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LTPP Data Analysis: Relative Performance of Jointed Plain Concrete Pavement with Sealed and Unsealed Joints

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LTPP Data Analysis:
Relative Performance of Jointed Concrete Pavement With Sealed and Unsealed Joints

Summary of Findings

NCHRP 20-50(2) was conducted to compare, based on the data available from the LTPP studies, the performance of JPCP designed and constructed with unsealed joints to that of JPCP with sealed joints. There are just five LTPP sites suitable for this analysis: the SPS-4 sites in Arizona, Colorado, and Utah. These sites are all located in the dry western region of the United States, for which reason it would be unwise to extrapolate the results of this analysis to other regions of the country that receive more precipitation. Nonetheless it is hoped that the analysis methods used in this study may serve as useful examples for future analyses of other sealed-versus-unsealed joint experiments in other climates.

Despite the conventional wisdom concerning the need to keep concrete pavement joints well sealed, previous studies on the subject have not demonstrated that JPCP with sealed joints and JPCP with unsealed joints perform differently in terms of spalling, faulting, IRI, or deflections.

The analyses conducted in this study do not indicate that unsealed joints are any more likely to develop joint spalling than sealed joints. The narrow unsealed-joint test sections did, in general, exhibit more faulting and higher rates of IRI increase than most other treatment groups. However, the same is true of one particular sealed-joint group: 9-mm silicone-sealed joint group. It would thus be inaccurate to conclude that the unsealed-joint treatment resulted in more faulting and roughness than the sealed-joint treatments. It is important to note as well that in every case, the order of treatment groups with respect to IRI was no different after several years of service than it was in the first year after construction.

The narrow unsealed-joint test sections have exhibited better deflection load transfer and other joint deflection responses than the sealed-joint test sections. At some sites, the unsealed-joint test sections exhibited higher total deflections (loaded plus unloaded sides of the joint) than the sealed-joint groups. However, it is not concluded on the basis of these analysis results that higher total joint deflection is correlated to higher faulting and IRI, because, for one thing, it would not explain the higher faulting and IRI in the 9-mm silicone-sealed test sections.

It should also be kept in mind that at three of the five sites, the sealed-joint test have moderate to severe joint seal damage. How well sealed the sealed-joint test sections really are is a factor that should be considered in future analyses of the longer-term performance of the pavements at these five sites, as well as analyses of sealed-versus-unsealed joint experiments in other climates.

Chapter I

Introduction and Research Approach

Research Objective

Currently, nearly all State highway agencies require sealing of transverse joints immediately after construcion for all new jointed concrete pavements (JPCP). Joint sealing is commonly believed to be beneficial to concrete pavement performance in two ways. Sealed joints are believed to reduce water infiltration into the pavement structure, thereby retarding the occurrence of moisture-related joint distresses such as pumping, faulting, corner breaking, and freeze-thaw damage (D cracking). Sealed joints are also believed to reduce or prevent the infiltration of incompressible materials (i.e., sand and small stones) into the joints, thereby reducing the likelihood of pressure-related joint distresses such as spalling and blowups.

Joint sealing is estimated to increase initial construction costs by approximately 2 to 7 percent. Joint resealing, which may be needed periodically over the service life of the pavement to maintain the integrity and/or effectiveness of the sealants, also increases the costs of these pavements. The expected return from this increased investment is enhanced performance, compared to JPCP performance with no joint sealant required.

A few highway agencies have designed and constructed jointed plain concrete pavements (JPCP) with unsealed joints for many years. The decisions by some States to eliminate joint sealant requirements were based on in-State research indicating that sealing and resealing transverse joints was not cost-effective. [1, 2] That is, the performance enhancement and/or life extension attributable to joint sealants did not offset the additional costs associated with sealant installation and maintenance. Wisconsin, in particular, reports having achieved excellent overall performance for up to 22 years in JPCP with narrow unsealed joints.[1] Some European countries, specifically Austria and Spain, also do not seal joints on some concrete pavements in dry areas.[3, 4] If these findings can be proven valid for other States as well, potential savings of millions of dollars in construction and maintenance costs may be realized. Reductions in traffic delays and increased worker safety are other potential benefits of eliminating joint sealant installation and maintenance requirements.

To date, a detailed comparison of the performance of JPCP with and without sealant requirements has yet to be completed on a national level. The objective of NCHRP Project 20-50(2) is to compare, based on the data available from the LTPP studies, the performance of JPCP designed and constructed with unsealed joints to that of JPCP with sealed joints.

Designing and Constructing Unsealed Joints

A joint that is designed and constructed to remain unsealed is typically formed by a single narrow sawcut (e.g., 3 mm, or approximately 1/8 inch, wide). The objective is to form the joint with a single sawcut, not necessarily to restrict the width to 3 mm, which may in fact be slightly greater than the thickness of a typical saw blade. The initial sawcut is not subsequently widened, as would be done if a sealant were to be installed. **For the purposes of this study, a narrow unsealed joint is defined as one that is formed by a single sawcut, with no secondary reservoir sawing or sealant installation.**

Both for joints intended to be left unsealed and those intended to be sealed, timing of the primary sawcut is critical. The goal in timing the sawcutting operation is not to saw too soon (which will cause ravelling of the concrete) nor too late (which may result in random cracking). Sawing of transverse and longitudinal construction joints should begin as soon as possible after adequate concrete strength is obtained, usually between 4 and 12 hours after placement.[5] However, the length of the "sawcutting window" is highly variable and depends on concrete mix properties, curing conditions, ambient temperatures during curing, the degree of friction between the concrete slab and underlying base.

Designing and Constructing Sealed Joints

Design and construction of sealed transverse joints in JPCP involve the following steps:

- Selection of an appropriate sealant material,
- Estimation of joint movements,
- Design of the joint sealant reservoir,
- Primary sawcutting to create the joint,
- Secondary sawcutting to create the joint sealant reservoir, and
- Installing the sealant.

References 4 and 5 provide more information on each of these steps in designing and constructing sealed transverse joints in JPCP.

Potential Effects of Unsealed Joints on JPCP Performance

By what specific measures should the effects of joint sealing on concrete pavement performance be evaluated? Observations related to sealant condition (e.g., cohesive failure, adhesive failure, extrusion of sealant from the joint, infiltration of incompressibles into the joint) reflect on the *effectiveness of the sealant installation*. These are not, however, necessarily directly related to the *effects of the sealant installation on performance*. These effects may be evaluated by the following measures:

1. **Blowups and other pressure damage:** Although less common in short-jointed pavements than in long-jointed pavements, the infiltration of incompressibles into poorly sealed joints could conceivably lead to blowups at joints or cracks, or damage to bridge structures.

2. **Joint faulting:** Infiltration of water into transverse joints is believed to contribute to pumping of fines beneath slab corners and to the buildup of fines under slab corners, which results in faulting. Joint faulting is believed to be a major contributor to roughness in jointed concrete pavements.

3. **Joint spalling:** Infiltration of incompressibles into joints is one of the possible causes of spalling at transverse joints. Spalling may have two adverse consequences: increased pavement roughness, and increased repair costs.

4. **Roughness:** From the users´ perspective, spalling and faulting are concerns only if they increase pavement roughness. A roughness measure such as the International Roughness Index (IRI) may be used to assess the impacts of spalling and faulting on ride quality.

Joint Deflection as an Indicator of Future Performance

Even in relatively young pavements that have not yet manifested much visible distress such as spalling or faulting, differences in deflection response may warn of future differences in performance. Among the deflection parameters that should be considered in evaluating the possible effects of joint sealing on pavement performance are the following:

Differential deflection, i.e., the difference in deflections of the loaded and unloaded sides of the joint. The magnitude of differential deflection is a measure of the deficiency in load transfer capability of the joint, and is related to the potential for the development of spalling, faulting, and corner breaks.

1. **Deflection load transfer**, i.e., the ratio of deflections on the loaded and unloaded sides of the joint. Like differential deflection, deflection load transfer is a measure of the load transfer capability of the joint. Deflection load transfer, expressed as a percentage, is considered important in evaluation of jointed concrete pavement performance because of its relationship to stress load transfer (although it is rarely actually used for this purpose). For the purposes of predicting joint distresses such as spalling, faulting, and corner breaks, it is arguably not as useful a measure of load transfer capability as differential deflection. Deflection load transfer is useful, however, in scaling load transfer measurements between 0 and 100 percent and thereby making adjustments for joint deflections measured at different temperatures.

2. **Total deflection**: of the loaded and unloaded sides of the joint. This is an indicator of the quality of support to the slab in the vicinity of the joint. Even when differential deflections are low (i.e., load transfer is very high), the loaded and unloaded sides of the joint can deflect excessively if the slab is too thin and/or the quality of support to the slab is poor. Total joint deflection may be compared to interior slab deflections to identify locations where pumping, base densification, and/or eventual slab cracking may occur.

Description of LTPP Experiment SPS-4

The Long-Term Pavement Performance (LTPP) Special Pavement Study SPS-4 (Preventive Maintenance Effectiveness of Rigid Pavements) was originally designed to assess the effects of selected rigid pavement maintenance treatments on performance, relative to the performance of untreated control sections. The experiment design was developed by the Texas Transportation Institute, under Strategic Highway Research Program (SHRP) Highway Operations contracts.[6]

The experiment design for the main SPS-4 experiment incorporated the same primary experimental factors as in the General Pavement Study (GPS) experiments: climatic zone, subgrade type, and traffic level. The original experimental design for SPS-4 included two second-level factors: type of subbase (granular or stabilized), and condition at the time of treatment (good, fair, or poor). Treatment type was another experimental design factor: the two maintenance treatments that were to be considered were (1) joint and crack sealing, and (2) undersealing.

The matrix of cells for this experiment, as originally designed, could not be filled out because, as Reference 6 explains:

> "…few agencies were willing to provide sites for the rigid pavement preventive maintenance (SPS-4) study. A primary concern was the use of undersealing as a preventive maintenance treatment. The rigid pavement preventive maintenance study was modified to allow agencies to participate in installation of sections with joint/crack sealing and undersealing, with joint/crack sealing only, or with undersealing only. This modification increased participation, but not enough to sufficiently fill the experimental design."

As a result, the experiment design for SPS-4 was reduced to the following factors: climatic zone (temperature and moisture), subgrade type, and subbase type. The SPS-4 projects that are jointed plain concrete pavement (JPCP) are those located in Arizona, California, Colorado, Indiana, Iowa, Kansas, Kentucky, Ohio, Oklahoma, Nebraska, Nevada, South Dakota, Texas, and Utah.

However, most of the JPCP SPS-4 projects are not relevant to the objective of NCHRP Project 20-50(2), because most of those projects do not have test sections constructed with narrow unsealed joints. Most of the SPS-4 projects are pavements that were already in service at the start of the experiment. The "control" or "unsealed" test sections on these projects are in fact sections in which the joints were previously sealed or filled, and the sealant or filler was later either removed or destroyed.

In fact, there are just five SPS-4 sites that have JPCP test sections constructed with narrow unsealed joints and that are therefore relevant to the objective of NCHRP Project 20-50(2). These five sites are the following:

- Mesa, Arizona;
- Campo, Colorado;
- Tremonton, Utah;
- Salt Lake City, Utah; and
- Heber City, Utah.

The five sites that are relevant to the objective of NCHRP Project 20-50(2) are located in the dry western region of the United States. It is important therefore not to assume that the findings that may be drawn from the performance of the pavements at these sites can be extrapolated to other regions of the country that receive more precipitation. Although field experiments of JPCP with sealed and narrow unsealed joints have been constructed in several other States in wetter regions of the country, evaluation of these other experiments was not within the scope of NCHRP Project 20-50(2).

Research Approach

Given the limited nature of the relevant data in the SPS-4 experiment, the research conducted for NCHRP Project 20-50(2) has been focused not on a nationwide statistical analysis of sealed versus unsealed joints, but rather on a detailed project-level examination of the sealed versus unsealed joints at these five sites. The findings from this examination are threfore likely to be of most immediate interest to the States in which these five projects were constructed and to States with similar climatic conditions. In addition, it is hoped that the methods used in this examination may serve as useful examples for future analyses of other sealed-versus-unsealed joint experiments located in other climates.

The research conducted for NCHRP Project 20-50(2) is presented in this report in the following sequence:

- A review of findings from previous studies on the effects of unsealed joints on JPCP performance. This literature review is presented in Chapter 2 of this report.
- Development of analysis methods to be used in assessing the effects of unsealed joints on JPCP performance. These analysis methods are described in Chapter 3 of this report.
- A project-level evaluation of the relative performance of JPCP with unsealed and sealed joints at each one of the five selected sites. These project-level evaluations are presented in Chapter 4 of this report.
- Synthesis of conclusions on the relative performance of JPCP with unsealed and sealed joints, drawn from the project-level evaluations, and recommendations for further research. These conclusions and recommendations are presented in Chapter 5 of this report.

Chapter 2

Observations from Previous Studies

The findings of previous studies of LTPP experiments that have addressed the relative performance of JPCP with and without joint sealing are summarized in this chapter. A number of State studies of unsealed JPCP pavement performance are also available; however a review of these studies is not provided because they do not involve LTPP sections.

The studies summarized in this chapter examined the performance of JPCP with unsealed joints, although not necessarily designed and constructed as narrow unsealed joints. Although these findings do not necessarily reflect the performance of JPCP with narrow unsealed joints, they do reflect past findings that have shaped public opinion regarding the need for sealing joints.

Effect of Unsealed Joints on Joint Spalling

Joint spalling in the sealed and unsealed sections of nine SPS-4 sites was evaluated subjectively by four Expert Task Groups (one in each LTPP region), in field visits made in the summer and fall of 1995. These field visits are described in Reference 7. No performance data are presented in this report for the SPS-4 projects visited. The following general observations were made:

- "Unsealed joints in the control sections contain significantly more debris than sealed joint sections.
- "Unsealed joint sections have significantly more spalling than the sealed joint sections.
- "Minor amounts of debris have lodged in the sealed joint sections, with little or no effect on pavement performance to date."

Reference 8 presents the results of analyses of SPS-4 section performance conducted using data drawn from the LTPP database. The performance indicators analyzed were joint spalling, joint

faulting, longitudinal profile, joint deflections, and joint seal damage. With respect to joint spalling, Reference 8 states:

> "…there is not a significant difference between the spalling of the different treatments. However, the control section does show more spalling than the associated joint seal or the underseal sections, which is the expected outcome and corresponds to the observations made by the ETG [Expert Task Groups]."

Reference 8 concludes that in the study described, "strong evidence was collected to support the positive contribution of joint sealing to pavement performance." This conclusion is not supported, however, by the analysis results presented in Reference 8. In fact, no significant performance differences were detected between the pavements examined with sealed joints and those with unsealed joints.

Reference 9 describes the development of a regression model for JPCP joint spalling, using data from the LTPP GPS-3 experiment. Most, but not all, of the pavements in the GPS-3 experiment have sealed joints. There are some GPS-3 sections with unsealed joints. Reference 9 does not provide details on how many, if any, of these unsealed-joint pavements were designed and constructed to be left unsealed. The sensitivity of is joint spalling model to joint sealing and sealant type is shown in Figure 1.

Effect of Unsealed Joints on Joint Faulting

In the analysis of SPS-4 performance through 1995, reported in Reference 8, no significant differences in joint faulting were detected between sealed-joint and unsealed-joint sections.

Neither presence nor type of sealant was found to be significant in the regression analysis of JPCP joint faulting in GPS-3 pavements, described in Reference 9. In other statistical analyses of GPS-3 performance data, reported in Reference 10, neither sealant presence nor sealant type was found to be a significant variable in the prediction of dowelled or undowelled joint faulting in JPCP.

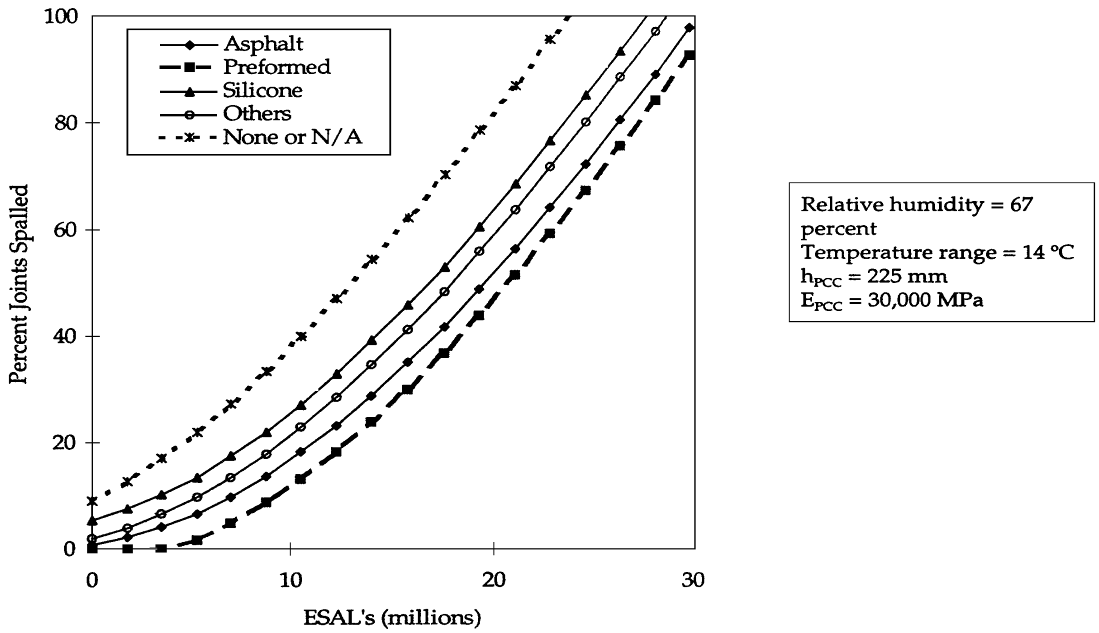


Figure 1. JPCP spalling model sensitivity to joint sealing and sealant type.[9]

Effect of Unsealed Joints on Roughness

In the analysis of SPS-4 performance through 1995, reported in Reference 8, no significant differences in roughness (as expressed by International Roughness Index) were detected between sealed-joint and unsealed-joint sections.

Neither presence nor type of sealant entered into the regression analysis of JPCP IRI in GPS-3 pavements, described in Reference 9.

Reference 10 presents a number of models, developed from LTPP GPS data, for IRI as a function of design features, as well as a model for JPCP for IRI as a function of distress (total faulting, spalling, and transverse cracking). The predictive capability (R^2) of this model and the relative significance (partial correlation coefficient) of the individual distresses such as faulting and spalling are not reported. Elsewhere in Reference 10, a strong positive correlation is reported between mean joint faulting and IRI. The IRI model does not appear to be sensitive to joint spalling: an increase from 0 to 100 percent spalled joints increases IRI by only 0.11 m/km.

Reference 11 presents analyses of trends in IRI data for the LTPP GPS experiments 1, 2, 3, 4, 5, 6, 7, and 9, as well as the SPS experiments 1, 2, 5, and 6. No analyses of SPS-4 IRI data are reported. For each GPS experiment, one or more models was developed for IRI as a function of design, traffic, climate, and soil variables, as well as backcasted initial IRI. Neither presence nor type of joint sealant entered into the IRI prediction models for the GPS-3 (JPCP) data set. For undoweled JPCP in the GPS-3 experiment, IRI was reported to be strongly related to faulting: 48 percent of the variation in observed IRI values could be explained by the magnitude of total faulting at joints and cracks.

Reference 12 describes additional analyses of GPS-3 data, following the research presented in Reference 11. Longitudinal profile data were analyzed to quantify the degree of slab curvature (due to curling, warping, etc.). Total faulting was found to be strongly related to the index of slab curvature, among the variables considered, followed by joint spacing, concrete elastic modulus, and subgrade moisture content. Similarly, the predictive capability of the model for IRI as a function of design and site variables was greatly improved when the slab curvature index was included. IRI was reported to be strongly related to joint faulting; no relationship of IRI to joint spalling was detected.

Effect of Unsealed Joints on Joint Deflections

Reference 8 reports on some attempts to analyze deflections measured on SPS-4 projects through 1995. This deflection analysis was focused primarily on the behavior of joints in the undersealed sections. With respect to the deflections measured in sealed-joint and unsealed-joint sections, the analysis was inconclusive. This is attributable to some of deflection data having been obtained at high temperatures when the joints were fully closed, no adjustments having been made for slab curling, and no statistical comparisons having been applied to the deflection measurements.

Summary of Observations from Previous Studies

The previous analyses of LTPP data summarized here have not found JPCP with sealed joints and JPCP with unsealed joints to perform differently in terms of spalling, faulting, or IRI. Joint faulting is reported to be a major contributor to roughness in JPCP, but joint spalling does not appear to contribute significantly to roughness in JPCP. Only one attempt has been made to analyze deflections measured on sealed joints and unsealed joints in the SPS-4 test sections, and the results of this analysis were inconclusive.

Chapter 3

Analysis Methods

The inventory and monitoring data for the five sites evaluated in this research were obtained from the three Microsoft Access databases incorporated in the DataPave 2.0 program, and a release of additional data, also in Microsoft Access format, from the LTPP Information Center. Additional climatic data were obtained from the National Oceanic and Atmospheric Administration (NOAA)´s database of U.S. divisional and station climatic data and normals.

The methods used to analyze the distress, roughness, and deflection data used in this study are described in this chapter. The statistical analysis methods used to assess significance of differences in behavior and performance are also described.

Distress Analysis

Measures of Joint Spalling

Joint spalling has been quantified by several measures in previous studies, including percent of joints spalled within a pavement section, total length of joint spalling, percent of total joint length spalled, and percent of individual joint length spalled. What severities of spalling are considered in these measures are sometimes unclear. For the purposes of this study, the following measures of joint spalling were used:

- **Weighted length of joint spalling**, an index that characterizes the overall transverse joint spalling condition of a pavement section as a weighted average of the length of low-, medium-, and high-severity joint spalling. Weighted length of spalling is calculated as:

$$WLS = 1(TLL) + 2(TLM) + 4(TLH)$$

where WLS = Weighted length of spalled joints, m
TLL = Total length of low-severity spalling, m
TLM = Total length of medium-severity spalling, m
TLH = Total length of high-severity spalling, m

- **Weighted number of spalled joints**, an index that characterizes the overall transverse joint spalling condition of a pavement section as a weighted average of the number of joints with low-, medium-, and high-severity joint spalling. Weighted number of spalled joints is calculated as:

$$WNS = 1(NLS) + 2(NMS) + 4(NHS)$$

where
WNS = Weighted number of spalled joints
NLS = Number of joints with low-severity spalling
NMS = Number of joints with medium-severity spalling
NHS = Number of joints with high-severity spalling

- **Number of medium- and high-severity spalled joints**, a measure that characterizes the amount of spalling on a project that (a) may be in need of repair, and (b) may be better correlated to pavement roughness than total spalling. This index is calculated simply as the sum of the numbers of medium- and high-severity spalled joints.

- **Length of medium- and high-severity spalled joints**, a measure that characterizes the length of joint spalling on a project that may be in need of repair. This index is calculated simply as the sum of the lengths of medium- and high-severity spalled joints.

The weighting coefficients used in these equations were selected based on a sensitivity analysis of the impacts of low-, medium- and high-severity joint spalling on pavement condition ratings calculated using the Pavement Condition Index (PCI) rating procedure. For sections with one specific type of distress, the PCI value can be computed as 100 minus the deduct value associated with that distress.

In this sensitivity analysis, varying densities (amounts) of joint spalling at each severity level were assigned to a representative pavement section to establish the PCI deduct value associated with that severity level. For each distress density considered, the deduct values for medium- and high-severity spalling were compared to the deduct value for low-severity spalling to compute associated deduct ratios. Figure 2 illustrates the trends of deduct ratios for the distress densities analyzed from which the coefficients of 2 and 4 were selected for weighting medium- and high-severity joint spalling, respectively.

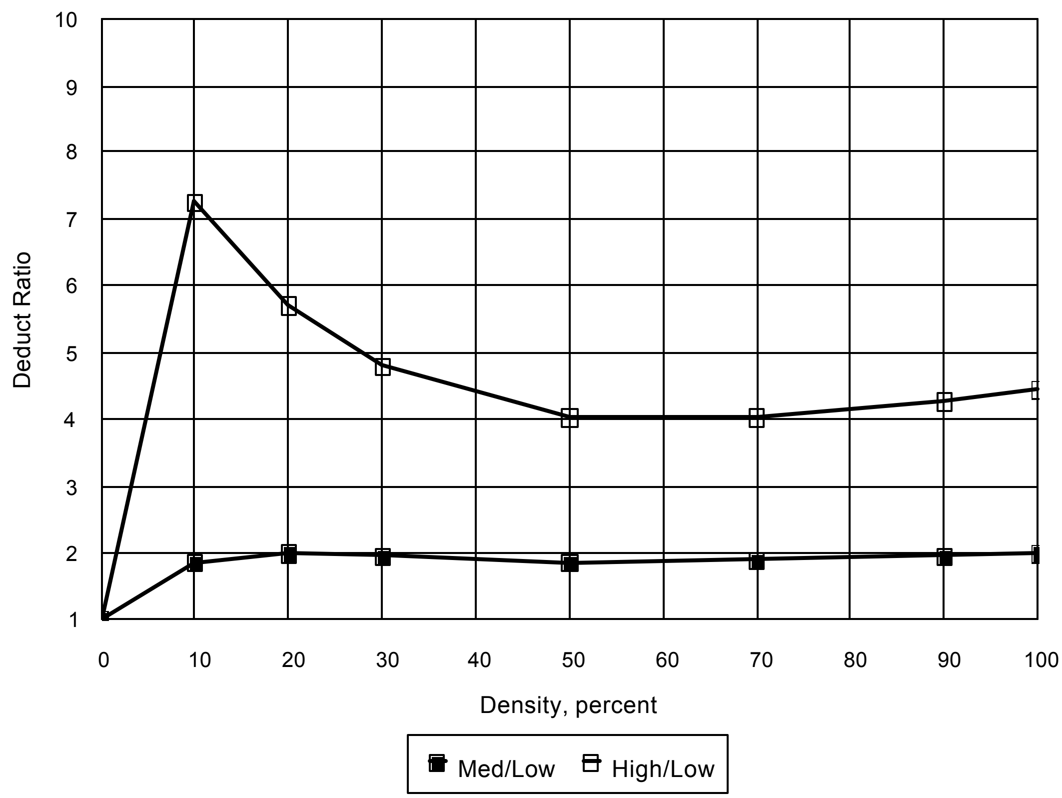


Figure 2. Weights for joint spalling severities based on analysis of Pavement Condition Index (PCI) deduct values.

