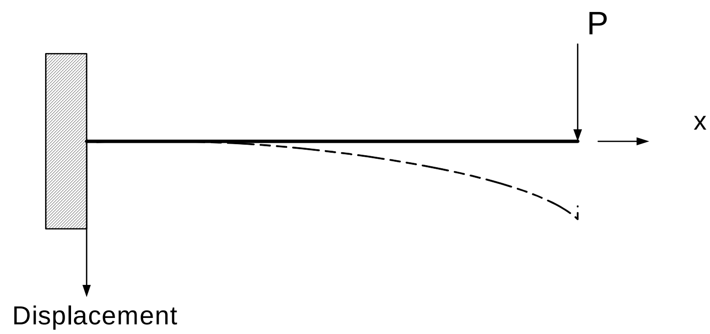

Figure 8-9 Beam Element Displacement

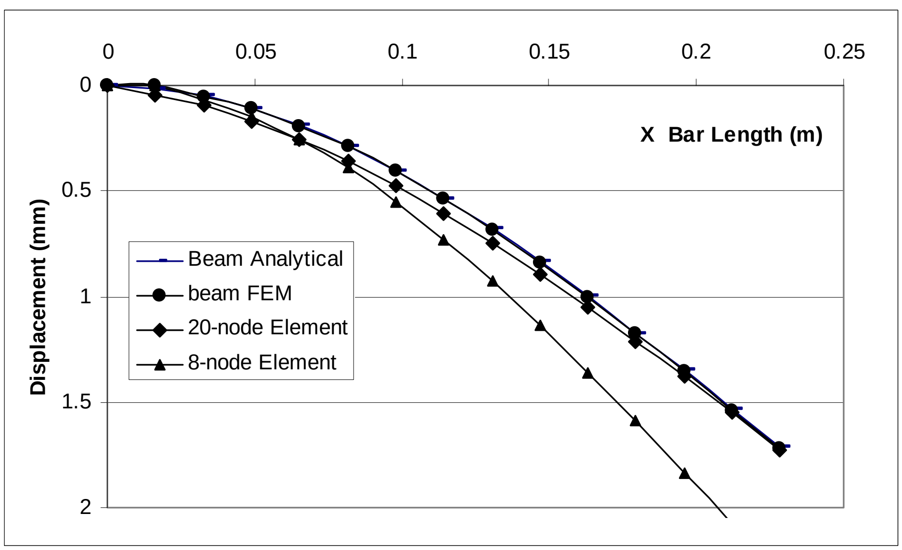

Figure 8-10 3D Bar Mesh Evaluation

Aggregate Interlock

In addition to the load transfer provided by dowel bars, the aggregate interlock between the two vertical surfaces of the adjoining slabs may also provide a measure of load transfer. This contact was modeled using the contact interface with friction interaction model in ABAQUS.

8.2.5 Element Type and Model Mesh

ABAQUS has an extensive element library to provide a powerful set of tools for solving many different problems. Solid (continuum) elements are the standard volume elements of ABAQUS and include both first-order and second-order interpolation elements in three dimensions. The 8-node linear brick, reduced integration (C3D8R) element was chosen from the ABAQUS library for the 3D pavement FEM model. This was done because solid elements can be composed of a single homogeneous material or can include several layers of different materials for the analysis. Further, first-order elements would provide less accurate results than second-order elements, but require much less computing time. Finally, reduced integration reduces running time, especially in three dimensions. The C3D8R element has been successfully applied in previous pavement analyses (Zaghloul 1994; and Huang 1995). Using the C3D8R element, the 3D pavement model has a fine mesh at the transverse joints and at the FWD loading area. Transition meshes are used to connect fine and coarse meshes, as shown in Figure 8-11.

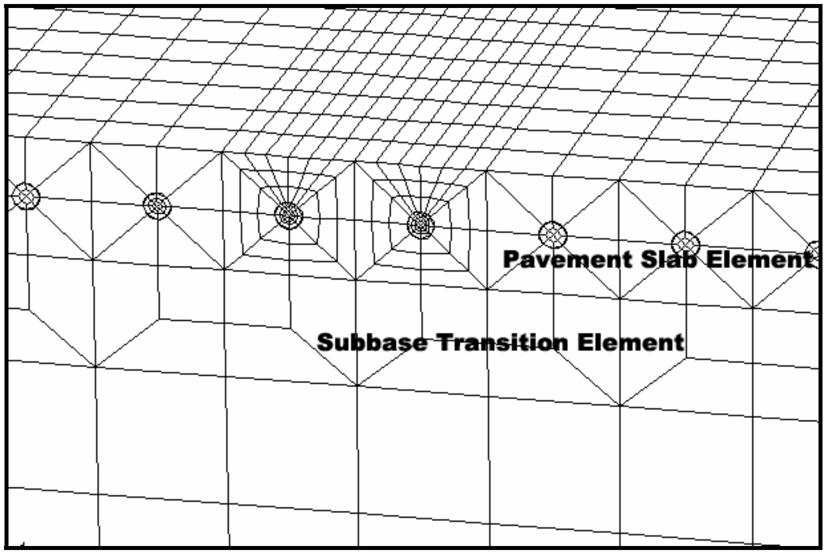

Figure 8-11 Transition Mesh with Fine and Coarse Mesh

8.2.6 FWD Load

For the FEM analyses, an FWD load of 40 KN (9,000 lb) was distributed on a circular plate having a 300 mm (12 in) diameter. In the simulation, because of the square shape of element faces, the loading area must be approximated using square brick element surfaces of 50 × 50 mm (2 × 2 in). As shown in Figures 8-12, 8-13 and 8-14, three different approximations of the loading area were used to evaluate the effects of different loading areas. Table 8-1 shows these three approximated loading areas and the distributed pressures, which were used in the FWD simulation to evaluate the effect. Due to the complexity of crack behavior under dynamic loading, only static loading was used.

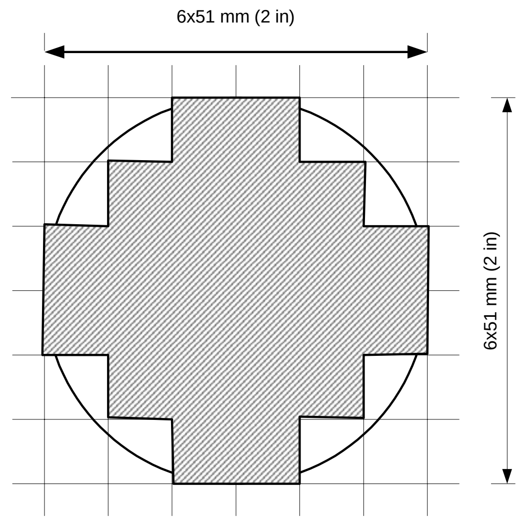

Figure 8-12   FWD Load Simulation Area #1

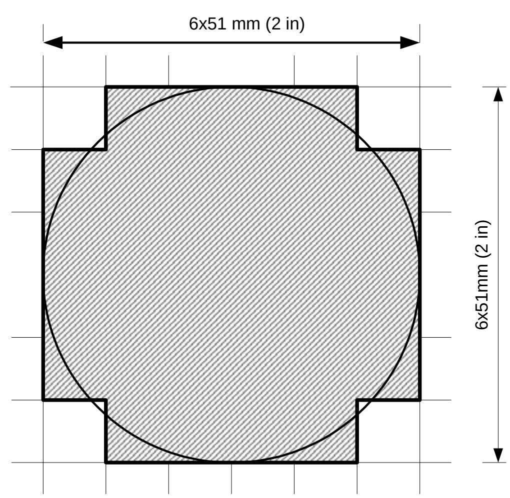

Figure 8-13   FWD Load Simulation Area #2

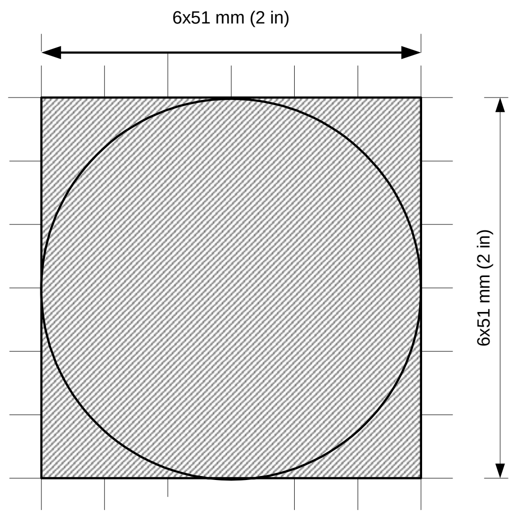

Figure 8-14 FWD Load Simulation Area #3

Table 8-1 Estimated Load Area and Pressure

Area Type	Load KN (lb)	Load Area mm^2 (in^2)	Pressure Pa (psi)
Area #1	40 (9000)	61940 (96)	636.4 (93.75)
Area #2	40 (9000)	82580 (128)	484.8 (70.31)
Area #3	40 (9000)	929900 (144)	430.9 (62.5)

8.2.7 Material Properties

PCC pavement, subbase and subgrade materials properties were considered as linear elastic and are represented by Young's modulus and Poisson's ratio. Zaghloul and White (Zaghloul and White, 1994) used the three stage model for PCC pavement, the Drucker-Prager model for base, subbase, and subgrade, and the Cam-Clay model for clays to simulate FWD deflection. However, compared to regular traffic loading, the relatively small FWD load produces very limited deflections, most of which are recoverable. The plastic deflections produced by the FWD load are therefore considered insignificant and all materials are reasonably modeled as linear elastic materials.

The back-calculated modulus at mid-slab was used as the reference modulus for the material properties. FWD field data was collected on ten selected pavement joints on US231. Data were taken at mid-slab position and across the joints. The mid-slab deflections were used to back-calculate the material properties. The deflections and those at the joints were used to calibrate the FWD model for the joints.

Table 8-2 lists the mean deflection values as well as the 95 percent confidence interval boundaries for the measured mid-slab deflections. Figure 8-15 shows these deflections plotted. Table 8-3 contains the back-calculated Young's modulus values generated by the ELMOD software.

In order to verify the back-calculated modulus values and calculate the final modulus values used in the FWD joint simulation, a 3D FEM model was developed to simulate the FWD test at the mid-slab position. Since the FWD load and the pavement structure are symmetric at mid-slab, the original pavement model, can be simplified with the roller support in the symmetric plane, shown in Figure 8-16.

Table 8-2 Measured FWD Deflections at Mid-Slab

Sensor Position mm (in) / Deflection ($mm \times 10^{-3}$)	-300 (12)	0 (0)	200 (8)	310 (12)	410 (16)	610 (24)	910 (36)	1220 (48)	1520 (60)
Mean	51.0	54.9	52.8	51.4	48.9	46.3	40.7	35.4	30.0
Stand. Deviation	4.4	4.4	4.6	3.9	4.1	3.6	3.5	2.9	2.5
95% Low bound	42.1	46.0	43.5	43.6	40.7	39.1	33.8	29.4	25.0
95% Up bound	59.9	63.8	61.8	59.3	57.2	53.4	47.7	41.1	35.1

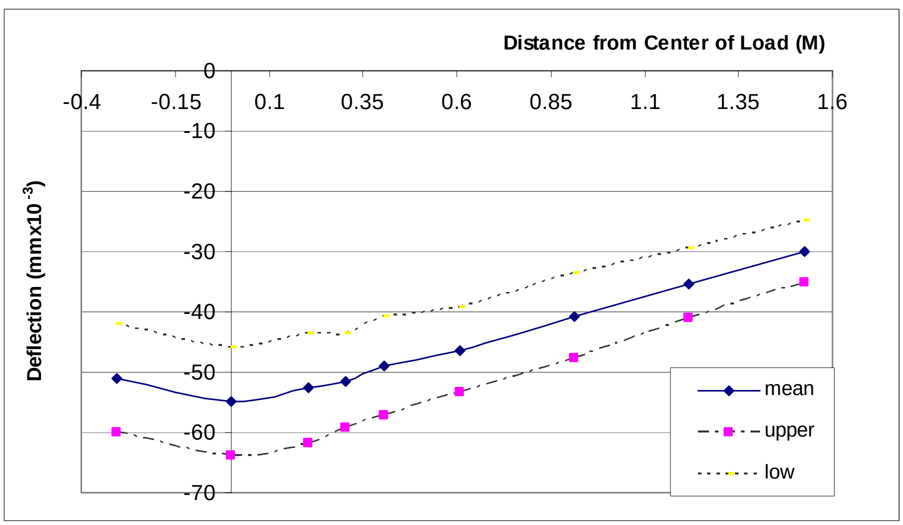

Figure 8-15 Measured FWD Deflections at Mid-Slab

Table 8-3 Back-Calculated Modulus Values

Layer	Thickness mm (in)	Poisson's Ratio	Young's Modulus MPa (kpsi)
Concrete	279 (11)	0.15	31874.7 (4,623.0)
Base	254 (10)	0.3	1468.6 (213.0)
Treat Subgrade	609 (40)	0.3	294.4 (42.7)
Subgrade	2540 (100)	0.3	294.4 (42.7)

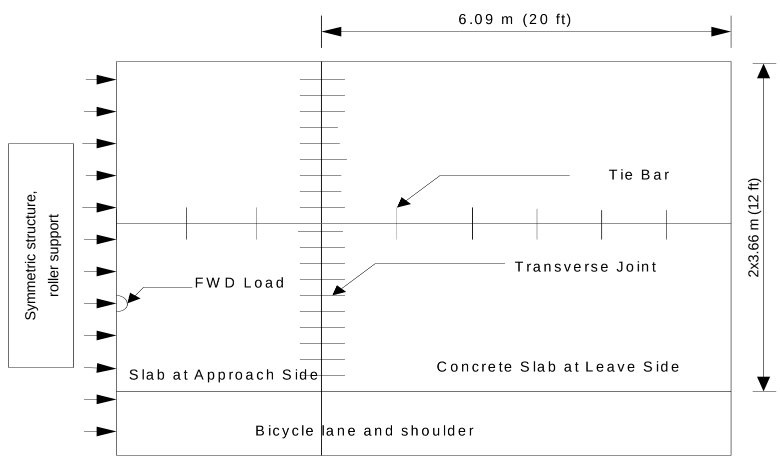

Figure 8-16 PlanView of Pavement Model with Symmetric Roller Support

With the back-calculated modulus values and the refined symmetric model, the effect of the three different approximated loading areas on simulated FWD deflections were evaluated. The results of FWD are shown Table 8-4. Note that there are no significant effects on the simulated FWD deflection results due to three approximated loading areas. Area #1 was therefore chosen for all subsequent FWD simulations.

Table 8-4 Simulated FWD Deflections at Mid-Slab with Different Load Areas

Dis. from Load Center (mm)		150	200	300	460	610	760	920	1070	1220	1370	1520
Deflection ($mm \times 10^{-3}$)	Area #1	53.4	50.5	46.1	42.8	39.1	35.7	32.4	29.4	26.5	24.0	21.7
	Area #2	53.9	50.7	46.4	43.0	39.3	35.8	32.5	29.4	26.7	24.1	21.8
	Area #3	55.3	51.3	46.2	43.2	39.2	36.0	32.5	29.6	26.6	24.1	21.8

The simulation deflections with back-calculated material properties are shown in Figure 8-17. Note that a portion of these deflections are out of the upper boundary of the FWD 95 percent confidence interval. To minimize the deflection simulation error, the back-calculated modulus values were adjusted so that the simulated deflections matched the field test deflection as much as possible. Three additional adjusted sets of modulus values were created based on the both simulation and field test deflection basins shown in Figure 8-17. These four sets of materials properties (the back-calculated values and the three additional cases) are listed in Table 8-5. The simulated deflections for each set are shown in Figure 8-17. The figure shows that simulation #3 provides the best match to field data. Therefore, the material properties from this simulation were used in the simulation for joint modeling.

Table 8-5 Material Properties Used in FEM Simulation

Layer	Thick mm (in)	ε	Young's Modulus MPa (kpsi)			
			Simulate#1	Simulate#2	Simulate#3	Back-calculated
Concrete	279 (11)	0.15	31716.1 (4,600.0)	324055.6 (4,700.0)	31716.1 (4,600.0)	31874.7 (4,623.0)
Base	254 (10)	0.3	1468.6 (213.0)	2068.4 (300.0)	1447.9 (210.0)	1468.6 (213.0)
Treat	609 (40)	0.3	294.5 (42.7)	344.7 (50.0)	296.5 (43.0)	294.4 (42.7)
Subgrade	2540 (100)	0.35	241.3 (35.0)	193.1 (28.0)	206.8 (30.0)	294.4 (42.7)

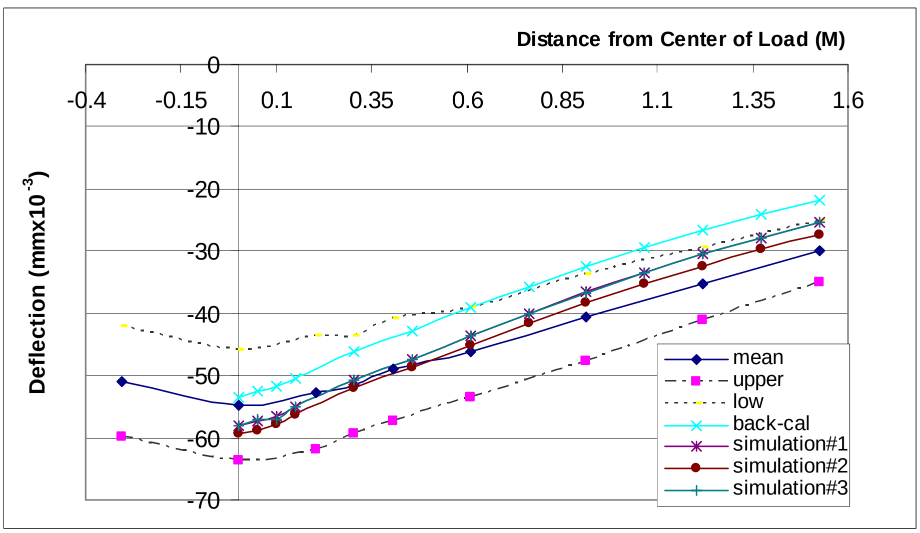

Figure 8-17 Simulated FWD Deflections Using Various Materials Properties

8.2.8 Boundary Condition and Evaluation

It was noted from the literature that previous research (Uddin 1995) did not include the passing lane slab in the analysis, but rather considered the lateral side of the travel lane as a roller support. Rather than accept this as correct, simulations were completed using three different boundary conditions on the lateral side of the travel lane, roller, free, and fixed supports. The plan view of the pavement model used for boundary condition evaluation is shown in Figure 8-18. The simulated deflection basins are shown in Figure 8-19. Based on these results it is concluded that the passing lane must be included in the model. Figure 8-19 clearly shows that the deflections with different supports at the lateral side of travel lane are significantly different than those with the passing lane support. The passing lane structure simulation shows less deflection than the fixed support simulation, and greater deflection than roller support and free support. The free support is higher than the deflection using roller support, but both provide very close deflection values.

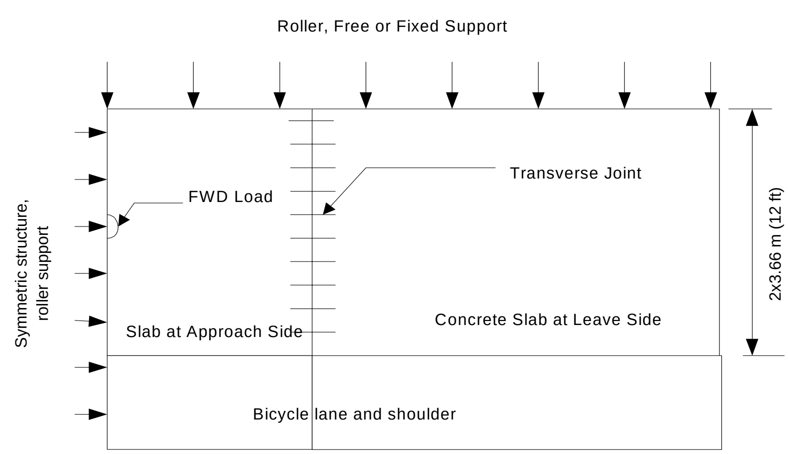

Figure 8-18 Plan View of Pavement Model Used in Boundary Evaluation

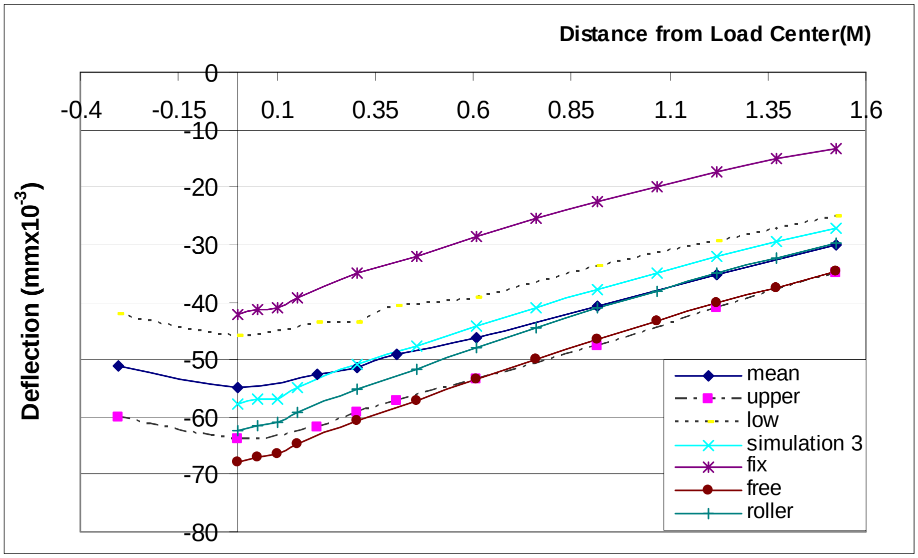

Figure 8-19 Simulated Deflections Using Various Boundary Conditions

8.2.9 Joint Simulation

FWD data was collected at 12 selected joints on US231 and used to calibrate the FWD simulation at the concrete joint position. Table 8-6 lists the mean deflection values as well as the 95 percent confidence interval boundaries for the data. The data is plotted in Figure 8-20. The measured load transfer from each joint is shown in Figure 8-21. Note that the average load transfer value is 93.5 percent.

Table 8-6 Measured Deflection at Selected PCC Pavement Joints

Sensor Position mm (in) / Deflection (mm×10^{-3})	-300 (12)	0 (0)	200 (8)	310 (12)	410 (16)	610 (24)	910 (36)	1220 (48)	1520 (60)
Mean	84.1	89.6	79.4	74.5	67.5	60.9	49.9	40.3	33.0
Stand. Deviation	7.60	7.01	6.35	5.85	5.37	4.83	3.88	3.02	2.62
95% Low bound	68.9	75.5	66.7	62.8	56.8	51.3	42.1	34.2	27.7
95% Up bound	99.3	104.	92.1	86.2	78.3	70.6	57.7	46.3	38.3

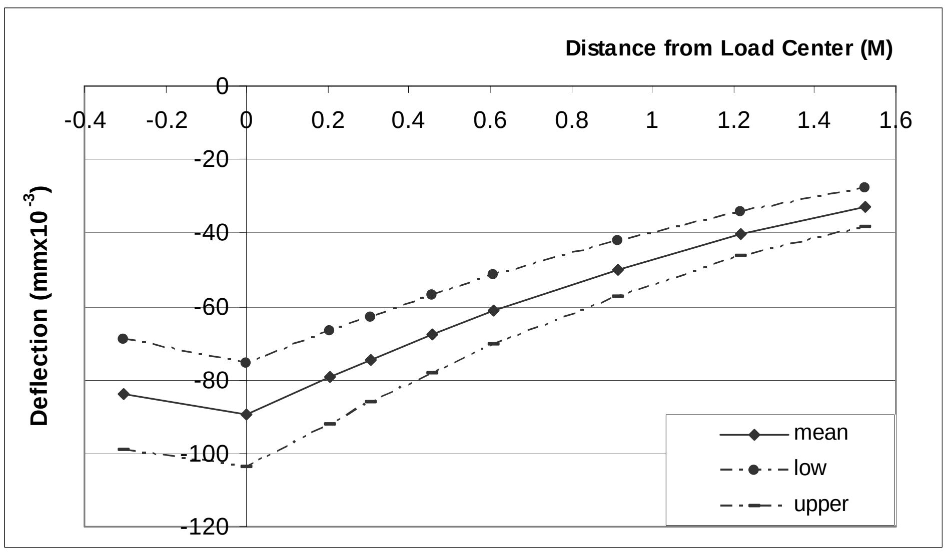

Figure 8-20 Measured Deflections at Selected Joints

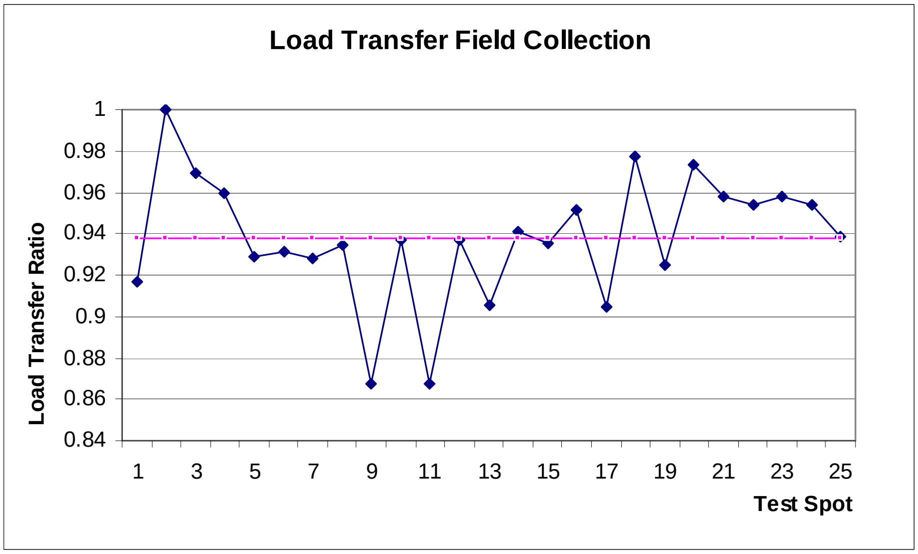

Figure 8-21 Measured Load Transfer

The materials of the four pavement layers were again considered linear elastic and the materials properties used for the mid-slab modeling at simulation #3 were used for the joint modeling as well.

The fixed end of dowel bar in the slab was simulated with TIE constraints provided by ABAQUS. TIE constraints allow two surfaces to be tied together for the duration of a simulation, even though the meshes created on those two surfaces may be dissimilar. The other (lubricated) end was simulated with contact pairs using a friction coefficient μ_{bar}. The aggregate interlock of the two contact interfaces of the slabs at the joint was simulated with contact pairs using a friction coefficient μ_{joint}. Since this bar is designed to move freely at the lubricated end, μ_{bar} should be relatively small. In the literature, it was assumed as 0 during the stresses analysis of dowel bars (Yoder 1975), and assumed as 0.05 during the temperature-induced stresses analysis for dowel bars (William, 2001). In this study, μ_{bar} is assumed as 0.1 and 0.3 to test the significance of effect on the load transfer. There is no information available in literature about the coefficient μ_{joint} except it was assumed as 0.5 by Uddin (1995). In this study, μ_{joint} is assumed as 0.5, 0.6, 0.8 and 0.9 to test the significance of effect on the load transfer. In order to choose reasonable coefficients, several FWD simulations were completed with these varying values of μ_{bar} and μ_{joint}. The results are shown in Table 8-7.

Table 8-7 Effect of the Contact Friction Coefficient on Simulated FWD Results

μ_{bar}	μ_{joint}	D_0(mm×10^{-3})	D_1(mm×10^{-3})	Ratio
0.3	0.5	71.2	61.8	0.868
	0.6	71.2	61.8	0.869
	0.8	71.1	61.94	0.870
	0.9	71.1	61.9	0.871
0.1	0.5	71.2	61.7	0.867

Both coefficients, μ_{bar} and μ_{joint}, do not appear to significantly affect the simulated FWD deflections. It may be that contact pairs in ABAQUS do not sufficiently simulate the aggregate interlock. Also, due to the dowel bars, the relative movement of the two vertical concrete contact surfaces at the joint is very small. Therefore, the contact interface provides only a limited effect on the relative movement between the two slabs. In this study, 0.3 and 0.5 were selected for μ_{bar} and μ_{joint} respectively during the simulations due to the fact that both coefficients do not significantly affect the simulation results.

As shown in Figure 8-22, the simulated FWD deflection basin does not fall within the 95 percent confidence intervals of the measured deflections. The relative simulation error at the load center is about 20 percent. Since the tested and simulated load transfers are 0.94 and 0.87 respectively, the error of the load transfer is about 7 percent. Possible reasons for this difference may be the presence of voids beneath the concrete joint. The location and quantity of voids varies and both are difficult to quantify. They are not considered in the simulation. The complicated behavior of the aggregate interlock at the joint could also affect the results. The aggregate interlock is hard to describe with one mechanical property. In this study, the contact interface with certain friction coefficients was used to simulate the performance. Unfortunately this simulation does not provide a good match to the measured FWD deflections.