In the above equations, severity levels for spalling are as defined by LTPP and reported in the LTPP database. It should be noted, however, that the definitions of joint spalling severities have not been consistent over the entire course of the LTPP study. The definitions of joint spalling severities are believed to have been modified one or more times, and these modifications have not been well documented.

Measures of Joint Sealant Damage

Joint sealant damage has also been quantified by various measures in previous studies. In detailed evaluations of joint sealant performance, several types of joint sealant distress (ahdesive failure, cohesive failure, oxidation, twisting, compression set, etc.) are recorded. However, in the test section monitoring being conducted in the LTPP studies, sealant damage is only given an overall rating of low, medium, or high severity.

An index of **weighted sealant damage** was used to quantify overall transverse joint sealant condition, as a weighted average of the numbers of joints within the section with low, medium, and high sealant damage ratings. Weights of 1, 2, and 4 were assigned to low-, medium-, and high-severity sealant damage, respectively, as shown below:

$$WSD = 1\left(\frac{NLSD}{TJ}\right) + 2\left(\frac{NMSD}{TJ}\right) + 4\left(\frac{NHSD}{TJ}\right)$$

where WSD = Weighted sealant damage (1 to 4)
NLSD = Number of joints rated with low-severity sealant damage
NMSD = Number of joints rated with medium-severity sealant damage
NHSD = Number of joints rated with high-severity sealant damage
TJ = Total number of joints rated

For example, a pavement section with joint sealant damage rated as low for all joints would have a weighted sealant damage index of 1, whereas a pavement section with joint sealant damage rated as high for all joints would have a weighted sealant damage index of 4. This index quantifies joint sealant damage within a confined 1 to 4 range, and gives more weight to higher severities of joint sealant damage.

The weighting coefficients used in the above equation were selected based on an analysis of the impacts of low-, medium- and high-severity joint sealant damage on pavement condition ratings calculated using the Pavement Condition Index (PCI) rating procedure. The PCI deduct values associated with low-, medium-, and high-severity joint sealant damage are 2, 4, and 8, respectively. The ratios of associated deduct values were used to establish the medium- and high-severity weighting coefficients of 2 (4/2) and 4 (8/2). Severity levels for joint sealant damage are as defined by LTPP and reported in the LTPP database.

Measures of Joint Faulting

Only one index of faulting, average joint faulting measured in the outer wheelpath, has been employed in most previous analyses of JPCP performance. Average joint faulting is arguably not the best available measure of faulting when the focus of the analysis is on the relationship between joint faulting and roughness. One reason for this is that negative faulting (the approach slab edge being lower than the leave slab edge) does sometimes occur, as a result of slab curling and/or warping. A simple arithmetic averaging of positive and negative joint faulting measurements

produces an average that does not really express the amount of roughness present. For this reason, average joint faulting was expressed in this study by a parameter called **average absolute faulting**. Absolute average faulting is calculated as the arithmetic average of the absolute values of the individual joint faulting measurements.

The second drawback to correlating average joint faulting to roughness is that the total amount of roughness present is a function not only of the magnitudes of the faults at the joints but also the joint spacing. For example, if two sections of the same length had the same average joint faulting, but the first section had a joint spacing longer than that of the second section, then the first section would have less roughness due to faulting than the second section. Again, the magnitudes of both negative and positive faults need to be considered appropriately in the calculation of the amount of roughness due to faulting. An appropriate parameter for this purpose is **total absolute faulting**, which is the arithmetic sum of the absolute values of the individual joint faulting measurements.

In general, average joint faulting (or average absolute faulting) is an inadequate indicator of relative roughness when comparing pavement sections of similar lengths but different joint spacings, while total faulting (or total absolute faulting) is an inadequate indicator of relative roughness when comparing pavement sections of similar joint spacings but different lengths.

In this study, joint spacing was constant within a site (or at least, in the case of sites with a repeating joint spacing pattern, mean joint spacing was constant). Furthermore, no comparisons of faulting among different sites (with different joint spacings) were conducted. On the other hand, the lengths of the test sections being compared were not always constant within a site. Therefore, average absolute faulting was selected as the faulting parameter for the purposes of this study.

Roughness Analysis

The roughness parameter used in this study is the International Roughness Index (IRI), which is calculated from the measured profile. It has been noted that IRI is not necessarily the roughness parameter that most closely correlates to the user's perception of ride quality. It may, however, be the best roughness parameter available in the LTPP database.

Deflection Analysis

It should be kept in mind that the pavement sections at the five study sites are relatively young and so far have exhibited little joint spalling and faulting. Conclusions that might be drawn at this time about the significance of any differences in distress or roughness among the test sections must be considered in that light: some significant performance differences may be manifested in the future, but not be evident yet. For that reason, it was considered important to perform a careful and thorough of the joint deflections measured on these test sections. Differences in joint deflection behavior may not only be related to currently observed differences in faulting or roughness, but may also be indicators of future performance differences.

Deflection Data Retrieval

The deflection data items extracted from the LTPP database for use in this analysis were the following:

- Testing date and time,
- Test location (position on the slab),
- Applied loads,
- Deflection sensor configurations,
- Peak deflection data,
- Air and pavement surface temperatures,
- Internal slab temperatures and depths of measurement, and
- Slab length data.

Analysis of Deflection Data

The deflection data collected at the five study sites permit several analyses of slab and foundation support parameters. Analysis of center slab (interior) deflections allows for the determination of dynamic foundation support, elastic modulus of the concrete slabs (based on a known or assumed thickness), and/or the effective thickness of the concrete slabs (based on an assumed modulus of elasticity). Analysis of transverse joint deflection data allows for the determination of deflection load transfer efficiency, total and differential edge deflections, dynamic edge foundation support, and/or edge support uniformity.

The use of multiple load levels allows for the allows for the analysis of response linearity as well as incremental load/deflection response. The incremental analysis provides a means for differentiating nonlinearities due to poor foundation support and/or temperature curling.

Analysis of Interior Slab Deflections

The interior deflections collected at the center-slab (J6) test location were used to determine the dynamic foundation support k-value and the effective thickness of the concrete slabs, based on an assumed modulus of elasticity of 5,000,000 psi. Separate analyses were conducted with and without the use of the deflections recorded at the center of the loading plate, to provide a comparative measure of slab compression effects.

Initially, interior deflections were used to compute the deflection basin AREA using the following equations:

$$AREA4 = \frac{6}{d_0}\left(d_0 + 2\, d_{12} + 2\, d_{24} + d_{36}\right) \qquad \text{Eqn 1}$$

$$AREA5 = \frac{3}{d_{12}}\left(1 + 2\, d_{18} + 3\, d_{24} + 6\, d_{36} + 4\, d_{60}\right) \qquad \text{Eqn 2}$$

where: AREA = deflection basin AREA, inches

d_i = surface deflection measured at i inches from the load center

The calculated AREA values were then used to backcalculate initial estimates of the radius of relative stiffness of the pavement system (dense-liquid foundation model), using the following equations:

$$l_{k4\text{-}est} = \left[\frac{\ln\left(\frac{36 - AREA4}{1812.279} \right)}{-2.559} \right]^{4.387} \qquad \text{Eqn 3}$$

$$l_{k5\text{-}est} = \left[\frac{\ln\left(\frac{48 - AREA5}{158.40} \right)}{-0.476} \right]^{2.220} \qquad \text{Eqn 4}$$

where: $\ell_{k\text{-}est}$ = estimated dense-liquid radius of relative stiffness, inches

The dense-liquid radius of relative stiffness is a combined term that incorporates both slab and subgrade properties and may be computed by the equation:

$$l_k = \sqrt[4]{\frac{E_c H_c^{3}}{12(1-\mu_c^{2})k}} \qquad \text{Eqn 5}$$

where: E_c = elastic modulus of concrete slab, psi
H_c = thickness of concrete slab, inches
μ_c = Poisson's ratio of concrete slab (assumed = 0.15)
k = dynamic subgrade k-value, psi/in

Initial estimates of the dynamic interior foundation k-value, based on infinite slab size assumptions, were then backcalculated using the following equations:

$$k\,4_{est} = \frac{0.1245\, e^{\left[-0.14707\, e^{\left(-0.07565\, l_{k4\text{-}est}\right)}\right]} P}{d_0\, l_{k4\text{-}est}^{2}} \qquad \text{Eqn 6}$$

$$k\,5_{est} = \frac{0.12188\, e^{\left[-0.79432\, e^{\left(-0.07074\, l_{k5\text{-}est}\right)}\right]} P}{d_{12}\, l_{k5\text{-}est}^{2}} \qquad \text{Eqn 7}$$

where k_{est} = estimated dynamic interior foundation k-value, psi/in
P = applied load, lb
D_I = utilized maximum deflection at i inches from the load, inches
$P_{k\text{-est}}$ = estimated radius of relative stiffness, inches

Based on the size of the slab tested, slab size correction factors for the estimated radius of relative stiffness and utilized maximum deflection (d_0 or d_{12}) were computed using the following equations:

$$CF_{lk\text{-}est} = 1 - 0.89434 \exp\left[-0.61662 \left(\frac{L_{eff}}{l_{k\text{-}est}}\right)^{1.04831} \right] \qquad \text{Eqn 8}$$

$$CF_{Di} = 1 - 1.15085 \exp\left[-0.71878 \left(\frac{L_{eff}}{l_{k\text{-}est}} \right)^{0.80151} \right] \qquad \text{Eqn 9}$$

where: $CF_{Pk\text{-}est}$ = correction factor for $P_{k\text{-}est}$
CF_{Di} = correction factor for utilized maximum interior deflection
L_{eff} = effective slab length, inches

The effective slab length was computed based on the length and width of the test slab, using the equation:

$$L_{eff} = \sqrt{L_s * L_w} \qquad \text{Eqn 10}$$

where: L_s = slab length, inches
L_w = slab width, inches

After computation of slab size correction factors, the adjusted radius of relative stiffness and dynamic foundation k-value were computed using the following equations:

$$l_{k\text{-}adj} = CF_{lk\text{-}est} * l_{k\text{-}est} \qquad \text{Eqn 11}$$

$$k_{adj} = \frac{K_{est}}{CF_{lk\text{-}est}^{\ 2}\, CF_{Di}} \qquad \text{Eqn 12}$$

After backcalculation of the adjusted radius of relative stiffness and dynamic foundation k-value, the effective thickness of the concrete slab was estimated from Eqn 5 as follows:

$$H_c = \sqrt[3]{\frac{11.73\, l_{k\text{-}adj}^{\ 4}\, k_{adj}}{E_c}} \qquad \text{Eqn 13}$$

where: H_c = effective slab thickness, inches
E_c = assumed slab modulus of elasticity, psi (= 5,000,000 psi)

Incremental analysis of deflection response was also conducted to provide a means of differentiating slab curling from poor foundation support. For those cases where the slab temperature gradient (top temperature - bottom temperature) is excessively positive and foundation support stiffness is high, the center of the slab may be lifted off the foundation. In these cases, the maximum deflection and the deflection basin AREA term increase, resulting in a reduced backcalculated foundation k-value and an increased backcalculated effective slab thickness. If, however, at least two of the load levels used during testing were sufficient to create maximum surface deflections exceeding the depth of curling-induced voids, incremental analysis should indicate an increased dynamic foundation k-value and a decreased effective slab thickness as compared to values backcalculated from individual load/deflection pairs.

For the purposes of this incremental analysis of interior deflections, the incremental maximum interior deflection and loading were computed as:

$$D_{inc} = \frac{D_{P2} - D_{P1}}{P2 - P1} \qquad \text{Eqn 14}$$

$$P_{inc} = P2 - P1 \qquad \text{Eqn 15}$$

where:
- D_{inc} = incremental maximum interior deflection, inches
- P_{inc} = incremental load, lb
- D_{P2} = maximum interior deflection at highest load level, inches
- D_{P1} = maximum interior deflection at second highest load level, inches
- P2 = maximum load level, lb (approximately 17,000 lb)
- P1 = second highest load level, lb (approximately 12,000 lb)

The incremental maximum interior deflections and loadings were then used in conjunction with Eqns 6, 7 and 13 to compute the incremental dynamic k-value and incremental effective slab thickness.

Analysis of Transverse Joint Deflections

The transverse joint deflections at slab positions J4 and J5 (approach side of joint and leave side of joint) were used to determine the joint load transfer efficiency, normalized total and differential edge deflection, dynamic edge foundation support, and transverse edge slab support ratios.

The transverse joint deflection load transfer efficiency is computed as the simple ratio of unloaded and loaded slab deflections, expressed as a percentage using the equation:

$$LT_d = \frac{D_u}{D_L} \, x \, 100\% \qquad \text{Eqn 16}$$

where: LT_d = deflection load transfer, %
D_U = unloaded slab deflection, mils (12 inches from the load center)
D_L = loaded slab deflection, mils (at the center of loading)

Deflection load transfer efficiency provides a measure of the competence of dowel bar and/or aggregate interlock interactions to effectively transfer edge loadings between adjacent slabs. In general, deflection load transfer efficiencies of 85 percent or greater are anticipated for effectively doweled joints.

The normalized total edge deflection is computed as the simple addition of unloaded and loaded slab deflections, normalized to a common load level of 9,000 lb, using the equation:

$$DT_e = \frac{9000 * (D_U + D_L)}{P} \qquad \text{Eqn 17}$$

where: DT_e = total edge deflection, inches, normalized to 9-kip load
D_U = unloaded slab deflection, inches (12 inches from the load center)
D_L = loaded slab deflection, inches (at the center of loading)
P = applied load, lb

The normalized total edge deflection should remain relatively constant regardless of available deflection load transfer, provided that slab thickness, elastic modulus, and foundation support remain constant. The total edge deflection can be used as a relative indicator of the overall edge structural capacity of a test section as well as an input for the backcalculation of edge foundation support.

The normalized differential edge deflection is computed as the simple subtraction of loaded and unloaded slab deflections, normalized to a common load level of 9,000 lb, using the following equation:

$$dD_e = \frac{9000 * (D_L - D_U)}{P} \qquad \text{Eqn 18}$$

where: dD_e = differential edge deflection, inches, normalized to 9-kip load
D_L = loaded slab deflection, inches (at the center of loading)
D_U = unloaded slab deflection, inches (12 inches from the load center)
P = applied load, lb

The differential edge deflection can be used as a relative indicator of the independent slab edge movement under loading, which may lead to spalling of the transverse joints.

The edge foundation support is backcalculated based on the assumption that each test slab is of uniform thickness and elastic modulus, using the following equation:

$$k_e = \frac{D_K}{\left[0.82\, a + \sqrt{\frac{2.32\, DT_e\, D_K}{P}} \right]^4} \qquad \text{Eqn 19}$$

where: k_e = transverse edge foundation k-value, psi/in
D_K = slab bending stiffness modulus, lb-in (= $P_{k4\text{-}adj}{}^4 * k_{4\text{-}adj}$)
a = radius of load plate, inches (= 5.9055 in)
DT_e = normalized total edge deflection, inches
P = normalized load value (= 9,000 lb)

The uniformity of support under the transverse edge, herein called the transverse edge slab support ratio, is computed as the ratio of backcalculated edge to interior dynamic foundation k-values using the equation:

$$SSR_e = \frac{k_e}{k_{4\text{-}adj}} \qquad \text{Eqn 20}$$

where: SSR_{et} = transverse edge slab support ratio
k_e = transverse edge foundation k-value, psi/in
$k_{4\text{-}adj}$ = interior foundation k-value of the same test slab, psi/in

In general, edge slab support ratios less than approximately 0.75 are indicative of slabs with poor edge support due to foundation densification/pumping and/or temperature curling.

Incremental analysis of transverse edge deflection response was also conducted to provide a means of differentiating slab curling from poor foundation support. For the purposes of this incremental analysis of edge deflections, the incremental normalized total edge deflection was computed as:

$$DT_{e\text{-}inc} = \frac{9000 * \left(DT_{e\text{-}P2} - DT_{e\text{-}P1}\right)}{P2 - P1} \qquad \text{Eqn 21}$$

where:
$DT_{e\text{-}inc}$ = incremental normalized (9-kip) total edge deflection, inches
$DT_{e\text{-}P2}$ = total transverse edge deflection at maximum load level, inches
$DT_{e\text{-}P1}$ = total transverse edge deflection at second highest load level, inches
P2 = highest load level, lb (approximately 17,000 lb)
P1 = second-highest load level, lb (approximately 12,000 lb)

The incremental normalized total edge deflection was then used in conjunction with Eqns 19 and 20 to compute the incremental transverse edge slab support ratio. In those cases where temperature curling alone was responsible for poor support, incremental slab support ratios should increase over those computed based on individual load levels, provided at least two load levels produced sufficient total edge deflection to close any curl-induced voids.

Statistical Analysis

The experimental test sections at the five SPS-4 supplemental study sites incorporate a wide range of combinations of joint forming method, joint width, and joint sealant type. The global matrix of joint designs is illustrated in Table 1. Those groups of cells that are not empty for all five of the sites are identified by the letters A through L. All other cells (representing joint design/sealant combinations not constructed at any of the five study sites) are shaded out.

Each of the five sites has some different combinations of joint design and joint sealant type (including “unsealed”), with the result that not all groups A through L are nonempty for all five sites. Also, the nonempty groups have different numbers of test sections at the different sites. A summary of which groups are nonempty at each site, and how many test sections is in each nonempty group, is provided in Table 2. These differences in the experimental design from site to site make it difficult to conduct an appropriate and meaningful statistical across all five sites.

Table 1. Global matrix of joint sealant-versus-design groups represented in the test sections at the the five SPS-4 supplemental study sites.

<table>
<tr><th colspan="2" rowspan="2"></th><th colspan="4">Conventional Saw</th><th>Soff-Cut Saw</th></tr>
<tr><th>3 mm</th><th>6 mm</th><th>9 mm</th><th>9 mm bevel</th><th>3 mm</th></tr>
<tr><td colspan="2">Unsealed</td><td>A</td><td></td><td></td><td></td><td>B</td></tr>
<tr><td rowspan="4">Asphalt</td><td>Crafco RS 221</td><td></td><td></td><td rowspan="4">C</td><td></td><td></td></tr>
<tr><td>Crafco SS 444</td><td></td><td></td><td></td><td></td></tr>
<tr><td>Koch 9005</td><td></td><td></td><td></td><td></td></tr>
<tr><td>Koch 9012</td><td></td><td></td><td></td><td></td></tr>
<tr><td rowspan="7">Silicone</td><td>Crafco 902</td><td rowspan="7">D</td><td rowspan="7">E</td><td rowspan="7">F</td><td rowspan="7">G</td><td rowspan="7">H</td></tr>
<tr><td>Crafco 903-SL</td></tr>
<tr><td>Dow 888</td></tr>
<tr><td>Dow 888-SL</td></tr>
<tr><td>Dow 890-SL</td></tr>
<tr><td>Mobay 960</td></tr>
<tr><td>Mobay 960-SL</td></tr>
<tr><td rowspan="7">Neoprene</td><td>DS Brown E-437H</td><td></td><td rowspan="7">I</td><td rowspan="7">J</td><td></td><td></td></tr>
<tr><td>DS Brown V-687</td><td></td><td></td><td></td></tr>
<tr><td>DS Brown V-812</td><td></td><td></td><td></td></tr>
<tr><td>Kold Seal Neo Loop</td><td></td><td></td><td></td></tr>
<tr><td>Esco PV 687</td><td></td><td></td><td></td></tr>
<tr><td>Watson Bowman 687</td><td></td><td></td><td></td></tr>
<tr><td>Watson Bowman 812</td><td></td><td></td><td></td></tr>
<tr><td>Polysulfide</td><td>Koch 9050-SL</td><td></td><td></td><td>K</td><td></td><td></td></tr>
<tr><td>Proprietary</td><td>Roshek</td><td></td><td></td><td>L</td><td></td><td></td></tr>
</table>

Table 2. Number of test sections in each joint sealant-versus-design group, at each SPS-4 supplemental study site.

Group	Mesa, AZ	Campo, CO	Tremonton, UT	Salt Lake City, UT	Heber City, UT
A	2	2	2	2	2
B			1	2	2
C	4		4	4	4
D	2	4	2	2	2
E	2	4			
F	10	5	6	4	4
G		2			
H			1	2	2
I		1	2	2	2
J	4	1	2	2	2
K				2	2
L			1		
Number of test sections	24	19	21	22	22
Number of nonempty groups	6	7	9	9	9

Statistical Analysis Approaches

Because the global matrix illustrated in Table 1 has several empty cells, it does not represent a full experimental factorial, which rules out a two-way (sealant presence/type versus joint design) analysis of variance (ANOVA).

Two separate one-way ANOVAs, one for sealant presence/type and another for joint design, may seem a feasible alternative. A one-way ANOVA tests whether the experimental factor being tested gives rise to significantly different responses among the treatment groups: that is, it seeks to determine whether all the observations are from one population, or from different populations. The limitation to the one-way ANOVA approach to analyzing the treatment effects of interest here is that an ANOVA willl determine whether at least one significant difference exists somewhere among the treatment means being compared, but it will not identify which differences are significant.

A similar statistical analysis approach is the use of a multiple comparison test method. Among the multiple comparison methods available are (in approximate decreasing order of sophistication and robustness) those associated with the names Newman-Keuls, Tukey, Fisher, Scheffé, and Duncan. These multiple comparison methods differ in their details, but in general they all seek to determine whether or not any significant differences exist among a set of more than two treatments.

In general, the multiple comparison tests mentioned previously are appropriate when analyzing a set of treatments among which no specific comparisons are of particular interest. The confidence interval represented by the multiple comparison test is sufficiently broad that it can encompass all possible pairwise comparisons that could conceivably be conducted between treatment means and combinations of treatment means.

This is not the case, however, in NCHRP Project 20-50(2). This study is concerned with specific comparisons, namely, how the performance of pavement sections with conventionally sawn narrow unsealed joints (group A) compares with the performance of pavement sections in the other nonempty groups (B through L). For any one performance indicator, such as average absolute joint faulting, at any given SPS-4 supplemental study site with *k* nonempty groups (one of which is always A), there are *k-1* possible pairwise comparisons of group A performance with the performance of other groups. The conclusions that may be drawn from these comparisons can be contained within a narrower overall confidence interval than that which would encompass all possible comparisons, i.e., comparisons among groups B through L as well.

Overall Confidence Level

It is important to note that if one wishes to make several comparisons among treatment means, and one wishes to have a certain overall level of confidence in the set of conclusions drawn from these comparisons, each individual comparison must be associated with a higher level of confidence than the desired overall level of confidence. The individual level of confidence required depends on the overall level of confidence desired and the number of comparisons planned.

For example, suppose one desires to conduct three comparisons of three individual treatment means: $:_1$ versus $:_2$, $:_1$ versus $:_3$, and $:_2$ versus $:_3$. From each of these three comparisons, one may draw a conclusion about whether or not the two means are significantly different. If these individual comparisons where each conducted at a confidence level of 95 percent, the overall confidence level, that is, how confident one can be that all three conclusions hold simultaneously, is $(0.95)^3$ = 0.857, or 85.7 percent. If one wishes to achieve an overall confidence level of 95 percent for a set of *n* comparisons, one must first determine the individual confidence level *1-∀*

such that $(1-\forall)^n$ = 0.95. For n = 3 comparisons, the individual confidence level $1-\forall$ must be 0.983.

Similarly, if a given SPS-4 supplemental study site has k nonempty groups in the experimental matrix, so that $k-1$ comparisons with group A (narrow unsealed joints) are of interest, the required individual confidence level $1-\forall$ can be calculated such that $(1-\forall)^{(k-1)}$ = 0.95 or whatever overall confidence level is desired.

The number of test section groups (k) for each of the five SPS-4 supplemental study sites, and the individual confidence level ($1-\forall$) required to obtain an overall confidence level of 95 percent, is summarized in Table 3 below.

Table 3. Confidence levels required for statistical tests of SPS-4 supplemental study site data.

SPS-4 site	Test section groups (k)	Confidence level ($1-\forall$)
Mesa, AZ	6	0.990
Campo, CO	7	0.991
Tremonton, UT	9	0.994
Salt Lake City, UT	9	0.994
Heber City, UT	9	0.994

Statistical Analysis Steps

For each of the five SPS-4 supplemental study sites, and for each of several distress, roughness, and deflection parameters, the objective of the statistical analysis is to determine whether or not the pavement test sections with narrow unsealed joints (group A) perform significantly differently than the pavement test sections in each of the other groups. This analysis proceeds according to the following steps.

1. For each test section, if the comparison parameter is determined from several individual measurements, the sample mean and sample variance of the measurements are calculated. Each test section sample variance s^2 is calculated from the following equation:

$$s^2 = \frac{E\,(x_i - \overline{x})^2}{(n-1)} \qquad \text{Eqn 22}$$

where x_i = individual measurement i

$\overline{x}$ = average of individual measurements

n = number of individual measurements

2. For each nonempty group, the group mean is calculated as the mean of the *m* test section sample means within the group, and the group, or pooled, variance according to the following equation:

$$s_p^2 = \frac{(n_1 - 1)\, s_1^2 + (n_2 - 1)\, s_2^2 + \ldots (n_m - 1)\, s_m^2}{(n_1 - 1) + (n_2 - 1) + \ldots\, (n_m - 1)} \qquad \text{Eqn 23}$$

3. For each possible pair of group A with a nonempty group j, the pooled variance of the two groups is calculated from the following equation:

$$s_p^2 = \frac{(n_A - 1)\, s_A^2 + (n_j - 1)\, s_j^2}{(n_A - 1) + (n_j - 1)} \qquad \text{Eqn 24}$$

4. For each nonempty group, a *t* test is conducted, at the preselected individual confidence level 1-∀, to determine whether that group's mean is significantly different than the group A mean. The null hypothesis is that the difference between the two means is zero. The null hypothesis is rejected, with ∀ chance of being rejected erroneously (type I error), if the absolute value of the *t* statistic, calculated from the following equation:

$$t_{calc} = \frac{(\overline{X}_A - \overline{X}_j)}{\sqrt{s_p^2 \left[(1/n_A) + (1/n_j) \right]}} \qquad \text{Eqn 25}$$

exceeds the positive value of $t_{\forall/2,(nA+nj-1)}$ (available from a table of percentage points of the t distribution, or as a built-in function in Microsoft Excel and similar spreadsheet programs), beyond which there is ∀ percent chance that the absolute value of t_{calc} could fall if the difference between the two means were not significantly different than zero.

5. For the site and the comparison parameter in question, the *k-1* individual conclusions that are drawn, each at a confidence level of *1-∀*, about the significance of differences between the mean of group A and the mean of each of the other *k-1* groups, can be stated together with a simultaneous confidence level of $(1-\forall)^{(k-1)}$.

Statistical Analysis Results

The results of the the statistical analyses of the monitoring data from the Mesa, Campo, Tremonton, Salt Lake City, and Heber City sites are provided in full in Appendices A, B, C, D, and E, respectively. These statistical analysis results are summarized in Chapter 4. Each appendix contains the following tables of information by test section group:

- Joint spalling and joint sealant damage group means,
- Total absolute and average absolute joint faulting group means (note: faulting means and standard deviations are reported in millimeters),
- Average absolute joint faulting group statistics,
- Average absolute joint faulting group statistical analysis,
- IRI group means (note: the IRI values shown are the averages, over five runs, of the averages of the inner and outer wheelpath IRIs),
- IRI group statistics,
- IRI group statistical analysis,
- Deflection parameter group means (dynamic k value, effective thickness, load transfer, total deflection, differential deflection, and edge support ratio),
- Deflection parameter group statistics, and
- Deflection paramenter group statistical analysis.

Chapter 4

Assessment of Unsealed Joint Effects on Performance Of Selected SPS-4 Sites

A project-level evaluation of the effects of unsealed joints on JPCP performance at each one of the five SPS-4 supplemental study sites is presented in this chapter. The objective of the project-level evaluation is to assess whether the performance of the test sections with unsealed narrow joints differs significantly from the performance of the other test sections at the same site.

In addition to the data sources mentioned earlier, information on the design and construction of the pavement at these five sites was obtained from the the LTPP report on the performance of joints sealants in the SPS-4 supplemental sections, and a related Transportation Research Board paper.[13,14] Detailed information on the design and construction of the Mesa, Arizona site was also obtained from the Arizona DOT's construction report for this experiment.[15]

Reference 13 reports on field evaluations of the joint sealant installations in the supplemental SPS-4 sections. The report details the sealant material and joint sawcut/reservoir configurations used in each of the supplemental sections, and details the performance of the sealant/configuration combinations for each site. Sealant performance was evaluated in terms of several specific sealant distresses (adhesion loss, tensile failure, sliver spalls, etc.). These field evaluations did not include measurement of ride quality, deflections, or faulting.

The term "sliver spall" is used in Reference 13 to refer to thin vertical spalling of the joint face attributable to damage during sawcutting and/or low concrete strength. Sliver spalling is distinguished from the larger-scale, predominantly horizontal spalling that emanates from joints and is commonly attributed to intrusion of incompressibles.

Although Reference 13 does not make any comparisons between the performance of unsealed joints and sealed joints, the data provided in the report show the occurrence of sliver spalling in unsealed-joint sections to be no different than in sealed-joint sections. The occurrence of sliver spalling is not believed to be related to the presence or type of sealant.

Reference 13 provides detailed performance data for the different types of sealants and joint sealant configurations used in the supplemental SPS-4 sections. The monitoring of these sections demonstrate that not all sealed joints are well sealed, and that the performance of unsealed joints should be compared against the performance that can realistically be expected to be achieved from the different joint sealant types and configurations in use.

Mesa, Arizona

The Arizona SPS-4 supplemental experiment is located on the Superstition Freeway (U.S. 60) in Mesa, Arizona, an eastern suburb of Phoenix. An illustration of the location and key location data are given in Figure 3.[1] Thirty-year average monthly temperature and precipitation normals for the weather station nearest the site are given in Figure 4. The site receives an average of 9.5 inches of precipitation annually, and the temperatures range from an average low of 38 °F in January to an average high of 105 °F in July.

This section of the freeway is a six-lane divided highway with concrete shoulders. The pavement is a 13-inch JPCP on 4 inches of aggregate base. The joints are undowelled, skewed, and spaced at 13, 15, 17, and 15 ft. The subgrade soils along the alignment are predominantly silty sand to sandy silt (Unified soil classifications SM to ML), which are uniform to a depth of at least 10 feet.

The experimental test sections are located in the eastbound lanes. The layout of the test sections is illustrated in Figure 5. The positions of these test sections in the global experimental matrix described earlier are illustrated in Table 3.

For the sake of consistency with the grouping of experimental test sections used in Reference 13, and for consistency in describing the experimental designs at the five different study sites, the joint widths are expressed in Table 4 in millimeters. The actual joint initial sawcuts were nominally 1/8 inch (this width corresponds to the column in Table 4 labelled “3 mm”). For test sections in which secondary sawing of sealant reservoirs was done, the nominal joint reservoir widths were 1/4 inch or 3/8 inch (indicated by “6 mm” or “9 mm” in Table 3.) Reference 15 provides detailed data on measurements of the joint widths during construction, and comparison of these as-constructed joint widths with the as-designed joint widths.

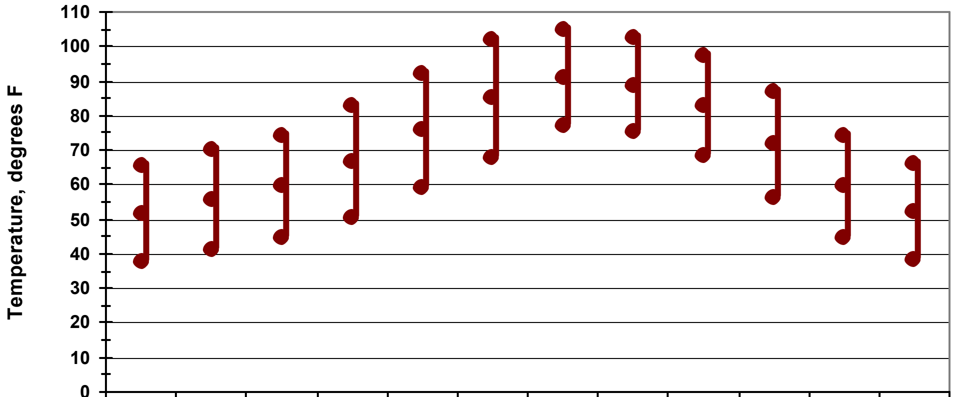
[1] All maps in this report were generated using *Microsoft Streets and Trips 2001*, copyright 1988-2000 Microsoft Corporation and/or its suppliers.

Location

SRHP ID	**04A400**	Linked GPS site	**47613**
State	**Arizona**	Route	**US 60**
County	**Maricopa**	Nearest city or town	**Mesa**
Latitude	**33.39 N**	Longitude	**111.84 W**

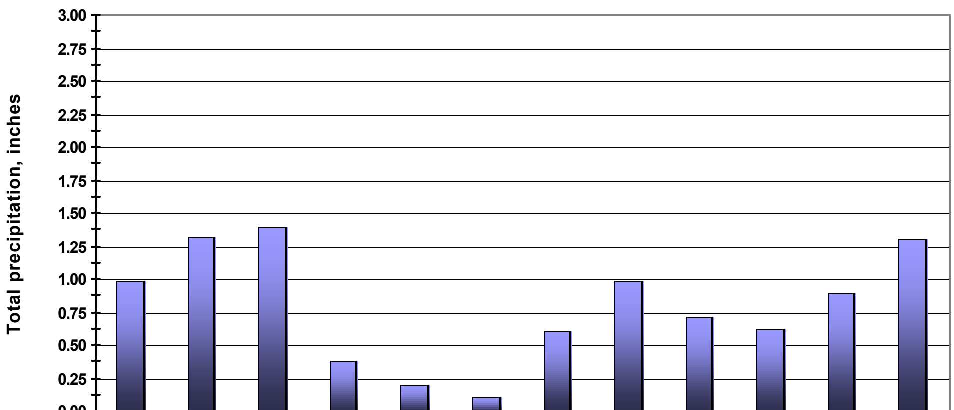
Location Notes: Eastbound on Route 60 (formerly numbered 360), between Power Road (exit 188) and Ellsworth Road (exit 191).

Figure 3. Location of Mesa, Arizona SPS-4 supplemental study site.



	J	F	M	A	M	J	J	A	S	O	N	D
Max	65.5	70.2	74	82.8	92.1	101.9	104.6	102.2	97.3	87	74.3	65.9
Mean	51.7	55.7	59.4	66.7	75.6	84.9	90.7	88.7	83	71.6	59.6	52.2
Min	37.9	41.2	44.7	50.6	59.1	67.9	76.8	75.1	68.6	56.2	44.8	38.5

Month



	J	F	M	A	M	J	J	A	S	O	N	D
Mean	0.99	1.32	1.39	0.38	0.19	0.11	0.61	0.98	0.71	0.62	0.89	1.31

Month

Climate

SRHP ID	**04A400**	Weather station ID	**22462**	
State	**Arizona**	Weather station name	**Deer Valley**	
County	**Maricopa**	Weather station latitude	**33.35 N**	
Nearest city or town	**Mesa**	Weather station longitude	**112.05 W**	
Mean annual temperature	**70.0** °F	Mean annual precipitation	**9.5**	in

Climate Notes: Data source is 1961-90 Monthly Station Normals, U.S. Divisional and Station Climatic Data and Normals, 1994, National Oceanic and Atmospheric

Figure 4. Mesa, Arizona temperature and precipitation normals.

US-60 Eastbound:

Eastbound

	SHRP ID	joint width (mm)	saw	sealant type	length (ft)
12	04A450	9	riding	Dow 890-SL	361
11	04A449	9	riding	Crafco RS-221	375
10	04A448	9	riding	Mobay 960-SL	373
9	04A447	3	riding	Dow 890-SL	375
8	04A446	9	riding	Crafco SS-444	375
7	04A445	9	riding	Dow 888	373
6	04A444	9	riding	Dow 888-SL	375
5	04A443	9	riding	Watson Bowman 687	359
4	04A442	9	riding	Dow 890-SL	375
3	04A430	3	riding	Unsealed	568
2	04A410	9	riding	Crafco 903-SL	511
1	04A441	9	riding	DS Brown V-687	374
					1954
24	04A462	6	riding	Dow 890-SL	360
23	04A461	9	riding	Crafco SS-444	373
22	04A460	9	riding	Watson Bowman 812	375
21	04A459	9	riding	Crafco RS-221	377
20	04A458	9	riding	Crafco 903-SL	375
19	04A457	9	riding	Dow 888-SL	373
18	04A456	9	riding	Dow 890-SL	371
17	04A455	3	riding	Unsealed	374
16	04A454	9	riding	Mobay 960-SL	373
15	04A453	9	riding	Dow 888	375
14	04A452	9	riding	DS Brown V-687	376
13	04A451	3	riding	Dow 890-SL	385

Figure 5. Mesa, Arizona SPS-4 supplemental test section layout.

Table 4. Positions of Mesa, Arizona SPS-4 supplemental test sections in global experimental matrix.

		Conventional Saw				Soff-Cut Saw
		3 mm	6 mm	9 mm	9 mm bevel	3 mm
Unsealed		A 04A430-03 04A455-17				B
Asphalt	Crafco RS 221			C 04A449-11 04A459-21		
	Crafco SS 444			04A446-08 04A461-23		
	Koch 9005					
	Koch 9012					
Silicone	Crafco 902	D	E	F	G	H
	Crafco 903-SL			04A410-02 04A458-20		
	Dow 888			04A445-07 04A453-15		
	Dow 888-SL			04A444-06 04A457-19		
	Dow 890-SL	04A447-09 04A451-13	04A450-12 04A462-24	04A442-04 04A456-18		
	Mobay 960					
	Mobay 960-SL			04A448-10 04A454-16		
Neoprene	DS Brown E-437H		I	J		
	DS Brown V-687			04A441-01 04A452-14		
	DS Brown V-812					
	Kold Seal Neo Loop					
	Esco PV 687					
	Watson Bowman 687			04A443-05		
	Watson Bowman 812			04A460-22		
Polysulfide	Koch 9050-SL			K		
Proprietary	Roshek			L		

The concrete pavement was placed on February 13, 1991. Primary sawcutting, to a depth of one third of the slab thickness, was done on February 13 and 14. Secondary sawcutting of sealant reservoirs was done on February 15. All joint sawing was done with conventional riding saws.

Although this pavement is located in a dry climate, several rainstorms occurred during its construction. Since the concrete was not paved in one pass along the full width of the roadway, some erosion of the prepared base material occurred at the edge of the paved slab. Reference 15 indicates that the contractor took action to correct this erosion. Nonetheless, it is conceivable that (a) the base and subgrade moisture content were unusually high at the time of paving, and (b) the quality of support along the longitudinal paving joint may have been nonuniform.

Mesa is one of only two of the five SPS-4 supplemental study sites for which some traffic data are available in the LTPP database. Figure 6 illustrates the estimated cumulative ESALs for 1991 and 1992 for Mesa's linked GPS site (047613), and the estimated cumulative ESALs for 1993 through 1998 for supplemental test section 04A441. The estimates shown in this figure are summarized in Table 5.

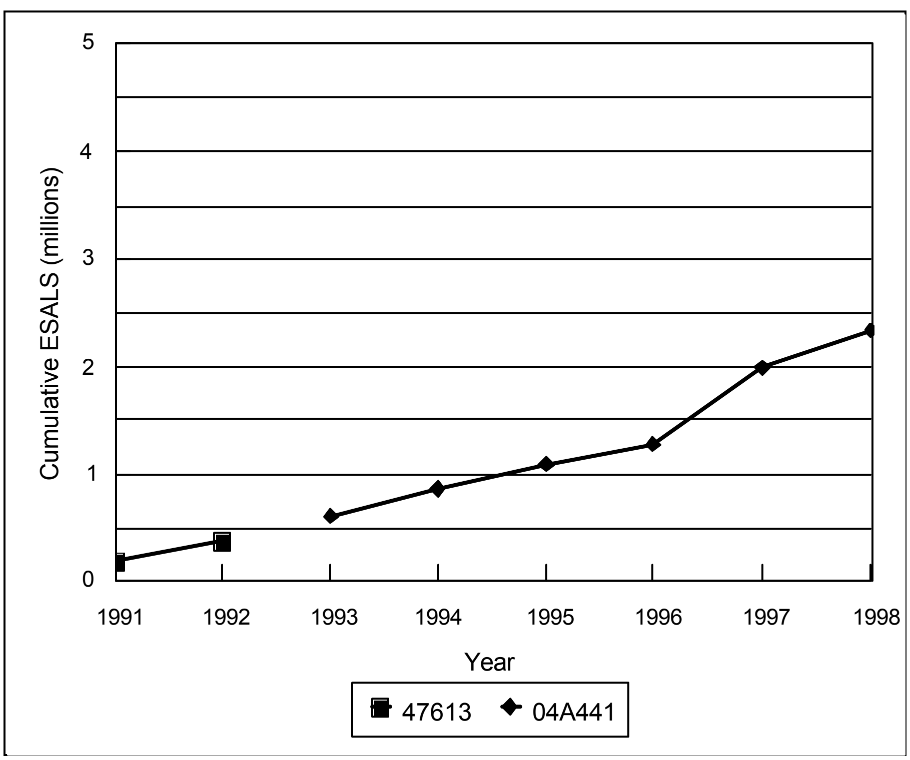


Figure 6. Estimated cumulative ESALs, Mesa SPS-4 supplemental study site.

Table 5. Estimated cumulative ESALs, Mesa SPS-4 supplemental study site.

SHRP ID	Year	Cumulative ESAL (10^6)
047613	1991	0.19
047613	1992	0.39
	1993	0.60
04A441	1994	0.87
04A441	1995	1.09
04A441	1996	1.29
04A441	1997	1.99
04A441	1998	2.35

The monitoring data from the Mesa site that were available for this study are summarized in Table 6 below.

Table 6. Monitoring data available for Mesa, Arizona SPS-4 supplemental test sections.

SPS-4 Site	Year Built	Data Type	Years with Data Available									
			‘90	‘91	‘92	‘93	‘94	‘95	‘96	‘97	‘98	‘99
Mesa, AZ	1991	Spalling						✔		✔		✔
		Sealant Damage						✔		✔		✔
		Faulting								✔		✔
		IRI			✔	✔			✔	✔	✔	✔
		Deflections		✔			✔	✔		✔		✔

Center-Slab Deflection Analysis

Deflection data were collected in April 1991, September 1994, July 1995, December 1997, and February 1999. The dynamic k value and effective slab thickness parameters were calculated starting from AREA4-based and AREA5-based solutions for the radius of relative stiffness. As described in Chapter 3, the AREA4 solution uses the deflection at the center of the load plate, whereas the AREA5 solution does not. The AREA4 solution may therefore more influenced by

compression in the slab. Although concrete slabs are commonly thought of and analyzed as plates, which experience only bending, in fact some compression does occur under load. This effect is greater for (a) thicker slabs, (b) lower concrete elastic moduli, and/or (c) stiffer foundations.

The overall project average average effective slab thickness values (calculated assuming a concrete elastic modulus of 5 million psi) obtained by the AREA4 and AREA5 methods are summarized by year in Table 7.

Table 7. Average effective thickness over time, Mesa, Arizona SPS-4 supplemental test sections.

Year	Hc4 (inches)	Hc5 (inches)
1991	12.0	14.2
1994	12.7	14.0
1995	11.7	13.8
1997	12.5	13.9
1999	12.5	13.9

The fact that the 1991 average effective thickness is less than the as-designed 13 inches for the AREA4 method and greater than the as-designed 13 inches for the AREA5 method suggests that for this project, (a) the concrete modulus was less than 5 million psi in 1991 and (b) the AREA4 method outputs are more realistic than the AREA5 method outputs. The alternate hypothesis, that the AREA5 method outputs are more realistic, would imply that the concrete modulus was greater than 5 million psi in 1991, which seems less likely. The increase over time in overall average effective slab thicknesses from the AREA4 method, in contrast to the decrease over time for the values from the AREA5 method, also supports the hypothesis that the AREA4 method outputs are the more realistic of the two. An increase in effective slab thickness for an assumed constant elastic modulus implies that the elastic modulus is increasing over time, as one would expect.

There are some significant differences between the group A effective thickness values and those of the other groups, but the significance of the differences is not consistent from year to year.

The results of the backcalculation of dynamic k value reveal a surpring trend: a decrease in k values over time. The overall project average dynamic k values obtained by the method are summarized by year in Table 8. The group mean k value results from the AREA4 solution method are shown in Figure 7.

Table 8. Overall average dynamic k value over time, Mesa, Arizona SPS-4 supplemental study site.

Year	K4 (psi/in)	K5 (psi/in)
1991	479	370
1994	317	264
1995	332	253
1997	241	207
1999	225	195

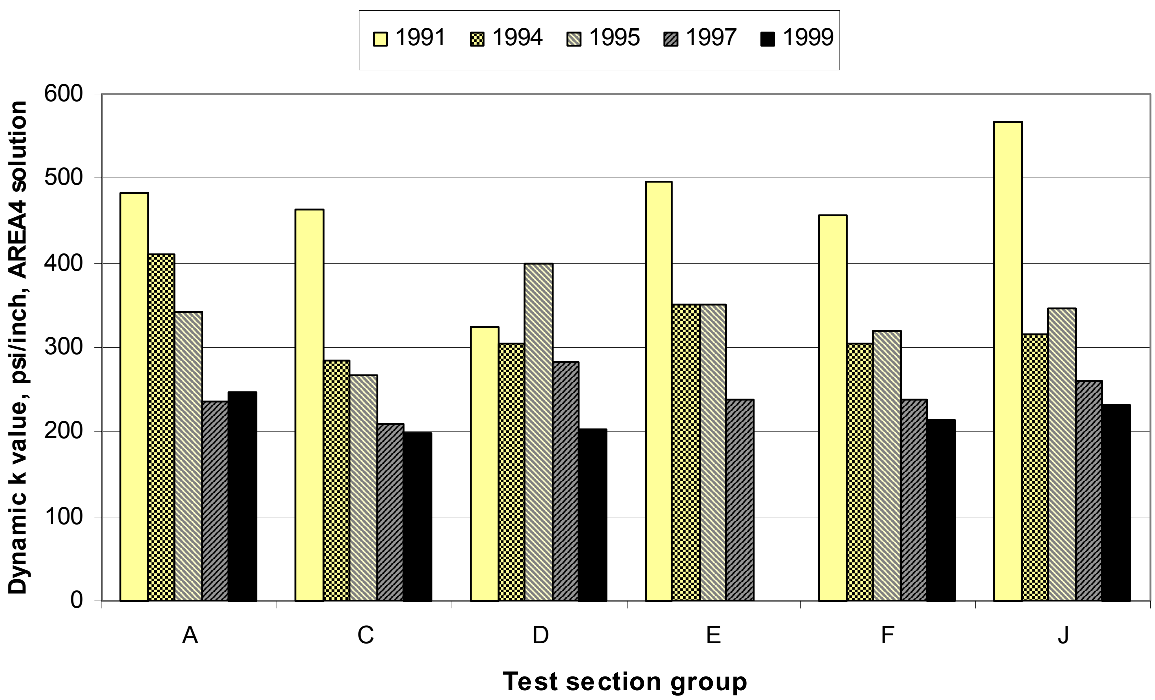


Figure 7. Change in dynamic k-value over time, Mesa, Arizona SPS-4 supplemental test section group means.

There are some significant differences detected between the group A average dynamic k values and those of the other groups, although the significance of these differences is not consistent from year to year. The magnitude of decrease in k values seems to be diminishing over time, but whether or not the 1999 values can be considered representative of stable long-term average k values is unknown.

Joint Spalling

Joint spalling data were collected in July 1995, December 1997, and February 1999. One section – 04A457, which is in the 9-mm silicone-sealed group (F) – had 0.8 meters of low-severity joint spalling reported in 1995, but zero spalling reported in subsequent surveys. All other test sections had zero spalling in all three surveys.

Joint Sealant Damage

Joint sealant condition data were collected in July 1995, December 1997, and February 1999. Sections 04A430 and 04A455 are the unsealed-joint sections (group A), so of course they have no joint sealant condition data associated with them.

For all of the sealed-joint sections for which sealant condition data are available, a weighted seal damage index is reported. Weighted seal damage, as discussed in Chapter 3, can range from 1 (all joints low severity) to 4 (all joints high severity). All sealed-joint test sections had a weighted seal damage index of 1.00 in 1995. In all sealed-joint test sections, the weighted seal damage index was greater in 1997 than in 1995, and in all but one section, the weighted seal damage index was equal or greater in 1999 than in 1997.

For the most recent survey, 1999, the sealed-joint test section groups had the following mean weighted joint seal damage index values, on a scale of 1 to 4:

- 9-mm asphalt-sealed (group C) = 3.93
- 3-mm silicone-sealed (group D) = 2.93
- 9-mm neoprene-sealed (group J) = 2.06
- 6-mm silicone-sealed (group E) = 1.88
- 9-mm silicone-sealed (group F) = 1.53

Joint Faulting

Faulting data were collected in December 1997 and February 1999. In every group, the average absolute joint faulting increased from 1997 to 1999. The average absolute faulting group means are illustrated in Figure 8. The results of the statistical comparisons of the 3-mm unsealed-joint test section group (A) with the sealed-joint test section groups are summarized in Table 9.