Although the FWD simulation is out of the 95 percent confidence boundary as shown in Figure 8-23, the simulation results for different locations might still provide an indication of the effect of test location on load transfer. Figure 8-23 shows six different loading locations used in simulations to evaluate the effect of test location on FWD deflection. Test locations #1, #2 and #3 are located at the center of the slab in the transverse direction, and are, respectively, 150 mm (6 in), 200 mm (8 in), and 250 mm (10 in) from the joint in the longitudinal direction. Test locations #4, #5 and #6 are located 150 mm (6 in) from the center of slab in the transverse direction, and are, respectively, 150 mm (6 in), 200 mm (8 in), and 250 mm (10 in) from the joint in the longitudinal direction. The simulated FWD deflections, D_0 and D_1, across the joint are listed in Table 8-8.

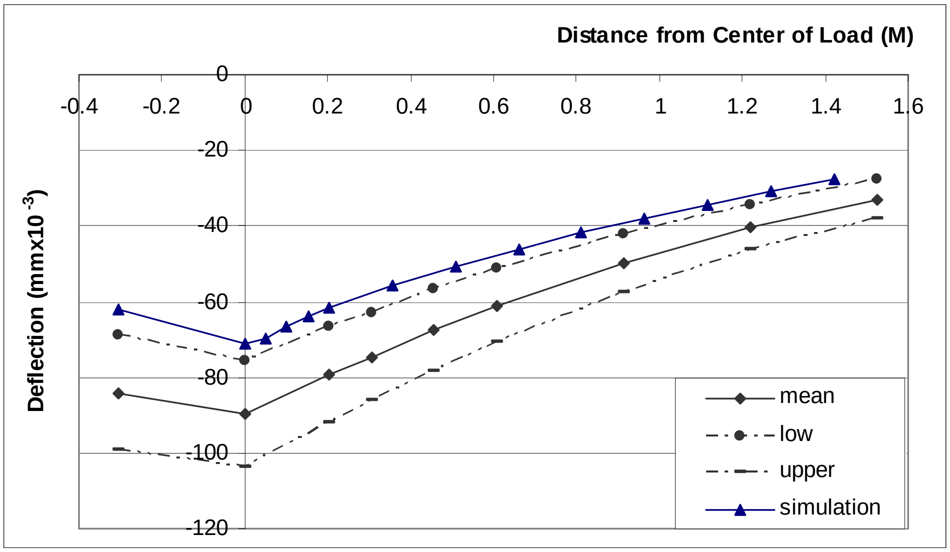

Figure 8-22 Simulated FWD Deflection at Joint

Table 8-8 FWD Load Location Effect on Simulation Result

Location	$D_0(mm\times10^{-3})$	$D_1(mm\times10^{-3})$	Ratio
1	73.2	63.4	0.866
2	71.2	61.8	0.868
3	69.5	60.6	0.872
4	73.1	62.1	0.849
5	71.1	61.1	0.859
6	69.4	60.5	0.872

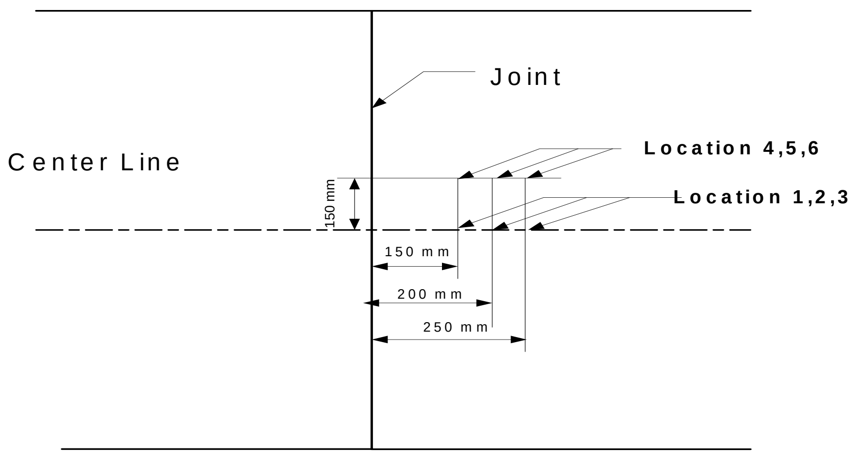

Figure 8-23 FWD Load Locations for FWD Simulation

As shown in Table 8-8, the test locations variation produces less than 4 percent difference in the load transfer during the FWD simulation. This may be an indication that a test location does not significantly affect the load transfer. In order to further evaluate the simulation result, the measured load transfer at locations #1, #2 and #3 are shown in Figure 8-24. No significant effect due to the test position is evident. Therefore, both simulation and field measurements appear to indicate that test position has no significant effect on load transfer.

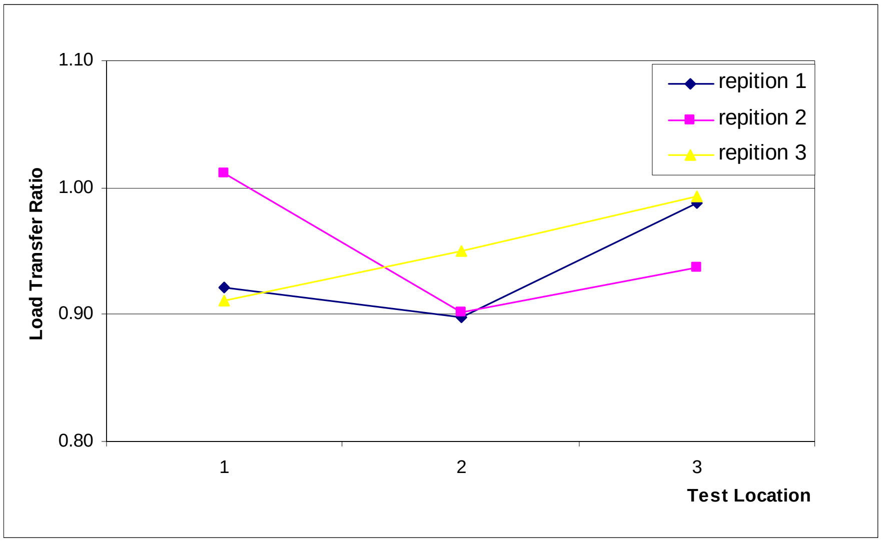

Figure 8-24 Measured Load Transfer Variation at Different Test Locations

8.3 Conclusions

A 3D pavement model was developed in order to evaluate the effect of test location on load transfer at joints. Using back-calculated material properties, based on measured FWD deflections at mid-slab, the effect of estimated load areas and different boundary conditions were evaluated on simulated FWD deflections.

The following are some findings and conclusions derived from this study.

1. Different approximated load areas for 3D FWD simulation do not have a significant effect on the results;

2. Different supports at the lateral side of the travel lane have a significant effect on the simulated FWD deflections. It is necessary to have the passing lane structure as the support to minimize simulation error;
3. Contact interface did not accurately simulate the behavior of aggregate interlock under load in this model. The aggregate interlock provides a measure of load transfer, but the contact interface insignificantly changes the load transfer in the simulation; and
4. Based on both simulation and field measurement, the test location has no significant effect on FWD deflection.

CHAPTER 9 TEMPERTURE EFFECT ON FWD MEASUREMENTS

9.1 Introduction and Objectives

The FWD has been used for nondestructive pavement structure testing for many decades. The measured pavement deflection under a particular load is used as a direct indicator of structural capacity. The typical FWD sensor spacing and geometry is shown in Figure 9-1. In the study, the FWD deflection under the load plate (D_0) was used as a measure of the pavement performance in both the sealed and unsealed sections. The load transfer value (D_1/D_0) of joints/cracks was used as an index of the condition of the joints/cracks.

The temperature gradient through a PCC slab can lead to tensile or compressive stresses at the top and bottom because of curling and warping. For HMA pavements, the asphalt binder is a viscoelastic material, its consistency changes with temperature. The pavement temperature may therefore significantly affect FWD deflections for both PCC and HMA pavements.

When analyzing FWD data the temperature effect must be minimized. This is typically done by converting the FWD deflections to a standard temperature condition, usually a pavement temperature of 20 or 25C (68 or 77F). However, when the literature was searched, no temperature correction information was found for PCC pavements.

Additionally, although D_0 temperature correction research abounds for HMA pavements, none of the literature dealt with converting the other sensors, such as D_1/D_0 ratio.

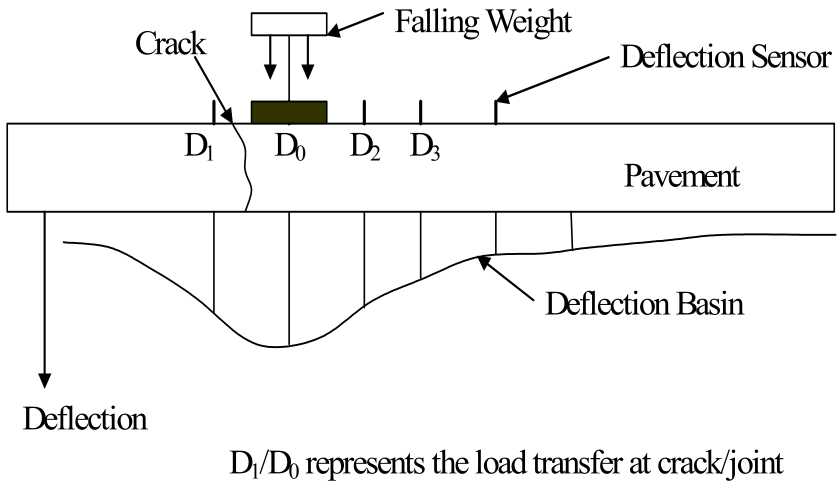

Figure 9-1 FWD Sensor Spacing and Geometry

9.2 <u>FWD Data Collection</u>

In order to evaluate the effect of temperature on FWD deflections, FWD test data were collected at five research test sites under different pavement temperature conditions. In September 2001, data were collected at two PCC pavement test sites, sites 1A1 and 1A2. In August 2002, data were collected at PCC pavement test site 1B, at full depth HMA pavement test site 5B, and at composite pavement test site 9A. For each site, FWD measurements were taken approximately every one and one-half hours between the early morning and the late afternoon. The pavement surface temperatures changed approximately 20C (15F) during data collection for each test site. Ten joints/cracks were selected from each test site as the data collection repetition. Table 9-1 shows the pavement and air temperatures for each test site at the time the data were collected.

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Table 9-1 Pavement and Air Temperature at Different Data Collection Time

Test Site	Time & Temp.	Test 1	Test 2	Test 3	Test 4	Test 5
1A1	Time	7:30am	9:30am	12:00pm	2:30pm	5:30pm
	Air Temp(C)	32.2	33.2	38.3	48.9	44.4
	Surf Temp(C)	29.6	32.5	34.7	38.2	34.2
1A2	Time	7:00am	9:00am	11:30am	2:00pm	5:00pm
	Air Temp(C)	25.8	32.5	35.0	47.6	42.9
	Surf Temp(C)	28.7	31.8	31.4	35.1	34.3
1B	Time	6:30am	9:30am	12:30pm	4:30pm	
	Air Temp(C)	15.7	28.9	38.9	39.1	
	Surf Temp(C)	17.0	25.3	28.7	33.6	
5B	Time	7:30am	10:00am	1:00pm	5:00pm	
	Air Temp(C)	17.8	32.9	44.8	41.4	
	Surf Temp(C)	19.5	26.9	33.9	32.6	
9A	Time	8:00am	10:30am	2:00pm	5:30pm	
	Air Temp(C)	21.4	37.2	48.8	39.4	
	Surf Temp(C)	21.2	28.0	33.6	31.9	

9.3 Pavement Temperature Model

Several models have been developed to predict the temperature gradient distribution through the pavement depth. One model for the PCC pavements and two models for HMA pavement are investigated and adapted to predict average pavement temperatures. FWD deflection variations due to pavement temperatures were then evaluated using the average pavement temperatures. In adapting the models, the pavement surface temperature, air temperature, pavement thickness, and FWD test time were selected as possible input variables. These variables not only provide appropriate estimates for the pavement temperature, but can also be easily obtained using FWD data. Infrared sensors mounted on the FWD frame measure the pavement surface and air temperatures. These temperature readings are recorded in the FWD data file with the corresponding test time.

9.3.1 PCC Pavement Temperature Model

The properties of portland cement concrete are usually considered temperature independent. However, the temperature gradient through the slab can lead to significant tensile or compressive stresses at the top and bottom of the slab (Chen 2001). As shown in Figure 9-2, at night when the slab surface is cooler than the slab bottom, the temperature gradient may cause the slab corners to curl upward creating tensile stresses at the slab surface and compressive stresses at the bottom of the slab. During the day, the slab surface is warmer than the slab bottom and the slab is mainly supported by the edges, as shown in Figure 9-3. The top of the slab is in compression while the bottom is in tension. FWD deflections can therefore vary significantly due to the temperature stresses in a slab.

The PCC pavement temperature gradient varies throughout the day and over the seasons. The typical temperature gradient was predicted by using the Climatic-Materials-Structural (CMS) computer model developed at the University of Illinois at Urbana Champaign (Thompson 1987). Figures 9-4, 9-5, and 9-6 show the typical temperature gradient distribution through a 230 mm (9 in) PCC slab during April, July and November, respectively. This nonlinear temperature distribution is usually approximated using a third degree polynomial,

$$T = A + Bz + Cz^2 + Dz^3 \qquad 9\text{-}1$$

where A, B, C and D are constant coefficients. Table 9-2 shows one example of curve-fitting coefficients for six times during a 24-hour period in April, at Urbana, Illinois (Thompson 1987).

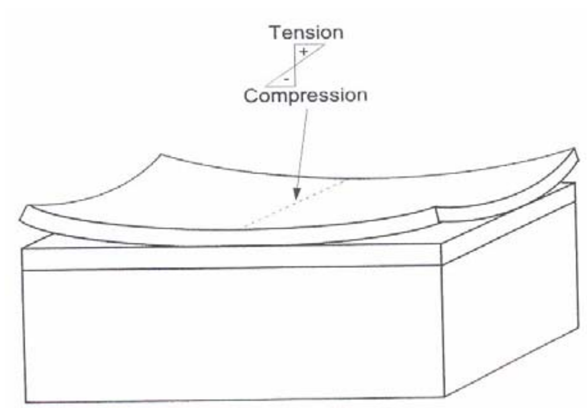

Figure 9-2 Concrete Slab Curling at the Night

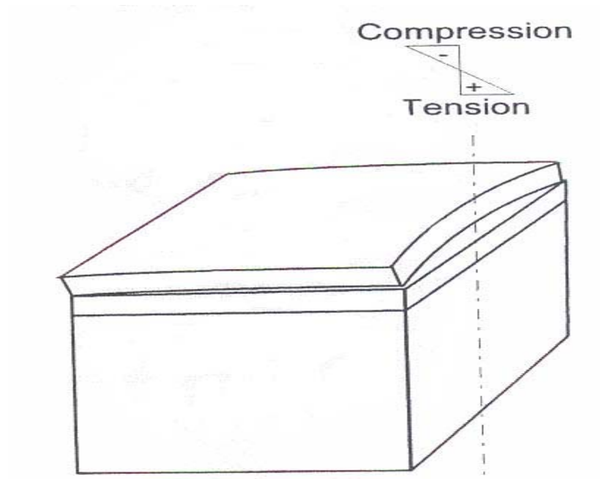

Figure 9-3 Concrete Slab Warping at Daytime

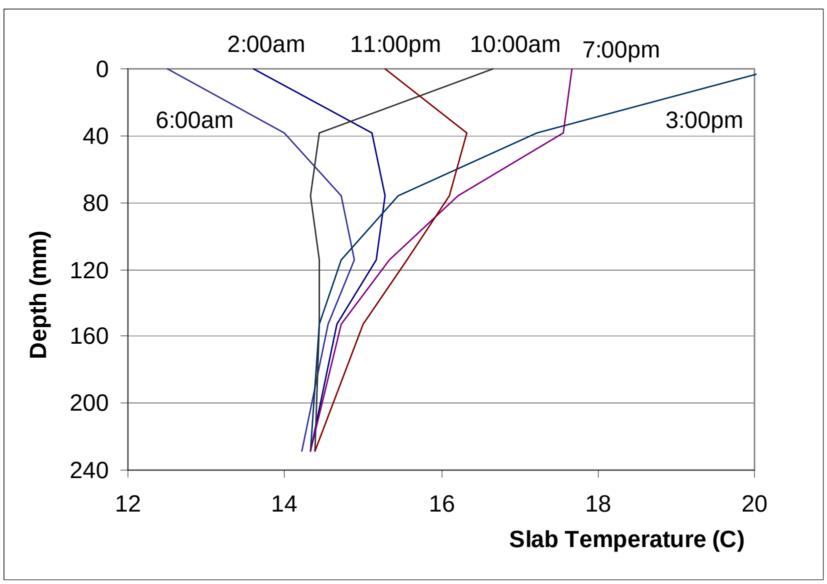

Figure 9-4 Temperature Distribution in PCC Slab (April, Urbana, Illinois)

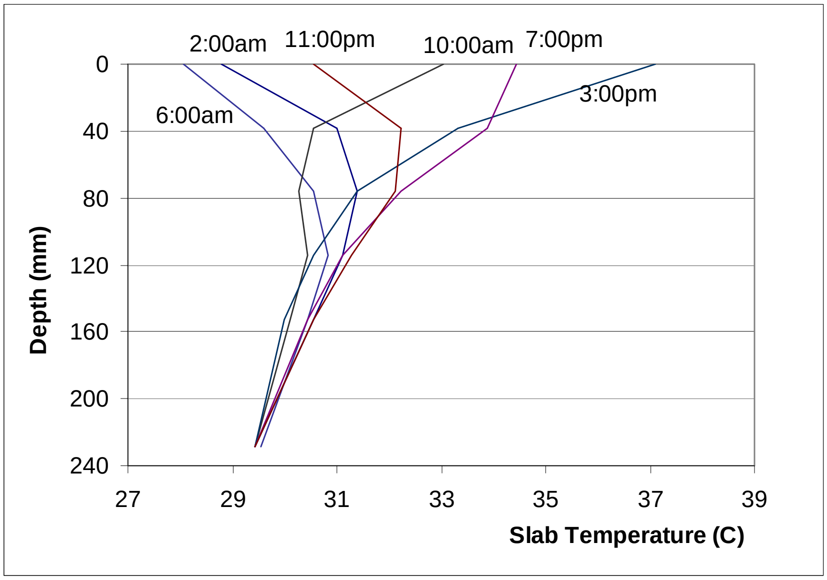

Figure 9-5 Temperature Distribution in PCC Slab (July, Urbana, Illinois)

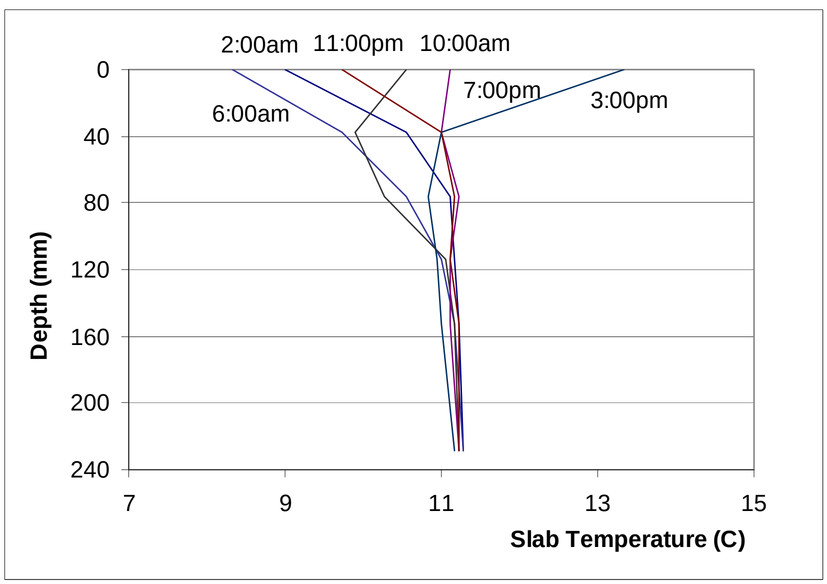

Figure 9-6 Temperature Distribution in PCC Slab (November, Urbana, Illinois)

Table 9-2 Curve-Fitting Coefficients for Temperature Data

Time		2:00am	6:00am	10:00am	3:00pm	7:00pm	11:00pm
Polynomial Coefficient	A	2.6905	1.8810	1.12143	1.4762	2.9286	3.3214
	B	-0.4947	0.09127	0.1720	-0.5291	-1.3717	-0.8723
	C	-0.16040	-0.16931	0.07937	0.26852	0.07937	-0.11905
	D	0.03292	0.01294	0.04115	-0.0514	0.02056	0.03479

9.3.2 HMA Pavement Temperature Model

No temperature gradient distribution research was found for the HMA pavements in Indiana. However, two temperature gradient models were selected from the literature and applied to the data.

Pavement Temperature Model I

Equation 9-2 is a temperature model of the pavement temperature gradient through the depth of an HMA pavement that was developed by Witzcak (1972).

$$T_{pave}(Z) = T_{air}\left[1 + (\frac{1}{0.394Z + 4})\right] - \frac{1.11}{(0.394Z + 4)} + 3.34 \quad \text{9-2}$$

where,

T_{pave} = mean monthly pavement temperature (C) at depth Z,

T_{air} = mean monthly air temperature (C), and

Z = depth within asphalt mix layer from the surface (mm).

In this study, the mean monthly temperatures were estimated using temperatures recorded during FWD data collection. In order to get a better fit between predicted and measured pavement surface temperatures, both the pavement surface temperature (T_{surf}) and the air temperature (T_{air}) were used to correct Equation 9-2. The relationship between the predicted and measured surface temperatures is shown in Figure 9-7 for the 9A test site. As shown in the Figure, the relationship is,

$$T_{surf\text{-}measured} = 1.592(T_{surf\text{-}predicted}) - 25.3 \qquad 9\text{-}3$$

and has an R^2 of 0.93. With the assumption that this correction is applicable though the entire depth of the pavement, Equation 9-2 can be corrected by Equation 9-3.

$$T_{pave}(Z) = 1.592 x T_{air}\left[1 + \left(\frac{1}{0.394Z + 4}\right)\right] - \frac{1.77}{(0.394Z + 4)} - 20.0 \qquad 9\text{-}4$$

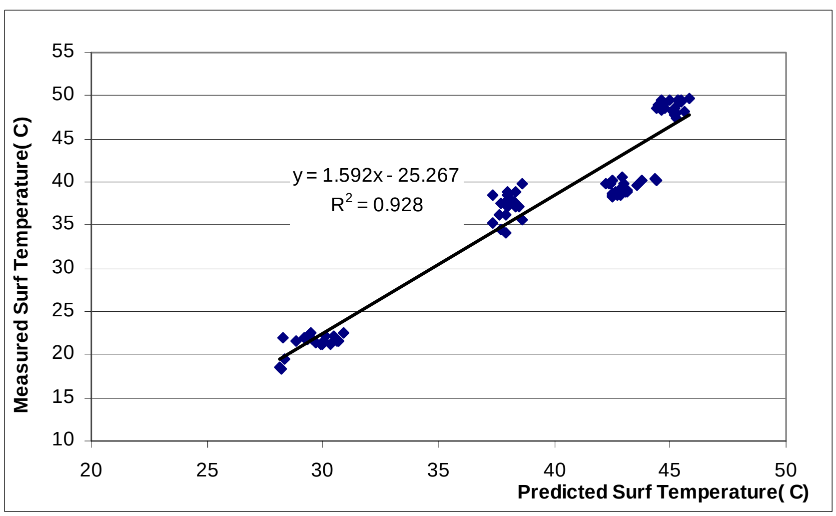

Figure 9-7 Relationship between Predicted and Measured Temperatures

The average temperature ($T_{average}$) through the pavement depth (H) is then,

$$T_{average} = \frac{1}{H}\int_{0}^{H}\left(1.592T_{air}\left[1+\left(\frac{1}{0.394Z+4}\right)\right]-\frac{1.77}{0.394Z+4}-20\right)dz \qquad 9\text{-}5$$

or,

$$T_{average} = \frac{1}{H}\ln(0.1H+1)\times(4.041T_{air}-4.49)+(1.592T_{air}-20) \qquad 9\text{-}6$$

From Equation 9-4, when Z equals 0, the surface temperature can be predicted using

$$T_{surf} = T_{pave}(Z=0) = 1.99T_{air}-20.44 \qquad 9\text{-}7$$

The air temperature can be predicted using

$$T_{air} = 0.503T_{surf}+10.27 \qquad 9\text{-}8$$

Substituting Equation 9-8 into Equation 9-6,

$$T_{average} = \frac{1}{H}\ln(0.1H+1)\times(2.033T_{surf}+37.01)+(0.801T_{surf}-3.65) \qquad 9\text{-}9$$

Equation 9-6 and Equation 9-9 can be used to estimate the average pavement temperature ($T_{average}$) through the entire pavement depth so long as the pavement thickness and either the air or the pavement surface temperature is known. As an example of Equation 9-6, the pavement temperature gradient through a 200 mm (8 in) thick HMA pavement is shown in Figure 9-8 for air temperatures of 30 and 50C (86 to 122F). Figure 9-9 shows the predicted and measured pavement surface temperatures for the 5B test site. The prediction errors are shown in Figure 9-10. This data shows that Equation 9-7 provides a good estimate of pavement surface temperature with an average error of approximately 5 percent.

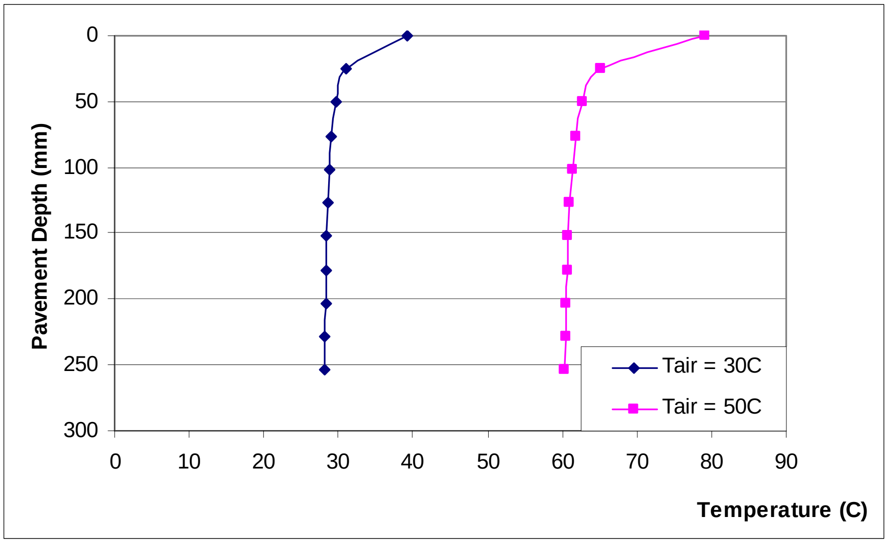

Figure 9-8 Predicted Pavement Temperature Gradient

This model is obviously restricted to surface temperatures from 10 to 60C (50 to 140F) due to the limited input data used in the regression model.

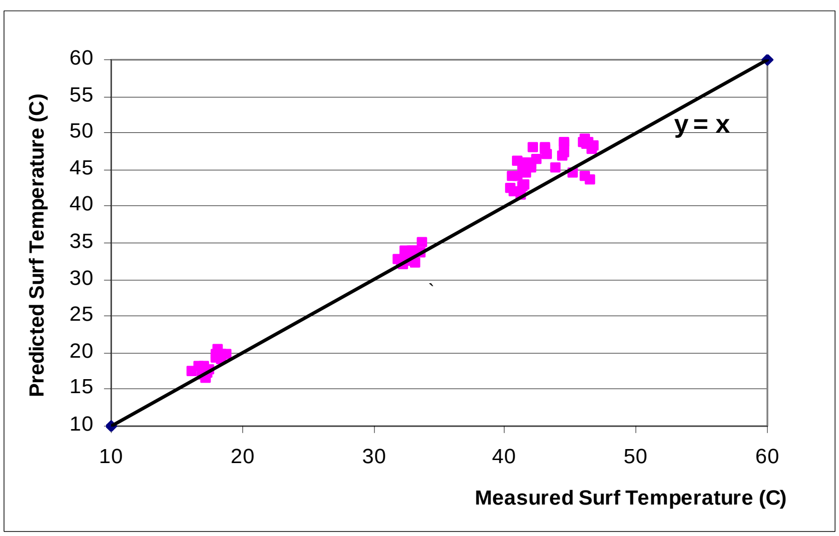

Figure 9-9 Pavement Surface Temperature Prediction for Test Site 5B

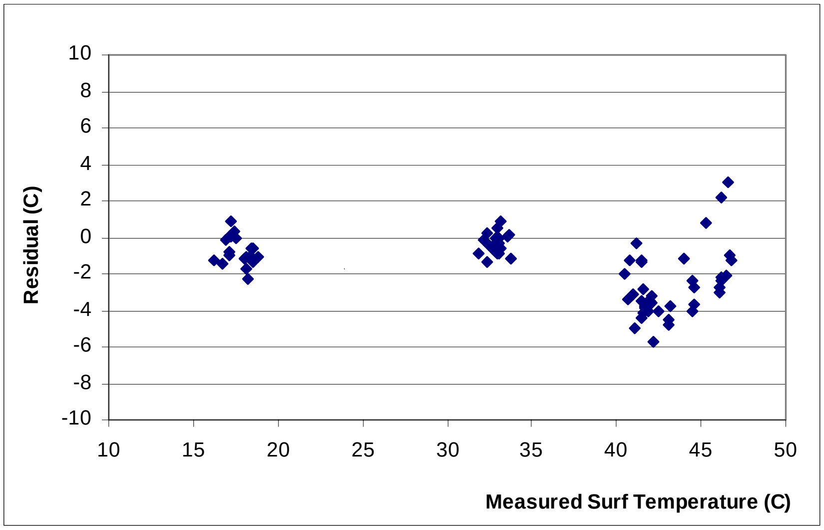

Figure 9-10 Pavement Surface Temperature Prediction Residual for Test Site 5B

Pavement Temperature Model II

A second model that uses time of day as a variable was also selected for investigation (Park 2001).

$$T_z = T_{surf} + (-0.3451z - 0.0432z^2 + 0.00196z^3)\sin(-6.3252t + 5.0967) \quad \text{9-10}$$

where,

T_z = pavement temperature at depth Z (C),

T_{surf} = pavement temperature at the surface (C),

Z = depth at which temperature is to be determined (cm), and

t = time when surface temperature was measured (days).

[days; 0< t < l(e.g., 1:30pm, = 13.5/24 = 0.5625 days)].

It was stated by Park that pavement surface temperatures used to develop the model were 19 to 43C (66 to 109F). Pavement thickness used to develop the model was 14 cm to 26 cm (5.5 to 10 in). The development temperature prediction model has an R^2 value greater than 0.90. Figure 9-11 shows the pavement temperature gradients though the pavement depth.

In this model, the middle depth temperature was chosen as representative of the pavement temperature. The estimated pavement temperature is then,

$$T_{pave} = T_{surf} + (-0.1726H - 0.0108H^2 + 0.000245H^3)\sin(-6.3252t + 5.0967) \quad 9\text{-}11$$

where,

H is the thickness of the pavement.

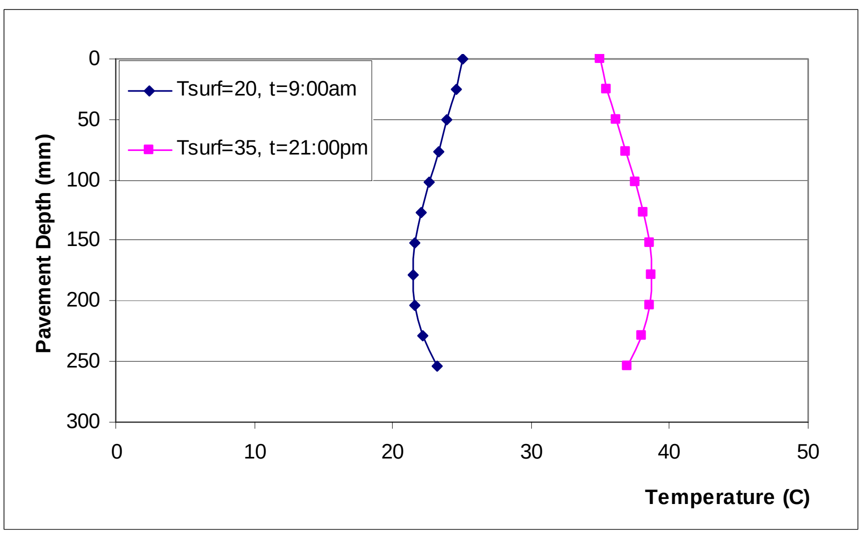

Figure 9-11 Pavement Temperature Gradient

9.4 Temperature Data Statistical Analysis

The temperature correction for FWD measurements taken at mid slab and for load transfer ratios at joints and cracks are discussed in this section.

9.4.1 Temperature Correction for PCC Pavement

FWD Deflection Correction

The factor k in Equation 9-12 is assumed to correct the FWD deflection measured under any conditions to the standard conditions.

$$u_0 = u_t \times k \qquad \text{9-12}$$

where,

μ_0= standard deflection at the reference temperature (T_0) and time (t_0),

μ_t= measured deflection at testing temperature T and time (t), and

k = the correction factor.

Based on the pavement temperature models presented in the previous section, the correction factor k should be dependent on:

1. Pavement surface temperature;
2. FWD collection season;
3. Time of day of FWD data collection; and
4. Concrete slab thickness.

Since the FWD measurement was usually collected in the fall, the collection season factor was not considered. The concrete slab thickness was also ignored to simplify the model, since the typical INDOT concrete pavement thickness ranges from 23 mm (9 in) to 300 mm (12 in). The time of day factor, pavement surface temperature and air temperature can be had from the FWD data file. Therefore, as shown in Equation 9-13, the FWD deflection is approximated using a first-order regression model with three variables, T_{surf}, T_{air}, and t (test time, 0< t < 1).

$$\mu_i = \beta_0 + \beta_1 T_{surf} + \beta_2 T_{air_i} + \beta_3 t_i + \varepsilon_i \qquad 9\text{-}13$$

where

μ_i = the FWD deflection,

$\beta_0, \beta_1, \beta_2, \beta_3$ = parameters,

T_{surf} = the pavement surface temperature,

T_{air}, = air temperature,

t_i = test time(0 < t < 1), and

ε_i = error term.

Tables 9-3 and 9-4 show the ANOVA table and parameter estimates for the 1A2 test site data.

Table 9-3 ANOVA Table for FWD Deflection Regression

Source	DF	SS	MS	F	Prob >F	R^2
Model	3	390.5935	130.1978	12.73612	3.46E-06	0.45
Error	46	470.2452	10.22272			
Total	49	860.8387				

Table 9-4 FWD Deflection Regression Parameter Estimation

Variable	Estimate	Std Err	T stat	P-Value
Intercept	50.87642	13.46424	3.778634	0.000452
T_{surf}	0.786841	0.220834	3.563044	0.000867
T_{air}	-1.08249	0.614166	-1.76254	0.084621
T	-7.55925	7.116533	-1.06221	0.293686

Since the P-value for variables T_{air} and t are 0.08 and 0.293 respectively, it is concluded with 95 percent confidence that T_{air} and t are not significant factors in the FWD deflection prediction model. Therefore, the FWD deflections are represented by pavement surface temperature, which is the only significant factor in Equation 9-13. The regressions between pavement surface temperature and FWD deflection for test sites 1A1, 1A2, and 1B are shown in Figures 9-12, 9-13 and 9-14, respectively. All three slopes in the regression models are significant, as shown in Table 9-5.

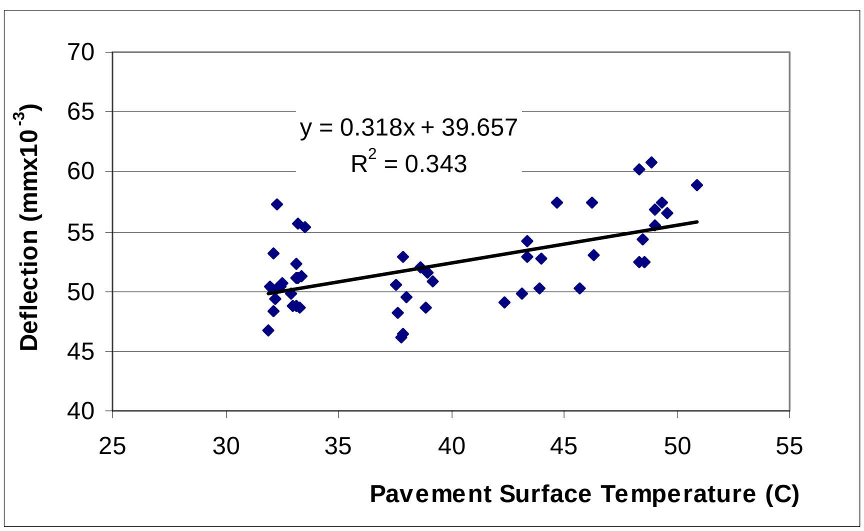

Figure 9-12 FWD Deflection at Test Site 1A1

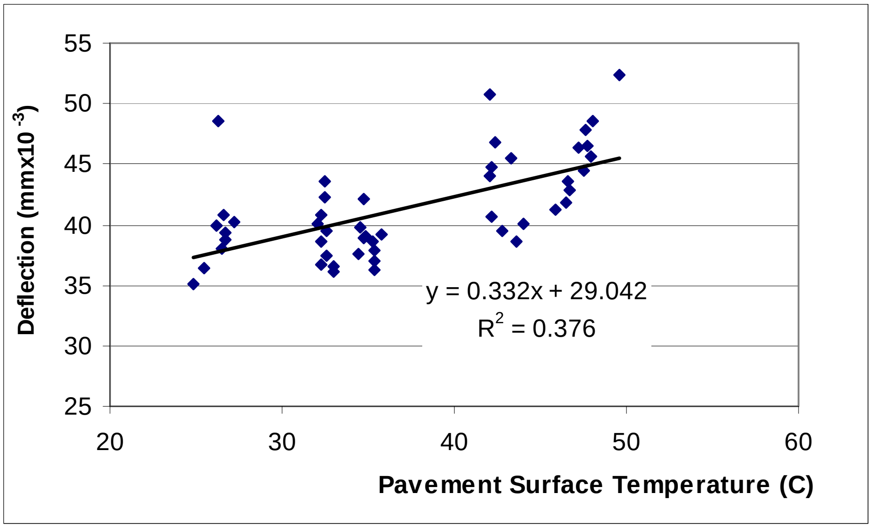

Figure 9-13 FWD Deflection at Test site 1A2

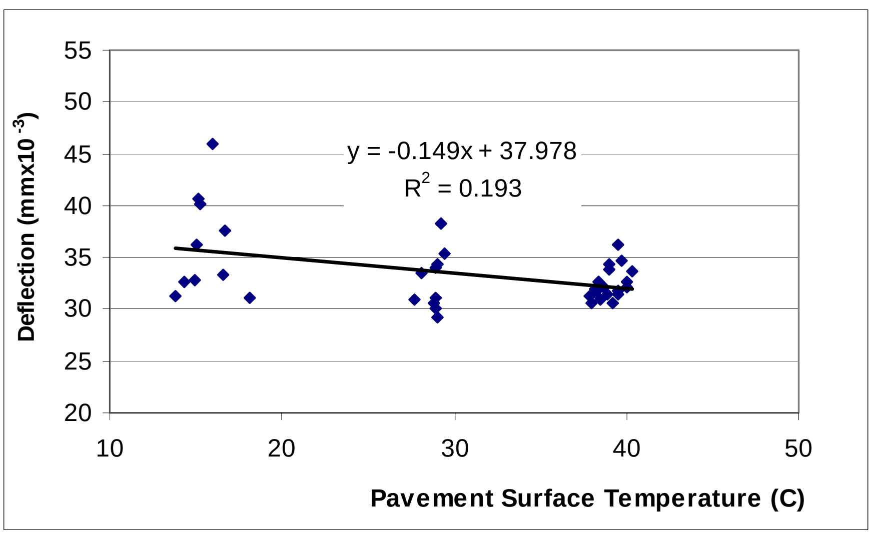

Figure 9-14 FWD Deflection at Test Site 1B

Table 9-5 Significance Test for FWD Deflection Regression Slope

Regression Slope	Coefficient	Std Err	T stat	P-Value
Site 1A1	0.318	0.0636	5.001	<0.0001
Site 1A2	0.332	0.0624	5.325	<0.0001
Site 1B	-0.149	0.0494	-3.011	0.00046

Note from the three figures that the R^2-values are relatively low indicating that pavement surface temperature does not completely explain deflection variation. Moreover, Figures 9-12 and 9-13 show that FWD deflections increase when pavement temperatures increase, while Figure 9-14 shows the opposite relation. This contradiction shows it is hard to develop a direct relation between pavement temperature and deflection for concrete pavement with the collected data. Therefore, no correction factors are considered for concrete pavement FWD deflections in this study.

Load Transfer Correction

The factor k in Equation 9-14 is assumed to correct the measured load transfer to standard conditions.

$$r_0 = r_t \times k \qquad \text{9-14}$$

where,

r_0 = standardized load transfer at the certain temperature (T_0) and time (t_0),

r_t = the measured load transfer at the collected temperature T and time (t), and

k = the correction factor.

As with the FWD mid-slab deflection, load transfer is approximated using a first-order regression model with three variables, T_{surf}, T_{air}, and t (test time, $0< t < 1$).

Using the FWD data collected at test site 1A2, Tables 9-6 and 9-7 show the ANOVA table and parameter estimates, respectively. Since the P-values for variables T_{air} and t are 0.092 and 0.394 respectively, it is concluded with 95 percent confidence that T_{air} and t are not significant factors in the deflection prediction model. Therefore, the load transfer is represented by pavement surface temperature, which is the only significant factor. The regression relation between pavement surface temperature and load transfer for test sites 1A1, 1A2 and 1B are shown in Figures 9-15, 9-16, and 9-17, respectively. As shown in Table 9-8, the regression slopes for test sites 1A1 and 1A2 are significant, while for 1B is insignificant.

Table 9-6 ANOVA Table for Load Transfer Regression

Source	DF	SS	MS	F	Prob >F	R^2
Model	3	0.006387	0.002129	13.72648	2.91E-05	0.65
Error	22	0.003412	0.000155			
Total	25	0.009799				

Table 9-7 Load Transfer Regression Parameter Estimation

Variable	Estimate	Std Err	T stat	P-Value
Intercept	0.99511	0.036661	27.14339	2.08E-18
T_{surf}	-0.0029	0.00113	-2.56618	0.017614
T_{air}	0.003143	0.001784	1.761945	0.09197
t	-0.02861	0.032966	-0.86794	0.394798

The results from all three test sites show reduced load transfer as surface temperatures increase. Theoretically, when the temperature increases, the load transfer between slabs should increase since the slabs expand and more contact occurs at the faces of the two slabs. For this research the impact of temperature on load transfer at the joint is ignored, and no correction factors are considered. The is because the small R^2 values in the regression results indicate there is no strong relation between test pavement surface temperature and load transfer at the joints. Additionally, load transfer reduction is very small, less than 10 percent from the lowest test temperature to the highest, and the reduction is opposite to what is expected.

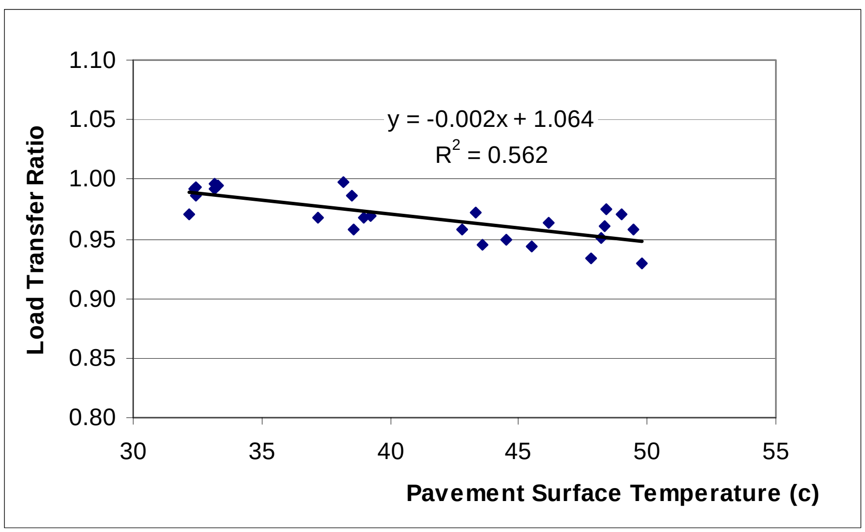

Figure 9-15 Load Transfer at Test Site 1A1

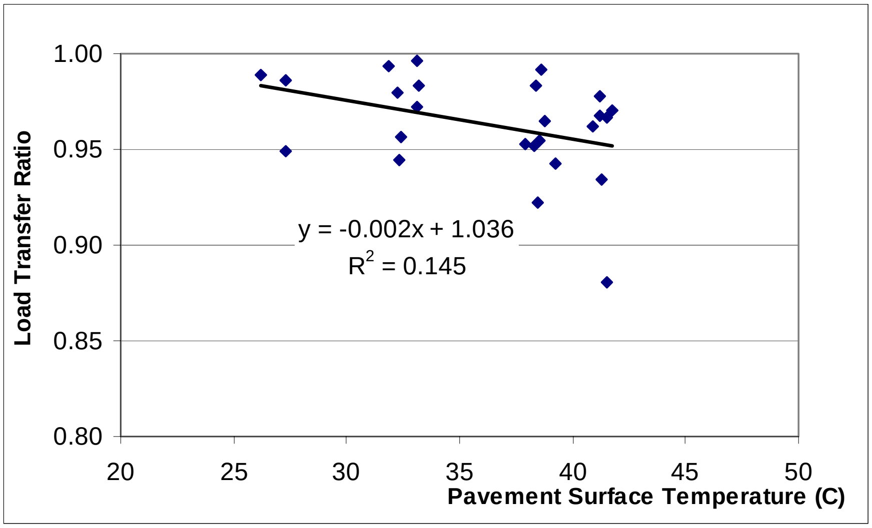

Figure 9-16   Load Transfer at Test Site 1A2

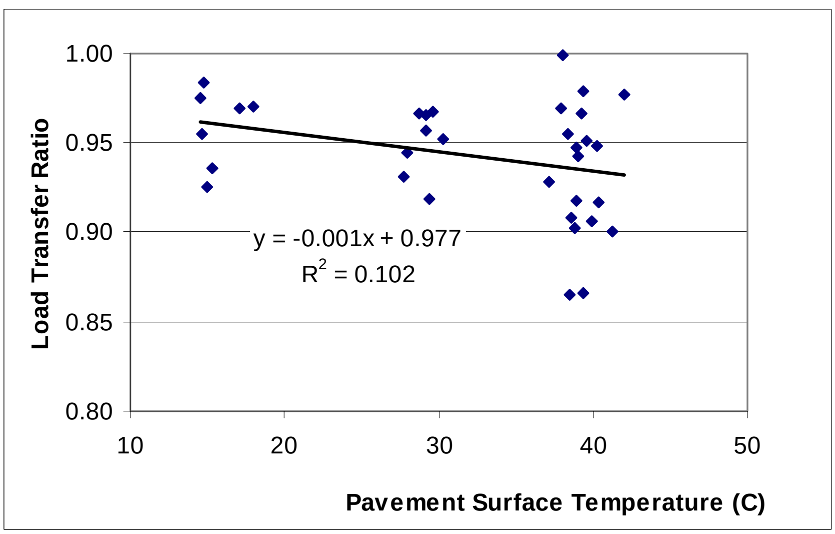

Figure 9-17   Load Transfer at Project 1B

Table 9-8 Significance Test for Load Transfer Regression Slope

Regression Slope	Coefficient	Std Err	T stat	P-Value
Site 1A1	-0.002	0.0419	-5.55	<0.0001
Site 1A2	-0.002	0.0001	-2.13	0.0435
Site 1B	-0.001	0.0001	-1.91	0.0655

9.4.2 Temperature Correction for HMA Pavement

Temperature Correction for Deflection with Temperature Model I

FWD deflections at test site 9A are presented in Figure 9-18 using the average pavement temperature estimated with the HMA pavement temperature model I.

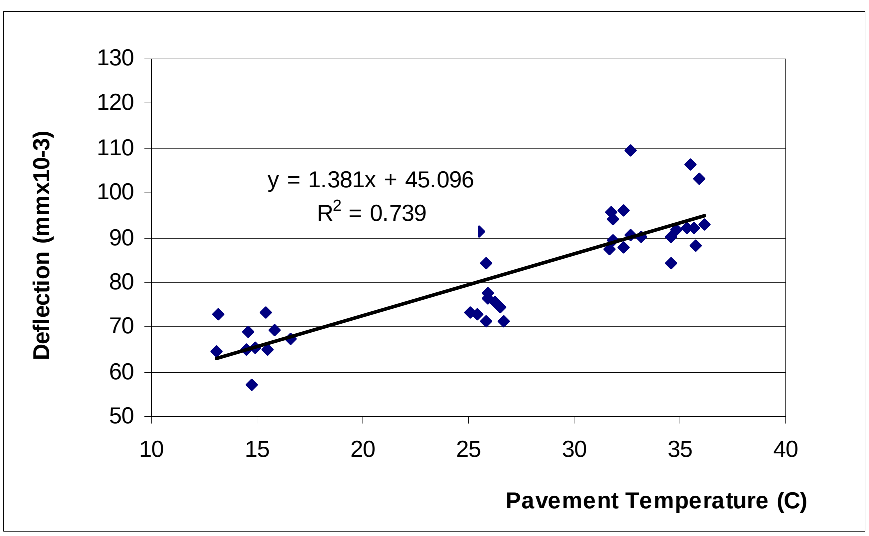

Figure 9-18 FWD Deflection at Test Site 9A

A linear regression relationship between deflection, μ_t, and pavement temperature, T_{pave}, was assumed. Tables 9-9 and 9-10 show the ANOVA table and parameter estimates, respectively.

Table 9-9 ANOVA Table for FWD Deflection Regression

Source	DF	SS	MS	F	Prob >F	R2
Model	1	4669.565	4669.565	104.8356	2.4010E-12	0.739
Error	37	1648.046	44.54178			
Total	38	6317.611				

Table 9-10 Deflection Regression Paramater Estimation

Variable	Estimate	Std Err	T stat	P-Value
Intercept	45.09562	3.786547	11.90943	3.19E-14
T_{pave}	1.381282	0.134905	10.23893	2.4E-12

The regression model for deflection and temperature is

$$\mu_t = 1.381 T_{pave} + 45.10 \qquad 9\text{-}15$$

Factor k corrects the measured FWD deflection to standard conditions, as shown in Equation 9-13. The factor k is

$$k = u_0 / u_t \quad . \qquad 9\text{-}16$$

When Equation 9-15 is substituted into Equation 9-17 the result is

$$k = \frac{1.381 T_{0\,pave} + 45.10}{1.381 T_{pave} + 45.10} \quad . \qquad 9\text{-}17$$

If T_{0pave} is set as 25C (77F), then Equation 9-17 can be transformed to

$$k = \frac{79.63}{\ln(0.1H + 1) \times (5.581 T_{air} - 6.201) / H + 2.199 T_{air} + 17.48} \qquad 9\text{-}18$$

or,

$$k = \frac{79.63}{\ln(0.1H + 1) \times (2.807 T_{surf} + 51.12) / H + 1.106 T_{surf} + 40.05} \qquad 9\text{-}19$$

Equations 9-18 and 9-19 can be used to estimate the FWD deflection correction factor for HMA pavements when surface temperatures (or air temperatures) and pavement thickness are given. Figure 9-19 shows how the correction factor varies with air temperature when pavement depth is between 50-500 mm (2-20 in), and Figure 9-20 shows how the correction factor varies with pavement surface temperature when pavement depth is between 50-500 mm (2-20 in).

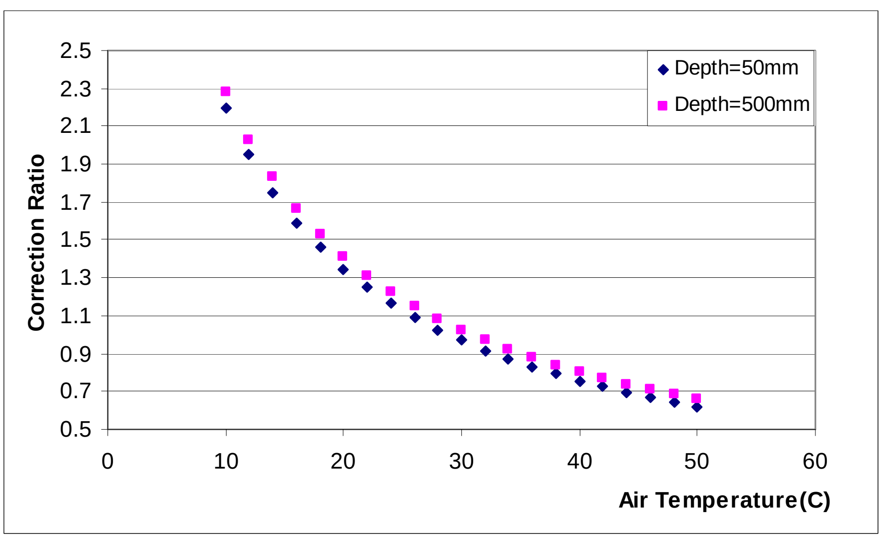

Figure 9-19 Correction Factor Relative to Air Temperature

Figures 9-21 and 9-22 show the FWD deflections before and after correction for test sites 9A and 5B, respectively.

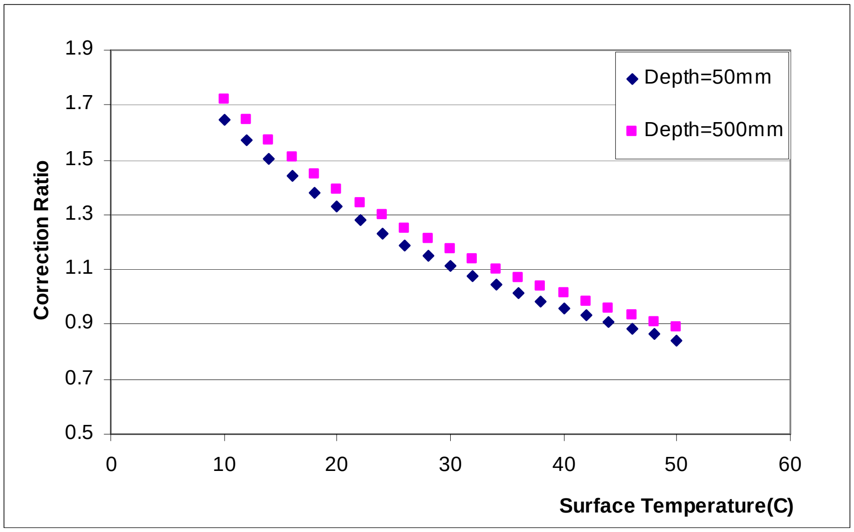

Figure 9-20 Correction Factor Relative to Pavement Surface Temperature

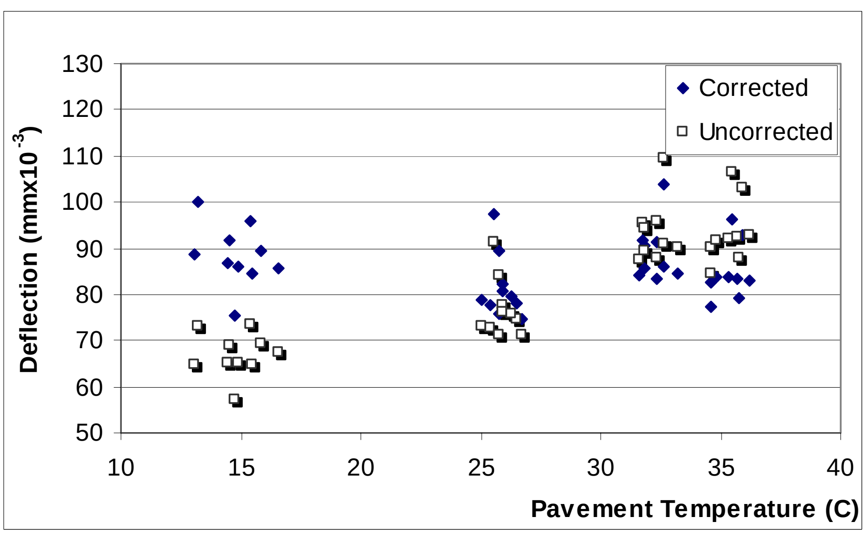

Figure 9-21 Deflection Correction for Test Site 9A

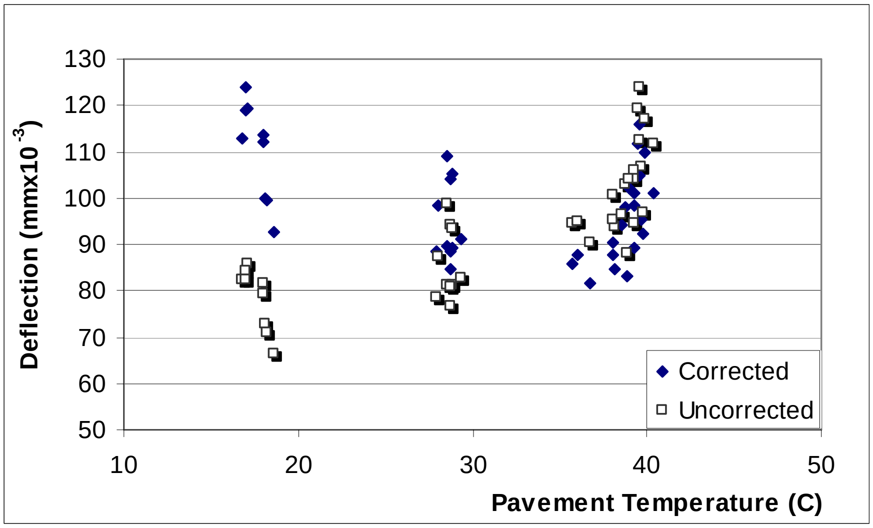

Figure 9-22 Temperature Correction for Test Site 5B

Temperature Correction for Deflection with Temperature Model II

FWD deflections at test site 9A are presented in Figure 9-23 using the average pavement temperature estimated with the HMA pavement temperature model II.