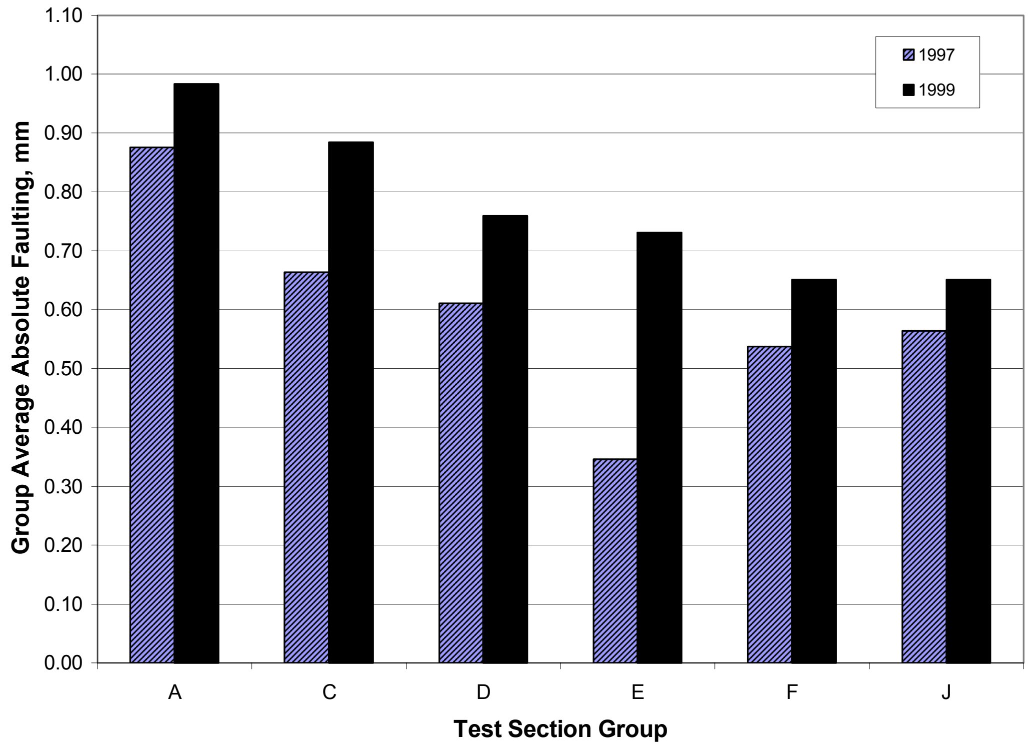


Figure 8. Average absolute faulting in 1997 and 1999, Mesa, Arizona SPS-4 supplemental test section groups.

Table 9. Average absolute faulting in unsealed versus sealed test section groups, Mesa, Arizona SPS-4 supplemental test section groups.

Test Section Group	1997		1999	
	Average Absolute Faulting, mm	Significantly Different than A?	Average Absolute Faulting, mm	Significantly Different than A?
3-mm unsealed (A)	0.876		0.984	
9-mm asphalt sealed (C)	0.663	NO	0.885	NO
3-mm silicone sealed (D)	0.611	NO	0.759	NO
6-mm silicone sealed (E)	0.346	YES	0.731	NO
9-mm silicone sealed (F)	0.537	YES	0.651	YES
9-mm neoprene sealed (J)	0.564	YES	0.651	YES

Roughness

Longitudinal profile data were collected in February 1992, February 1993, April 1996, January 1997, January 1998, and February 1999.

No significant differences are detected between group A and any of the other five groups in any year, except for one: a significant difference, in the 1998 data only, with respect to the 6-mm silicone sealed (group E) sections. It is not surprising that the groups are not significantly different in IRI, despite some significant differences in faulting, because the levels of faulting are still so low. The overall average absolute faulting in all groups was 0.78 mm.

The increases in group mean IRI between 1992 and 1999 are listed in Table 10. The increase in group mean IRI for the narrow unsealed test sections (group A) was greater than or equal to the increase for every one of the sealed-joint test sections. The trends in group mean IRI are illustrated in Figure 9.

Table 10. Increase in IRI between 1992 and 1999,
Mesa, Arizona SPS-4 supplemental test section groups.

Group	Description	IRI average 1992	IRI average 1999	IRI increase
A	3-mm unsealed	0.700	0.958	0.258
C	9-mm asphalt sealed	0.700	0.960	0.260
D	3-mm silicone sealed	0.709	0.906	0.197
E	6-mm silicone sealed	0.738	0.858	0.120
F	9-mm silicone sealed	0.760	0.985	0.225
J	9-mm neoprene sealed	0.718	0.926	0.208

Joint Deflection Analysis

Deflection data were collected in April 1991, September 1994, July 1995, December 1997, and February 1999. The transverse joint deflection parameters were calculated from deflections measured at slab positions designated J4 and J5 in LTPP deflection testing, namely, the approach and leave sides of the joint, in the outer wheelpath.

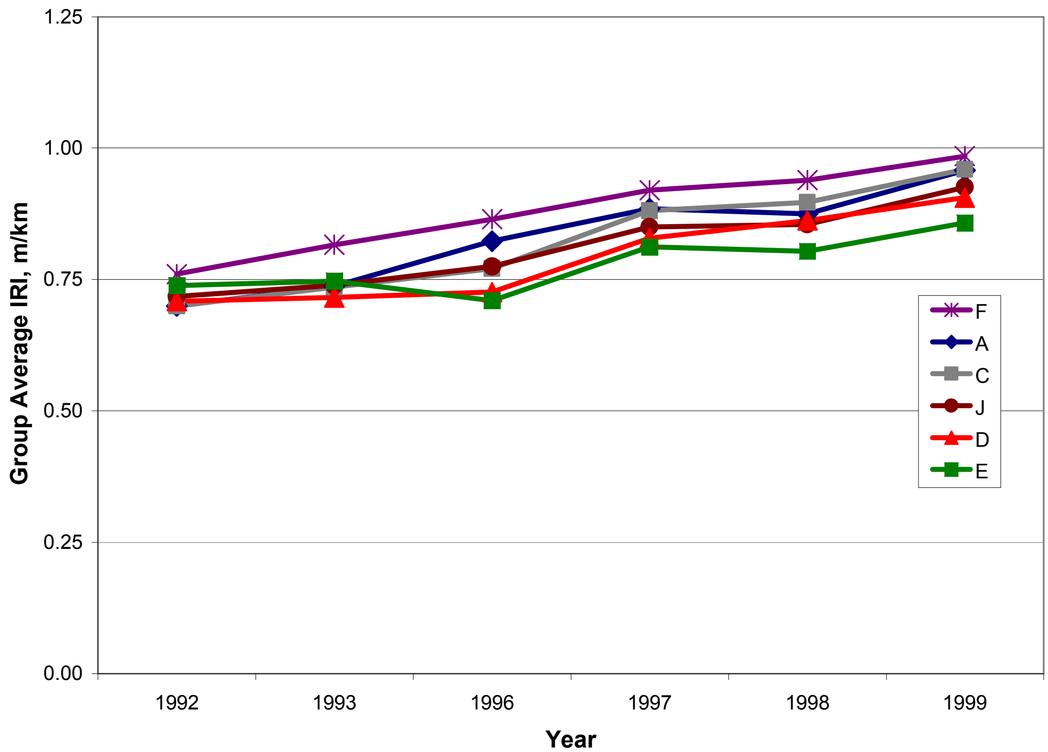


Figure 9. Trends in group mean IRI , Mesa, Arizona SPS-4 supplemental test section groups.

Deflection Load Transfer

The minimum and maximum pavement temperatures recorded during each of the five deflection surveys are summarized in Table 11.

Table 11. Pavement temperatures during deflection testing, Mesa, Arizona SPS-4 supplemental test sections.

Month and Year	Minimum (deg F)	Average (F)	Maximum (deg F)
Apr 1991	70	91	104
Sep 1994	88	106	127
Jul 1995	100	127	149
Dec 1997	50	70	100
Feb 1999	55	75	93

The group A average deflection load transfer values are significantly higher than the average deflection load transfer values for every sealed-joint group. However, deflection load transfer data should be examined in light of the pavement temperatures during testing, so that differences that may be partially due to temperature differences are not erroneously attributed entirely to treatment differences.

The availability of deflection load transfer measurements over a wide range of temperatures permits the determination of a characteristic curve of deflection load transfer as a function of temperature, for each of the treatment groups. The following model form was used for this purpose (note that the temperature input to this equation is in degrees Celsius):

$$LT\% = \frac{1}{a_1 + a_2 * a_3^{(-temperature)}} \qquad \text{Eqn 26}$$

The A, B, and C coefficients determined for each treatment group, and the square of the correlation coefficient (R^2) between estimated and observed deflection load transfer values, are summarized in Table 12. The curves are illustrated in Figure 10.

Table 12. Coefficients of equations for deflection load transfer versus temperature, Mesa, Arizona SPS-4 supplemental test section groups.

	A	C	D	E	F	J
a_1	0.0107	0.0107	0.0112	0.0107	0.0101	0.0105
a_2	0.4864	0.4864	17.1928	3.7232	1.0090	0.5199
a_3	1.2580	1.2065	1.5405	1.2682	1.2034	1.1798
R^2	0.978	0.765	0.990	0.928	0.731	0.882

As Figure 10 illustrates, the two groups with the highest deflection load transfer over the widest range of temperatures are groups A (3-mm unsealed) and D (3-mm silicone-sealed). The next-best load transfer is exhibited by group C (9-mm asphalt-sealed), and the remaining three groups (E, F, and J; 6-mm silicone-sealed, 9-mm silicone-sealed, and 9-mm neoprene sealed) exhibit lower, and similar, deflection load transfers over much of the temperature range.

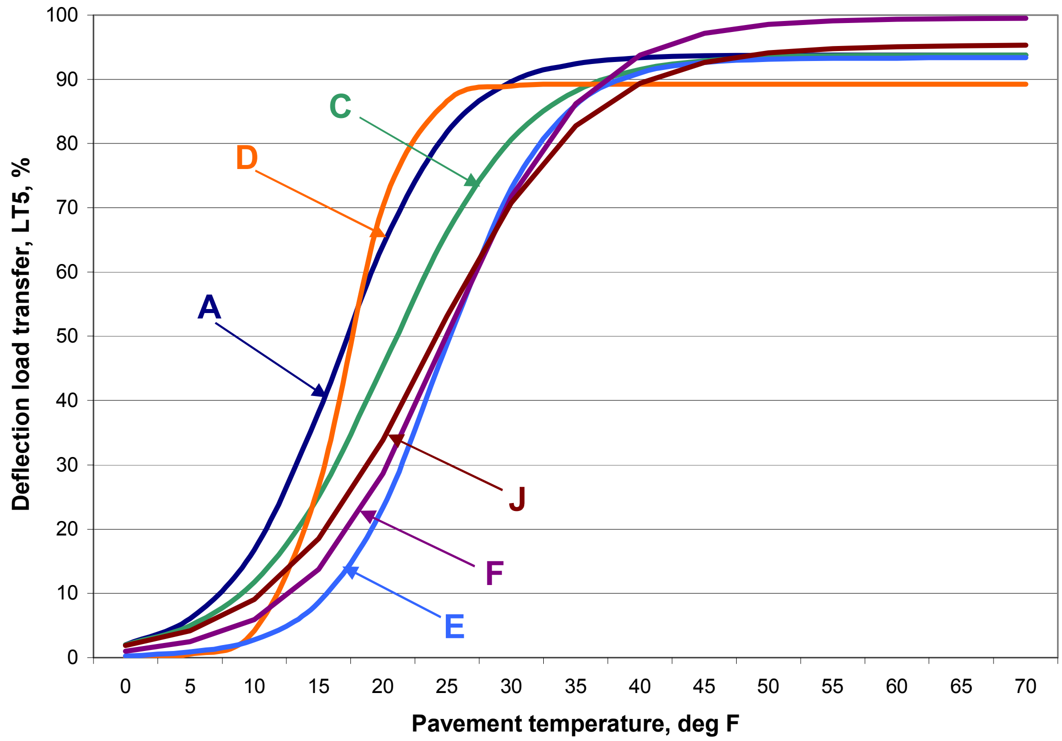


Figure 10. Deflection load transfer versus temperature,
Mesa, Arizona SPS-4 supplemental test section groups.

Total Edge Deflection

The 9000-pound normalized total edge deflection, as explained in Chapter 3, is a relative indicator of the overall structural capacity of the pavement at the transverse joint. Total edge deflection is also an input to the backcalculation of the foundation support (k value) at the transverse joint. The normalized total edge deflection should remain relatively constant despite variations in deflection load transfer, so long as slab thickness, concrete elastic modulus, and foundation support remain constant.

In the case of the Mesa site, not all of these assumptions hold: there has been a notable decrease in k value in all of the test section groups between 1991 and 1999. There has also been a slight increase in effective slab thickness (from the AREA4 solution method) for an assumed constant concrete modulus of 5 million psi – which implies an increase in concrete modulus for a constant slab thickness. Decreasing foundation k value and increasing concrete elastic modulus can be expected to have opposing, but not necessarily counterbalancing, effects on total edge deflection.

The group A average total edge deflection was significantly lower than the total edge deflection for every sealed-joint group, in 1991 and 1994. In the later sets of deflection data, the group A total edge deflection results are higher than some groups and lower than others, but these differences are not consistent in the 1995 and 1997 surveys, nor necessarily statistically significant. The 1999 deflection results indicate that group A's total edge deflection was not significantly different than that of groups C and D (9-mm asphalt-sealed and 3-mm silicone sealed) but was significantly lower than that of groups F and J (9-mm silicone-sealed and 9-mm neoprene sealed). Overall, all of the groups have exhibited similar increases in total edge deflection between 1991 and 1999. This suggests similar net decreases in edge structural capacity over this time period.

Differential Edge Deflection

The 9000-pound normalized differential edge deflection is similar to deflection load transfer in that it is an indicator of the relative movement of the loaded and unloaded slabs at a transverse joint. Like deflection load transfer, differential edge deflection is expected to vary with pavement temperature. Differential edge deflection is arguably a better indicator of the true magnitude of relative slab movement at the joint, since it is not expressed as a percentage of the deflection of one of the slabs.

The group A differential edge deflection was significantly lower than the differential edge deflection for every sealed-joint group, in 1991 and 1994. As with the total edge deflection results, the differences between group A and the other groups are not consistent in the 1995 and 1997 data. The 1999 indicate that differential edge deflection was lower in group A than in every sealed-joint group with which a comparison was possible (C, D, F, and J).

Transverse Edge Slab Support Ratio

The uniformity of support under the transverse edge, as explained in Chapter 3, is referred to as the transverse edge slab support ratio and is calculated as the ratio of the slab edge k value to the slab interior k value for the same slab. In general, edge slab support ratios less than about 0.75 indicate poor edge support, which may be due to foundation erosion, densification, and/or slab curling.

Most of the Mesa test section groups have experienced similar declines in edge slab support ratios over time, from values in the range of 1.2 to 1.4 in 1991, to values in the range of 0.6 to 0.9 in 1999. Group J (9-mm neoprene-sealed) was already exhibiting an edge support ratio of 0.9 in 1991, but this edge support ratio remained fairly constant through 1999.

Summary Observations

After eight years in service, the Mesa, Arizona SPS-4 supplemental test sections do not exhibit any joint spalling, but the sealed-joint test sections do exhibit, to different degrees, joint seal damage such as adhesive and cohesive failure. The sealed-joint test sections cannot be considered to have been kept well sealed.

Although average absolute joint faulting is low overall, as of the most recent survey it was significantly higher in the 3-mm unsealed test section group (A) than in two of the other test section groups: the 9-mm silicone-sealed (F) and 9-mm neoprene-sealed (J) group. The Mesa test sections had impressively low and uniform IRI values in 1991. Although IRI levels remain low, the 3-mm unsealed test section group has exhibited a greater increase in IRI than most other groups.

The deflection analysis reveals some significant nonuniformity in effective slab thicknesses and foundation support levels, even in the first year's deflection survey. Even more surprising are the decreases in k value that have occurred in every test section group over time. Such inconsistencies in effective slab thickness and slab support must be taken into consideration whenever (a) comparing the relative structural capacities of the test sections, and (b) judging the significance of differences in parameters that relate to transverse joint behavior, particularly for those parameters computed using slab-interior value as points of reference.

The 3-mm unsealed and 3-mm silicone-sealed test section groups (A and D) actually seem to have exhibited the best deflection load transfer over the range of temperatures at which load transfer has been measured. The 3-mm unsealed test section group also has exhibited the lowest differential transverse joint deflections.

Nearly all of the Mesa test section groups have exhibited declines in slab edge support ratios. This means that not only have the interior-slab k values declined over time, but the k values at the transverse joints have declined even more. These trends are rather surprising, considering the project is located in one of the country's driest climates. Both the interior and edge k values may be approaching stable long-term values.

The narrow unsealed test sections would seem to have the advantage over the other treatment groups in terms of deflection load transfer at transverse joints, but this has not necessarily resulted in lower faulting than in the other groups. Overall, the narrow unsealed test sections and the various sealed-joint test sections at the Mesa site are all exhibiting similarly good performance thus far, with respect to joint spalling, joint faulting, and IRI.

Campo, Colorado

The Colorado SPS-4 supplemental experiment is located on U.S. 287 in the southeast corner of Colorado. An illustration of the location and key location data are given in Figure 11. Thirty-year average monthly temperature and precipitation normals for the weather station nearest the site are given in Figure 12. The site receives an average of 15.4 inches of precipitation annually, and the temperatures range from an average low of 17°F in January to an average high of 91°F in July.

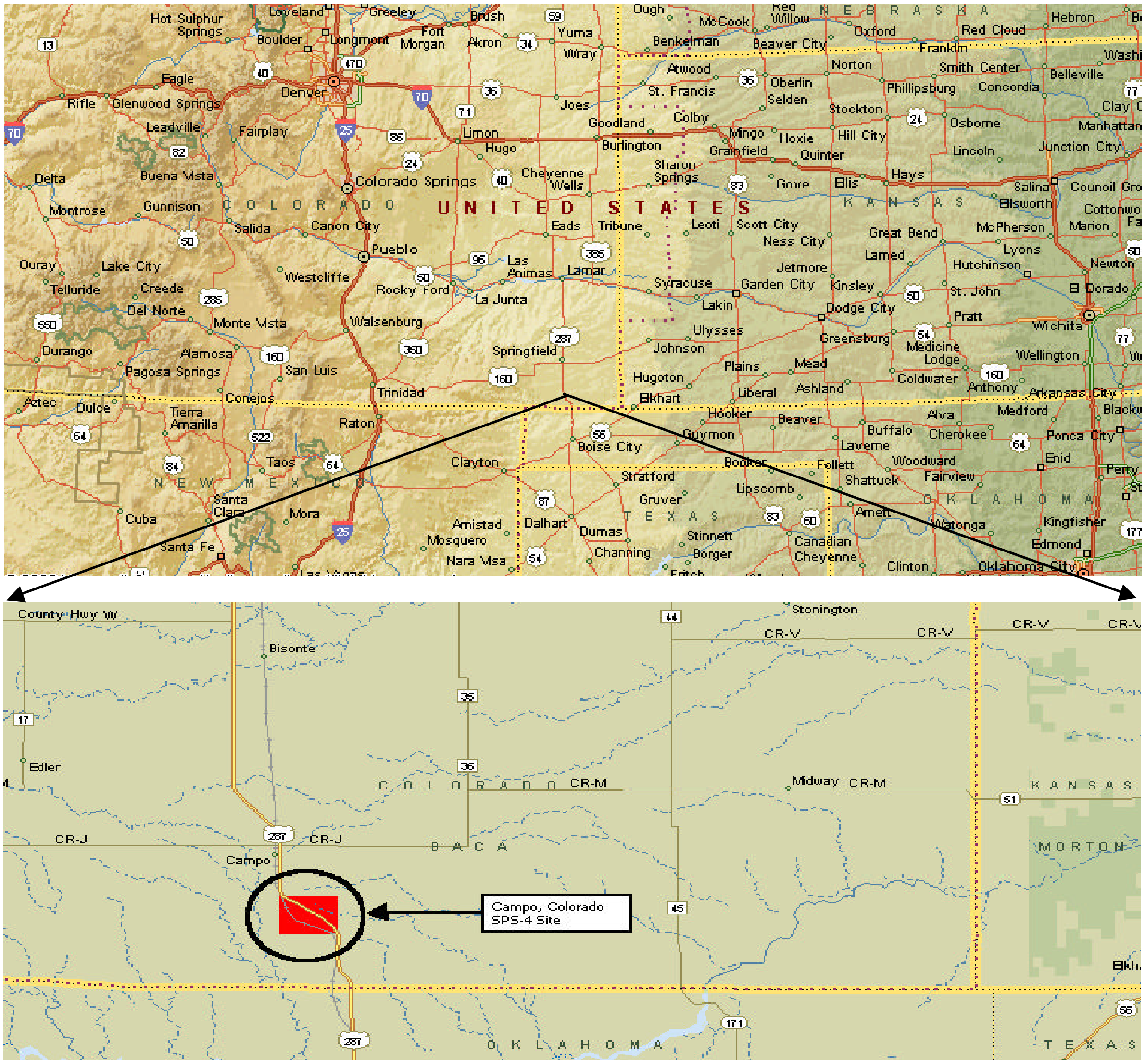
In the vicinity of the experiment site, U.S. 287 is a two-lane highway, with 12-ft-wide traffic lanes and 10-ft-wide paved shoulders. The Campo test sections were constructed in October and November of 1995. The pavement is a 10-inch JPCP on 2 ft of select soil and a sandy subgrade. The joints are dowelled, unskewed, and spaced at 15 ft. No information on the diameter of the dowels is available in the LTPP database.

The monitoring data from the Campo site that were available for this study are summarized in Table 13 below.

Table 13. Monitoring data available for Campo, Colorado SPS-4 supplemental test sections.

SPS-4 Site	Year Built	Data Type	Years with Data Available									
			'90	'91	'92	'93	'94	'95	'96	'97	'98	'99
Campo, CO	1995	Spalling							✔		✔	
		Sealant Damage							✔		✔	
		Faulting							✔		✔	
		IRI								✔	✔	✔
		Deflections							✔		✔	

The experimental test sections are located in the northbound lane. The layout of the test sections is illustrated in Figure 13. The positions of these test sections in the global experimental matrix described earlier are illustrated in Table 14.

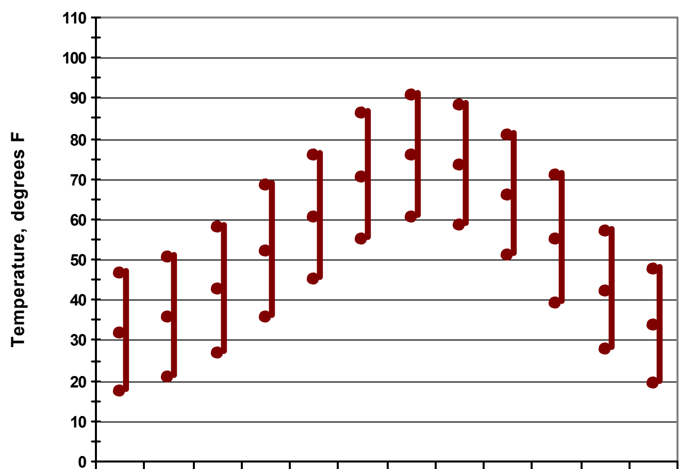


Location

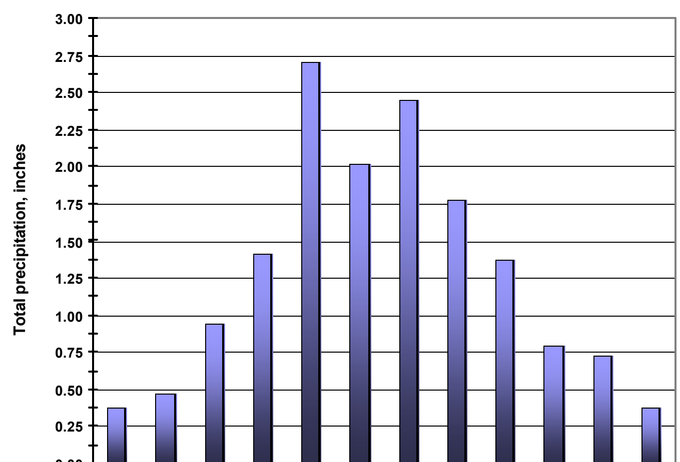
SRHP ID	**08A400**	Linked GPS site	**None**
State	**Colorado**	Route	**US 287**
County	**Baca**	Nearest city or town	**Campo**
Latitude	**37.06**	Longitude	**102.57**

Location Notes: Northbound on Route 287, between Campo and Oklahoma state line.

Figure 11. Location of Campo, Colorado SPS-4 supplemental study site.