It was assumed that a linear relationship exists between deflection, μ_t, and pavement temperature, T_{pave}. Tables 9-11 and 9-12 show the ANOVA table and parameter estimates, respectively.

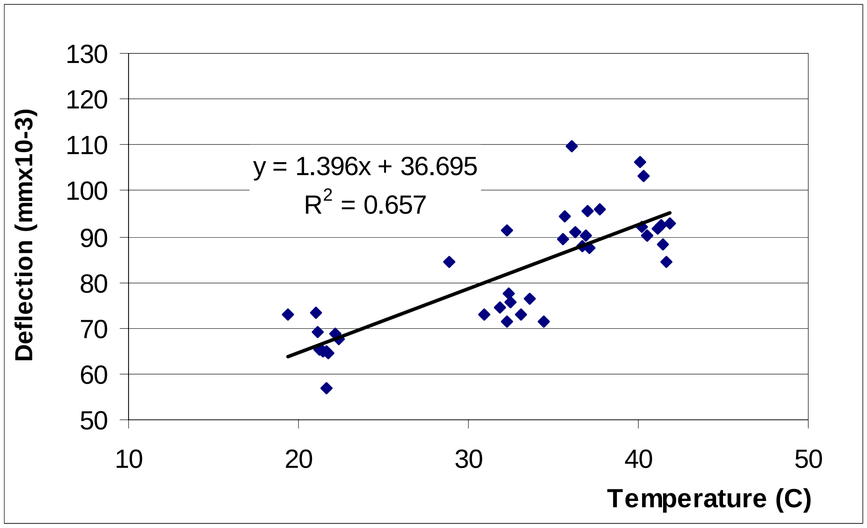

Figure 9-23 FWD Variation with Pavement Temperature at Test Site 9A

Table 9-11 ANOVA Table for FWD Deflection Regression

Source	DF	SS	MS	F	Prob >F	R2
Model	1	4148.282	4148.282	70.75295	4.1E-10	0.657
Error	37	2169.329	58.63052			
Total	38	6317.611				

Table 9-12 Deflection Regression Paramater Estimation

Variable	Estimate	Std Err	T stat	P-Value
Intercept	36.69535	5.557427	6.602939	9.64E-08
T_{pave}	1.395555	0.165911	8.411477	4.1E-10

The regression model for deflection and temperature is

$$\mu_t = 1.396\, T_{pave} + 36.70\ . \qquad 9\text{-}20$$

As in model I, the correction factor can be transformed to

$$k = \frac{1.396\, T^0_{pave} + 36.70}{1.396\, T_{pave} + 36.70} \qquad 9\text{-}21$$

or,

$$k = 1 - \frac{1.396 T_{surf} - 34.90}{1.396 T_{surf} + 36.70 + (-0.2409H - 0.01508H^2 + 0.000342H^3)\sin(-6.253t + 5.0967)} \qquad 9\text{-}22$$

Equation 9-22 can be used to estimate the FWD deflection correction factor for HMA pavement when pavement surface temperature (or air temperature), pavement thickness and time of day are known. Figure 9-24 shows how the correction factor varies with pavement surface temperature at various times of the day for a 300 mm (12 in) pavement depth. Time of day t does not significantly affect the correction ratio. Therefore, the regression model shown in Figure 9-25 ignores the variable t.

The correction factor is approximated using a second degree polynomial as

$$k = aT_{surf}^{\ 2} - bT_{surf} + c \qquad 9\text{-}23$$

where,

a, b, and c = constants.

The estimated values for the constants are shown in Table 9-13 for different pavement depths.

Table 9-13 Correction Factor Regression Parameter Estimation

Parameter		a	b	c	R^2
Pavement Depth	100 mm	0.0003	-0.0354	1.7114	0.999
	300 mm	0.0003	-0.0402	1.7994	0.9973
	500 mm	0.0003	-0.0373	1.7464	0.9985

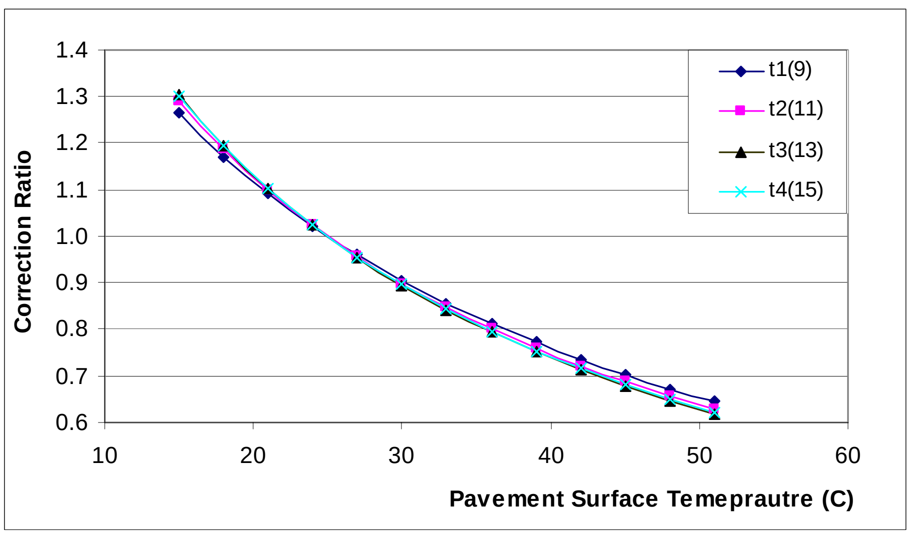

Figure 9-24 Correction Factor Variation with Pavement Surface Temperature

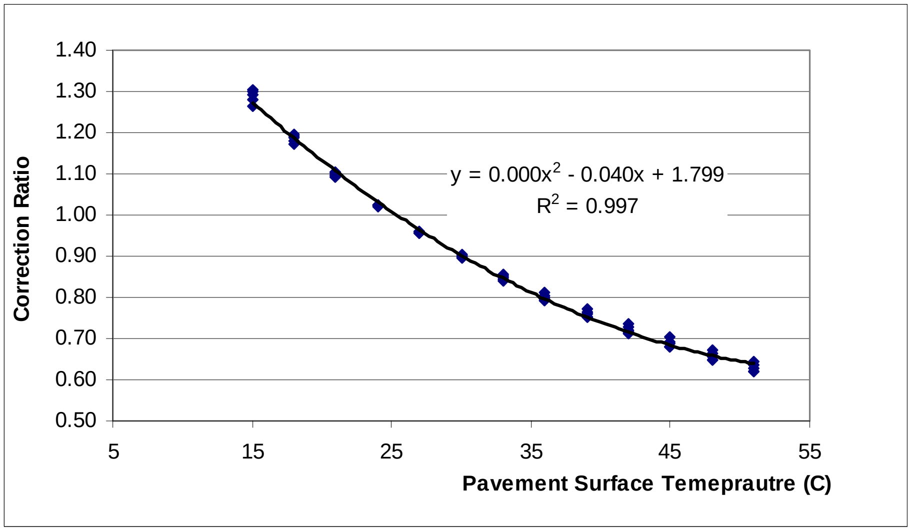

Figure 9-25 Correction Factor and Pavement Surface Temperature

Figures 9-21 and 9-22 show the FWD deflection before and after the correction for test sites 9A and 5B, respectively.

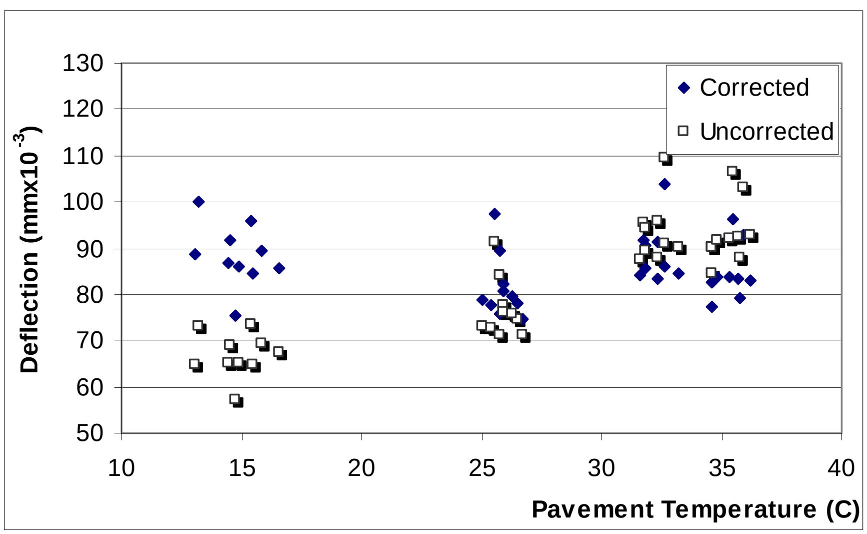

Figure 9-26 Deflection Variation before and after Correction at Test Site 9A

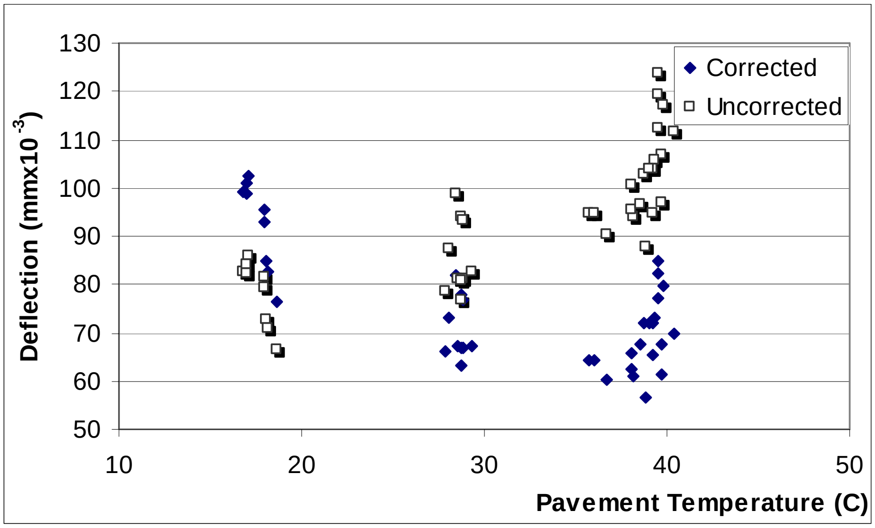

Figure 9-27 Deflection Variation before and after Correction at Test Site 5B

Temperature Correction for Load Transfer

The load transfer variations with pavement temperature for test sites 9A and 5B are presented in Figures 9-28 and 9-29 respectively. The temperature was estimated with HMA pavement temperature model I. In this study, the temperature correction for load transfer is ignored. This is because the R^2 values for the regression between load transfer and pavement temperature are very low for the two test sites, only 0.222 and 0.279 respectively. Pavement temperature cannot completely explain the load transfer variation and no strong relation between load transfer and pavement temperature was found. Furthermore, although these figures show that load transfer is reduced for higher pavement temperatures, the absolute load transfer change is very small, less than 0.1, for the pavement temperature range of 20 degrees difference.

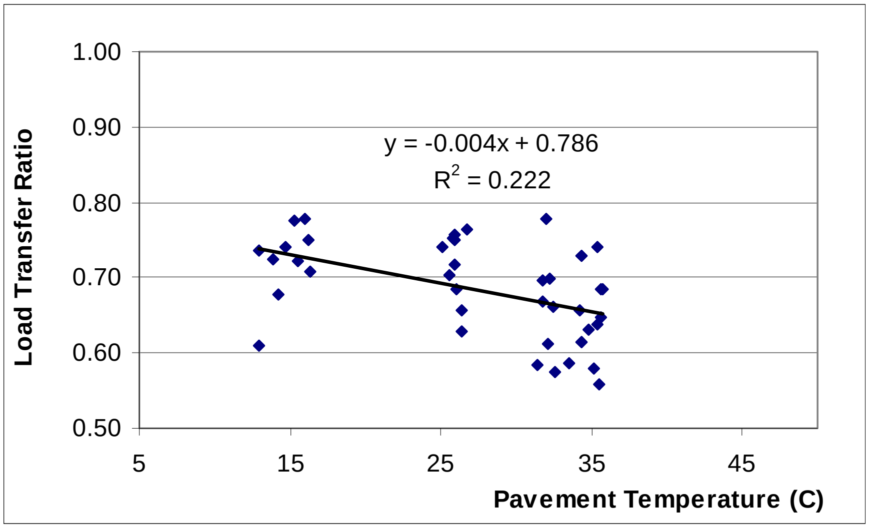

Figure 9-28 Load Transfer Variation at Test Site 9A

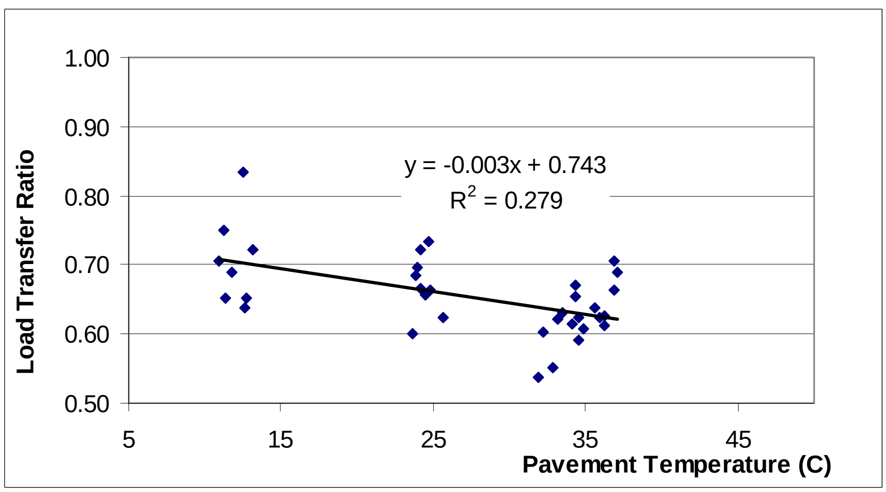

Figure 9-29 Load Transfer Variation at Test Site 5B

9.5 Conclusions

Pavement temperature may affect the FWD deflection, due to the temperature gradient in a PCC slab, and to the viscoelastic property of HMA. In this study, statistical analyses were conducted to evaluate the temperature effects. FWD deflection data were collected at different temperatures on five test sites in order to evaluate the temperature effect. The following conclusions are made based on the statistical results.

1. For PCC pavements, the pavement temperature does not completely explain the FWD deflection. No strong relationship between temperature and FWD deflection was found. For this reason, no temperature correction of FWD deflection data was made for PCC pavement.
2. It is widely believed that load transfer at PCC pavement joints should increase when pavement temperature increases. However, the FWD data appears to show an opposite trend. More field data should be collected to verify this contradiction.
3. The correction factor can properly correct the FWD deflection at the mid-panel cracks within a particular temperature range for HMA pavement. However, no correction factor was used for load transfer across HMA pavement cracks. The correction factor derived by the pavement temperature model I is used to corrected the FWD deflection in this study.

Given these conclusions, two recommendations can be made:

1. No pavement temperature gradient research is available in Indiana. This data should be collected to provide prediction models for pavement temperature in Indiana.

2. All conclusions are based on statistical analyses of a limited data set from five test sites. Additional data collection at various temperatures is suggested in order to develop more reliable temperature correction models for FWD measurements.

CHAPTER 10 SUMMARY AND CONCLUSIONS

The literature review considered over one hundred potential references and revealed only eighteen that specifically discussed the cost-effectiveness of joint and crack sealing. Of these, only four provided useful quantitative information related to the cost-effectiveness of joint and crack sealing. This effort revealed little quantitative evidence to prove the cost-effectiveness of joint and crack sealing and suggested the need for additional research. The literature review also showed that only two studies relative to joint and crack sealing have been conducted in Indiana. However, neither of these considered the overall pavement performance as influenced by sealing and the cost effectiveness of joint and crack sealing in Indiana.

A survey of practice, having eleven questions, was conducted. Forty-two of the fifty state highway agencies responded. The survey revealed that like most other agencies, the INDOT joint and crack sealing policy is based on long standing policy rather than research. The statistical results of the survey also showed that most states, including Indiana, do not have quantitative justification for sealing policies nor do they know the cost-effectiveness of the operations. Thus, a well designed field experiment was strongly recommended to investigate the cost-effectiveness of joint and crack sealing relative to pavement performance in Indiana.

An experimental design for a field study was developed through a series of meetings with pavement technologists and a statistician. Three main factors, roadway classification (national and state routes), pavement type (PCC, HMA, and composite), and drainage (drained and undrained), were included in the design as they were expected to have the greatest influence on pavement performance relative to joint and crack sealing effectiveness. The objective of the experiment was to provide adequate evidence to determine if joint and crack sealing is cost effective and under what conditions. For each cell in the design, two test sites, each with two test sections (one sealed and one unsealed), were investigated. The sealed and unsealed sections were rigorously maintained throughout the duration of the performance monitoring period.

Nineteen test sites were selected under the principle that the sites must conform as closely as possible to the proposed experimental design. All joints and cracks in sealed sections were sealed using typical quality materials and practice; all transverse joints and cracks were kept open in unsealed sections. FWD measurements on both the sealed and the unsealed sections were used to measure load transfer at the joints and cracks and deflections at the mid-panel locations. Roughness values were also measured on both the sealed and the unsealed sections. Visual condition surveys (distress surveys) were conducted to assess the severity and extent of individual distresses such as faulting and cracking. Core samples were collected near joints and cracks to investigate both physical and mechanical properties of the pavement.

A statistical model was developed to compare the pavement performance between sealed and unsealed sections for three pavement types, PCC, HMA and composite. The FWD deflections, load transfer, and IRI statistical analyses were conducted and analyzed in order to detect changes between the sealed and unsealed test sections. Based on the analyses of these measurements, the following conclusions can be made:

1. The IRI data indicates that the roughness for both sealed and unsealed sections was the same at the beginning of the research. However, no conclusions can be made concerning sealed or unsealed performance since only one year of data was available. Additional IRI data should be collected and analyzed to see if the roughness between the sealed and unsealed sections changes over a longer period of time.
2. The FWD measurements indicate that there is no significant difference between the performance of sealed and unsealed sections regardless of pavement type, drainage condition, and road classification. However, only two years of data has been collected to date. Additional data gathered in future years could indicate differences.
3. No cost-effectiveness analysis for joint and crack sealing can be conducted with the limited pavement performance data and statistical analysis results available to date.

Since pavement temperature may significantly affect the FWD deflection for both PCC and HMA pavement, the effect of temperature was evaluated by statistical analyses based on a sample of FWD deflections collected at different temperatures at five test sites. It was concluded that no temperature corrections are needed for FWD deflections and load transfer measurements for PCC pavements. For HMA pavements, a correction factor can properly correct the FWD deflection over a particular temperature range. No correction was considered for load transfer across HMA pavement cracks.

Finally, it is recommended that this study be extended, and the monitoring of the pavement test sites be continued so that the long-term performance can be measured and further conclusions can be drawn regarding the cost-effectiveness of joint and crack sealing.

LIST OF REFERENCES

"ABAQUS/Standard User's Manual", Version 6.4, Hibbitt, Karlsson & Sorensen, Inc. 2001.

Armaghani, J. M., Larsen, T. J. and Smith, L. L., "Temperature Response of Concrete Pavements", in Transportation Research Board 1121, TRB, National Research Council, Washington D. C., 1987.

Ali, H. A. and Lopez, A., "Statistical Analyses of Temperature and Moisture Effects on Pavement Structural Properties Based on Seasonal Monitoring Data", in Transportation Research Record 1540, TRB, National Research Council, Washington D. C.,1996

Blais, E.J., "Value Engineering Study of Crack and Joint Sealing", FHWA-TS-84-221, Federal Highway Administration, December, 1984.

Chen, Hung-Ming, Dere, Yunus, Sotelino, Elisa and Archer, Graham, "Mid-Panel Cracking of Portland Cement Concrete Pavements in Indiana", FHWA,/IN/JTRP-2001/14, SPR-2493, 2001.

Cho, Y., Mccullough, B. F., and Weissmann, J., "Consideration on Finite-Element Method Application in Pavement Method Application in Pavement Structural Analysis", in Transportation Research Board 1539, TRB, National Research Council, Washington D. C., 1996.

Chong, G. J., "Rout and Seal Cracks in Flexible Pavement-A Cost-effective Preventive Maintenance Procedure", Ontario Ministry of Transportation, December, 1989.

Chong, G.J. and Phang, W.A, "Improved Preventive Maintenance: Sealing Cracks in Flexible Pavements in Cold Regions", Transportation Research Record 1205, TRB, National Research Council, Washington, D.C., 1988.

Chou, Yu T., "Estiamting Load Transfer From Measured Joint Efficiency in Concrete Pavement", in Transportation Research Board 1482, TRB, National Research Council, Washington D. C, 1995.

Choubane, B., Tai, M., "Analysis and Verification of Thermal Gradient Effects on Concrete Pavement", ASCE, Journal of Transportation Engineering, 1994.

Dunn, K.H., “The Effect of Sealed vs. Unsealed Joints on the Performance of Jointed PCC Pavement – A Synthesis”, Wisconsin Department of Transportation, June, 1987.

Eacher, M. J., and Bennett, A. R., “Bituminous Crack Filling Test Section on US-10 Near Evart”, Michigan Department of Transportation, April, 1998.

Fang, Hongbin, “Rational Approach to Rutting Rehabilitation Decision”, Ph.D. Thesis, Purdue University, West Lafayette, IN, 2001.

Federal Highway Administration, “Techniques for Pavement Rehabilitation - Reference Manual, 6th Edition”, FHWA HI-98-033, Washington, D.C., August, 1998.

Foinquinos, R., Roesset, J.M., and Stokoe, K. H., “Response of Pavement Systems to Dynamic Loads Imposed by Nondestructive Test”, in Transportation Research Record 1504, TRB, National Research Council, Washington D. C.,1997

Freeman, R. B., and Johnson, D., “Cost Effectiveness of Crack Sealing Materials and Techniques for Asphalt Pavements”, Montana State University, February, 1999.

Hand, A. J. Galal, K. A., Ward, D. R., and Fang, C., “Cost-Effectiveness of Joint and Crack Sealing, Synthesis of Practice”, ASCE, Journal of Transportation Engineering, 2000.

Hua, Jianfeng, “Dynamic Analysis of Rigid Pavement Response Under Standard Nondestructive Testing Equipment Using a Three Dimensional Finite Element Model By Abaqus”, Master Thesis, Purdue University, West Lafayette, IN, 1998.

Hua, Jianfeng, “Finite Element Modeling and Analysis of Accelerated Pavement Testing Devices and Rutting Phenomenon”, Ph.D. Thesis, Purdue University, West Lafayette, IN, 2000.

Huang, Y. H., “Pavement Analysis and Design”, Prentice Hall Inc., New Jersey, 1993.

Hudson, R. W. (1981). “Road Roughness: Its Elements and Measurement”, Transportation Research Record 836, TRB, National Research Council, Washington D.C., 1981.

Kim, Y. R., Lee, Y. C., Park, S., and Ranjithan, S. R., “Interpretation of FWD Data When Pavement Layers Are Not Intact”, Final Report Submitted to the North Carolina Department of Transportation, Department of Civil Engineering, North Carolina State University, 1999.

Maina, J. W. and Yokota, H., “Dynamic Backcalculation and Overlay Design Based on FWD Data”, in “Nondestructive Testing of Pavements and Backcalculation of Moduli”, Tayabji, S. D., and Lukanen, E. O. , third Volume, ASTM, 2000.

Mallela, J., George, K. P., “Three-Dimensional Dynamic Response Model for Rigid Pavements”, Transportation Research Record 1448, TRB, National Research Council, Washington, D.C., 1995.

Masd, E., Taha, R. and Muhunthan, B., “Finite-Element Analysis of Temperature Effects on Plain-Jointed Concrete Pavements”, Journal of Transportation Engineering, September/October, 1996.

Montgomery, D. C., “Design and Analysis of Experiments”, fourth edition, John Wiley & Sons, New York, 1996.

Mohamed, A. R. and Hansen, W., “Effect of Nonlinear Temperature Gradient on Curling Stress in Concrete Pavements”, Transportation Research Record 1568, TRB, National Research Council, Washington, D.C., 1997.

Morian, D. A. and Stoffels, S., “Joint Seal Practices in the United States Observation and Consideration”, Transportation Research Record 1627, TRB, National Research Council, Washington, D.C., 1998.

Morian, D. A., Epps, J. A., and Gibson, S.D., “Pavement Treatment Effectiveness, 1995 SPS-3 and SPS-4 Site Evaluation”, National Report, Nichols Consulting Engineers, May, 1997.

Morian, D. A., Gibson, S. D. and Epps, J.A., “Concrete Pavement Maintenance Treatment Performance Review: SPS-4 5 Year Data Analysis”, Federal Highway Administration, FHWA-RD-97-155, Washington, D.C., July, 1998.

Neter, John, Kutner, M. H., Nachtsheim, C. J. and Wasserman W., “Applied Linear Statistical Models”, Fourth Edition. Irwin, 1990.

Nouredlin, A. S., Leahy, R. B., and Lundy, J. R., “Development of Temperature Coefficients for the AASHTO Flexible Pavement Design Equation”, Association of Asphalt Paving Technologists Journal, Volume 66, March 1996.

Noureldin, A. S. “Temperature Gradient in a Full Depth Asphalt and its Effect on Modulus and Shear Gradients”, 6th conference on Asphalt Pavement for Southern Africa.

Park, Dong-Yeob, Buch, Neeraj and Chatti, KArim, “Effective Layer Temperature Prediction Model and Temperature Correction via Falling Weight Deflectometer Deflections”, in Transportation Research Board 1764, TRB, National Research Council, Washington D. C.,2001.

Ponniah, J. E. and Kennepohl, F. J., “Crack Sealing in Flexible Pavements: A Life-Cycle Cost Analysis”, Transportation Research Record 1129, TRB, National Research Council, Washington, D. C., 1996.

Queiroz, C.A.V. and Hudson, W.R., “A Stable, Consistent, and Transferable Roughness Scale for Worldwide Standardization,” Transportation Research Record 997, TRB, National Research Council, Washington, D. C., 1984

Richardson, J. M. and Armaghani, J. M., “Stress Caused by Temperature Gradient in Portland Cement, Concrete, Pavements”. Transportation Research Record 1121, TRB, National Research Council, Washington, D. C., 1991.

Roberts, F. L., Kandhal, P. S., and Brown, E. R., “Hot Mix Asphalt Materials, Mixture Design, and Construction”, NAPA Research and Education Foundation, 1991.

Rutkowski, T. S., “Evaluation of Asphaltic Concrete Crack Sealing”, WI-08-96, Wisconsin Department of Transportation, January, 1998.

Rutkowski, T.S., “Joint Sealant Study, Interim Report. Report WI-02-90”, Wisconsin Department of Transportation, May 1990.

Rutkowski, T.S., S.F. Shober, and R.B. Schmeidlin, “Performance Evaluation of Drained Pavement Structures”, Wisconsin Department of Transportation, December, 1998.

Sayers, M.W. and Karamihas, S.M., “The Little Book of Profiling” Transportation Research Institute, University of Michigan, 1998.

Seraaly, B. E., Mamlouk, M. S., and Davies, T. G., “Dynamic Analysis of Falling Deflectometer Data”, in Transportation Research Board 1070, TRB, National Research Council, Washington D. C., 1986.

Shahin, M.Y., “Pavement Management for Airport, Roads, and Parking Lots”, Chapman and Hall, New York, 1994

Shalaby A., A. Hallm, O. and Easa, S. M., “Low-Temperature Stresses and Fracture Analysis of Asphalt Overlays”, in Transportation Research Board 1539, TRB, National Research Council, Washington D. C., 1996.

Sharaf, E. A., and Sinha, K. C., “Energy Saving from Increased Preventive Maintenance on Indiana highways”, Transportation Research Record 1102, TRB, National Research Council, Washington, D.C., 1986.

Shober, S. F., “The Great Unsealing—A Perspective on PCC Joint Sealing”, Wisconsin Department of Transportation, April, 1997.

Strategic Highway Research Program, “Distress Identification Manual for the Long-Term Pavement Performance Project”, Washington, DC 1993.

Thompson, M. R., Dempsey, B. J., Hill, H. and Vogel, J., "Charactering Temperature Effects for Pavement Analysis and Design", Transportation Research Record 1121, TRB, National Research Council, Washington D. C, 1987.

Uddin, Waheed, Hackett, R. M. , Joseph, A. , Pan, Zhou and Crawley, A. B., "Three-Dimensional Finite-Element Analysis of Jointed Concrete Pavement with Discontinuities", Transportation Research Record 1482, TRB, National Research Council, Washington D. C, 1994.

Uddin, Waheed, Zhang, Dingming, and Fernandez, Francisco, "Finite Element Simulation of Pavement Discontinuities and Dynamic Load Response", Transportation Research Record 1448, TRB, National Research Council, Washington D. C, 1994.

Uddin, W., Hackett, R. M., Noppakunwijai, P., and Pan,Z., "Three-dimensional Finite-Element Simulation of FWD Loading on Pavement Systems". Transportation Research Record 1482, TRB, National Research Council, Washington D. C, 1995.