	J	F	M	A	M	J	J	A	S	O	N	D
Max	46.5	50.4	58.2	68.2	76.0	86.1	90.9	88.2	80.8	71.0	56.8	47.7
Mean	31.8	35.7	42.4	52.0	60.6	70.5	75.6	73.5	65.8	55.1	42.3	33.6
Min	17.1	20.8	26.6	35.8	45.1	54.8	60.4	58.7	50.8	39.3	27.7	19.5



	J	F	M	A	M	J	J	A	S	O	N	D
Mean	0.4	0.5	0.9	1.4	2.7	2.0	2.5	1.8	1.4	0.8	0.7	0.4

Climate

SRHP ID	**08A400**	Weather station ID	**57866**
State	**Colorado**	Weather station name	**Springfield 7 WSW**
County	**Baca**	Weather station latitude	**37.23 N**
Nearest city or town	**Campo**	Weather station longitude	**102.44 W**
Mean annual temperature	**53.2** °F	Mean annual precipitation	**15.42** in

Climate Notes: Data source is 1961-90 Monthly Station Normals, U.S. Divisional and Station Climatic Data and Normals, 1994, National Oceanic and Atmospheric Administration

Figure 12. Campo, Colorado temperature and precipitation normals.

US-287 Northbound:

Northbound

Section	SHRP ID	joint width (mm)	saw	sealant type	length (ft)
					315
10B	08A416	9	riding	Crafco RS-902	60
9B	08A455	9	riding	Crafco RS-902	375
8B	08A446	9	riding	Crafco RS-903-SL	375
7B	08A415	9	riding	Crafco RS-902	375
	08A454				375
5B	08A445	6	riding	Crafco RS-903-SL	375
4B	08A414	6	riding	Crafco RS-902	375
3B	08A444	3	riding	Crafco RS-903-SL	375
2B	08A413	3	riding	Crafco RS-902	375
1B	08A431	3	riding	Unsealed	375
					315
10A	08A453	9	riding	Crafco RS-902	60
9A	08A452	9	riding	DS Brown V-687	375
8A	08A443	9	riding	Crafco RS-903-SL	375
7A	08A412	9	riding	Crafco RS-902	375
6A	08A451	6	riding	DS Brown E-437-H	375
5A	08A442	6	riding	Crafco RS-903-SL	375
4A	08A411	6	riding	Crafco RS-902	375
3A	08A441	3	riding	Crafco RS-903-SL	375
2A	08A410	3	riding	Crafco RS-902	375
1A	08A430	3	riding	Unsealed	375

Figure 13. Campo, Colorado SPS-4 supplemental test section layout.

Table 14. Positions of Campo, Colorado SPS-4 supplemental test sections in global experimental matrix.

		Conventional Saw				Soff-Cut Saw
		3 mm	6 mm	9 mm	9 mm bevel	3 mm
Unsealed		A 08A430-1A 08A431-1B				B
Asphalt	Crafco RS 221			C		
	Crafco SS 444					
	Koch 9005					
	Koch 9012					
Silicone	Crafco 902	D 08A410-2A 08A413-2B	E 08A411-4A 08A414-4B	F 08A412-7A 08A415-7B 08A455-9B	G 08A453-10A 08A416-10B	H
	Crafco 903-SL	08A441-3A 08A444-3B	08A442-5A 08A445-5B	08A443-8A 08A446-8B		
	Dow 888					
	Dow 888-SL					
	Dow 890-SL					
	Mobay 960					
	Mobay 960-SL					
Neoprene	DS Brown E-437H		I 08A451-6A	J		
	DS Brown V-687			08A452-9A		
	DS Brown V-812					
	Kold Seal Neo Loop					
	Esco PV 687					
	Watson Bowman 687					
	Watson Bowman 812					
Polysulfide	Koch 9050-SL			K		
Proprietary	Roshek			L		

Center-Slab Deflection Analysis

Deflection data were collected in May 1996 and July 1998. It should be noted that no deflection data are yet available for the two main LTPP SPS-4 experimental test sections at this site, 08A410 and 08A430, and that deflection data are lacking in one or both years for some other test sections as well.

The overall average thicknesses by the AREA4 method were 9.54 and 9.51 inches in 1996 and 1998 respectively, and by the AREA5 method were 11.40 and 10.26 inches in 1996 and 1998 respectively. Among these results, the only ones that seem unrealistic are the 1996 effective thicknesses, which are considerably higher than the as-designed thickness of 10 inches, which suggests that as-constructed thicknesses considerably greater than 10 inches and/or a concrete elastic modulus considerably greater than 5 million psi early in the life of the pavement.

The overall average dynamic k value from the AREA4 method was 296 psi/in in 1996 and 226 psi/in in 1998. Similarly, the overall average dynamic k value from the AREA5 method was 222 psi/in in 1996 and 204 psi/in in 1998. In general, the 1996 k values were not significantly different by group, but in 1998, the mean dynamic k value of the 3-mm unsealed joint group (A) was significantly lower than those of five of the six sealed-joint groups.

However, the lower k values in 1998 may be partially due to incomplete contact between the slabs and the foundation. Slab temperatures were much higher during the 1998 deflection testing than during the 1996 deflection testing, with positive temperature gradients on the order of 1 to 1.5 degrees F/inch recorded for all but one section (08A441, one of the group D sections). No slab temperature gradients were recorded during the 1996 survey. The 1998 temperature gradients might have produced downward curling that may not have been fully compensated for in the incremental analysis of the deflection measurements. For section 08A441, where the temperature gradient was slightly negative (-0.7 deg F/inch) the average dynamic k value reduced only slightly (from 326 psi/in in 1996 to 283 psi/in in 1998), based on the AREA4 analysis, and remained essentially constant (239 psi/in in 1996 and 241 psi/in in 1998) based on the AREA5 analysis.

Joint Spalling

Joint spalling data were collected in May 1996 and August 1998. In the 1998 survey, one spalled joint was noted in each of two test sections: 08A446 (9-mm silicone-sealed joints) and 08A453 (9-mm beveled silicone-sealed joints). All other test sections had zero spalling in both surveys.

Joint Sealant Damage

Joint sealant condition data were collected in May 1996 and August 1998. As of the 1998 survey, the sealed joint test section groups had the following mean weighted joint seal damage index values, on scale of 1 to 4:

- 3-mm silicone sealed (group D) = 1.07
- 6-mm silicone sealed (group E) = 1.04
- 9-mm silicone sealed (group F) = 1.00
- 9-mm beveled silicone sealed (group G) = 1.00
- 6-mm neoprene sealed (group I) = 1.12
- 9-mm neoprene sealed (group J) = 1.00

Joint Faulting

Faulting data were collected in May 1996 and July 1998. The average absolute faulting group means are illustrated in Figure 14. The results of the statistical comparisons of the 3-mm unsealed-joint test section group (A) with the sealed-joint test section groups are summarized in Table 15.

In nearly every test section and nearly every group, the average absolute faulting was <u>lower</u> in the 1998 survey than in the 1996 survey. Among the possible explanations for the lower mean values in average absolute faulting in the later survey are the following:

- A change in the measurement equipment used or its calibration,
- Different degrees of slab curling during the two surveys, or
- Negative faulting turning into positive faulting, that is, decreases in the absolute values of negative faulting measurements.

The faulting data from both the 1996 and 1998 surveys show no significant difference in average absolute faulting between the narrow unsealed-joint group (group A) and any of the sealed-joint groups.

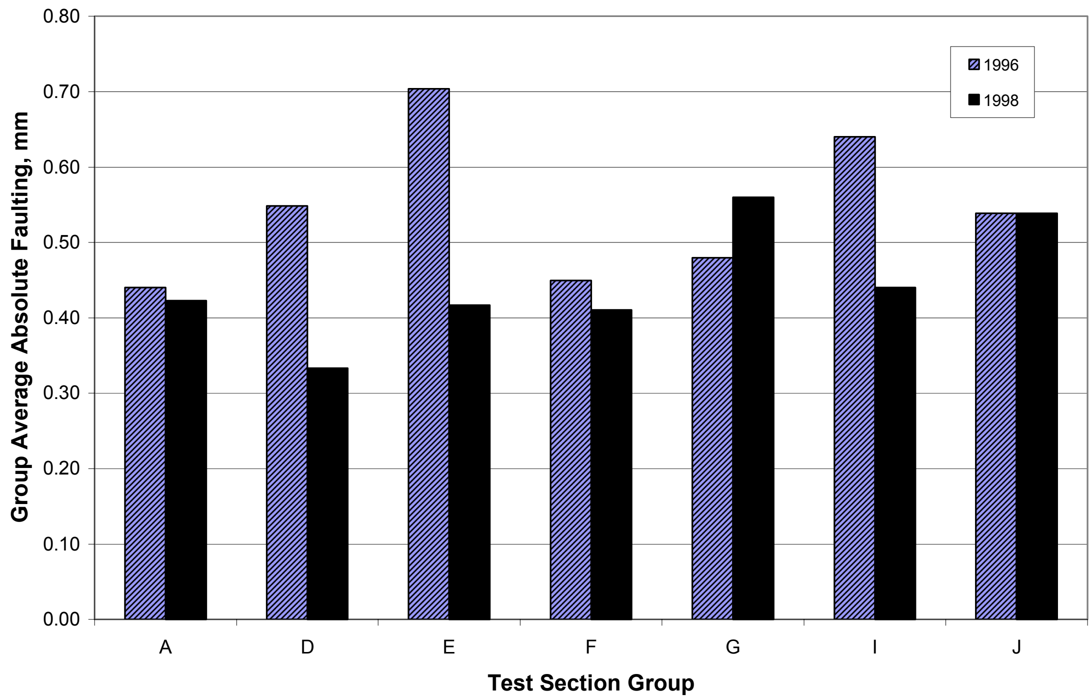


Figure 14. Average absolute faulting in 1996 and 1998, Campo, Colorado SPS-4 supplemental test section groups.

Table 15. Average absolute faulting in unsealed versus sealed test section groups, Campo, Colorado SPS-4 supplemental test section groups.

Test Section Group	1996		1998	
	Average Absolute Faulting, mm	Significantly Different than A?	Average Absolute Faulting, mm	Significantly Different than A?
3-mm unsealed (A)	0.440		0.423	
3-mm silicone sealed (D)	0.549	NO	0.333	NO
6-mm silicone sealed (E)	0.704	NO	0.417	NO
9-mm silicone sealed (F)	0.449	NO	0.411	NO
9-mm beveled silicone sealed (G)	0.480	NO	0.560	NO
6-mm neoprene sealed (I)	0.640	NO	0.440	NO
9-mm neoprene sealed (J)	0.538	NO	0.538	NO

Roughness

Longitudinal profile data were collected in November 1997, August 1998, and July 1999. Note that no IRI data are available for test sections 08A430 (narrow unsealed joints, group A) and 08A410 (narrow silicone-sealed joints, group D). The increases in group mean IRI between 1997 and 1999 are listed in Table 16. The trends in group mean IRI are illustrated in Figure 15.

In every group, the group average IRI steadily increased from 1997 to 1999. It is interesting to note that the Campo test sections have higher IRI values at any given age than the Mesa test sections, even though the faulting averages at Campo are lower than the faulting averages at Mesa. Since the Campo test sections have no other distress, this suggests that the measured roughness is primarily that which was "built in" at construction. This casts doubt on whether any significant differences that might be detected between group average IRIs can be attributed to the differences in treatments (i.e., sealant presence, joint width, sealant type) applied to the groups.

The narrow unsealed group (A) exhibited a smaller increase in IRI between 1997 and 1999 than any of the sealed-joint groups. It should be noted that IRI data are currently only available for one of the two test sections in group A, 08A431. It would be interesting to see what longer-term trends in IRI increase are observed in these test sections, especially when IRI data also become available for the other group A test section, 08A430.

Joint Deflection Analysis

Deflection Load Transfer

With few exceptions, the 1996 and 1998 deflection data indicate slightly higher load transfer with the load plate on the leave side of the joint (the J5 position) than with the load plate on the approach side of the joint (the J4 position). The 1998 load transfer values tend to be higher than the 1996 values, which is to be expected since the temperatures were considerably higher during the 1998 testing. The approach-side deflection load transfer of the 3-mm unsealed joint group (A) was comparable to that of every other group in 1996 and better than that of every other group but one in 1998. The leave-side deflection load transfer of the 3-mm unsealed joint group (A) was better than that of every other group in 1996, and comparable to that of every other group but one in 1998. Overall, all of the groups exhibit comparable deflection load transfer values in these two sets

Table 16. Increase in IRI between 1997 and 1999, Campo, Colorado
SPS-4 supplemental test section groups.

Group	Description	IRI average 1997	IRI average 1999	IRI increase
A	3-mm unsealed	1.577	1.687	0.110
D	3-mm silicone sealed	1.505	1.683	0.178
E	6-mm silicone sealed	1.429	1.603	0.174
F	9-mm silicone sealed	1.396	1.565	0.169
G	9-mm beveled silicone sealed	1.433	1.701	0.268
I	6-mm neoprene sealed	1.568	1.852	0.284
J	9-mm neoprene sealed	1.443	1.739	0.296

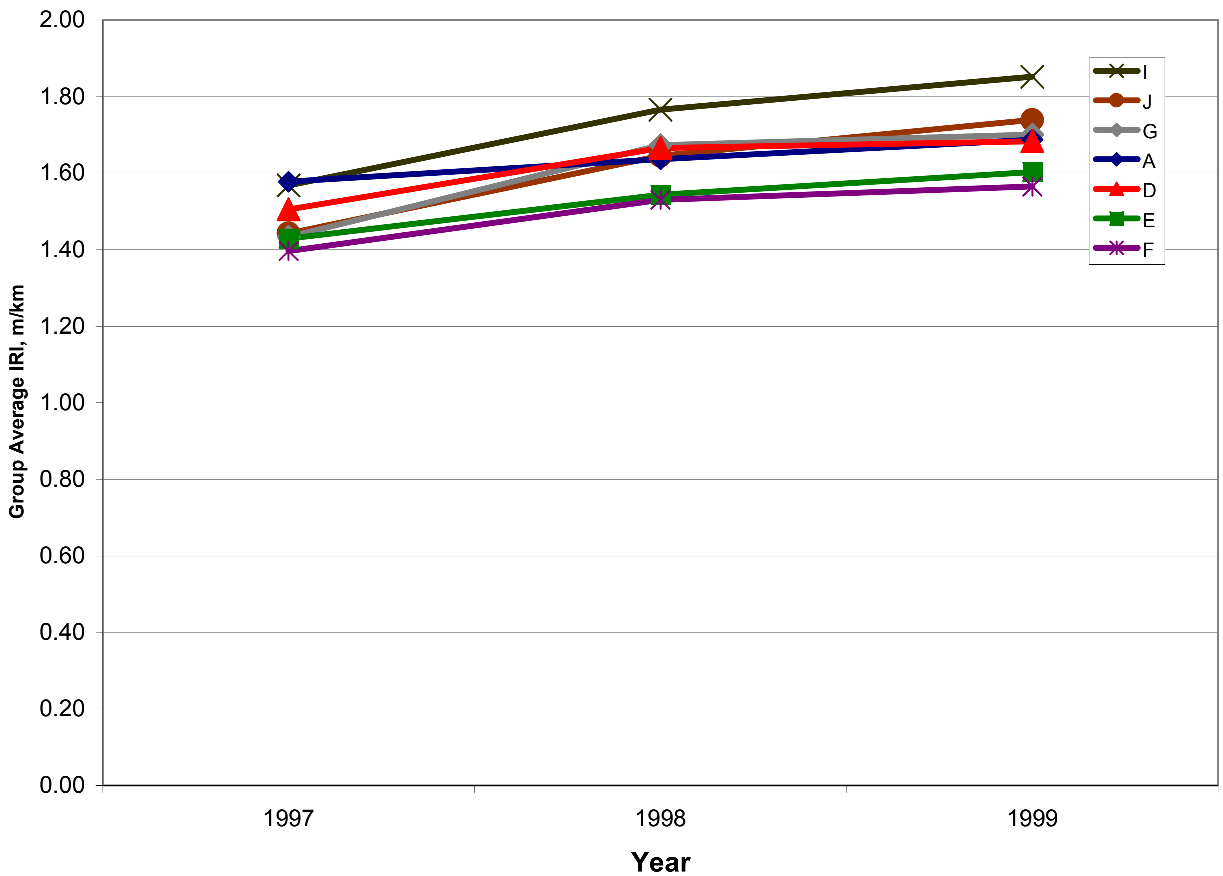


Figure 15. Trends in group mean IRI, Campo, Colorado
SPS-4 supplemental test section groups.

of summertime deflection testing. Deflections measured in other times of the year would be more useful in determining whether significant differences exist in deflection load transfer among the different treatment groups.

Total Edge Deflection

Total deflection of the loaded and unloaded sides of the joint is not expected to vary with pavement temperature, i.e., with joint opening, so long as the slabs remain in the contact with the foundation or any loss of contact due to curling is accounted for in the incremental load-deflection analysis. The 1998 deflection data indicate the total joint deflection in the 3-mm unsealed test section (group A) was significantly higher than that in each of the sealed-joint test section groups, except the 6-mm neoprene-sealed section (group I) in 1996 and the 9-mm neoprene section (group J) in 1998, for which significant differences were not detected. The 6-mm silicone-sealed sections (group E) had exhibited higher total edge deflection than the narrow unsealed section in 1996, but this reversed in 1998.

Differential Edge Deflection

The differential edge deflections measured in the 1996 and 1998 surveys are all very low, which is to be expected considering the temperatures at which they were measured. A deflection survey conducted when the pavement temperature is lower would be more useful in determining whether any significant differences in differential edge deflection exist by treatment group.

Transverse Edge Slab Support Ratio

The 1998 slab support ratios at the transverse joints were significantly higher in the 3-mm unsealed test section (group A), than those in the 3-mm, 6-mm, and 9-mm silicone-sealed groups (D, E, and F) and the 9-mm neoprene-sealed group (J). There was no significant difference in slab support ratio between group A and the 9-mm bevelled silicone-sealed and 6-mm neoprene-sealed groups (G and I).

Summary Observations

After three years of service, the sealed-joint test sections at the Campo site were still well sealed, and no significant joint spalling had occurred in either the unsealed-joint test section or any of the sealed-joint test sections.

The faulting data from both the 1996 and 1998 surveys show no significant difference in average absolute faulting between the narrow unsealed-joint (group A) test section and any of the sealed-joint groups. The main source of roughness measured in the first few years of the life of this pavement is believed to be associated with construction rather than with faulting or any other distresses. The narrow unsealed (group A) test section has exhibited the smallest increase in IRI over time of any of the treatment groups.

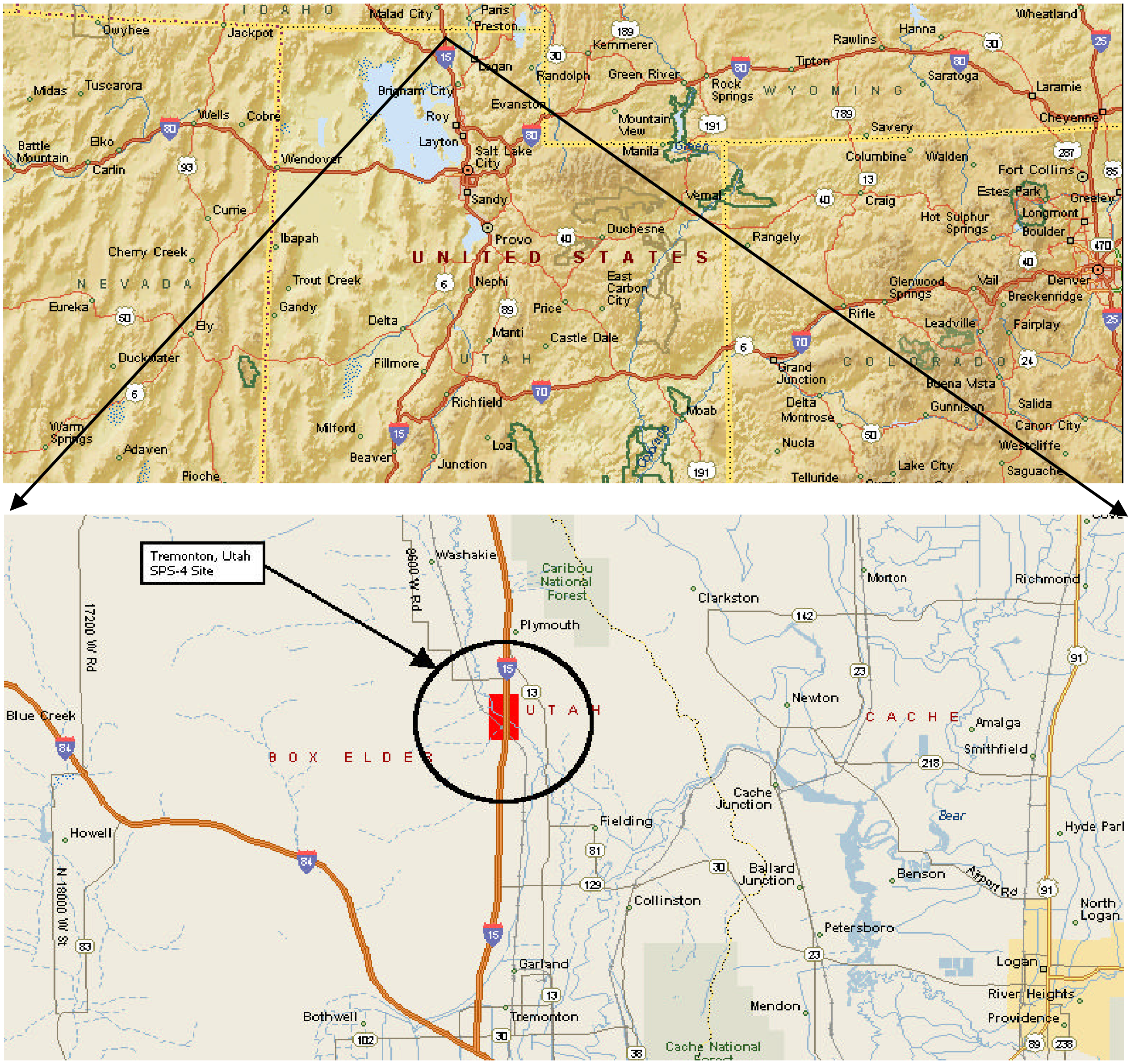
All of the treatment groups exhibited high load transfer and low differential deflections in the two deflection surveys, which is not surprising considering the young age of the pavement and the high temperatures at which the deflections were measured. Additional joint deflection data measured at lower temperatures is needed to determine whether or not any significant differences in joint load transfer by treatment type exist. The only deflection-based indicator of potential future differences in joint behavior between the unsealed-joint and sealed-joint test sections are the higher total edge deflections (loaded side plus unloaded side) measured in the unsealed-joint test section. On the other hand, the transverse joint slab support ratios calculated for the unsealed-joint test section are as good as or better than those of the sealed-joint test section groups.

Tremonton, Utah

The Tremonton SPS-4 supplemental experiment is located on Interstate 15 in northern Utah, between the town of Tremonton and the Idaho state line. An illustration of the location and key location data are given in Figure 16. Thirty-year average monthly temperature and precipitation normals for the weather station nearest the site are given in Figure 17. The site receives an average of 12.7 inches of precipitation annually, and the temperatures range from an average low of 9 °F in January to an average high of 90 °F in July.

This section of I-15 is a four-lane divided highway, with 12-ft-wide concrete traffic lanes, 3-ft-wide concrete inner shoulders, and 8-ft-wide concrete outer shoulders. The test sections were constructed in October 1990. The pavement is a 10-inch JPCP on 4 inches of lean concrete base and 4 inches of crushed gravel subbase. The joints are undowelled, skewed, and spaced at 10, 15, 11, and 14 ft. The subgrade materials are well-graded gravels.

The experimental test sections are located in the northbound and southbound lanes. The layout of the test sections is illustrated in Figure 18. The positions of these test sections in the global experimental matrix described earlier are illustrated in Table 17.

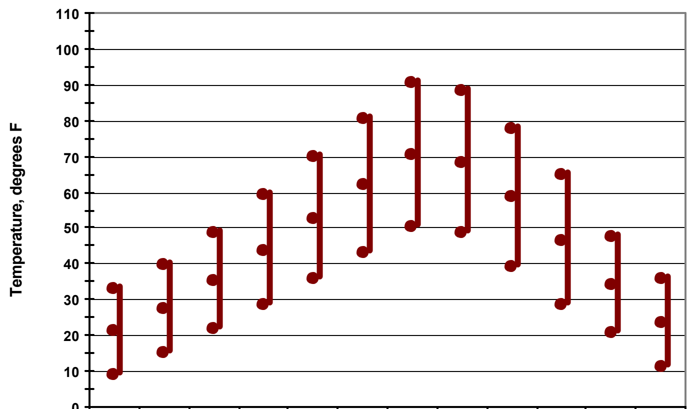


Location

SRHP ID	**49C400**	Linked GPS site	**497082**
State	**Utah**	Route	**I-15**
County	**Box Elder**	Nearest city or town	**Tremonton**
Latitude	**41.85**	Longitude	**112.17**

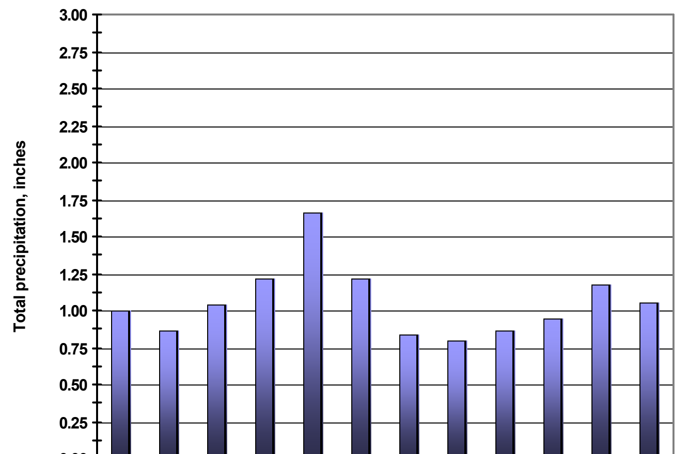
Location Notes Northbound and southbound on I-15, about 9 miles north of Tremonton.