Ward, D. R., "Evaluation of Crack Sealing Performance on Indiana's Asphalt Concrete Surfaced Pavements", Indiana Department of Transportation, October, 1993.

William, G. W. and Shoukry, S. N. "3D Finite Element Analysis of Temperature-Induced Stresses in Dowel Jointed Concrete Pavements", The International Journal of Geomechanics, Volume 1, Number 3, 291-207, 2001.

Witczak, M. W., "Design of Full Depth Asphalt Airfield Pavement", Third International Conference on the Structural Design of Asphalt Pavements, London, England, 1972.

Yoder, E. J., and Hampton, D. "Pavement Profile and Roughness Measurements: A Review of Method." Purdue University, West Lafayette, Indiana,1958.

Yoder, E. J., and Witczak, M. W. "Principles of Pavement Design". John Wiley &Sons, Inc, New York, 1975.

Zaghloul, S. M., and White, T. D., "Dynamic Analysis of FWD Loading and Pavement Response Using a Three-Dimensional Dynamic Finite Element Program", Nondestructive Testing of Pavements and Back-calculation of Moduli (Second Volume), ASTM STP 1198, American Society for Testing and Materials, Philadelphia, 1994.

Zaghloul, S. M., White, T. D., "Use of a Three-Dimensional, Dynamic Finite Element Program for Analusis of Flexible Pavment", Transportation Research Record 1388, TRB, National Research Council, Washington D. C, 1994.

Zaghloul, S. M., White, T. D., Drnevich, V. P., and Coree, B., "Dynamic Analysis of FWD Loading and Pavement Response Using a Three-Dimensional", in "Nondestructive Testing of Pavements and Backcalculation of Moduli", H. V. Quintas, A. J. Bush, and G. Y. Baladi , Second Volume, ASTM, 1994.

Appendix A Joint and Crack Sealing Questionnaire

1. Do you seal new concrete pavements?

   Yes No

2. How wide is your saw cut for joints on new concrete pavements?

3. Do you reseal older concrete pavements?

   Yes No

   If yes, a. When do you perform the first resealing (as needed)?

   b. How often do you reseal? (For Example, every 5 years)

4. Do you reseal bituminous pavements?

   Yes No

   If yes, a. When do you perform the first resealing (as needed)?

   b. How often do you reseal? (For Example, every 5 years)

5. How was the decision made to conduct joint or crack sealing?
   a. long standing policy
   b. research results
   c. unsure

6. Do you install subsurface drains on new pavements?

   Yes No

7. Has your DOT studied the effect of joint and crack sealing with regard to the impact it has on the performance of your concrete, asphalt or composite pavement? If yes, please give the title of the project, name of the principal investigators and how we can get a copy of this research?

   a. Title of the project:

b. Principal investigators:
c. Availability of the report:

8. Does your DOT plan on investigating the cost of joint/crack sealing in the near future?

   Yes No

   If so, how?

   a. in house research (name)
   b. consultant (name)
   c. university research (name)

9. How do you define traffic level in terms of ESALS and/or truck count/truck factor?

10. How do you define thick vs. thin pavement (concrete, flexible or composite)? For example: concrete pavement less than 6” thin, greater than 6” thick…etc.

11. Do you have FWD criteria that define performing joints or cracks? If so, please state the criteria? For example: ratio between any sensors or difference between any sensors.

Appendix B Survey Results of Questionnaire on Joint and Crack Sealing

Table B-1 Survey Responses

State	1. Do you seal new concrete pavements?	2. How wide is your saw cut for joints on new concrete pavements?	3. Do you reseal older concrete pavements? If yes,a. When do you perform the first resealing (as needed)? b. How often do you reseal? (For Example, every 5 years)	4. Do you reseal bituminous pavements?	5. How was the decision made to conduct joint or crack sealing? a. long standing policy b. research results c. Unsure	6. Do you install subsurface drains on new pavements?
Alabama	Yes	3/8" (9.53mm)	Yes a. No uniform criteria. Resealing is performed when deemed necessary by the division maintenance Engineer or a District Engineer b. No set time interval.	Yes a. same as question 3a b. Same as question 3b.	c.	Yes
Alaska	No, we don't use concrete pavement	N/A	No a. N/A b. N/A	Yes, sometimes a. Chip seal + 8 yr old pavements without ruting or IRI problems b. No criteria set, depends on condition.	b.	Yes A Few
Arkansas	Yes	Standard drawing attached	Yes a. When the joint sealer begins to lose adhesion to joint surfaces b. 5-8 years.	Yes a. When enough cracks or joints open to justify b. 3-5 years.	a.	Some
Arizona	Yes	nominal 1/8"(3.18mm) (width of saw Blade)	Yes a. Generally after 10 years of service b. Approximate 10 year cycle	Yes a. 7-10 years cycle b. As needed-generally 3 year cycle	a.	No
California						
Colorado	Yes	Single cut, 1/8" (3.18mm) wide.	Yes a. As needed	Yes a. as needed b. Unsure	a.	No. However, through LTPP we have installed edge drains with permeable asphalt treated base (PATB).
Connecticut						
Delaware	Yes	3/8"(9.53mm) Joint - Neoprene seals	Yes a. When Moderate to severe failure of Joint material is observed b. As needed	Yes a. When moderate failure of crack sealant is observed b. ~5 years as need	a. -also, pavement managemnet road raters note sealant condition during annual road rating.	Yes

Table B-1 Survey Responses (Cont.)

Florida	Yes	3/8"(9.53mm)	Yes a. When minor CPR needed (3% slab replacement) b. ~10 years, when CPR needed	No	b.	Yes - Rigid only
Georgia	Yes	3/8" (9.53mm) transverse joint; 1/4"(6.35mm) longitudinal joint	Yes a. Based on annual inspections of PCC pavement condition. No written criteria b. 7 to 10 years	occasional crack sealing	a.	Yes - only on an as-needed basis
Hawaii	No We use permeable bases and draw our pavements.	N/A	No Our older pavements were never sealed.	No	N/A	Yes, with permeable bases.
Idaho						
Illinois	Yes	3/8"(9.53mm)	Yes a. We reseal concrete pavements when deemed necessary. We do not have any set policy on resealing concrete pavements. Resealing is decided based on the appearance of the joint and/or the surrounding pavement b. When necessary.	Illinois Department of Transportation does not seal bituminous pavements initially. Bituminous pavements are sealed after cracks appear a. The first sealing of bituminous pavements is done about three to five years after construction b. Usually bituminous pavements are overlaid or replaced before cracks are sealed a second time.	a.	Yes
Indiana	Yes	initial 1/8"(3.18mm)	No	No	a.	Yes
Iowa	Yes	1/4"(6.35mm)	Yes.(Infrequently) a. age b. Should be every 7 years but is not that frequent. (Once the maintenance management system is in place we should be able to better determine when these activities are being done.)	Yes a. age b. should be every 10 years. (Once the maintenance management system is in place we should be able to better determine when these activities are being done.)	a.	Yes

Table B-1 Survey Responses (Cont.)

Kansas	Yes	3/8"(9.53mm) (15' Plain PCCP w Dowels)	Yes a. as needed b. Usually once in life of a pavement	Yes a. When cracks (unsealed) reach 1/4" or wider and do not exhibit roughness (noticable). b. As needed	b. (SHPP SPS-3 results)	Yes
Kentucky						
Louisiana	Yes	3/8"(9.53mm)	No	Yes as determined by the maintenance engineer.	This is also determined by the maintenance egineer, if the cracking is extensive and it is associated with raveling and pitting (about 20% of the area), a seal coat may be applied.	Yes, for interstate highways
Maine	N/A Do not construct PCC Pavement	N/A	N/A	No Not much if any resealing has been done to my knowledge a. We do have a fairly aggregate under sealing program along interstate. Also Bureau of Maintenance & operations has crack sealing program.	c. Decision to crack seal is made based on concensus approach with planning, project development and maintenance team members.	Yes, install under drain systems where needed. Do not use edgedrain systems.
Maryland	Yes	1/8"(3.18mm) - contraction joints	Yes a. When seals are demaged and to be replaced b. Varies - not a routine preventive measure	Yes a. No criteria is established although we are developing guidelines this summer. Typically seal tight environment or joint reflection cracks b. Varies - not routine	currently based on local expertise - will become part of our pavement management decision process.	Yes - not all pavements - those that require outlets for drainage.
Massachusetts						

Table B-1 Survey Responses (Cont.)

Michigan	Yes	3mm relief cut 10mm final width for neoprene (compression) seal	Yes a. MDOT will look at resealing concrete joints at approximate 12-15 year of pavement life b. A second cycle of resealing may or may not occur depending on the deterioration of the pavement at the time of consideration	Yes, both flexible and rigid Flexible - MDOT will crack seal at year 5 to year 10 depending on the pavement condition. A second cycle of resealing may or may not occur depending on the rate of pavement deterioration of the pavement at the time of consideration Composite - MDOT will crack seal at year 2 to year 3 depending on the pavement condition. A second cycle of resealing may or may not occur depending on the rate of pavement deterioration of the pavement at the time of consideration	Recent policy directive, research and informal field observations of the benefits obtained from joint and crack sealing.	Yes, 100mm and 140mm circular
Minnesota	Yes	3/8"(9.53mm)	Yes a. When sealant fails b. Varies with life of sealant (3yrs-25yrs)	Yes a. 1) Reseal joints when sealant fails. 2) Seal cracks when new transverse cracks develop b. Reseal varies, we typically seal new transverse cracks two years after paving.	b.	Yes On high volume and highways with non granular subgrade.
Mississippi	Yes	1/2"(12.7mm)	Yes a. No timetable; when needed b. N/A	No	a.	Yes
Missouri	Yes	3/8"(9.53mm)	No	Yes a. Seal cracks as they occur on an annual basis b. Seal cracks as they occur on an annual basis	a.	Yes Heavy Duty PVMT - Longitudinal edge drains on both sides of dual PVMT. Medium Duty PVMT - longitudinal edge drains on outside of dual PVMT. Light Duty PVMT - No longitudinal edge drains.

Table B-1 Survey Responses (Cont.)

Montana	Yes	1/4"(6.35mm)	Yes a. Hot Pour = 5 to 10 yrs b. Silicone = longer 8 to 14 yrs	Yes a. 2 to 3 years b. 2 to 3 years	b.	No
Nebraska	Yes	3/16"(4.76mm)	Yes a. District engineer's judgement b. A 5-year cycle has been proposed for the interstate	Yes a. District Engineer's judgement b. A 3-year cycle is being used for the Interstate	Engineering judgement.	Yes
Nevada	Yes	3/8"(9.53mm)for transverse joints, 1/4"(6.35mm)for logitudinal joints.	Yes	Yes	a.	Yes,under PCCP only
New Hampshire	NH has not placed a concrete pavement in 50 years		No	Yes b. 5-8 years	b. & c.	Yes
New Jersey	Yes	Formed Expansion Joint - 3/4" (19.05mm)	Yes a. As needed b. 5-6 years; Depends on funding.	Yes a. As needed b. 5-6 years; Depends on funding.	c.	1. Pavement Drainage system for concrete pavement 2. Subsurface, cross-drains every 250' +/- for bituminous pavement
New Mexico	Yes	20mm	Yes a. Our District Maintenance Engineers decide when this is necessary based on in-field reviews. b. Same as 3(a) - no set average interval.	see answer to question 3	see answer to question 3	No
New York	Yes	First Stage: 3-6mm Second Stage: 10mm+/-1 mm Bevel (Transverse Joints only): 3mm x 3mm	Yes a. Between years 8-12. After sealer sidewall adhesion starts to fall as determined by field Inspection b. 8-12 years	See attached	a.	Yes

Table B-1 Survey Responses (Cont.)

North Carolina	Yes	Currently we use two saw cuts. (see figure) But are considering using a single 1/8" (3.18mm) cut.	Yes. a & b. this varies greatly depending on the area of the state.	Yes. a & b. this varies greatly depending on the area of the state.	No response---	Yes
North Dakota	Yes	For joints we use both silcone and preformed. Saw cuts for silicone is 3/8" (9.53mm) wide and for preformed is 1/4" (6.35mm) wide.	Yes a. When doing a concrete pavement repair or dowel bar retrofit project on a section of highway the joints will be resealed.	Yes a. After the appearance of the initial crock pattern b. Varies, normally be the appearance of the pavement.	c.	Yes, only in concrete pavements
Ohio						
Oklahoma	Yes	1/4"(6.35mm)	Yes a. Usually as part of a AC rehab. Project b. 10-15 years	Yes a. severe cracking is usually seal by state Maintenance forces b. 5 to 20 years, as needed.	a.	Yes - only on very high type facilities (Interstate Hwys.)
Oregon	Yes	3-6mm	Yes, only on rare ocassions	Yes, a. when crack become a problem.	c.	Yes, most new pavements.
Pennsylvania	Yes Neoprene Transverse Joints Seals For Interstate Highways	1/8" (3.18mm) initial reservoir 1/2"(12.7mm)~5/8"(15.9mm)	Yes Joints and Cracks a. As needed b. As needed	Yes a. 3-5 years b. usually every 5 years.	a.	Yes
Rhode Island	N/A	N/A	N/A	Yes a. Please see additional information b. see additional information	Experience and judgement	occasionally
South Carolina	Yes	3/8" (9.53mm)	Yes a. Resealing is performed as part of general rehabilitation. These projects are generally driven by other distresses such as faulting or broken slabs rather than by seal condition. The first rehabilitation is generally at 18-30 years after construction, so the seals are generally gone by the time we get to them. (This is not a good practice, but this is what we do.) b. No set period.	No	a.	Yes, for rigid. No, for flexible

Table B-1 Survey Responses (Cont.)

South Dakota	Yes	Contraction Joints - 3/8"(9.53mm)	Yes a. Approximate 15 years b. After the first reseal it is about 10-15 year intervals.	Yes a.Based on inspection - the 1st seal is at 2 years b. As required.	b.- informal research test site results	Yes - very limited in number and location of any new installations we would do.
Tennessee						
Texas	Yes	Contraction Joints - 3/8"(9.53mm) Logitudinal Joints - 1/4"(6.35mm)	Yes a. Local decision b. No scehedule, judgement	Yes a. Generally 8 to 15 years for seal coats b. Generally 5 to 10 years for seal coats	Local engineering judgement	No
Utah	Yes	1/8" (3.18mm)	Yes a. Scheduled every ten years	Yes a. As needed	a & b	Yes Not used in the past. They are starting to be used now.
Vermont	Yes	1/2"(12.7mm)	Yes a. 5 years b. every 5 years	Yes a. 2-3 years b. every 5 years	c.	As necessary.
Virginia	Yes	See the attached Standard PR-2	Yes a. 8th year b. 10 years	Yes a. 8th year b. 10 years	a.	Yes
Washington	Yes	3/16"(4.76mm) - 5/16"(7.94mm)	Yes a. As part of other rehabilitation - Dowel Bar Retrofit, Diamond Grinding.	No In general, some maintenance areas are sealing cracks as part of maintenance work a. No specific criteria has been established.	Engineering judgement - Minimize Ability of incompressibles & Moisture into Joint - Minimize spalling/faulting/cracking Potential. (new pavements) still investigating resealing of existing Joints/cracks.	Yes Only when part of larger drainage plan - Typically in Urban Areas.
West Virginia						

Table B-1 Survey Responses (Cont.)

Wisconsin	No	1/8"(3.18mm) - 3/16"(4.76mm)	Not generally - If it had a wide saw cut, we may a. Hit or miss	Yes a. Counties do work for us - in first 5 years b. every 5 years +/-	b. -50 years of research of PCC says it is not cost-effective. AC research says it may be cost-effective.	Yes - PCC only
Wyoming	Yes	Transverse- 3/8"(9.53mm) use preformed seals Longitudinal- 1/8"(3.18mm) use hot pour seal	Yes a. usually not until other CPR is being performed, such as grinding b. Propably 15 years	Yes a. Propably 5 to 10 years after construction b. Only when sealant and crack is in poor condition	a.	Yes, On most concrete pavements. On some flexible sections where earth widing will create a bath tub.

Table B-1 Survey Responses (Cont.)

7. Has your DOT studied the effect of joint and crack sealing with regard to the impact it has on the performance of your concrete, asphalt or composite pavement? If yes, please give the title of the project, name of the principal investigators and how we can get a copy of this research?	8. Does your DOT plan on investigating the cost of joint/crack sealing in the near future?If so, how? a. in house research b. consultant c. university research	9. How do you define traffic level in terms of ESALS and/or truck count/truck factor?	10. How do you define thick vs. thin pavement (concrete, flexible or composite)?	11. Do you have FWD criteria that define performing joints or cracks? If so, please state the criteria? For example: ratio between any sensors or difference between any sensors.	Additional information
No	No	Low traffic level < 1,000,000 ESALs over 20 years <= medium traffic Level < 10,000,000 over 20 year ESALs<= High traffic level	Thin PCC <8" Thick PCC >10" Thin HMA <6" Thick HMA >8" Thin AC/PCC <10" Thick AC/PCC >12"	No	
No	No, it costs approximately $0.27/SY	ESALs	Flexible: Thin <= 2" (50.8mm) Thick > 2" (50.8mm)	No	
No	Currently have data to compute cost in house.	ESALs	No criteria	No	A drawing of Transverse & longitudinal Joints for Concrete Pavement is attached.
Ongoing LTPP Studies of Test Sections	Yes a. Larry Scofield. Research ongoing as part of LTPP study.	ESALs	Concrete - thin less than 10"(254mm) Flexible - thin less than 5"(127mm)	No criteria.	
NO	No	ESAL's	For concrete 0-4"(101.6mm) =ultra thin 4-7"(77.8mm) =Thin & anything over 8" (203.2mm) is full depth	Yes, CDOT uses FWD to examine load transfer efficiency (LTE) between slabs & shoulders. FWD is also used as an indicator of load carrying capacity of rigid pavements.(AA). No stated criteria (GL)	(AA)=Ahmad Ardani (GL)=Greg Lowery

Table B-1 Survey Responses (Cont.)

No	No	ESALS	Don't Define thick vs. thin. Classify by PCC, flexible and composite.	Have no FWD equipment. Use a consultant on as-needed basis (very infrequently)	
Yes a. Evaluation of Surface Sealing Techniques b. Jim Musselman, Gale Page c. FDOT Materials Office	No	Total ESALs in design period.	Flexible, thick>4"(101.6mm) Rigid, thick>9"(228.6mm), Ultrathin<=4"(101.6mm)	No	
No formal studies since there are many interacting factors which affect performance	No	Use ESALs in design, AADT and percent trucks in studies which use existing traffic levels - Depends on what tpye of study	Do not use such definitions	None	Questionnaire completed by Wouter Gulden Georgia DOT State Material and Research Engineer 404 363-7512
No	No	ESALs and Truck count	We don't have a difinition. All of our PCC pavements are greater than 6".	No	We don't seal joints because we believe they require high maintenance and our maintenance crew won't maintain them. We believe draining the pavement is the best alternative. We are interested in your study if available.

Table B-1 Survey Responses (Cont.)

Yes a. Repair of longitudinal Cracks in CRPCC Pavement, February 1984 b. John L. Saner, Illinois Department of Transportation, Bureau of Materials and Physical Research c. Please request a copy, if one is desired. Contact: Tessa Volle, IDOT-Bureau of Materials & physical Research 126E. Ash street, Springfeild, Illinois 62704 (217)782-7200	Possibly. Illinois DOT just began a study that is similar to Wisconcin DOT's seal?no seal study. Cost may be one of the topics of study. a. For more information on the study. Please contact Mark Gawedzinski, P.E. IDOT-Bureau of Materials and Physical Research 126 E. Ash St, Springfield, Illinois 62704 (217)782-7200	Illinois Department of Transportation defines traffic levels in terms of ESALs.	IDOT does not label pavements "thick" or "thin". Bituminous pavements are at least 6"(152.4mm) thick. Concrete pavements are at least 6.5"(165.1mm) thick. Design thicknesses greater than the minimums are based on traffic that exceeds the minimum design traffic.	The FWD criteria that defines performing joints or cracks is load transfer efficiency: the ratio between deflection under load and deflection of the sensor on the other side of the joint or crack. IDOT does not have any absolute criteria, but follows the following guidelines: Below 50% Failed 50%-65% Poor 65%-85% Moderate 85%-100% Good	
See Dave Ward INDOT Research---No	a.	ESALs	All thick 12"(304.8mm) or greater	No	
No	Yes a. We plan on trying to use the maintenance management system to get information on frequency and costs.	ESALs and Truck count/axle loadings	PCC thin 8" (203.2mm)and less, thick >10" (254mm) ACC thin 11" (279.4mm)and less, thick >13" (330.2mm)	No	
Yes a. "Final Report of North Central Region Tour of SPS-3 Projects-1995" b. Ron Shuberg, Engr. Of Maint.; KDOT (Retired) c. Report may be available through FHWA; Summary of all regions pulished as "Pavement Treatment Effectiveness, 1995 SPS-3 and SPS-4 Site Evaluations, National Report". Pub. No. FHWA-RD-96-208 May 1997	No - satisfied with SHRP findings	ESALs/ day in design lane	PCCP>=8"(203.2mm) - thick Bit.>=5.5"(139.7mm) - thick Comp>2"(50.8mm) - thick	AASHTO Guide criteria - "Performing" joints have deflection load transfer (^LT)>70% where:^LT=100*(^uI / ^I)*B ^uI=unloaded side deflection (in.) ^I=loaded side deflection (in.) B=slab bending & AC compression correction factor	Tessa Volle's e-mail: VolleTH@nt.dot,state.il.us

Table B-1 Survey Responses (Cont.)

Although, we have done research on the joint material performance, we have not look at the effect of joint sealing on the performance of the pavements.	We are initiating to look at performance of sealed and unsealed narrow joints in concrete pavements. LTRC in-house research.	Modified AASHTO procedures.	For concrete pavements, our concrete overlays are arround 4"(101.6mm) thick, that can be considered thin. For our concrete pavements, the minimum is 10"(254mm) thick. For flexible pavements, the minimum hot mix overlay is about 1.5"(38.1mm), a typical value for our AC overlay is 3.5"(88.9mm), any thickness greater than this will be considered a thick pavement.	We have used the FWD to determine the load carrying efficiency of load transfer devices at concrete pavement joints, please refer to the attached diagram for information.	Diagram attached.
No	No	18K equivalents, ESALs	Don't really have definitions. Generally speaking use 6"(152.4mm) hot mix asphalt on new construction, 3"(76.2mm) HMA over full depth reclaim on highway improvements, and use 1.5(38.1mm) to 3"(76.2mm) HMA overlays.	N/A	
No	No	We calculate ESALs based on traffic counts, truck counts and truck weights.	Only define Flexible: Thick - >4"(101.6mm) overlay Med - 2.5"(63.5mm)-4"(101.6mm) overlay Thin - <2.5"(63.5mm) overlay	Load transfer <70% requires load transfer repair. Load transfer <70% on concrete/composite pavement requires PCC repair instead of Full depth AC repair.	
Yes, enclosed is a study titled "Bituminous Crack Filling - Test Section on US-10 Near Evart" Please contact Mr. Mike Eacker, Pavement Rehabilitation Engieer, 517-322-5673 for information on this report or additional informal research on joint /crack sealing.	The department continually track cost of various sealants. Please contact Mr. Mike Eacker, Pavement Rehabilitation Engineer, 517-322-5673 for information on this report or additional informal research on joint/crack sealing.	Ambiguous	Composite & Flexible - A thin overlay is considered a one course overlay (40 to 50mm). Rigid - thin overlay is considered less than 100mm thick.	Yes, a performing joint or crack is defined by MDOT to have a load transfer efficiency of 70% or greater. If efficiency is lower, we consider a joint/crack retrofit to restore joint/crack integrity.	

Table B-1 Survey Responses (Cont.)

Yes a. 1) Sawing and Sealing Joints in Bituminous Pavements to Control Cracking #96-27 2) Joint and Crack Filler #93-11 3) Evaluation of Materials and Methods for Bituminous Pavement Crack Sealing and Filling #89-19 b. 1) David W. Janisch and Cutis M. Turgeon 2) Mark Hagen 3) Curtis M. Turgeon c. Contact Mn/DOT Office of Materials & Road Research, Lisa Bilotta (651) 779-5500	Yes a. Roger Olson (651)779-5517 Monitor and evaluate pavement performace of test sections and pavements.	No criteria for joint & crack sealing. Mn/DOT uses ESALs for Pavement Design and Rehab, ie Superpave Levels 1-7.	No criteria for joint & Crack sealing Asphalt Pavements: Thin<2"(50.8mm), Medium =2"(50.8mm)-4"(101.6mm), Thick>4"(101.6mm)	No criteria.	
No	No	ESALs for design life.	MDOT makes no such determination.	No	
No	Yes We currently have research underway (in our Northwest District) evaluating the effectiveness of unsealed joints in PCCP. At this time, the study has not been under investigation long enough to draw any conclusions. a. Patricia Lemongelli - Director/Research	ESALs	PCCP < 8"(203.2mm) - thin	No	
Yes a. Crack sealing Cost Effectiveness b. Dave Johnson, Montana State c. Progress report is available. Last progress report is attached. Final report due March 2002.	Yes. It is currently ongoing, see #7 above c. Montana State University	ESALs	concrete = our new standard is 9"(228.6mm). Flexible = Interstate minimum = 4.6"(116.84mm). Primary = 3.6"(91.44mm). Composite = White on top = 2"(50.8mm) to 4"(101.6mm) = Ultra-thin. 4"(101.6mm) to 6"(152.4mm) = whitetopping	Yes, load transfer, ratio of deflections on each side of the joint.	
No	No	ESALs	8"(203.2mm) or less - thin greater than 8"(203.2mm) - thick	No	

Table B-1 Survey Responses (Cont.)

No	No	ESALs	Concrete Pavement less than 10"(254mm) is thin, Flexible less than 4"(101.6mm) is thin	Use FWD to measure load transfer at the conc. Joint. Less than 70% load transfer is considered poor joint.	
No	No	ESALs	Flexible thin: 4"(101.6mm) or less	No	
No	No	ESALs for pavement design as determined from traffic/Truck %/Truck Factor	Don't Define	1. 60% load transfer for joint replacement 2. Slab intercept angle for sub sealing	
No	No	ESALs based on a minimum of a 48 hour continuous traffic count	We have no formal definition. Our minimum PCCP thickness is 180mm. For asphalt pavements, our minimum are based on the AASHTO 1993 design guide recommendations supplemented by actual minimum construction requirements (I.e. a 19mm Superpave minimum constructed thickness would be 65 mm). In general, we build what is required, regardless if it is a construction or maintenance project, to support a given design life need.	No.	New Mexico is basically an asphalt state with only 2% of our system being PCCP. If you should have any questions concerning this response, please contact me. John Tenison, P.E. Section Head Pavement, Investigation and Design Section, New Mexico State Highway Department, State Materials Bureau, P.O. Box 1149, Santa Fe, New Mexico 87504 (505)827-87504
No	No	We use ESALs in thickness desion and superpave mix design	We do not define "Thin". We define pavement later thickness based on ESALs.	We have used our FWD to determine tranverse joint load transfer efficiency.	

Table B-1 Survey Responses (Cont.)

No	No	ESALs (Two classes of trucks: Single unit and tractor-Trailor, each with a truck factor)	>=8" (203.2mm)concrete is think	No	
No	No	ESALs	Concrete - greater than 6" (152.4mm) is thick Flexible - greater than 3" (76.2mm) is thick	We test joints with a FWD to determine load transfer efficiency using the ratio between sensors that are a foot apart.	
No	No	Simple ADT's are evolving into ESAL counts.	AC- Thin less than 6"(152.4mm), Thick 6"(152.4mm) or Greater PC- Thin less than 8"(203.2mm), Thick 8"(203.2mm) or Greater	Joint efficiency = 75-100% - good 50-75% - fair 0-50% - very poor	New Pavement Engineer is Masoud Pajoh.
No	No	We compute ESALs for all projects.	We don't have that definition.	None	
No	No, we currently are using Neoprene Joints on Interstate Highways.	Volume? Or Loading 18KIPS/ESALs	Thin : Asphalt ~3.5"(88.9) or less Concrete ~4" or less Thick: Asphalt 8" = Full Depth Concrete Full Depth 6"-14"	If deflection is acceptable if it is 0.02 IN or less or JT effection *see attachment (For Ultra Thin Whitetopping)	With Attachment
No	No`	Both	Asphalt concrete: less than 4"(101.6mm) thin	No	Accompanying this is your completed questionnaire. In Rhode Island, we rarely construct concrete pavements. However, last year we began a pavement preservation program which includes the crack-sealing of many roads and state Highways.

					Those to be crack-sealed were selected based on their condition. (i.e., a moderate amount of cracking), not their age. We plan to continue our crack sealing program, but have not yet determine an interval at which we will seal or reseal pavements.
No	No	Our Planning office makes traffic counts and uses the estimated truck percentage and a truck load factor determined from previous weight studies to calculate future ESALs for 10 and 20 year periods.	We don't use thick and thin as descriptive factors in our pavement design and analysis. Consequently, we do not have formal definitions for these terms. However, what follows are my own rules of thumb for South Carolina. Concrete-Thin, less than 9" (228.6mm). Thick, more than 12"(304.8mm) Asphalt-Thin, less than 3"(76.2mm) over base; Thick, more than 12" (304.8mm) over base Composite-Thin, less than 4"(101.6mm) overlay on existing PCC, thinck, more than 6"(152.4mm) on existing PCC.	No	

Table B-1 Survey Responses (Cont.)

Yes a.1. (SD 90-13) PCC/AC Shoulder Joint Sealants 2. (SD96-10) Evaluation of PCC/AC Joint Sealant 3.(SD 92-03) Evaluation of silicone Joint Sealant Performance b. 1.Hal Rumpca (SD96-10) 2. Dan Johnston (SD90-13) 3. Arial Soriano(SD92-03) c. SDDOT	No	ESALs	PCC Concrete: <8"(203.2mm) thin >8"(203.2mm) thick Asphalt concrete: <4"(101.6mm) thin >8"(203.2mm) thick	PCC Pavement - Testing is done 4" on either side of the joint to deermine load transfer efficiency and inplace concrete strength at the joint.	
No	No	18kip ESALs	Define surfacing thickness ACP: thin<=2", thick ACP>=7". Do not defined thick/thin for PCC. But thickness of most PCC lies between 10" (254mm) and 13" (330.2mm). Some extreme thickness of PCC may lies between 8"(203.2mm) to 15"(381mm).	No criteria - Project judgement decision	
Sort of. An engineer working for UDOT write his thesis on this subject. a. Field Performance Study of Selected Potland Cement Concrete joint Sealants in Utah. b. Tim Biel c. He can send a copy. ph.(801) 975-4928.	No Also, Lynn Evans from ERES is monitoring our LTPP sealant sites. We have not seen a report from them.	ESALs	We don't distinguish between thick and thin.	Ratio of deflection of the average of 1st and 3rd sensors across the joint and 1 st sensor.	
No	No	ESALs	PCC > 8"(203.2mm) BCP > 6"(152.4mm) BCP/PCC > 12"(304.8mm)	No	
No, but conventional wisdom told us that this is the best practice. We may not have the exact number, but we know the benefit is there.	No	For 30 year life: 20 million low 21-50 million medium 51-100million heavy	Concrete 8"(203.2mm) thin, 11"(279.4mm) thick Asphalt 4"(101.6mm)-5"(127mm) thin, 13"(330.2mm) thick (this is asphalt courses only) Composite (only asphalt overlaying concrete) 2"(50.8mm) thin, 6"(152.4mm) thick	The load transfer criteria is: 0-50% poor Require full depth patching and dowel bars replacement 51-75% fair-good (no action taking, but will watch for future faulting) 76-100% excellent	Sent by Mike Jennings for Mohamed Elfino

Table B-1 Survey Responses (Cont.)

No	Yes a. Linda Pierce	Primarily use ESALs, however the pavement mansgement systems contain all terms.	ACP<4"(101.6mm) Thin, >4"(101.6mm) Thick PCCP - Not Defined. Typical Range 8"(203.2mm)-12"(304.8mm).	Load transfer less than ~70% Faulting greater than 1/8"(3.18mm)	
Yes a. 1. The Great Unsealing-TRR 1597 2. Evaluation of AC Crack Sealing b. S.F. Shober & Terry Rutkouski c. 1. TRR 1597 2. From WISDOT	No, It is done in WI. NCHRP is doing it	ESALs	___________	No. Just evaluate Actual performance (not a surrogate like FWD) in terms of ride and distress!	
No	No	Daily ESALs	Do not make a distinction.	No	

Appendix C Traffic Control Information

Table C-1 Traffic Control Information

Site	Road	Site Dir	Site Location MM+ .Mi or feet	Length of Site	Pave Type	Dist	Sub / Unit	Contact Name	Phone
4a	SR38	EB	MM 4 +67	469	JCP	CVI	Fowler	Ross Kurtz	(765)884-1500
1a1	US231	NB	S of Riv Brdg	409	JCP	CVI	Fowler	Ross Kurtz	(765)884-1500
1a2	US231	NB	108' N of Riv	602	JCP	CVI	Fowler	Ross Kurtz	(765)884-1500
2a	SR63	NB	MM 82 +667	560	JCP	CVI	Fowler	Ross Kurtz	(765)884-1500
10a	SR63	NB	MM 93 +7	642	OVER	CVI	Fowler	Ross Kurtz	(765)884-1500
4b	SR63	SB	MM 91 +4255	619	JCP	CVI	Fowler	Ross Kurtz	(765)884-1500
7a	US421	NB	MM 127 +1063	854	ASPH	CVI	Frankfort	Randy Large	(765)659-3369
9b	I74	WB	MM 31 +1673	2223	OVER	CVI	CVI sub	Gordon Burns	(765)362-9484
12b	SR9	NB	MM 175 + 560	1687	OVER	FTW	Goshen	Doug Mickem	(219)533-9578
2b	SR3	SB	MM 193 +395	418	JCP	FTW	Goshen	Doug Mickem	(219)533-9578
11b	US30	WB	MM 103 -217	1322	OVER	FTW	Warsaw	Terry Hatfield	(219)267-8571
8a	SR18	EB	MM 64 +454	768	ASPH	LAP	Monticello	Jim Miller	(219)583-4171
11a	US24	EB	MM 32+.7 (3902)	2101	OVER	LAP	Monticello	Jim Miller	(219)583-4171
1b	I-65	NB	MM 217 +930	733	JCP	LAP	Rensselaer	William Swartz	(219)866-7422
5b	I-65	NB	MM 224 +0	1517	ASPH	LAP	Rensselaer	William Swartz	(219)866-7422
7b	US35	SB	0.3 Mi S. of RR @450	775	ASPH	LAP	Laporte	Michael Fraze	(219)362-3520
9a	I-65	NB	MM 232 +68	1227	OVER	LAP	Rensselaer	William Swartz	(219)866-7422
12a	SR25	NB	MM 78 +2746	2168	OVER	LAP	Monticello	Jim Miller	(219)583-4171
8b	SR29	SB	MM 29 +919	1413	ASPH	LAP	Monticello	Jim Miller	(219)583-4171
10b	SR62	WB	MM 3 +0	2055	OVER	VIN	Evansville	Mike Walters	(812)867-9016