Figure 16. Location of Tremonton, Utah SPS-4 supplemental study site.



	J	F	M	A	M	J	J	A	S	O	N	D
Max	33.1	39.8	48.3	59	69.6	80.4	90.4	88.3	77.8	64.7	47.3	35.6
Mean	21.2	27.3	35.1	43.7	52.6	61.8	70.3	68.4	58.4	46.6	34.2	23.4
Min	9.2	14.8	21.7	28.3	35.7	43.1	50.3	48.5	39	28.4	20.9	11.1

Month



	J	F	M	A	M	J	J	A	S	O	N	D
Mean	1.00	0.86	1.04	1.22	1.66	1.22	0.84	0.8	0.86	0.94	1.18	1.05

Month

Climate

SRHP ID	**49C400**	Weather station ID	**427931**
State	**Utah**	Weather station name	**Snowville**
County	**Box Elder**	Weather station latitude	**41.58 N**
Nearest city or towr	**Tremonton**	Weather station longitud	**112.43 W**
Mean annual temperature	**45.3** °F	Mean annual precipitation	**12.67** in

Climate Notes: Data source is 1961-90 Monthly Station Normals, U.S. Divisional and Station Climatic Data and Normals, 1994, National Oceanic and Atmospheric Administration

Figure 17. Tremonton, Utah temperature and precipitation normals.

I-15 Southbound and Northbound:

Northbound

Section	SHRP ID	joint width (mm)	saw	sealant type	length (ft)
1	49C440	9	riding	Koch 9012	400
2	49C441	9	riding	Koch 9005	400
3	49C410	9	riding	Dow 888-SL	500
	Bridge				2452
4	49C430	3	riding	Unsealed	600
5	49C443	9	riding	Mobay 960	400
6	49C444	9	riding	Esco PV-687	400
7	49C445	6	riding	Kold Seal Neo Loop	400
8	49C446	3	riding	Dow 890-SL	400
9	49C447	9	riding	Mobay 960	400
	OverPass				5047

Southbound

Section	SHRP ID	joint width (mm)	saw	sealant type	length (ft)
21	49C455	9	riding	Dow 888-SL	400
20	49C454	3	riding	Dow 890-SL	400
19	49C453	9	riding	Mobay 960	400
	Bridge				2552
18	49C431	3	riding	Unsealed	600
17	49C452	9	riding	Koch 9012	400
16	49C451	9	riding	Mobay 960	400
15	49C450	6	riding	Kold Seal Neo Loop	400
14	49C449	9	riding	Esco PV-687	400
13	49C448	9	riding	Koch 9005	400
	OverPass				3539
12	49C458	3	Soff Cut	Unsealed	406
11	49C457	9	riding	Roshek	591
10	49C456	3	Soff Cut	Dow 890-SL	511

Figure 18. Tremonton, Utah SPS-4 supplemental test section layout.

Table 17. Positions of Tremonton, Utah SPS-4 supplemental test sections in global experimental matrix.

		Conventional Saw				Soff-Cut Saw
		3 mm	6 mm	9 mm	9 mm bevel	3 mm
Unsealed		A 49C430-04 49C431-18				B 49C458-12
Asphalt	Crafco RS 221			C		
	Crafco SS 444					
	Koch 9005			49C441-02 49C448-13		
	Koch 9012			49C440-01 49C452-17		
Silicone	Crafco 902	D	E	F	G	H
	Crafco 903-SL					
	Dow 888					
	Dow 888-SL			49C410-03 49C455-21		
	Dow 890-SL	49C446-08 49C454-20				49C456-10
	Mobay 960			49C443-05 49C447-09 49C451-16 49C453-19		
	Mobay 960-SL					
Neoprene	DS Brown E-437H		I	J		
	DS Brown V-687					
	DS Brown V-812					
	Kold Seal Neo Loop		49C445-07 49C450-15			
	Esco PV 687			49C444-06 49C449-14		
	Watson Bowman 687					
	Watson Bowman 812					
Polysulfide	Koch 9050-SL			K		
Proprietary	Roshek			L 49C457-11		

Tremonton is one of only two of the five SPS-4 supplemental study sites for which some traffic data are available in the LTPP database. Figure 19 illustrates the estimated cumulative ESALs for 1991 through 1996 for Tremonton's linked GPS site (497802). The estimates shown in this figure are summarized in Table 18.

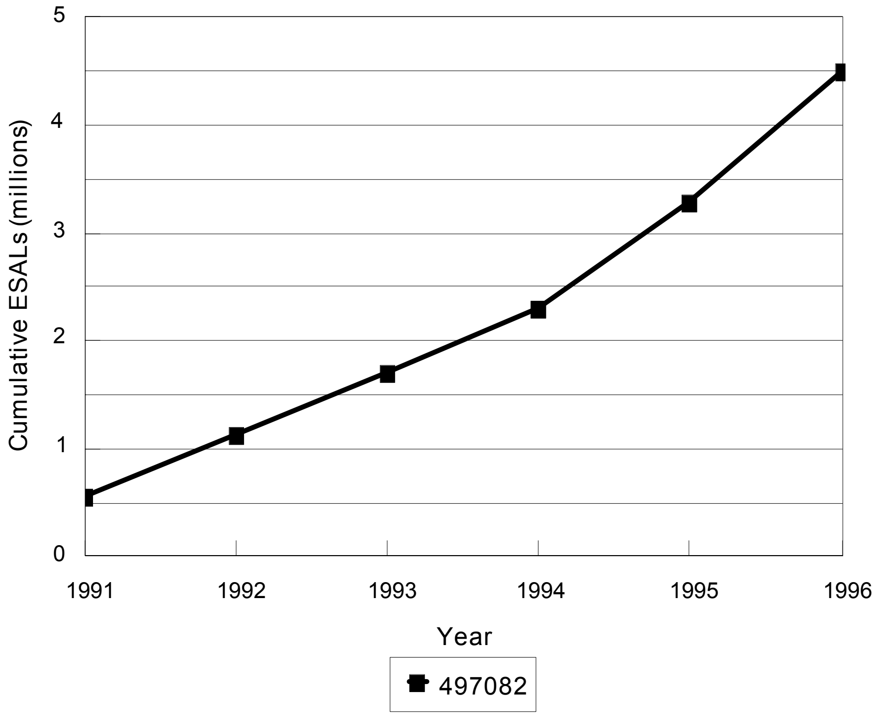


Figure 19. Estimated cumulative ESALs, Tremonton, Utah SPS-4 supplemental study site.

Table 18. Estimated cumulative ESALs, Tremonton, Utah SPS-4 supplemental study site.

SHRP ID	Year	Cumulative ESAL (10^6)
047613	1991	0.19
047613	1992	0.39
	1993	0.60
04A441	1994	0.87
04A441	1995	1.09
04A441	1996	1.29
04A441	1997	1.99
04A441	1998	2.35

The monitoring data from the Tremonton site that were available for this study are summarized in Table 19.

Table 19. Monitoring data available for Tremonton, Utah SPS-4 supplemental test sections.

SPS-4 Site	Year Built	Data Type	Years with Data Available									
			'90	'91	'92	'93	'94	'95	'96	'97	'98	'99
Tremonton, UT	1990	Spalling			✔	✔				✔		
		Sealant Damage			✔	✔				✔		
		Faulting			✔	✔				✔		
		IRI		✔	✔	✔	✔	✔		✔	✔	
		Deflections	*			✔				*		

An asterisk (*) indicates a year in which very limited data are available.

Center-Slab Deflection Analysis

Deflection data were collected in December 1990, July 1993, and September 1997, although very few sections have any deflection data available from the 1990 or 1997 surveys. This leaves only the 1993 deflection data, which lends itself to analysis.

The overall average effective slab thickness in 1993 was 12.32 inches according to the AREA4 method and 13.84 inches according to the AREA5 method. The corresponding overall averages in 1997, for those sections with deflection data available, were 10.31 and 11.30 inches. In every test section for which an effective slab thickness could be calculated for both 1993 and 1997, the effective thickness decreased between these two years. In some test sections the effective thickness decreased by more than two inches. This suggest that frictional resistance to bending between the slab and the cement-treated base diminished between 1993 and 1997.

The overall average dynamic k values calculated from the 1993 deflection data were 432 psi/in by the AREA4 method and 366 psi/in by the AREA5 method. The high group mean dynamic k values are reasonable for the type of subgrade at the site.

Joint Spalling

Joint spalling data were collected in January 1992, July 1993, and September 1997. It is evident that some spalling has occurred in some test sections, but it is difficult to see what meaningful analysis of these data can be conducted, since they contain so many abnormalities. Several sections have spalling reported in the first survey year only, or in the second survey year only. In no case does a section with nonzero spalling show a normal progression of more spalling in the second survey year than the first, and more in the third survey year than the second. These abnormalities are believed to be due to undocumented changes in the spalling severity definitions and/or inconsistency in the subjective assessment of spalling severities.

Both of the narrow, riding saw, unsealed-joint (group A) test sections, 49C430 and 49C431, had some nonzero amount of spalling reported in every survey. The narrow, Soff-Cut, unsealed-joint (group B) test sections, 49C458, had some nonzero spalling reported in the first two surveys, but zero spalling reported in the most recent survey.

Joint Sealant Damage

Joint sealant condition data were collected in January 1992, July 1993, and September 1997. Reference 13 reports that many of the joint sealants have not performed well at the Tremonton site. The highest incidence of joint sealant distresses have been observed in the asphalt-sealed, neoprene-sealed, and proprietary "Roshek"-sealed groups (C, J, and K). The silicone-sealed groups (D, F, and H) have also exhibited lesser amounts of joint sealant distress.

As of the 1997 survey, the sealed-joint test section groups had the following mean weighted joint seal damage index values, on a scale of 1 to 4:

- 9-mm asphalt-sealed (group C) = 3.98
- 3-mm silicone-sealed (group D) = 3.00
- 9-mm silicone-sealed (group F) = 2.93
- 3-mm Soff-Cut silicone-sealed (group H) = 2.84
- 6-mm neoprene-sealed (group I) = 3.97
- 9-mm neoprene-sealed (group J) = 3.97
- 9-mm Roshek-sealed (group L) = 4.00

Joint Faulting

Faulting data were collected in January 1992, July 1993, and September 1997. Faulting data from the 1993 survey are unavailable for several of the test sections. The average absolute faulting group means are illustrated in Figure 20. The results of the statistical comparisons of the 3-mm unsealed joint test section group (A) with the sealed-joint test section groups are summarized in Table 20.

The analysis indicates that in 1997, average absolute faulting in the 3-mm, riding saw, unsealed test section group (A) was significantly higher than in every other group but one, the 6-mm neoprene-sealed group (I), in which the average absolute faulting was similar.

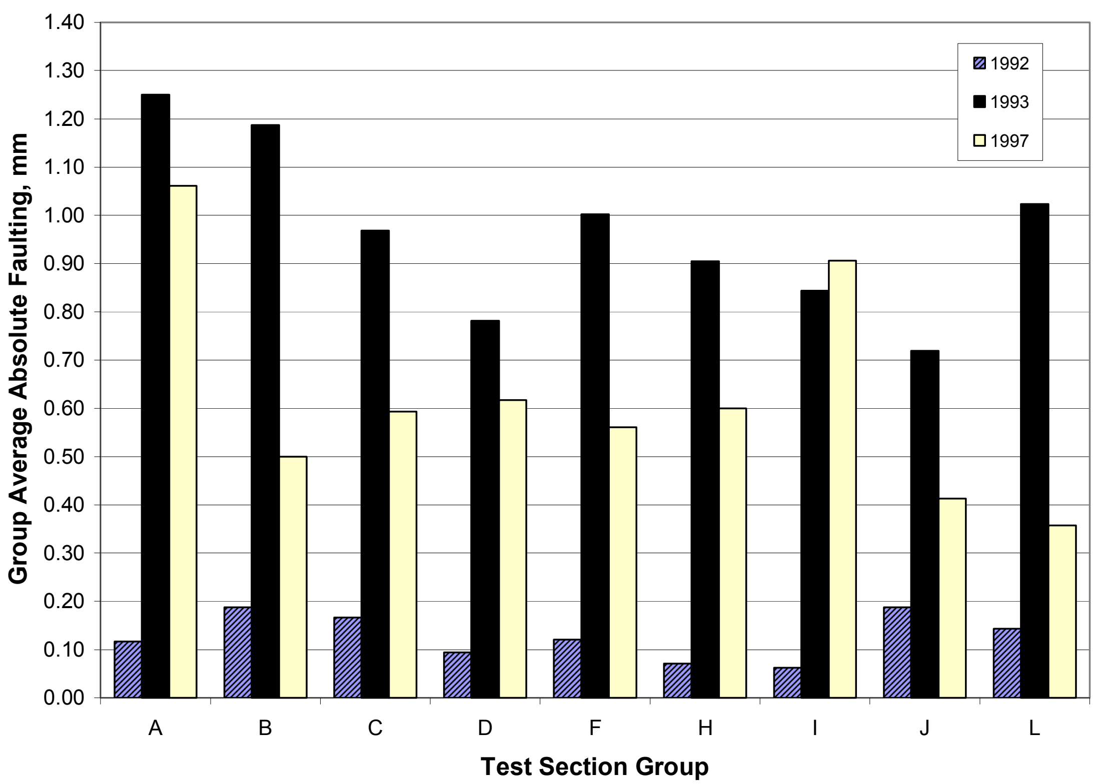


Figure 20. Average absolute faulting in 1992, 1993, and 1997, Tremonton, Utah SPS-4 supplemental test section groups.

Table 20. Average absolute faulting in unsealed versus sealed test section groups, Tremonton, Utah SPS-4 supplemental test section groups.

Test Section Group	1992		1993		1997	
	Average Absolute Faulting, mm	Significantly Different than A?	Average Absolute Faulting, mm	Significantly Different than A?	Average Absolute Faulting, mm	Significantly Different than A?
3-mm unsealed (A)	0.117		1.250		1.061	
3-mm Soff-Cut unsealed (B)	0.188	NO	1.188	NO	0.500	YES
9-mm asphalt sealed (C)	0.167	NO	0.969	NO	0.594	YES
3-mm silicone sealed (D)	0.094	NO	0.781	YES	0.617	YES
9-mm silicone sealed (F)	0.121	NO	1.002	YES	0.561	YES
3-mm Soff-Cut silicone sealed (H)	0.071	NO	0.905	NO	0.600	YES
6-mm neoprene sealed (I)	0.063	NO	0.844	YES	0.906	NO
9-mm neoprene sealed (J)	0.188	NO	0.719	YES	0.413	YES
9-mm Roshek sealed (L)	0.143	NO	1.024	NO	0.357	YES

Roughness

Longitudinal profile data were collected in September 1991, November 1992, December 1993, September 1994, July 1995, October 1997, and August 1998. The increases in group mean IRI between 1992 and 1998 are listed in Table 21. The trends in group mean IRI are illustrated in Figure 21.

Table 21. Increase in IRI between 1991 and 1998, Tremonton, Utah SPS-4 supplemental test section groups.

Group	Description	IRI average 1991	IRI average 1998	IRI increase
A	3-mm unsealed	1.523	1.790	0.267
B	3-mm Soff-Cut unsealed	1.429	1.516	0.087
C	9-mm asphalt sealed	1.516	1.741	0.225
D	3-mm silicone sealed	1.473	1.590	0.117
F	9-mm silicone sealed	1.716	1.828	0.112
H	3-mm Soff-Cut silicone sealed	1.150	1.265	0.115
I	6-mm neoprene sealed	1.496	1.637	0.141
J	9-mm neoprene sealed	1.557	1.726	0.169
L	9-mm Roshek sealed	1.388	1.480	0.092

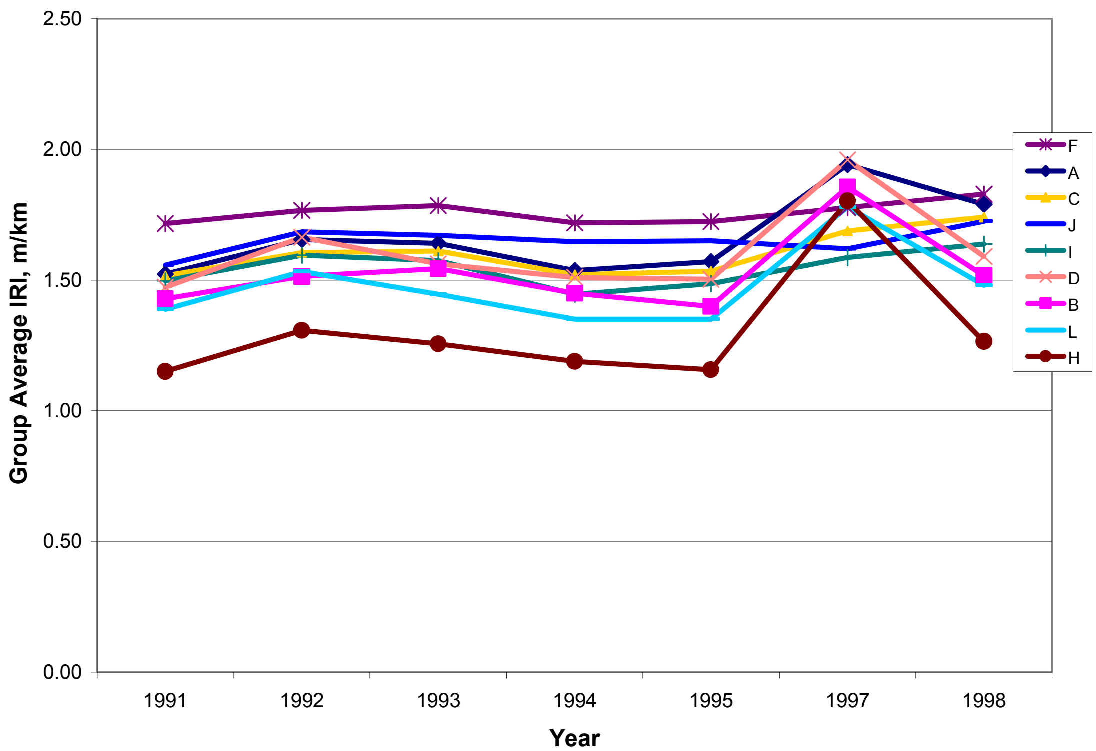


Figure 21. Trends in group mean IRI, Tremonton, Utah
SPS-4 supplemental test section groups.

As this plot illustrates, the order of the groups, from highest to lowest IRI, was not much different in 1998 than in 1991. The two groups with IRI trends notably different than the rest of the pack are group F (9-mm silicone sealed), which has consistently had the highest IRI, and group H (3-mm Soff-Cut, silicone-sealed), which has had the lowest IRI in nearly every year. Five of the nine groups had a jump in IRI in 1997 and a lower IRI in the following year. It is not known what produced these higher IRI values for some, but not all, of the groups in 1997.

Joint Deflection Analysis

Deflection Load Transfer

In the 1993 survey, deflection load transfer was significantly higher in the 3-mm, riding-saw, unsealed test section group (A) than in most other groups, except the groups two neoprene-sealed groups (I and J), which had comparable load transfer. Deflection load transfer was most notably low, compared to that in group A, in the 3-mm, Soff-Cut, unsealed group (B), the 3-mm, Soff-Cut, silicone sealed group (H), and the 9-mm Roshek-sealed group (L).

Total Edge Deflection

For those groups with sufficient data for analysis in 1993 and 1997, total edge deflections are observed to increase between these two years. In 1993, the 3-mm unsealed (group A) total edge deflection was significantly higher than that of two groups (3-mm Soff-Cutt unsealed B, and 9-mm Roshek L), and significantly lower than that of five groups (asphalt-sealed C, 3-mm silicone-sealed D, 9-mm silicone-sealed F, 6-mm neoprene-sealed I, and 9-mm neoprene sealed J). The group A total edge deflection is not significantly different than that of the 3-mm Soff-Cutt silicone-sealed group (H) for the approach-side loading position, but it is significantly lower for the leave-side loading position.

Differential Edge Deflection

In the 1993 survey, the 3-mm unsealed (group A) differential edge deflection was substantially lower than that of every other group except the 9-mm neoprene-sealed group (J). The highest differential edge deflections in the 1993 survey were in groups B, H, and L: the 3-mm Soff-Cut unsealed and silicone-sealed groups, and the Roshek-sealed group.

Transverse Edge Slab Support Ratio

In the 1993 survey, group A had the second-highest slab support ratio, at 1.3, second only to group B. All other groups had slab support ratios between 0.9 and 1.0. Slab support ratios were higher in 1997 than in 1993, for every group for which 1997 data were available to permit a comparison.

Summary Observations

After seven years of service, the sealed-joint test sections at the Tremonton site are all exhibiting moderate to severe joint seal damage. Joint spalling has been reported in some years in some test sections, including the narrow unsealed-joint test sections, but the joint spalling survey data are considered too unreliable for analysis.

The group mean IRI values shortly after construction were rather high for new construction, about 1.5 to 1.75 m/km, but the IRI values have not increased much in the subsequent seven years. The ranking of the treatment groups by IRI is essentially the same in 1998 as in 1991.

The faulting analysis indicates that average absolute faulting in the 3-mm, riding saw, unsealed test sections (group A) was higher in 1997 than in every other treatment group but one. However, both group A and group B (3-mm Soff-Cut unsealed) have consistently had IRI values in the middle of the pack of IRIs of the other groups.

The site has a gravel subgrade and correspondingly high backcalculated dynamic k values. The effective thickness backcalculation results indicate that the combined effective thickness of the slab and cement-treated base decreased between 1993 and 1997.

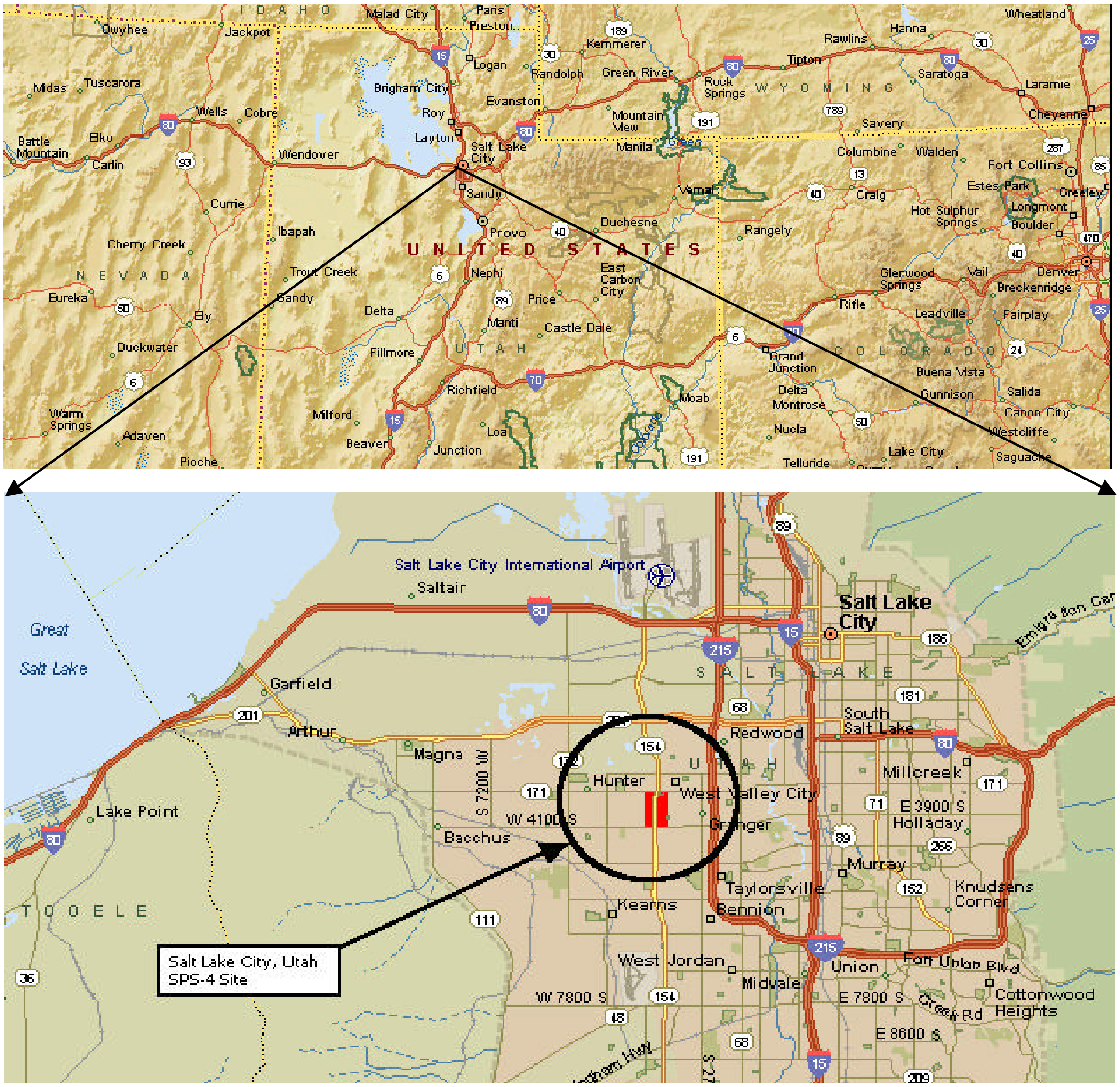
The 3-mm unsealed test sections (group A) have exhibited higher deflection load transfer, lower total joint deflections, and lower differential joint deflections than most of the other treatment groups. The 3-mm unsealed Soff-Cut test section (group B) has exhibited considerably lower deflection load transfer and higher differential deflections than group A. Total joint deflections, on the other hand, are lower in group B than group A. Together, groups A and B have exhibited higher transverse edge slab support ratios than the other treatment groups.