Appendix D Test Site Sketch

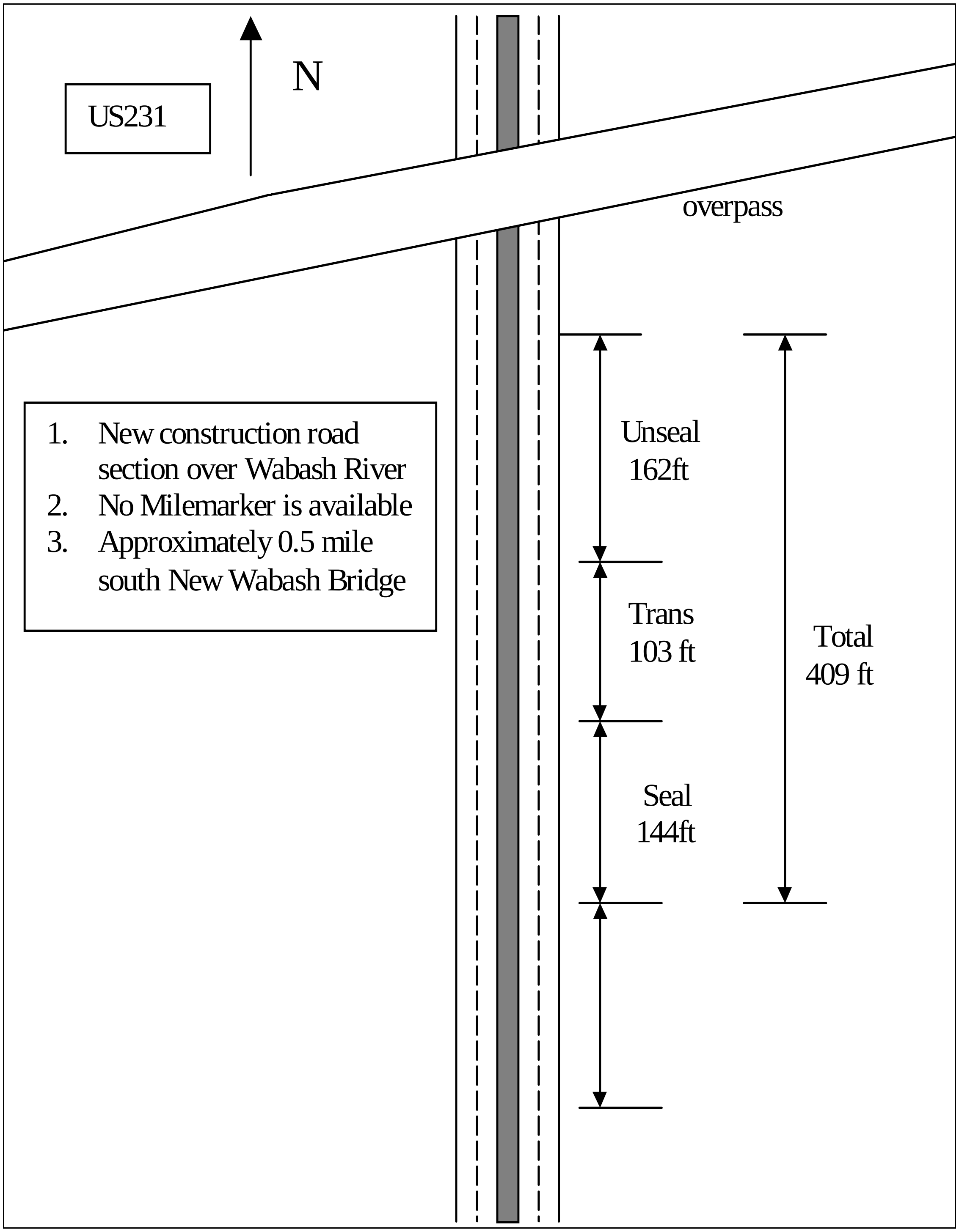

Figure D-1 Sketch for Test Site 1A-1

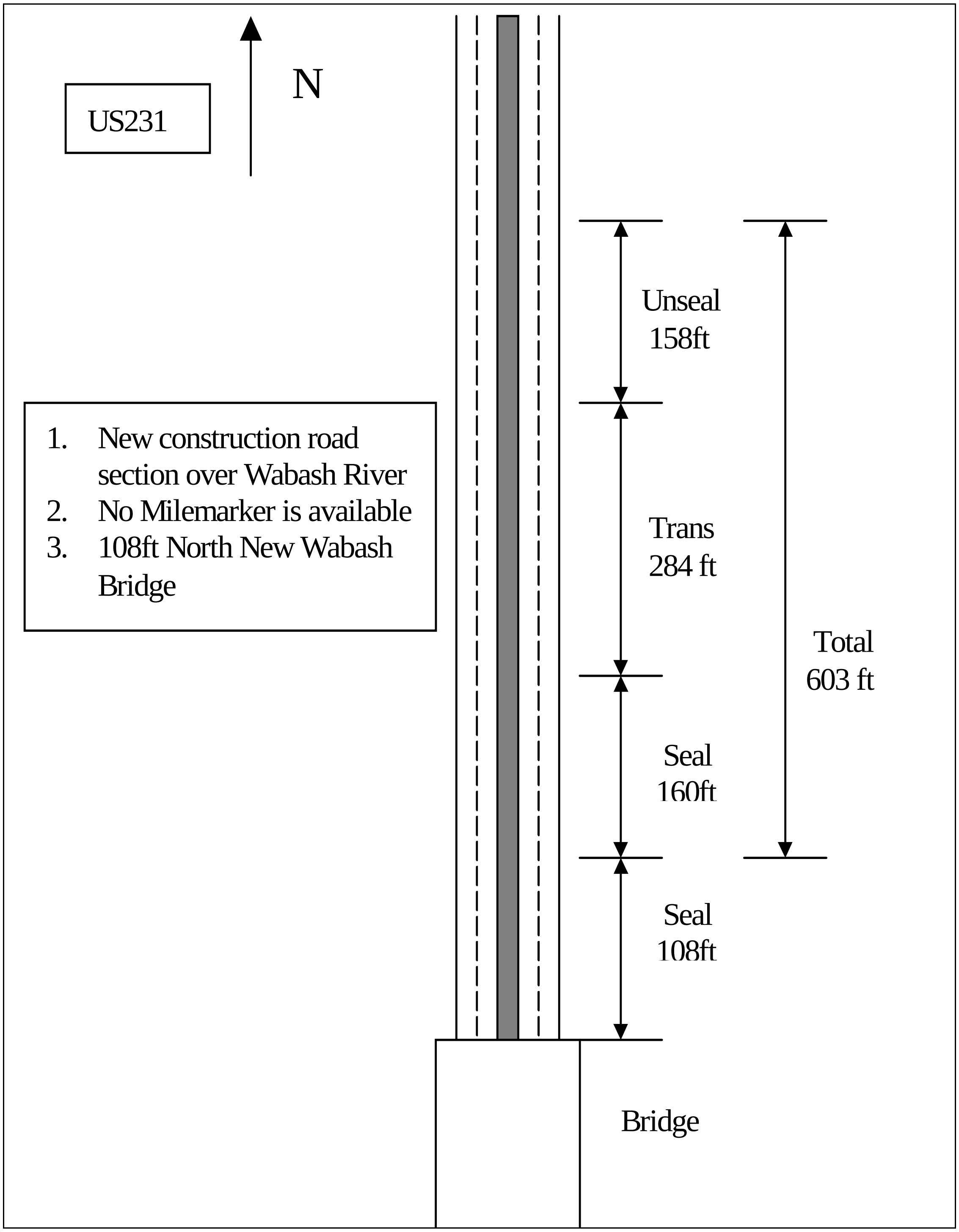

Figure D-2 Sketch for Test Site 1A-2

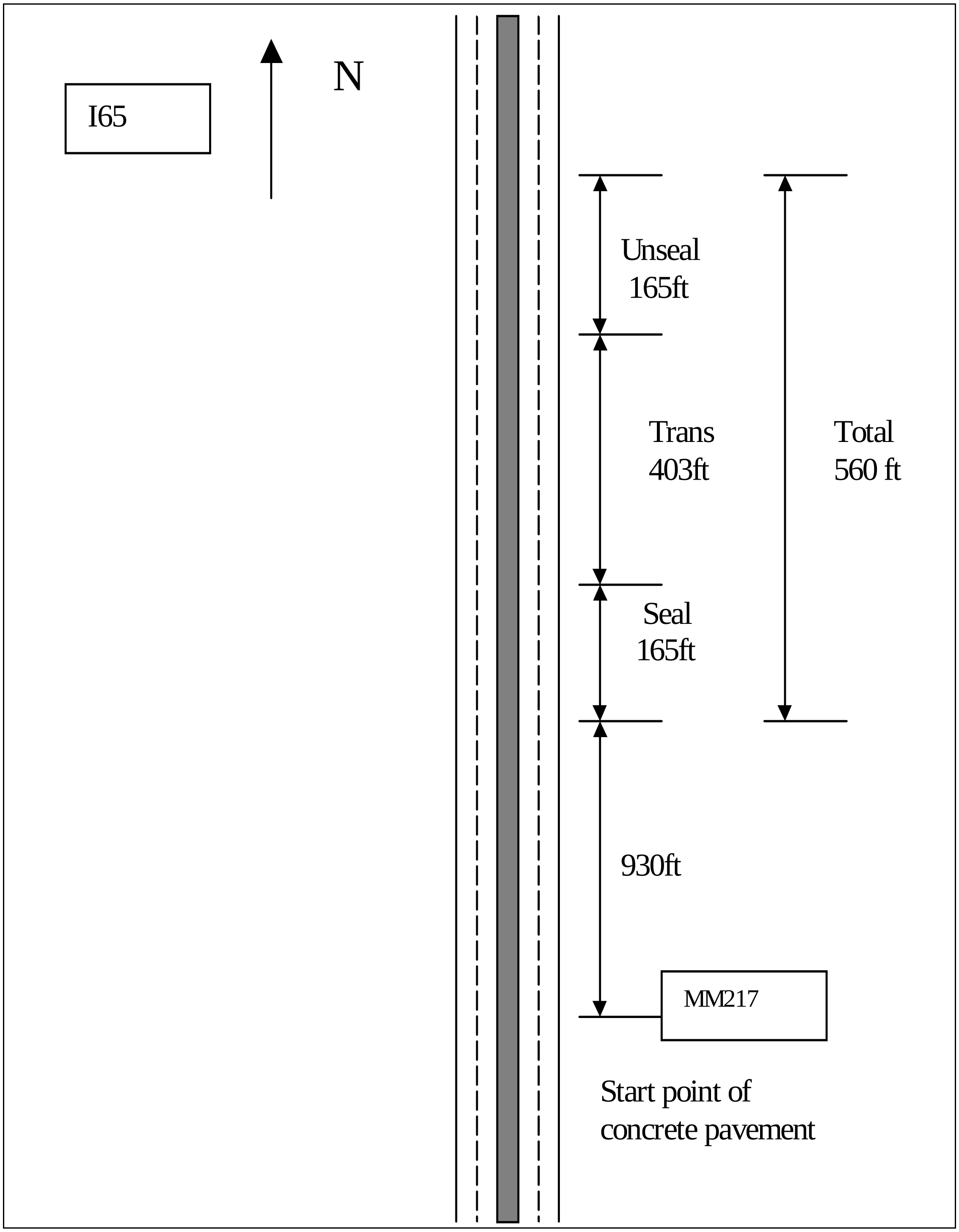

Figure D-3 Sketch for Test Site 1B

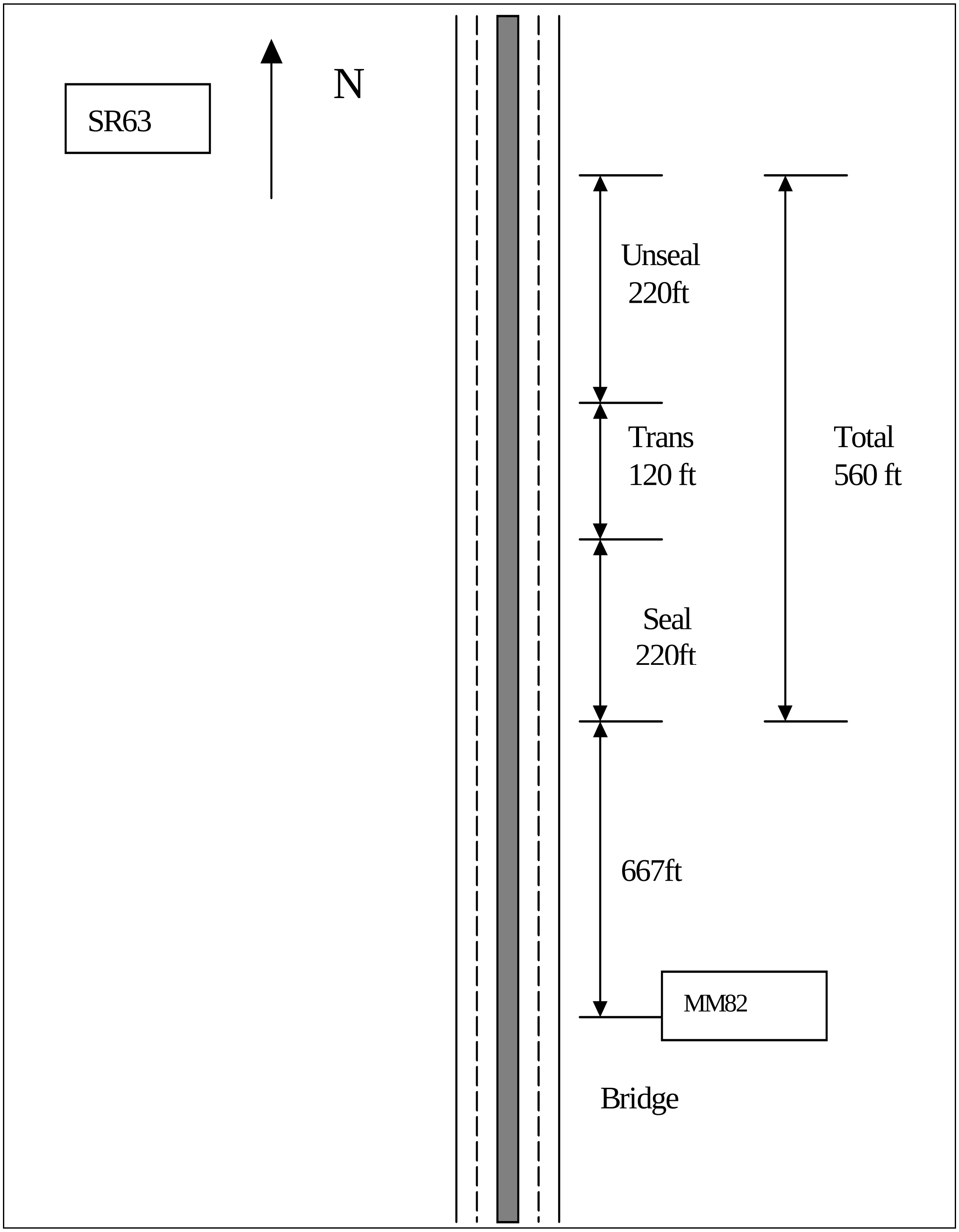

Figure D-4 Sketch for Test Site 2A

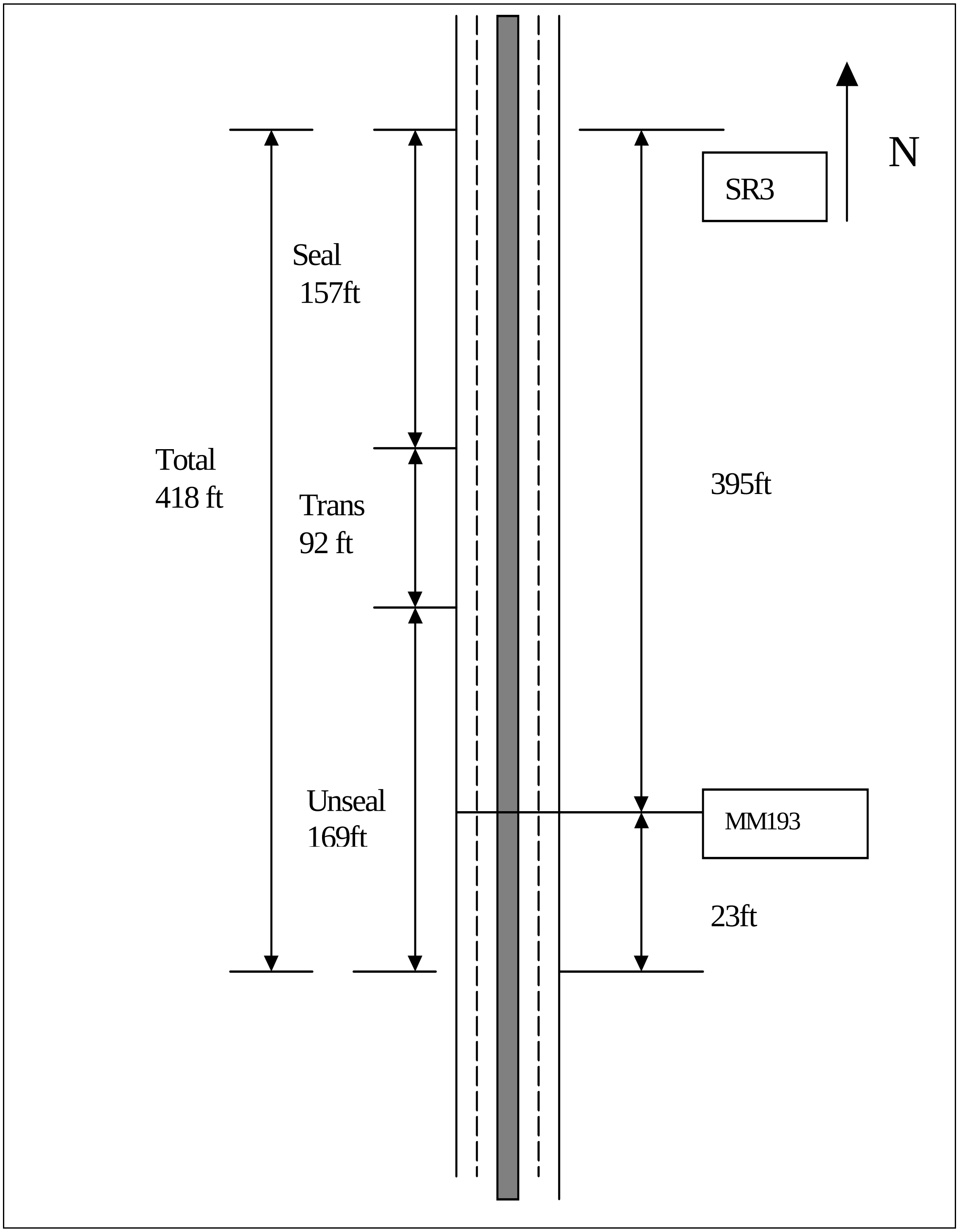

Figure D-5 Sketch for Test Site 2B

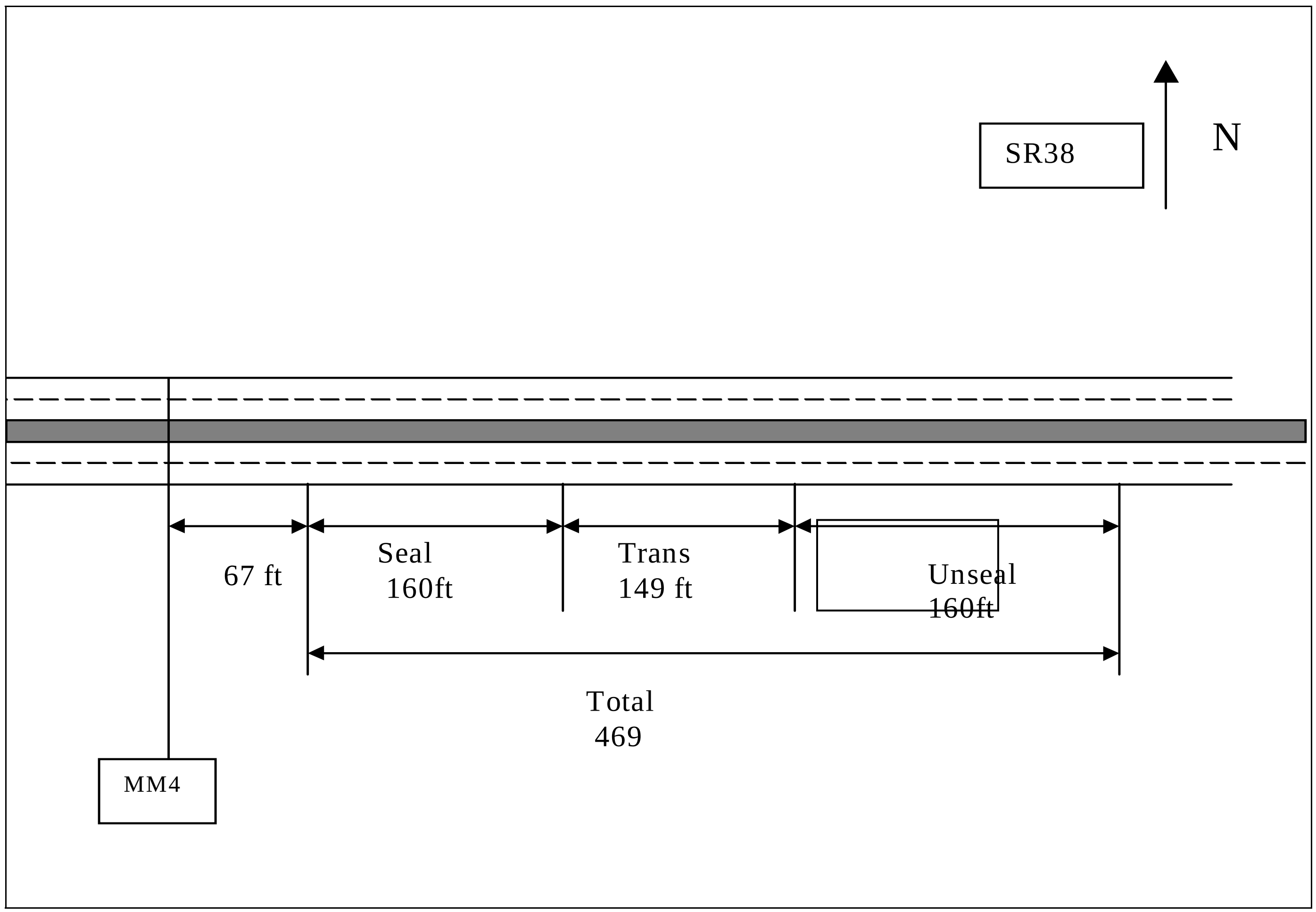

Figure D-6 Sketch for Test Site 4A

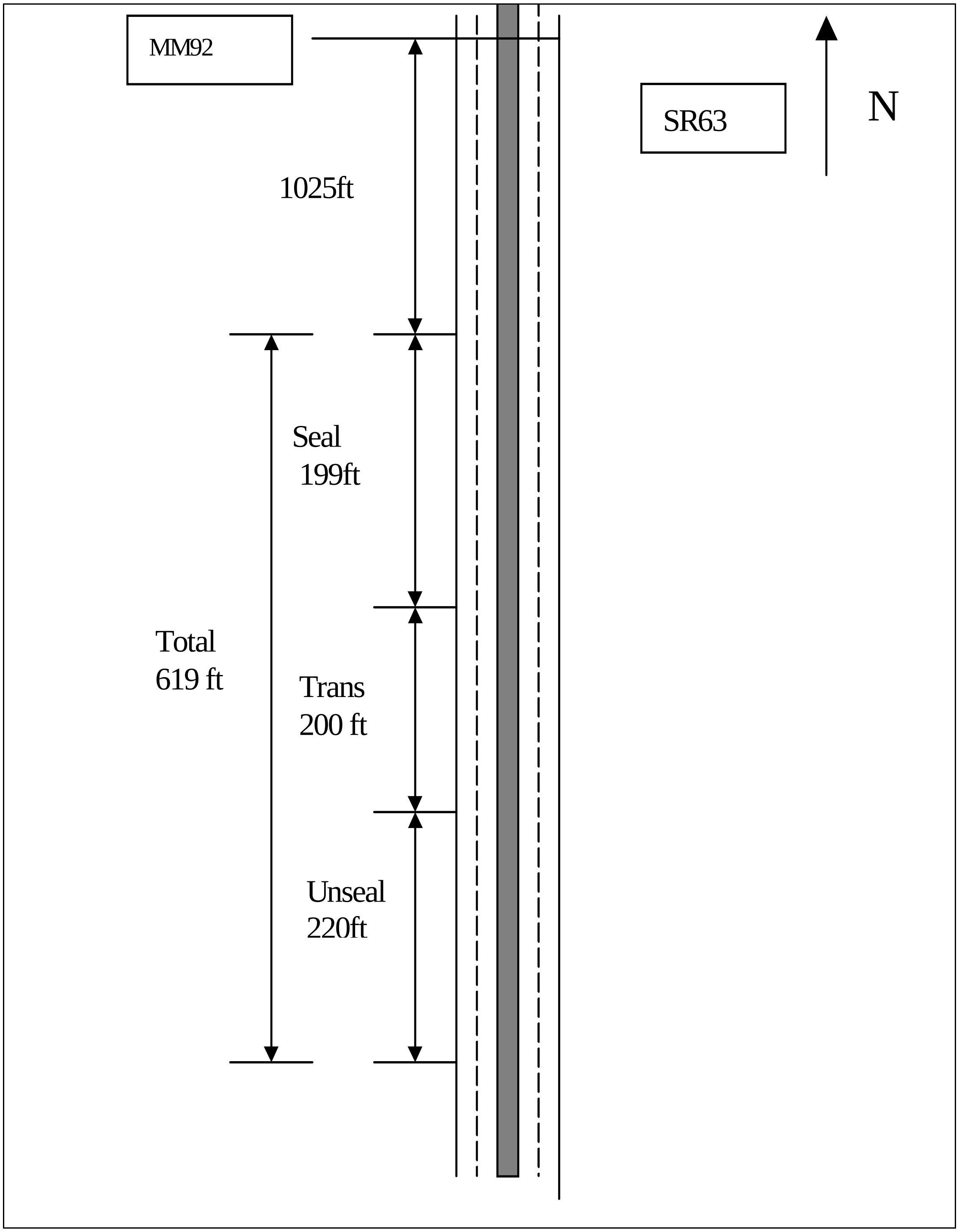

Figure D-7 Sketch for Test Site 4B

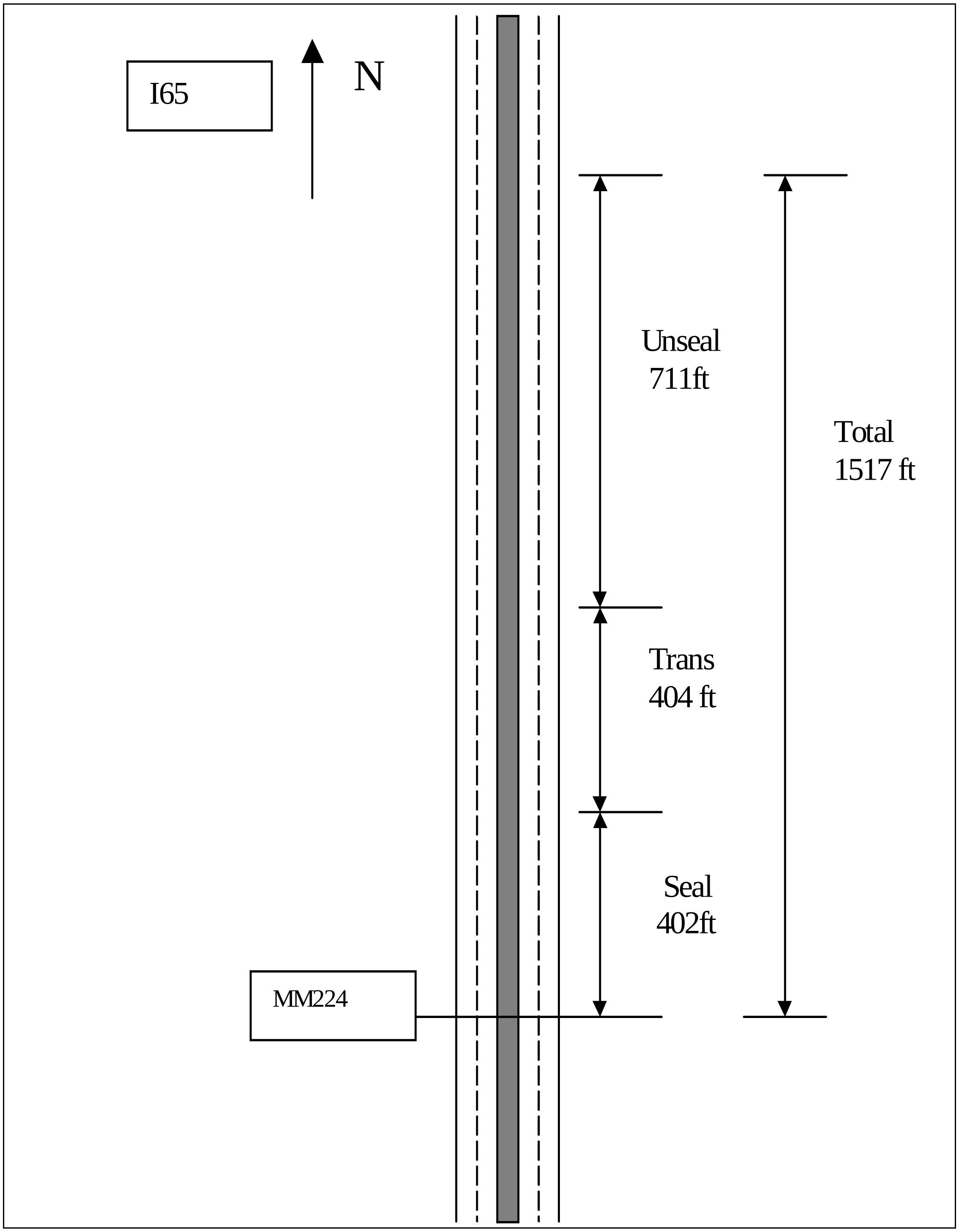

Figure D-8 Sketch for Test Site 5B

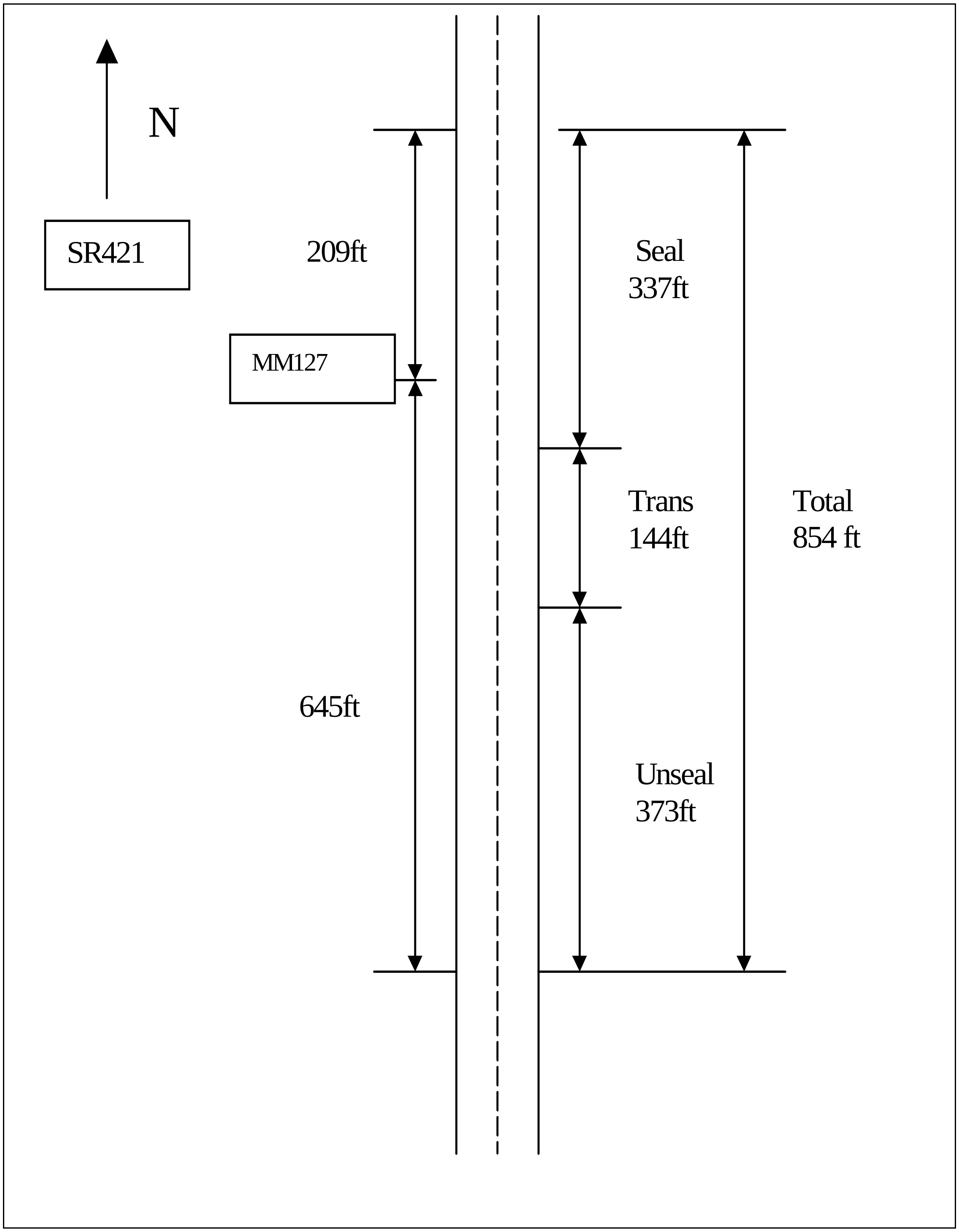

Figure D-9 Sketch for Test Site 7A

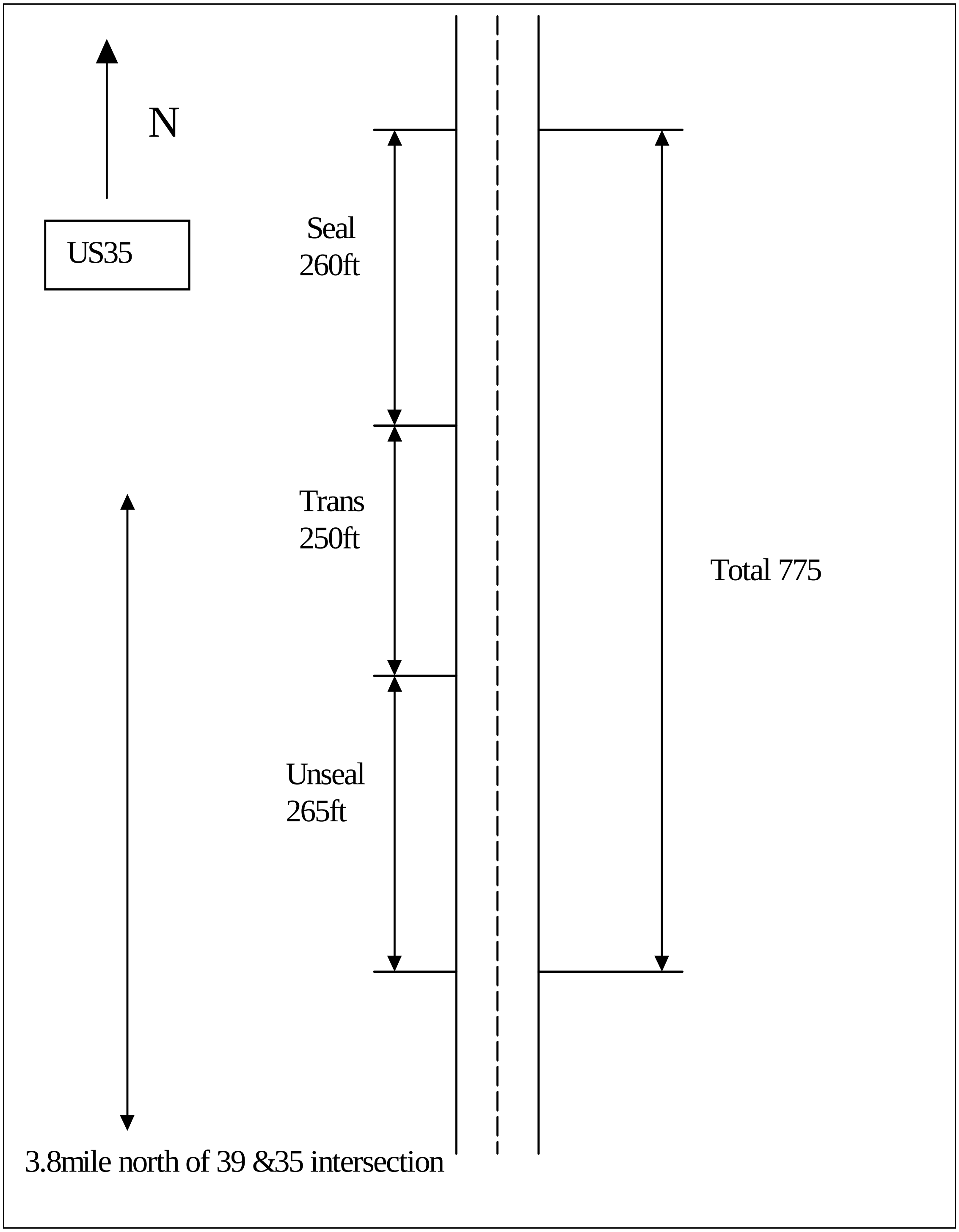

Figure D-10 Sketch for Test Site 7B

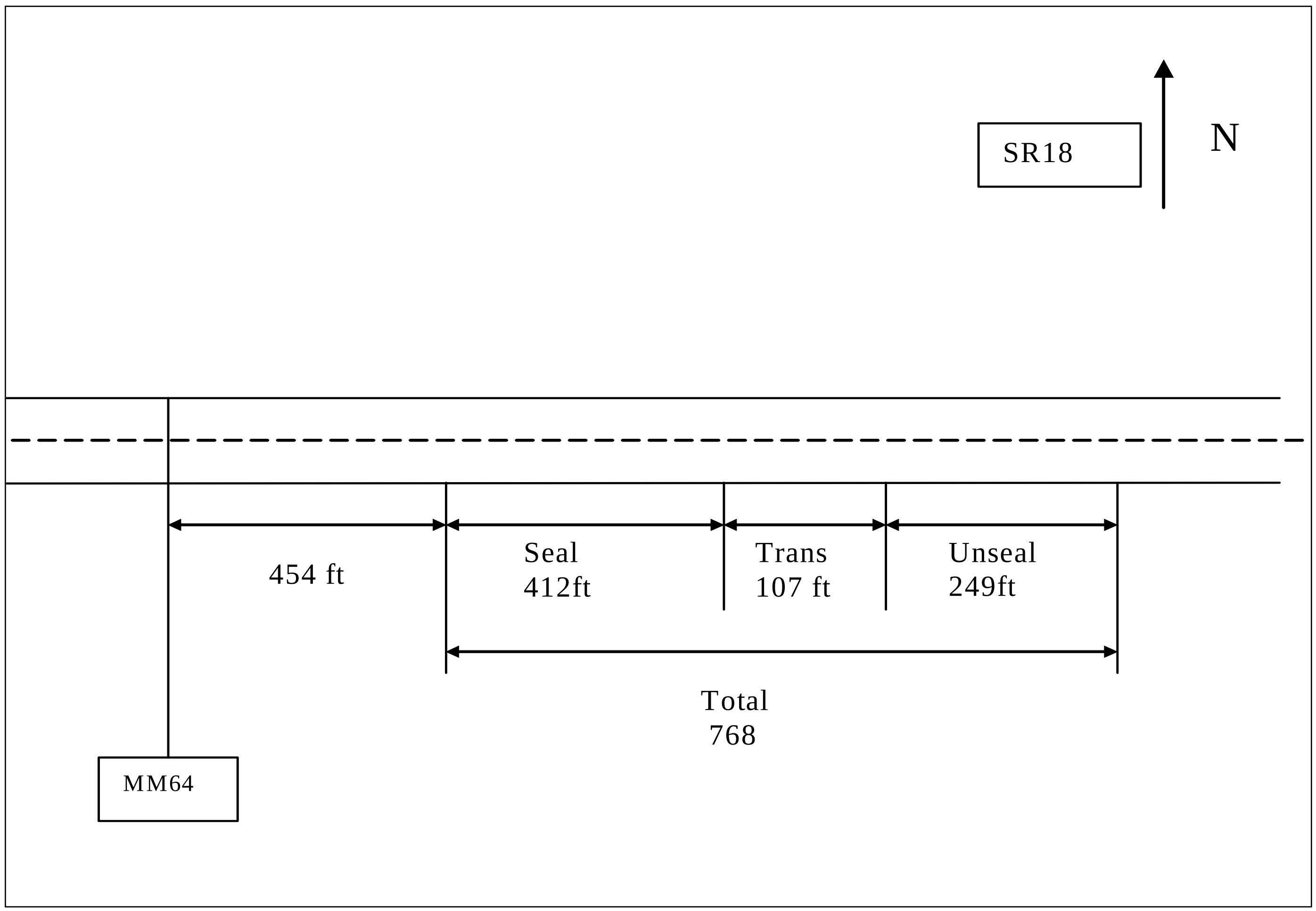

Figure D-11 Sketch for Test Site 8A

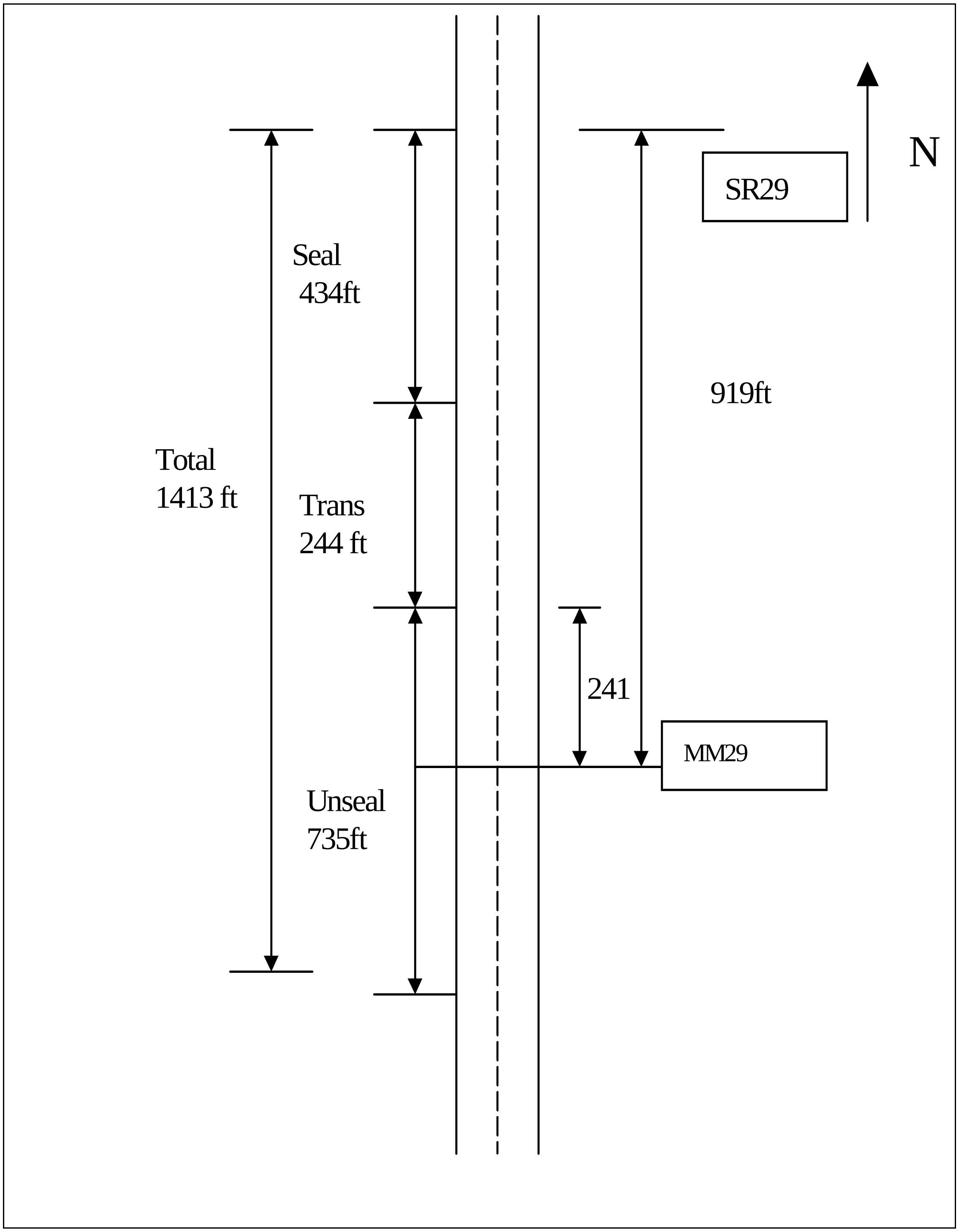

Figure D-12 Sketch for Test Site 8B

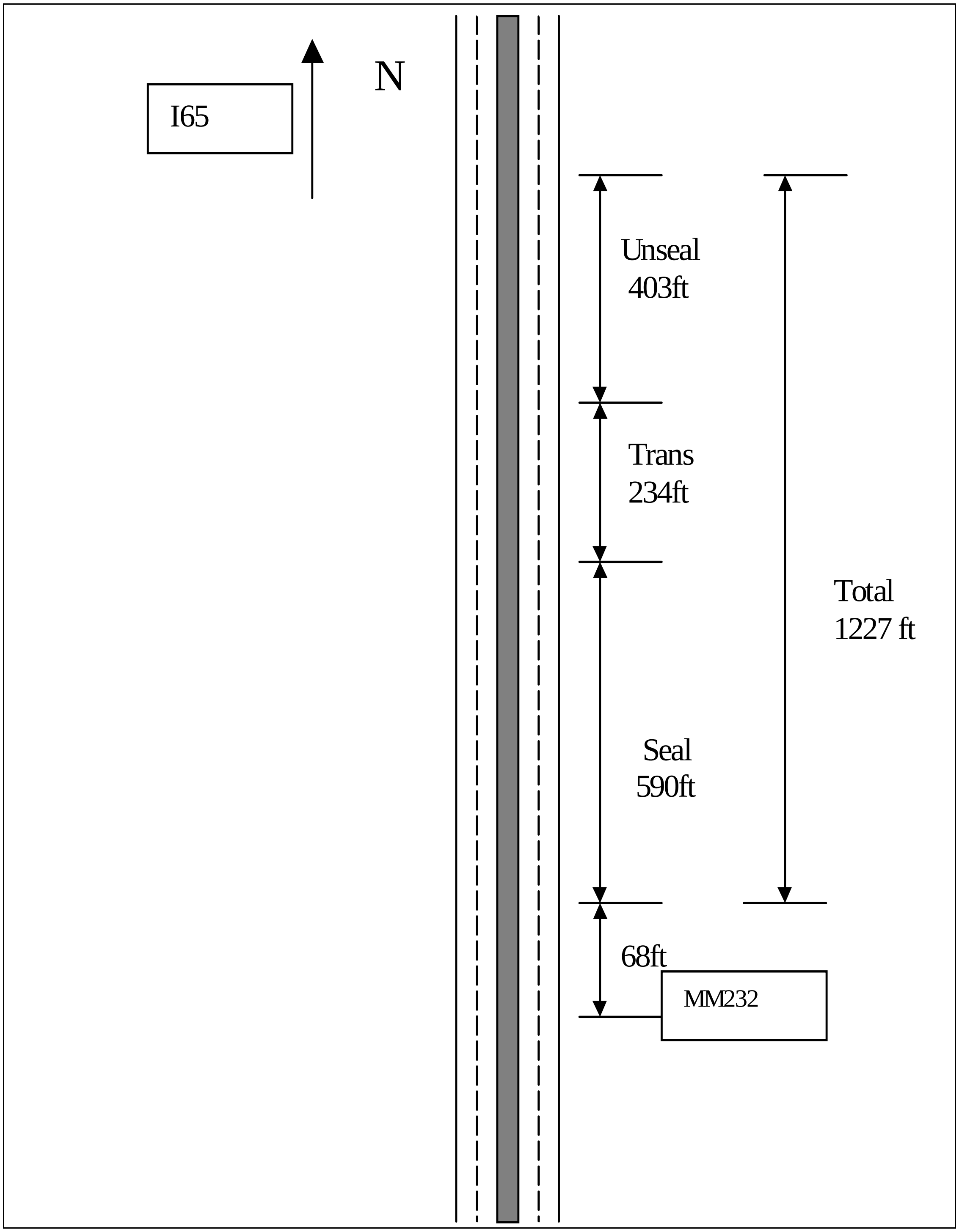

Figure D-13 Sketch for Test Site 9A

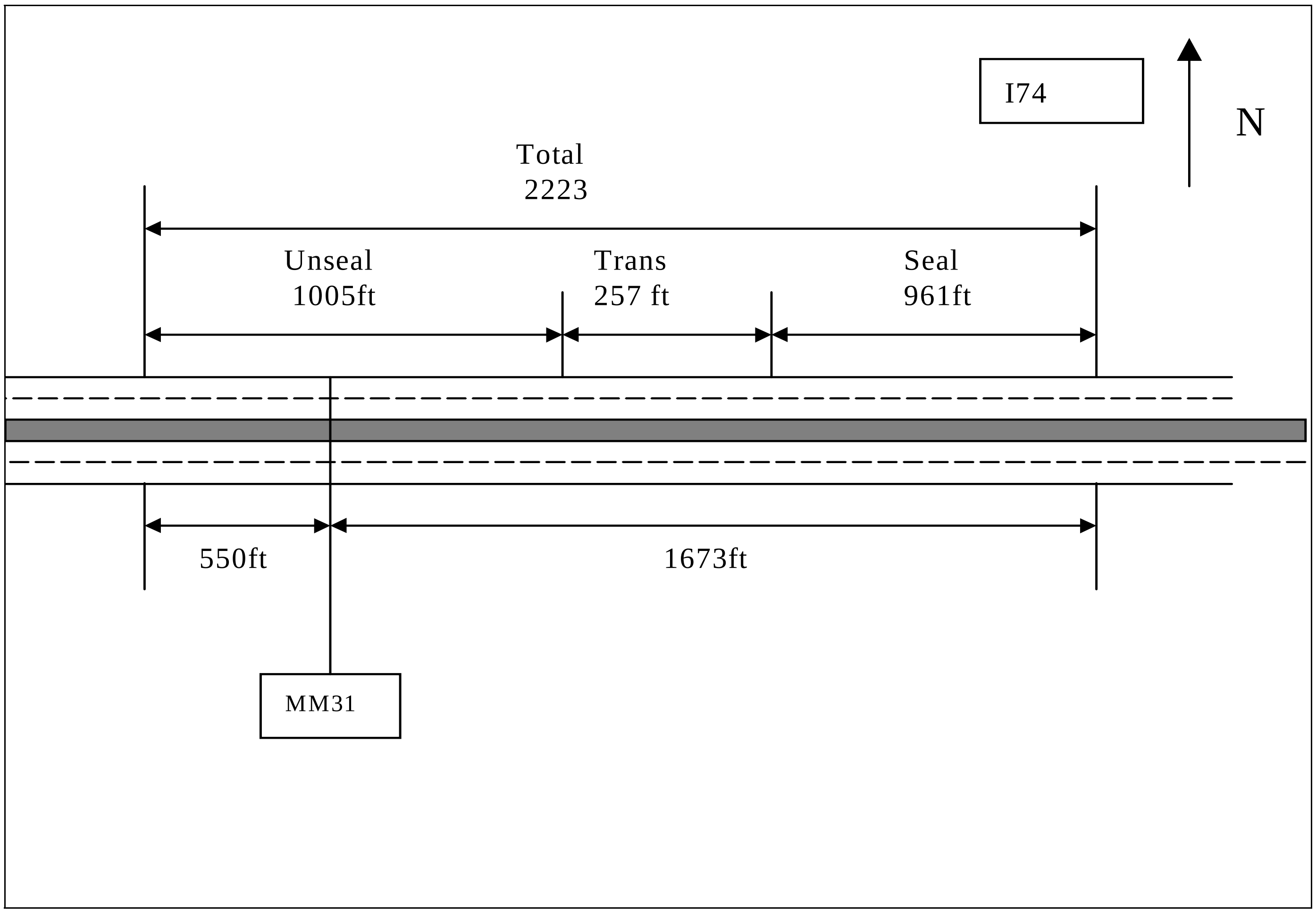

Figure D-14 Sketch for Test Site 9B

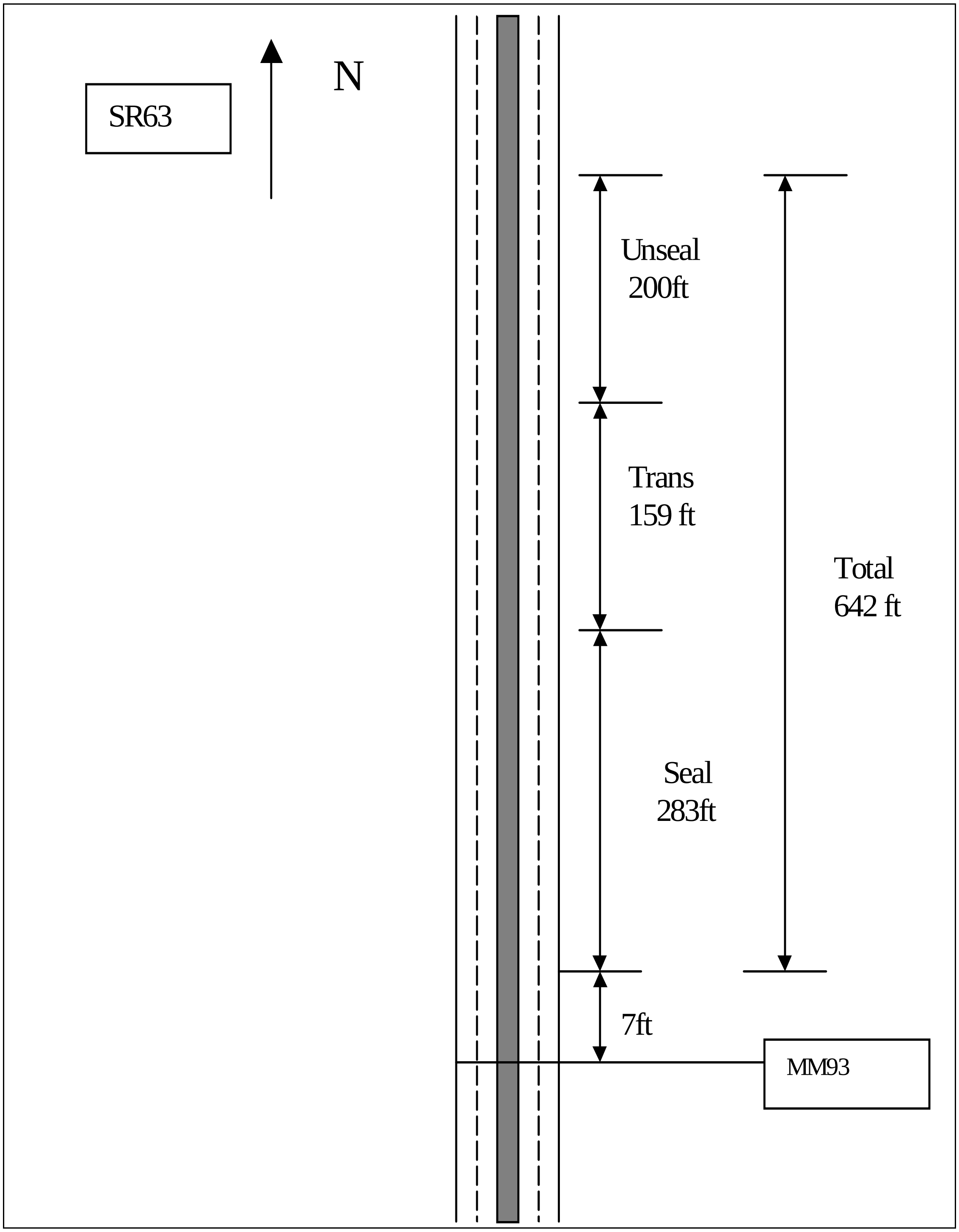

Figure D-15 Sketch for Test Site 10A

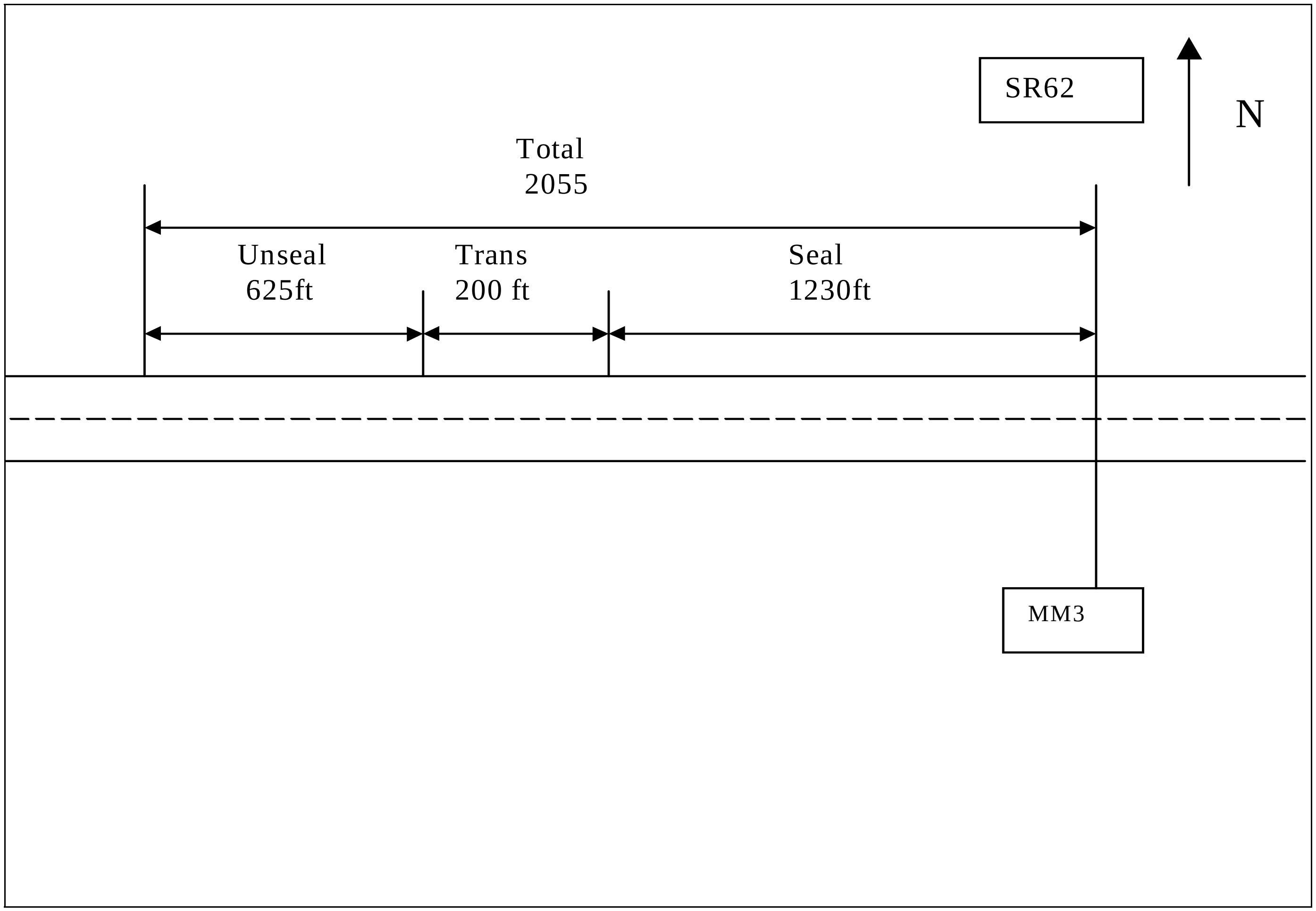

Figure D-16 Sketch for Test Site 10B

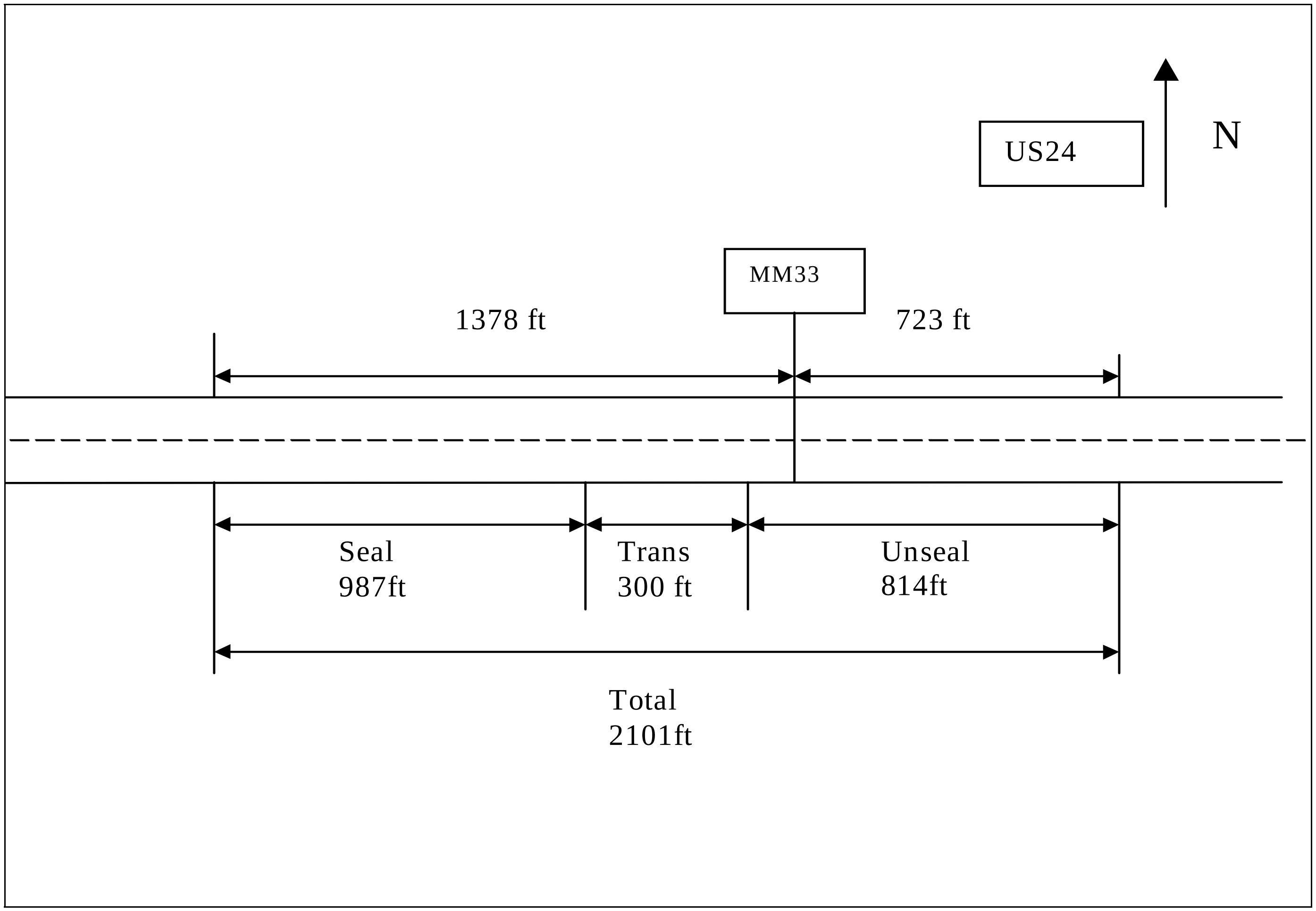

Figure D-17 Sketch for Test Site 11A

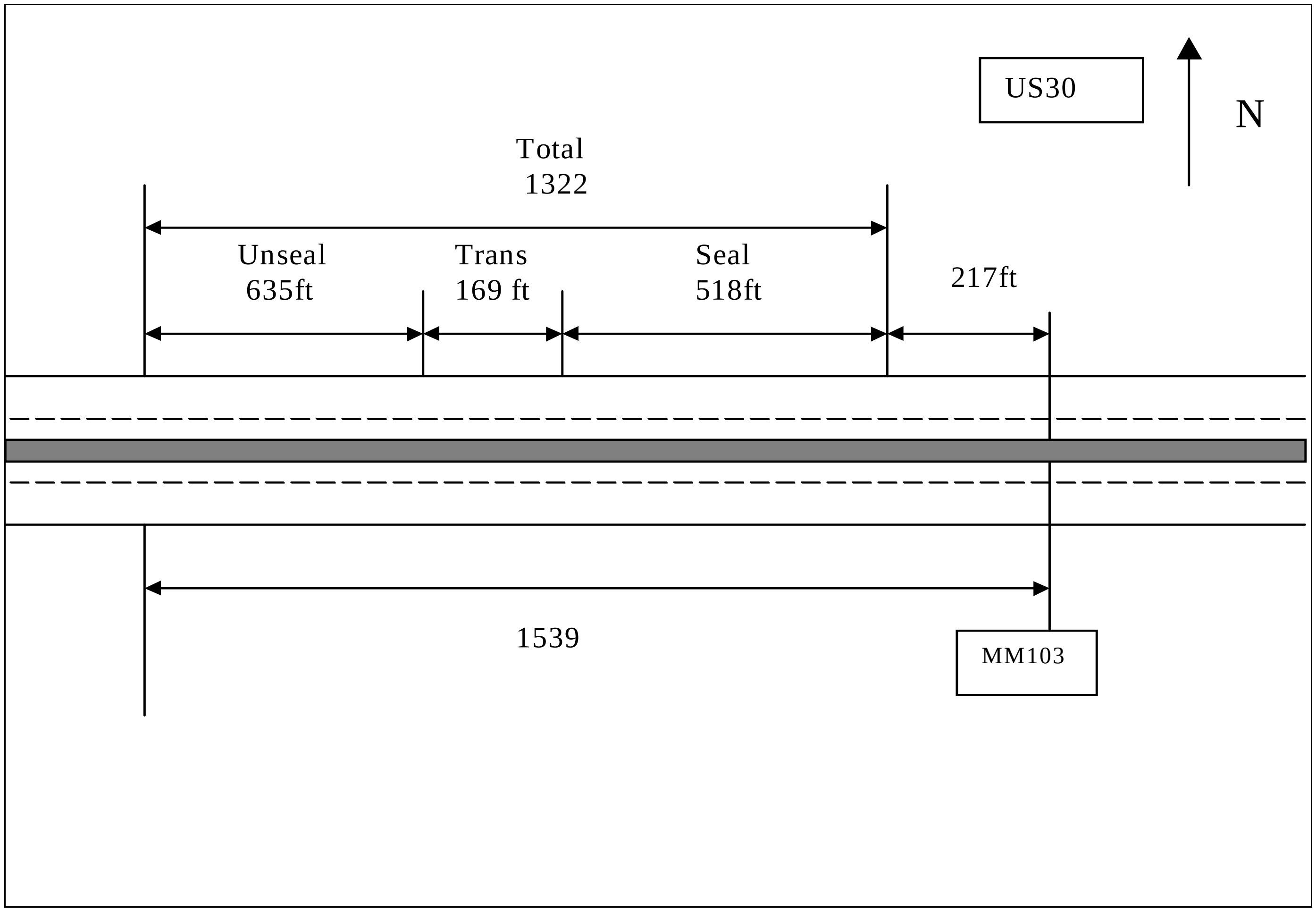

Figure D-18 Sketch for Test Site 11B

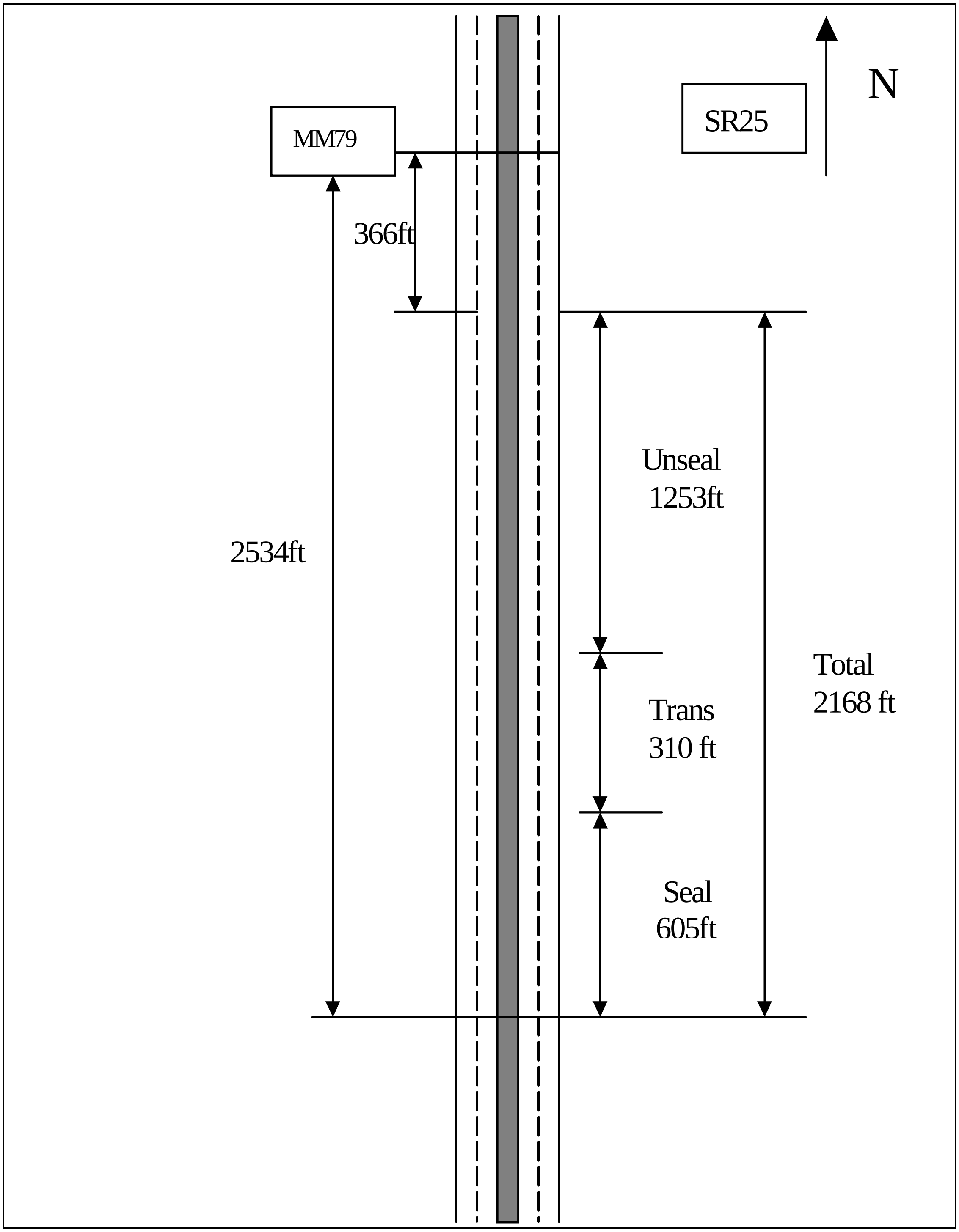

Figure D-19 Sketch for Test Site 12A

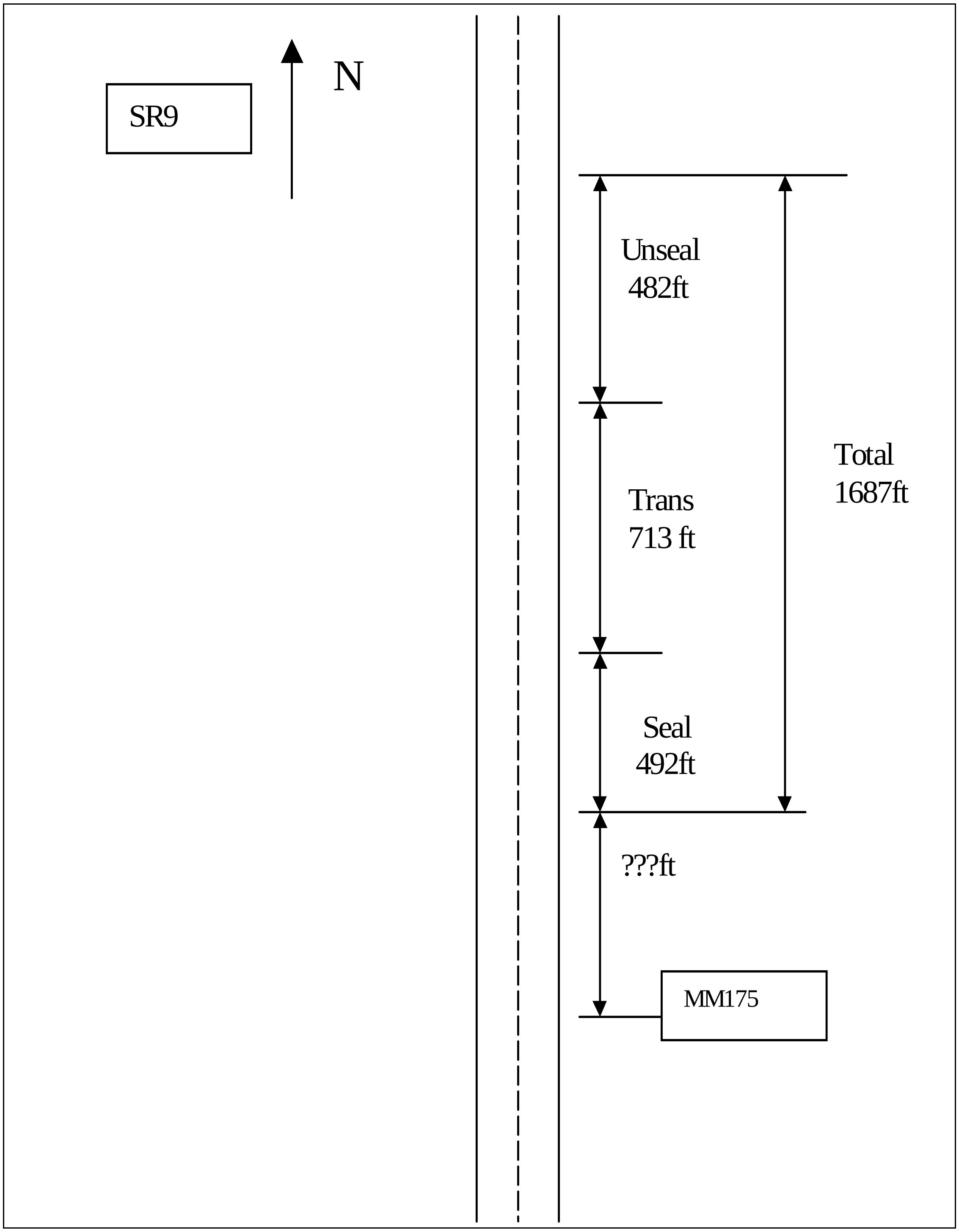

Figure D-20 Sketch for Test Site 12B

Appendix E Core Samples

1. Figure E-1 shows the core sample taken from on northbound SR25 at milemaker (MM) 79 on 12 April 2000. Figure E-2 shows the pavement layer structure. This is a composite pavement with apparently three layers of HMA on top of the PCC. Unfortunately, the core sampler was not able to cut deeper in order to obtain the thickness of PCC or the material of the subbase.