Salt Lake City, Utah

The Salt Lake City SPS-4 supplemental experiment is located on the Bangerter Expressway (Utah Route 154) on the west side of Salt Lake City. An illustration of the location and key location data are given in Figure 22. Thirty-year average monthly temperature and precipitation normals for the weather station nearest the site are given in Figure 23. The site receives an average of 13.6 inches of precipitation annually, and the temperatures range from an average low of 18 °F in January to an average high of 90 °F in July.

This section of Route 154 is a six-lane divided highway, with 12-ft-wide concrete traffic lanes, 12-ft-wide concrete inner shoulders, and concrete curb and gutter along the outer lanes. The test sections were constructed in the fall of 1991 and the spring of 1992. The pavement is a 10-inch JPCP on 4 inches of lean concrete base, 4 inches of crushed gravel subbase, and another 12 inches of poorly graded gravel subbase. A 12-inch geogrid layer of clean, free-draining gravel and filter fabric separate the poorly graded gravel subbase from the sandy clay subgrade. The joints are undowelled, skewed, and spaced at 10, 15, 11, and 14 ft.

The experimental test sections are located in the northbound and southbound lanes. The layout of the test sections is illustrated in Figure 24. The positions of these test sections in the global experimental matrix described earlier are illustrated in Table 22.

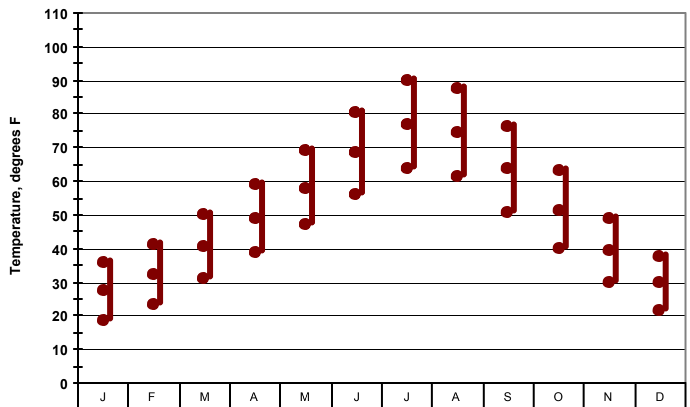


Location

SRHP ID	**49D400**	Linked GPS site	**497086**
State	**Utah**	Route	**Utah 154**
County	**Salt Lake**	Nearest city or town	**Salt Lake City**
Latitude	**40.72**	Longitude	**111.98**

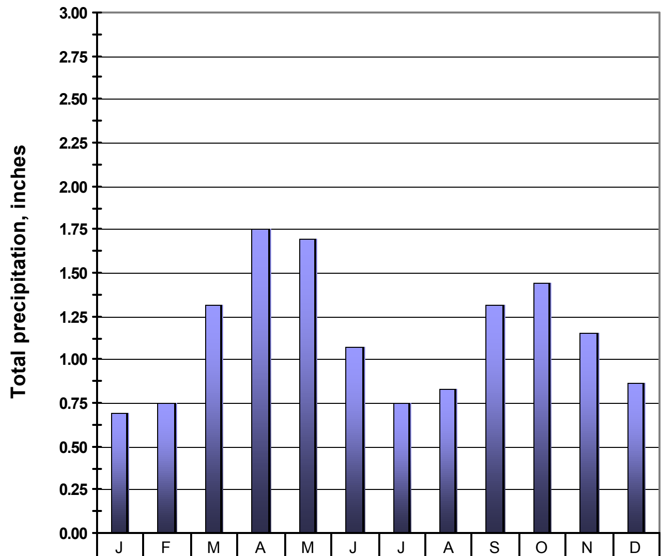
Location Notes: Northbound and southbound on Bangerter Expressway (Utah Route 154), between 3500 S Street and 4100 S Street.

Figure 22. Location of Salt Lake City, Utah SPS-4 supplemental study site.



	J	F	M	A	M	J	J	A	S	O	N	D
Max	35.7	41.2	49.8	58.7	68.7	80.2	89.7	87.2	76	63	49	37.7
Mean	27.1	32.3	40.4	48.7	57.8	68.1	76.6	74.3	63.5	51.3	39.5	29.5
Min	18.4	23.3	31	38.6	46.9	55.9	63.5	61.3	50.8	39.6	29.9	21.3

Month



	J	F	M	A	M	J	J	A	S	O	N	D
Mean	0.69	0.75	1.31	1.75	1.7	1.07	0.75	0.83	1.31	1.44	1.15	0.86

Month

Climate

SRHP ID	**49D400**	Weather station ID	**427578**
State	**Utah**	Weather station name	**Saltair Salt Plant**
County	**Salt Lake**	Weather station latitude	**40.46 N**
Nearest city or town	**Salt Lake City**	Weather station longitude	**112.06 W**
Mean annual temperature	**50.8** °F	Mean annual precipitation	**13.61** in

Climate Notes: Data source is 1961-90 Monthly Station Normals, U.S. Divisional and Station Climatic Data and Normals, 1994, National Oceanic and Atmospheric Administration

Figure 23. Salt Lake City, Utah temperature and precipitation normals.

UT-154 Southbound and Northbound:

Northbound

Section	SHRP ID	joint width (mm)	saw	sealant type	length (ft)
					300
22	49D460	3	Soff Cut	Unsealed	400
21	49D459	3	Soff Cut	Dow 890-SL	400
20	49D462	9	riding	Koch 9050-SL	400
19	49D450	6	riding	DS Brown E-437H	400
18	49D452	9	riding	Koch 9012	400
17	49D454	3	riding	Dow 890-SL	400
16	49D448	9	riding	Crafco RS-221	400
15	49D449	9	riding	DS Brown V-687	400
14	49D451	9	riding	Dow 888	400
13	49D455	9	riding	Dow 888-SL	500
12	49D431	3	riding	Unsealed	600
					300

Southbound

Section	SHRP ID	joint width (mm)	saw	sealant type	length (ft)
					300
11	49D458	3	Soff Cut	Unsealed	400
10	49D456	3	Soff Cut	Dow 890-SL	400
9	49D461	9	riding	Koch 9050-SL	400
8	49D445	6	riding	DS Brown E-437H	400
7	49D440	9	riding	Koch 9012	400
6	49D446	3	riding	Dow 890-SL	400
5	49D441	9	riding	Crafco RS-221	400
4	49D444	9	riding	DS Brown V-687	400
3	49D443	9	riding	Dow 888	400
2	49D410	9	riding	Dow 888-SL	500
1	49D430	3	riding	Unsealed	600

Figure 24. Salt Lake City, Utah SPS-4 supplemental test section layout.

Table 22. Positions of Salt Lake City, Utah SPS-4 supplemental test sections in global experimental matrix.

		Conventional Saw				Soff-Cut Saw
		3 mm	6 mm	9 mm	9 mm bevel	3 mm
Unsealed		A 49D430-01 49D431-12				B 49D458-11 49D460-22
Asphalt	Crafco RS 221			C 49D441-05 49D448-16		
	Crafco SS 444					
	Koch 9005					
	Koch 9012			49D440-07 49D452-18		
Silicone	Crafco 902	D	E	F	G	H
	Crafco 903-SL					
	Dow 888			49D443-03 49D451-14		
	Dow 888-SL			49D410-02 49D455-13		
	Dow 890-SL	49D446-06 49D454-17				49D456-10 49D459-21
	Mobay 960					
	Mobay 960-SL					
Neoprene	DS Brown E-437H		I 49D445-08 49D450-19	J		
	DS Brown V-687			49D444-04 49D449-15		
	DS Brown V-812					
	Kold Seal Neo Loop					
	Esco PV 687					
	Watson Bowman 687					
	Watson Bowman 812					
Polysulfide	Koch 9050-SL			K 49D461-09 49D462-20		
Proprietary	Roshek			L		

The monitoring data from the Salt Lake City site that were available for this study are summarized in Table 23 below.

Table 23. Monitoring data available for Salt Lake City, Utah SPS-4 supplemental test sections.

SPS-4 Site	Year Built	Data Type	Years with Data Available									
			'90	'91	'92	'93	'94	'95	'96	'97	'98	'99
Salt Lake City, UT	1991-1992	Spalling				✔				✔		
		Sealant Damage				✔				✔		
		Faulting				✔				✔		
		IRI				✔	✔	✔		✔	✔	
		Deflections				✔				*		

An asterisk (*) indicates a year in which very limited data are available.

Center-Slab Deflection Analysis

Deflection data were collected in July 1993 and September 1997. Several sections have no deflection data available from the 1997 survey, but the only groups that have no deflection data at all in 1997 (and which therefore cannot be compared with group A) are groups C, I, and J, which are the asphalt-sealed and neoprene-sealed groups. All other groups have 1997 deflection data for at least one test section.

The overall average effective slab thickness in 1993 was 11.53 inches according to the AREA4 method and 12.59 inches according to the AREA5 method. The corresponding overall averages in 1997, for those sections with deflection data available, were 10.60 and 11.41 inches. The decreases in effective thickness between these two years suggest that frictional resistance to bending between the slab and the cement-treated base diminished between 1993 and 1997.

The overall average dynamic k values calculated from the 1993 deflection data were 331 psi/in by the AREA4 method and 293 psi/in by the AREA5 method. The overall average dynamic k values calculated from the 1997 deflection data were 276 psi/in by the AREA4 method and 249 psi/in by the AREA5 method.

Joint Spalling

Joint spalling data were collected in July 1993 and September 1997. The following mean weighted spall lengths were calculated for the treatment groups:

- 3-mm unsealed (group A) = 1.40
- 3-mm Soff-Cut unsealed (group B) = 0
- 9-mm asphalt-sealed (group C) = 2.90
- 3-mm silicone-sealed (group D) = 0
- 9-mm silicone-sealed (group F) = 2.85
- 3-mm Soff-Cut silicone-sealed (group H) = 0
- 6-mm neoprene-sealed (group I) = 1.30
- 9-mm neoprene-sealed (group J) = 0
- 9-mm polysulfide-sealed (group K) = 0.40

Joint Sealant Damage

Joint sealant condition data were collected in July 1993 and September 1997. As of the 1997 survey, the sealed-joint test section groups had the following mean weighted sealant damage index values, on a scale of 1 to 4:

- 9-mm asphalt-sealed (group C) = 2.02
- 3-mm silicone-sealed (group D) = 1.00
- 9-mm silicone-sealed (group F) = 1.75
- 3-mm Soff-Cut silicone-sealed (group H) = 1.00
- 6-mm neoprene-sealed (group I) = 1.00
- 9-mm neoprene-sealed (group J) = 1.87
- 9-mm polysulfide-sealed (group K) = 2.50

Groups D, H, and I have no joint sealant damage, whereas groups C, F, J, and K have a moderate amount. An abnormality in the distress survey procedure is illustrated in the joint sealant data: Group A has a weighted sealant damage index greater than 1.0, specifically 1.33, because of some 6 joints in section 49D431 reported to have high-severity joint sealant damage – even though neither section 49D431 nor the other test section in group A, 49D430, has sealed joints.

Joint Faulting

Faulting data were collected in July 1993 and September 1997. Faulting data from the 1993 survey are unavailable for several of the test sections. The average absolute faulting group means are illustrated in Figure 25. The results of the statistical comparisons of the 3-mm unsealed joint test section group (A) with the sealed-joint test section groups are summarized in Table 24.

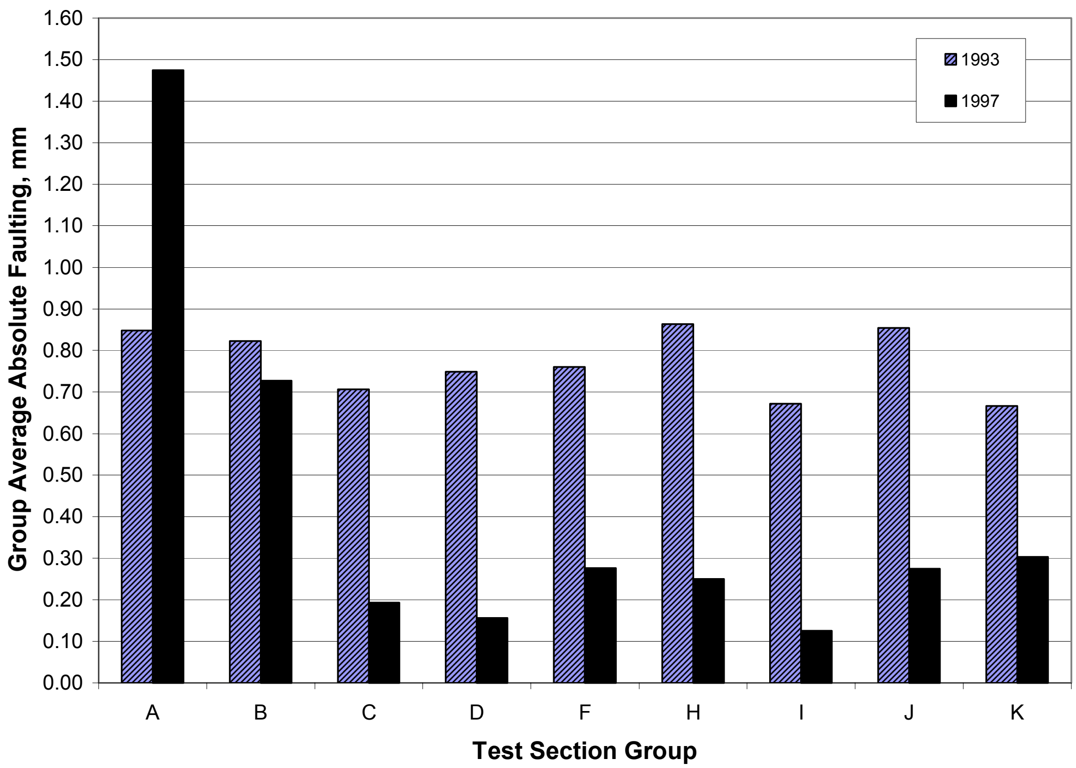


Figure 25. Average absolute faulting in 1993 and 1997, Salt Lake City, Utah SPS-4 supplemental test section groups.

Table 24. Average absolute faulting in unsealed versus sealed test section groups, Salt Lake City, Utah SPS-4 supplemental test section groups.

Test Section Group	1993		1997	
	Average Absolute Faulting, mm	**Significantly Different than A?**	**Average Absolute Faulting, mm**	**Significantly Different than A?**
3-mm unsealed (A)	0.848		1.474	
3-mm Soff-Cut unsealed (B)	0.823	NO	0.727	YES
9-mm asphalt sealed (C)	0.706	NO	0.192	YES
3-mm silicone sealed (D)	0.749	NO	0.156	YES
9-mm silicone sealed (F)	0.761	NO	0.276	YES
3-mm Soff-Cut silicone sealed (H)	0.864	NO	0.250	YES
6-mm neoprene sealed (I)	0.672	NO	0.125	YES
9-mm neoprene sealed (J)	0.855	NO	0.274	YES
9-mm polysulfide sealed (K)	0.667	NO	0.303	YES

In 1993, average absolute faulting in group A was not significantly different than in any other group, but that in 1997, average absolute faulting in group A was significantly higher than in every other group. All of the sealed-joint sections exhibited a decrease in average absolute faulting between 1993 and 1997, which may be due to a change in the measurement equipment or calibration, different degrees of slab curling during the two surveys, or decreases in negative faulting. Since not every group exhibited a decrease in faulting, the last of these three possible explanations is considered to be the most likely in this case.

Roughness

Longitudinal profile data were collected in December 1993, September 1994, July 1995, November 1997, and December 1998. The increases in group mean IRI between 1993 and 1998 are listed in Table 25. The trends in group mean IRI are illustrated in Figure 26.

As Figure 26 illustrates, the order of the groups, from highest to lowest IRI, has not varied much between 1993 and 1998. The three groups with the highest IRIs have been group A (3-mm unsealed), group B (3-mm Soff-Cut unsealed), and group F (9-mm silicone-sealed. The group A mean IRI has increased more than the other group mean IRI values from 1995 on. A drop in IRI was seen for all treatment groups in 1995.

Table 25. Increase in IRI between 1993 and 1998, Salt Lake City, Utah SPS-4 supplemental test section groups.

Group	Description	IRI average 1993	IRI average 1998	IRI increase
A	3-mm unsealed	1.573	2.194	0.621
B	3-mm Soff-Cut unsealed	1.433	1.975	0.542
C	9-mm asphalt sealed	1.352	1.659	0.307
D	3-mm silicone sealed	1.439	1.807	0.368
F	9-mm silicone sealed	1.559	1.931	0.372
H	3-mm Soff-Cut silicone sealed	1.350	1.738	0.388
I	6-mm neoprene sealed	1.405	1.845	0.440
J	9-mm neoprene sealed	1.256	1.639	0.383
K	9-mm polysulfide sealed	1.238	1.724	0.486

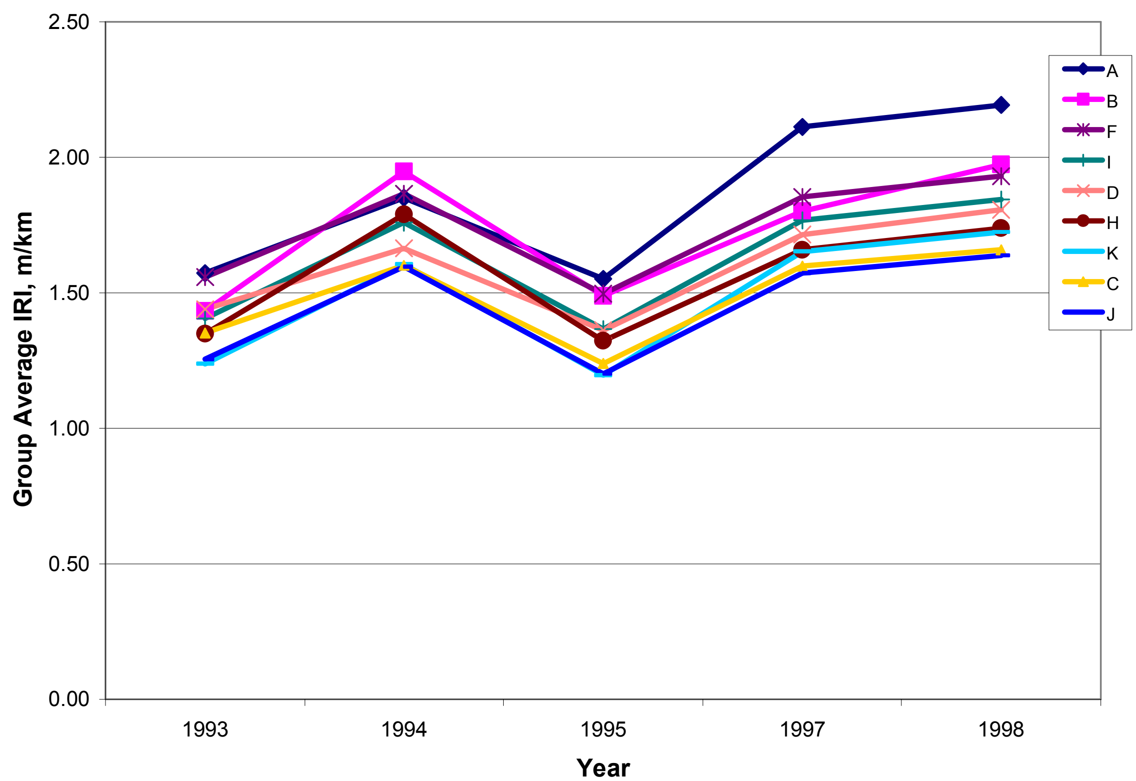


Figure 26. Trends in group mean IRI, Salt Lake City, Utah SPS-4 supplemental test section groups.

Joint Deflection Analysis

Deflection Load Transfer

In the 1993 survey, deflection load transfer was significantly higher in the 3-mm, riding-saw, unsealed test section group (A) than in every other groups. In the 1997 survey, deflection load transfer in group A was comparable to that of the other groups, except that group K (9-mm polysulfide-sealed) had considerably higher deflection load transfer, and group D (3-mm silicone-sealed) had considerably lower deflection load transfer.

Total Edge Deflection

For all groups except group D (3-mm silicone-sealed), total edge deflections increased between 1993 and 1997. In 1993, the group A (3-mm unsealed) total edge deflection was significantly higher than that of most other groups. In 1997, the group A (3-mm unsealed) total edge deflection was significantly higher than that of groups B and D (3-mm Soff-Cut unsealed and 3-mm silicone-sealed), but lower than that of groups F and H (9-mm silicone-sealed and 3-mm Soff-Cut silicone sealed).

Differential Edge Deflection

In both the 1993 and 1997 surveys, the group A (3-mm unsealed) differential edge deflection was similar to that of most other groups. The group that has exhibited differential deflections most notably different than those of the other groups is D (3-mm silicone-sealed).

Transverse Edge Slab Support Ratio

Transverse edge slab support ratios in 1993 and 1997 were acceptable in all groups but one: the group F (9-mm silicone-sealed) slab support ratio dropped to 0.4 in 1997. The highest mean slab support ratios in 1997 was in group B (3-mm Soff-Cut unsealed).

Summary Observations

After six years of service, some of the sealed-joint test sections at the Salt Lake City site are exhibiting minor to moderate joint seal damage. Joint spalling has been observed in one of the group A (3-mm unsealed) test sections, but zero joint spalling is reported for the other three narrow-joint test section groups: B (3-mm Soff-Cut unsealed), D (3-mm silicone-sealed), and H (3-mm Soff-Cut silicone-sealed).

The group mean IRI values increased in all of the treatment groups between 1993 and 1998. The ranking of the treatment groups by IRI is very similar in 1998 as the ranking in 1993. The mean IRI of group A (3-mm unsealed) test sections increased more than the other group mean IRIs after 1995. This is consistent with the finding that faulting in group A was not significantly different than that in the other groups in 1993, but was significantly higher than that in every other group in 1997.

In 1993, group A had better deflection load transfer than all of the sealed-joint groups, but it also had higher total edge deflections than most of the other groups. Deflection load transfer in group A dropped by 1997 to levels comparable to those of most of the other groups. Overall, the 1997 joint deflection behavior of group A test sections is similar to that of the other groups, so it is difficult to identify any clear relationship between the increase in faulting and IRI and some deterioration in joint deflection behavior.

Heber City, Utah

The Heber City SPS-4 supplemental experiment is located on U.S. 40, southeast of Salt Lake City. An illustration of the location and key location data are given in Figure 27. Thirty-year average monthly temperature and precipitation normals are given in Figure 28 for the weather station nearest the site with both temperature and precipitation data available. The site receives an average of 24.1 inches of precipitation annually, and the temperatures range from an average low of 13 °F in January to an average high of 89 °F in July.

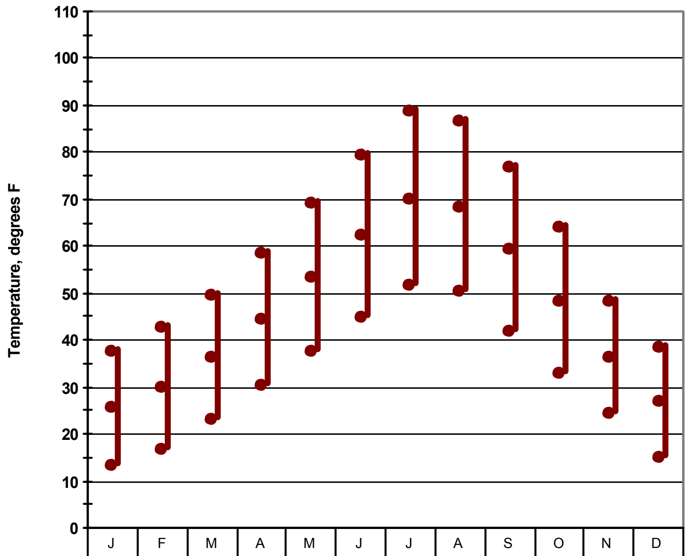
This section of U.S. 40 is a four-lane divided highway, with 12-ft-wide concrete traffic lanes, 4-ft-wide concrete inner shoulders, and 8-ft-wide concrete outer shoulders. The test sections were constructed in September 1990. The pavement is a 10-inch JPCP. The base layer in the westbound lanes is an unknown thickness of asphalt concrete, while the base layer in the eastbound lanes is 4 inches of lean concrete. The subbase in both directions is 4 inches of crushed gravel subbase, and another 18 inches of silty sandy gravel. The subgrade is a poorly graded gravel. The joints are undowelled, skewed, and spaced at 10, 15, 11, and 14 ft.