Figure E-1 Pavement Core on Northbound SR25

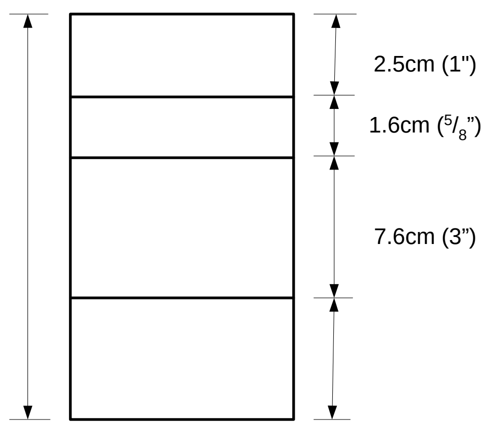

Figure E-2 Pavement Layer Structure on Northbound SR25

2. Figure E-3 shows the core sample taken from on eastbound SR18 at MM65 on 12 April, 2000. Figure E-4 shows the pavement layer structure. This pavement is full depth HMA with approximately six layers. No subbase sample was taken.

Figure E-3 Pavement Core on Eastbound SR18

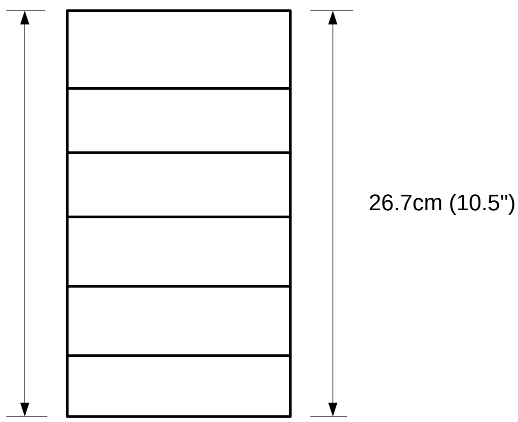

Figure E-4 Pavement Layer Structure on Eastbound SR18

3. Figure E-5 shows the core sample from southbound SR29 at MM29 on 12 April 2000. Figure E-6 shows the pavement layer structure. This is a full depth HMA pavement with four layers.

Figure E-5 Pavement Core on Southbound SR29

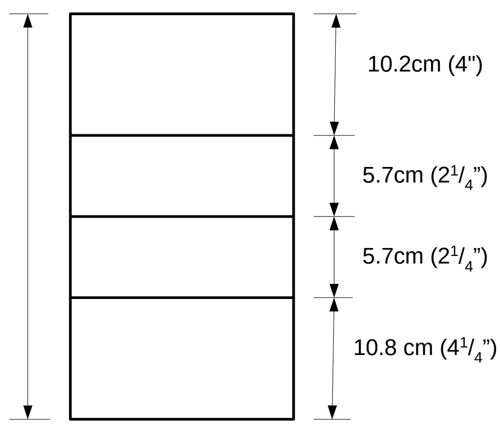

Figure E-6 Pavement Layer Structure on Southbound SR29

4. Figure E-7 shows the core sample from eastbound US24 at MM33 on 12 April 2000. Figure E-8 shows the pavement layer structure. This is a composite pavement with 17.1 cm (6.5 in) HMA on the top and 18.1 cm (7.13in) at the bottom.

Figure E-7 Pavement Core on Eastbound US24

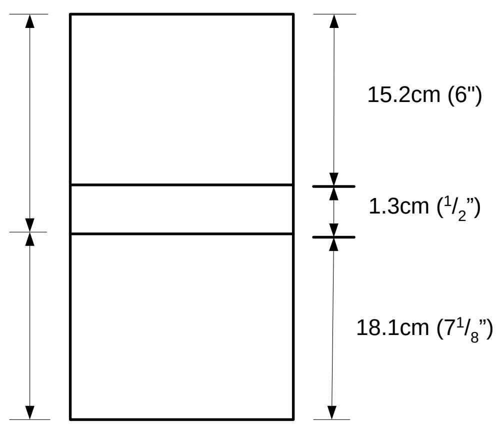

Figure E-8 Pavement Layer Structure on Eastbound US24

5. Figure E-9 shows the pavement layer structure from southbound US421 at MM116. This is a composite pavement with one PCC layer at bottom and three layers on the top. Two subbase samples were taken. Both samples contained mostly dark clay.

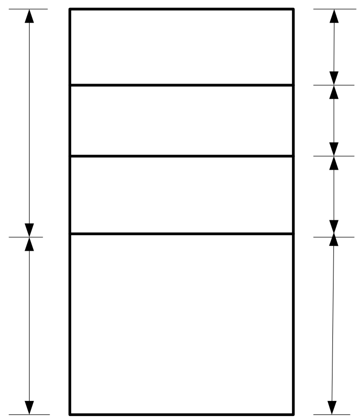

Figure E-9 Pavement Layer Structure on Southbound US421

6. Figure E-10 shows the pavement layer structure from southbound US421 at MM129 on 27 April 2000. This is a full depth HMA pavement with five layers. One subbase sample was taken, which contained course aggregate with sand.

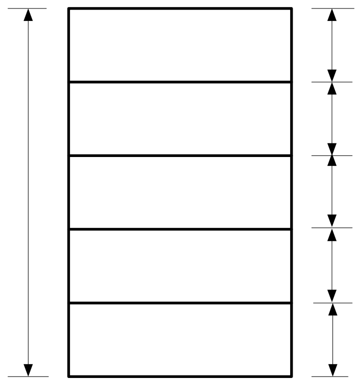

Figure E-10   Pavement Layer Structure on Northbound US421