Location

SRHP ID	**49E400**	Linked GPS site	**497085**
State	**Utah**	Route	**US 40**
County	**Wasatch**	Nearest city or town	**Heber City**
Latitude	**40.57**	Longitude	**111.43**

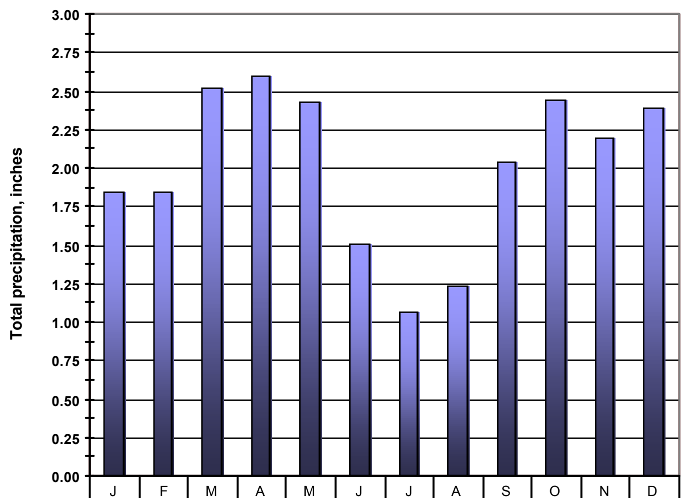
Location Notes: Eastbound and westbound on US 40, technically an east-west route, although it runs north-south in the vicinity of the test site.

Figure 27. Location of Heber City, Utah SPS-4 supplemental study site.



	J	F	M	A	M	J	J	A	S	O	N	D
Max	37.7	42.8	49.4	58.6	69	79.5	88.6	86.4	76.6	63.8	48.1	38.3
Mean	25.4	29.7	36.3	44.5	53.4	62.1	70.1	68.3	59.2	48.3	36.1	26.7
Min	13.1	16.6	23.1	30.3	37.7	44.6	51.5	50.1	41.9	32.7	24.1	15.1

Month



	J	F	M	A	M	J	J	A	S	O	N	D
Mean	1.84	1.85	2.52	2.6	2.43	1.51	1.07	1.24	2.04	2.44	2.19	2.39

Month

Climate

SRHP ID	**49E400**	Weather station ID	**425892**
State	**Utah**	Weather station name	**Mountain Dell Dam**
County	**Wasatch**	Weather station latitude	**40.45 N**
Nearest city or town	**Heber City**	Weather station longitude	**111.50 W**
Mean annual temperature	**46.7** °F	Mean annual precipitation	**24.12** in

Climate Notes: Nearest weather station is 421446, City Creek Water Plant, but only precipitation data are available for this station. Second-nearest weather station is 425892, Mountain Dell Dam. Data source is 1961-90 Monthly Station Normals, U.S. Divisional and Station Climatic Data and Normals,

Figure 28. Heber City, Utah temperature and precipitation normals.

The monitoring data from the Heber City site that were available for this study are summarized in Table 26 below.

Table 26. Monitoring data available for Heber City, Utah SPS-4 supplemental test sections.

SPS-4 Site	Year Built	Data Type	Years with Data Available									
			'90	'91	'92	'93	'94	'95	'96	'97	'98	'99
Heber City, UT	1990	Spalling		*		*				✔		
		Sealant Damage		*		*				✔		
		Faulting				✔				✔		
		IRI			✔	✔		✔		✔	✔	
		Deflections				✔				*		

An asterisk (*) indicates a year in which very limited data are available.

The experimental test sections are located in the eastbound and westbound lanes. The layout of the test sections is illustrated in Figure 29. The positions of these test sections in the global experimental matrix described earlier are illustrated in Table 27.

Center-Slab Deflection Analysis

Deflection data were collected in July 1993 and September 1997. Several sections have no deflection data available from the 1997 survey, so the only groups that can be compared with group A are groups B, D, F, and H.

For the eastbound sections with the lean concrete base, the overall average effective slab thickness in 1993 was 10.57 inches according to the AREA4 method and 11.25 inches according to the AREA5 method. For the westbound sections with the asphalt concrete base, the overall average effective slab thickness in 1993 was similar: 10.40 inches according to the AREA4 method and 11.37 inches according to the AREA5 method.

For the 1997 data collected within selected eastbound test sections constructed over the lean concrete base, the AREA4 and AREA5 analyses yield average effective thicknesses of 9.43 inches and 9.76 inches, respectively, compared to the 1993 averages of 10.82 inches and 11.45 inches, respectively, for the same five sections. The decrease in effective slab thickness could be attributed in part to loss of slab/base bond, but the k value backcalculation results suggest that this bond was already diminished in 1993.

US-40 Eastbound and Westbound:

Westbound

Section	SHRP ID	joint width (mm)	saw	sealant type	length (ft)
					12
1	49E460	3	Soff Cut	Unsealed	488
2	49E459	3	Soff Cut	Dow 890-SL	500
3	49E462	9	riding	Koch 9050-SL	500
4	49E449	9	riding	DS Brown V-687	464
					36
5	49E448	9	riding	Koch 9005	500
6	49E450	6	riding	DS Brown E-437H	500
7	49E454	3	riding	Dow 890-SL	500
8	49E452	9	riding	Koch 9012	500
					75
9	49E451	9	riding	Dow 888	425
10	49E455	9	riding	Dow 888-SL	500
					100
11	49E431	3	riding	Unsealed	500

Eastbound

Section	SHRP ID	joint width (mm)	saw	sealant type	length (ft)
22	49E458	3	Soff Cut	Unsealed	500
21	49E456	3	Soff Cut	Dow 890-SL	500
20	49E461	9	riding	Koch 9050-SL	500
19	49E445	6	riding	DS Brown E-437H	500
18	49E440	9	riding	Koch 9012	500
17	49E446	3	riding	Dow 890-SL	500
16	49E444	9	riding	DS Brown V-687	500
15	49E441	9	riding	Koch 9005	500
14	49E443	9	riding	Dow 888	500
13	49E410	9	riding	Dow 888-SL	500
					100
12	49E430	3	riding	Unsealed	500

Figure 29. Heber City, Utah SPS-4 supplemental test section layout.

Table 27. Positions of Heber City, Utah SPS-4 supplemental test sections in global experimental matrix.

		Conventional Saw				Soff-Cut Saw
		3 mm	6 mm	9 mm	9 mm bevel	3 mm
Unsealed		A 49E431-11 49E430-12				B 49E460-01 49E458-22
Asphalt	Crafco RS 221			C		
	Crafco SS 444					
	Koch 9005			49E448-05 49E441-15		
	Koch 9012			49E452-08 49E440-18		
Silicone	Crafco 902	D	E	F	G	H
	Crafco 903-SL					
	Dow 888			49E451-09 49E443-14		
	Dow 888-SL			49E455-10 49E410-13		
	Dow 890-SL	49E454-07 49E446-17				49E459-02 49E456-21
	Mobay 960					
	Mobay 960-SL					
Neoprene	DS Brown E-437H		I 49E450-06 49E445-19	J		
	DS Brown V-687			49E449-04 49E444-16		
	DS Brown V-812					
	Kold Seal Neo Loop					
	Esco PV 687					
	Watson Bowman 687					
	Watson Bowman 812					
Polysulfide	Koch 9050-SL			K 49E462-03 49E461-20		
Proprietary	Roshek			L		

The westbound test sections at the Heber City site have an asphalt concrete base, while the eastbound test sections have a lean-concrete base. The overall average dynamic k values calculated for the westbound test sections from the 1993 deflection data were 594 psi/in by the AREA4 method and 513 psi/in by the AREA5 method. The overall average dynamic k values calculated for the eastbound test sections from the 1993 deflection data were 460 psi/in by the AREA4 method and 414 psi/in by the AREA5 method.

The reason for the difference between eastbound and westbound k values is not fully known. Base type is expected to influence the backcalculated effective modulus or effective thickness of the slab, but is not expected to significantly influence backcalculated k value, as long as the slab and base are in full contact. However, downward curling caused by a positive temperature gradient could have a more noticeable effect in reducing k values for slabs over stiffer bases.

Slab surface temperature data collected during the 1993 survey indicate general uniformity between eastbound and westbound section pairs. For the most part, slab temperature gradient data were collected only in the eastbound lanes, those constructed over the lean concrete base. For the limited temperature gradient data collected in the westbound lane, only one section (49E452, 9-mm asphalt-sealed) has a corresponding eastbound section (49E441) with temperature gradient data. For this section pair, the eastbound surface temperature and slab temperature gradient were 93°F and 0.96°/inch, respectively, while the westbound surface temperature and slab temperature gradient were 102°F and 0.96°F/inch, respectively. The average incremental dynamic foundation k value backcalculated for the westbound section 49E452, constructed over the asphalt concrete base, was 504 psi/inch from the AREA4 analysis and 437 psi/inch from the AREA5 analysis. The average incremental dynamic foundation k value backcalculated for the eastbound section 49E441, constructed over the lean concrete base, was 388 psi/inch from the AREA4 analysis and 348 psi/inch from the AREA5 analysis. If the underlying subgrade and subbase materials are similar in both directions, the lower backcalculated k values for the eastbound section is most likely due to downward curling of the slab and some loss of contact between the slab and lean concrete base.

The dynamic foundation k values backcalculated from the 1997 data, collected only in five sections over the lean concrete base, have an overall average of 360 psi/inch from the AREA4 analysis and 336 psi/inch from the AREA5 analysis – lower than the 1993 average k values of 451 psi/inch and 405 psi/inch, respectively, for these same five test sections. Slab surface temperatures were higher during the 1993 survey than during the 1997 survey, but positive temperature gradients of similar magnitudes were recorded during both surveys.

Joint Spalling

Joint spalling data were collected in July and September of 1991, July 1993 and September 1997. Nonzero spalling was reported in the 1997 survey only in one test section each in groups C, D, and F.

Joint Sealant Damage

Joint sealant condition data were collected in July and September of 1991, July 1993 and September 1997. The following mean weighted sealant damage index values, on a scale of 1 to 4, were calculated for the treatment groups:

- 9-mm asphalt-sealed (group C) = 3.83
- 3-mm silicone-sealed (group D) = 3.75
- 9-mm silicone-sealed (group F) = 3.35
- 3-mm Soff-Cut silicone-sealed (group H) = 2.57
- 6-mm neoprene-sealed (group I) = 3.98
- 9-mm neoprene-sealed (group J) = 4.00
- 9-mm polysulfide-sealed (group K) = 3.96

An abnormality in the distress survey procedure is illustrated in the joint sealant data: one of the test sections in Group A was reported to have no joint sealant damage of any severity, while the other test section in group A was reported to have high-severity joint sealant damage at 48 joints – even though neither of the two test sections in group A has sealed joints.

Joint Faulting

Faulting data were collected in July 1993 and September 1997. The average absolute faulting group means are illustrated in Figure 30. The results of the statistical comparisons of the 3-mm unsealed-joint test section group (A) with the sealed-joint test section groups are summarized in Table 28.

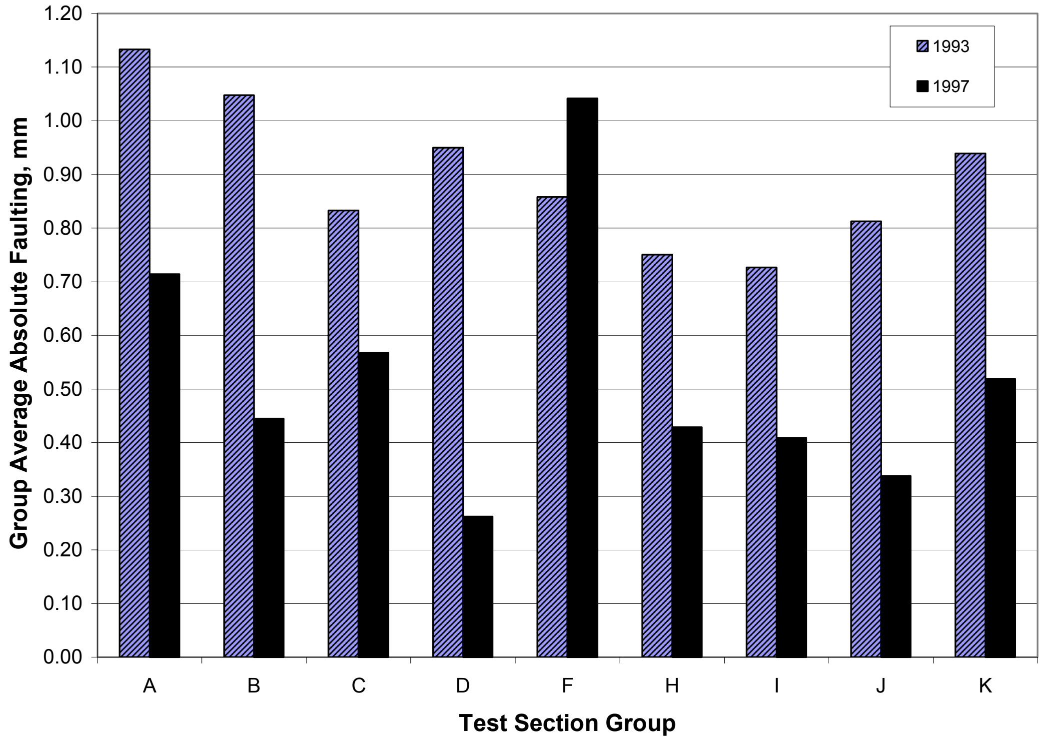


Figure 30. Average absolute joint faulting in 1993 and 1997, Heber City, Utah SPS-4 supplemental test section groups.

Table 28. Average absolute faulting in unsealed versus sealed test section groups, Heber City, Utah SPS-4 supplemental test section groups.

Test Section Group	1993		1997	
	Average Absolute Faulting, mm	Significantly Different than A?	Average Absolute Faulting, mm	Significantly Different than A?
3-mm unsealed (A)	1.133		0.714	
3-mm Soff-Cut unsealed (B)	1.048	NO	0.445	NO
9-mm asphalt sealed (C)	0.833	YES	0.568	NO
3-mm silicone sealed (D)	0.950	NO	0.263	YES
9-mm silicone sealed (F)	0.858	YES	1.042	NO
3-mm Soff-Cut silicone sealed (H)	0.751	YES	0.429	NO
6-mm neoprene sealed (I)	0.727	YES	0.409	YES
9-mm neoprene sealed (J)	0.813	YES	0.338	YES
9-mm polysulfide sealed (K)	0.939	NO	0.519	NO

The majority of the test sections had lower average absolute faulting in 1997 than in 1993, which may be due to a change in the measurement equipment or calibration, different degrees of slab curling during the two surveys, or decreases in negative faulting. Since not every group exhibited a decrease in faulting, the last of these three possible explanations seems the most likely in this case. In 1997, the 9-mm silicone-sealed group (F) had the highest average absolute faulting, followed by the 3-mm unsealed group (A).

Roughness

Longitudinal profile data were collected in November 1992, November 1993, July 1995, November 1997, and August 1998. The increases in group mean IRI between 1992 and 1998 are listed in Table 28. The trends in group mean IRI are illustrated in Figure 31.

Table 28. Increase in IRI between 1992 and 1998, Heber City, Utah
SPS-4 supplemental test section groups.

Group	Description	IRI average 1992	IRI average 1998	IRI increase
A	3-mm unsealed	1.429	1.831	0.402
B	3-mm Soff-Cut unsealed	1.239	1.708	0.469
C	9-mm asphalt sealed	1.248	1.518	0.270
D	3-mm silicone sealed	1.188	1.273	0.085
F	9-mm silicone sealed	1.415	1.745	0.330
H	3-mm Soff-Cut silicone sealed	1.176	1.332	0.156
I	6-mm neoprene sealed	1.234	1.413	0.179
J	9-mm neoprene sealed	1.081	1.304	0.223
K	9-mm polysulfide sealed	1.200	1.384	0.184

As Figure 31 illustrates, the order of the groups, from highest to lowest IRI, has not varied much between 1991 and 1998. The three groups with the highest IRIs have been group A (3-mm unsealed), group F (9-mm silicone-sealed, and group B (3-mm Soff-Cut unsealed).

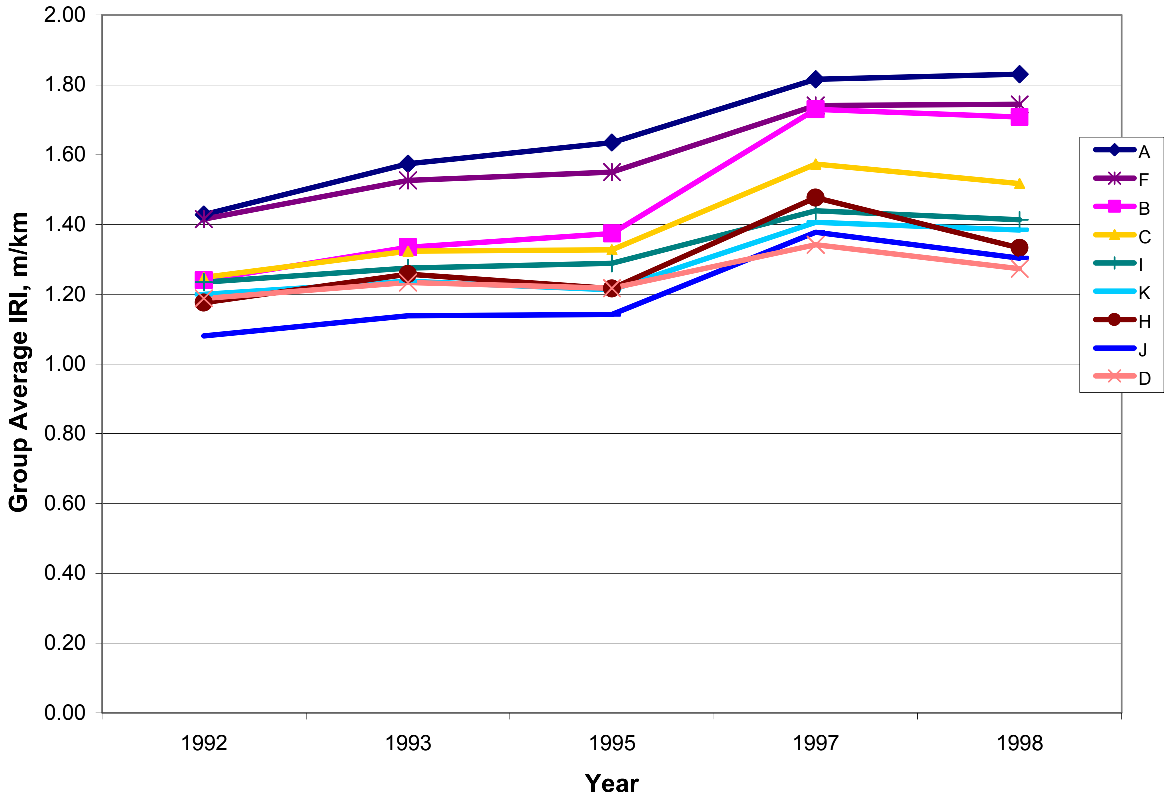


Figure 31. Trends in group mean IRI, Heber City, Utah
SPS-4 supplemental test section groups.

Joint Deflection Analysis

Deflection Load Transfer

In the 1993 survey, deflection load transfer was significantly higher in the 3-mm, riding-saw, unsealed test section group (A) than in all but one of the other groups. Similarly, in the 1997 survey, deflection load transfer in group A was higher than in every other group with which a comparison is possible.

Total Edge Deflection

In the 1993 survey, total edge deflection was significantly higher in the 3-mm, riding-saw, unsealed test section group (A) than in all but one of the other groups. In the 1997 survey, however, total

edge deflection in group A was higher than that in two groups and (D and F) and lower than that in two other groups (B and H).

Differential Edge Deflection

In both the 1993 and 1997 surveys, differential deflections in all groups were comparable.

Transverse Edge Slab Support Ratio

Transverse edge slab support ratios in 1993 and 1997 were acceptable in all groups.

Summary Observations

After six years of service, all of the sealed-joint test sections at the Heber City site are exhibiting moderate to severe joint seal damage. Joint spalling has been observed in three sealed-joint test sections, but not in any unsealed-joint test sections.

The group mean IRI values increased in all of the treatment groups between 1992 and 1998. The ranking of the treatment groups by IRI is very similar in 1998 as the ranking in 1992. The three groups with the highest IRIs, and highest rates of IRI increase, have been group A (3-mm unsealed), group F (9-mm silicone-sealed, and group B (3-mm Soff-Cut unsealed). Groups F and A also had the highest faulting in 1997. However, it should be noted that these two groups had notably higher IRI than all of the other groups even in 1992, within a year of construction.

The foundation k value and effective slab thickness backcalculation results suggest that downward curling may be responsible for greater loss of contact between the slab and base in the eastbound test sections, which were built on a lean concrete base.

In 1993 and 1997, group A had better deflection load transfer than all of the sealed-joint groups. However, in 1993 total edge deflections in group A were higher than in most other groups. 1997 total edge deflections were highest in the two 3-mm Soff-Cut groups (B, unsealed, and H, silicone-sealed), followed by group A. In terms of other joint deflection parameters, the behavior of all groups is comparable.

Chapter 5

Conclusions

Joint sealing is commonly believed to be beneficial to concrete pavement performance in two ways. Sealed joints are believed to reduce water infiltration into the pavement structure, thereby retarding the occurrence of moisture-related joint distresses such as pumping, faulting, corner breaking, and freeze-thaw damage (D cracking). Sealed joints are also believed to reduce or prevent the infiltration of incompressible materials (i.e., sand and small stones) into the joints, thereby reducing the likelihood of pressure-related joint distresses such as spalling and blowups.

NCHRP 20-50(2) was conducted for the purpose of comparing, based on the data available in the LTPP database, the performance of JPCP designed and constructed with unsealed joints to that of JPCP with sealed joints. There are just five LTPP sites suitable for this analysis: the SPS-4 supplemental test sites in Arizona, Colorado, and Utah. Obviously these sites are all located in the dry western region of the United States, for which reason it would be unwise to extrapolate the results of this analysis to other regions of the country that receive more precipitation. Nonetheless it is hoped that the analysis methods used in this study may serve as useful examples for future analyses of other sealed-versus-unsealed joint experiments in other climates.

Despite the conventional wisdom concerning the need to keep concrete pavement joints well sealed, previous studies on the subject have not demonstrated that JPCP with sealed joints and JPCP with unsealed joints perform differently in terms of spalling, faulting, IRI, or deflections.

The analyses conducted in this study do not indicate that unsealed joints are any more likely to develop joint spalling than sealed joints. At two of the five sites considered, no joint spalling was observed in any test sections. At two other sites, minor joint spalling was reported in some unsealed-joint and sealed-joint test sections, and at one site, the only spalling observed was in sealed-joint test sections.

The youngest site (Campo, Colorado) as yet shows no significant differences in faulting among any of the joint sealant treatment groups. It should be noted that this is also the only one of the five sites at which the joints are doweled. At the other four sites, faulting tends to be highest in two groups: the 3-mm unsealed group (A), and the 9-mm silicone-sealed group (F).

At three of the fives sites, the rate of IRI increase was highest in the 3-mm unsealed group (A), but at one site, it was lowest in this group, and at the fifth site, it was no different than in the other groups. Groups A and F, and in some cases group B (3-mm Soff-Cut unsealed), have had the highest IRIs among the treatment groups. It is important to note, however, that in every case, the order of treatment groups by IRI is no different after five, or seven, or nine years of service than it was in the first year after construction. This underscores the importance of analyzing data on initial IRI values and rates of IRI increase, rather than just IRI magnitudes in later years, to detect significant differences in roughness development by treatment type.

The narrow unsealed-joint test sections did, in general, exhibit more faulting and higher rates of IRI increase than most other treatment groups. However, the same is true of one particular sealed-joint group: the 9-mm silicone-sealed group (F). Why this sealed-joint design would differ from the others in terms of faulting and IRI is unknown. The signficance of this finding is that it would be an inaccurate overgeneralization to conclude that the unsealed-joint treatment resulted in more faulting and roughness than the sealed-joint treatments.

The narrow unsealed-joint test sections (groups A and B, formed using riding saws and Soff-Cut saws, respectively) have actually tended to exhibit better deflection load transfer and other joint deflection responses than the sealed-joint test sections. At some sites, one or both of these unsealed-joint groups exhibited higher total deflections (loaded plus unloaded sides of the joint) than the sealed-joint groups. Total joint deflection is the only joint deflection parameter that may potentially be correlated to the higher faulting and IRI in the unsealed-joint test sections. However, it is not concluded on the basis of these analysis results that such a correlation exists. One reason to doubt that it exists is that it would not explain the higher faulting in the group F test sections, which did not necessarily have higher total joint deflections.

It should also be kept in mind that at three of the five sites, the sealed-joint test sections are not necessarily well sealed – they have moderate to severe joint-seal damage. How well-sealed the sealed-joint test sections really are is a factor that should be considered in future analyses of the longer-term performance of the pavements at these five sites, as well as analyses of sealed-versus-unsealed joint experiments in other climates.

References

[1] Schober, S. F., "The Great Unsealing: A Perspective on Portland Cement Concrete Joint Sealing," Transportation Research Record 1597, 1997.

[2] Scofield, L. A., "Results from the Western States´ Sealant Experiment," Arizona Transportation Research Center.

[3] Federal Highway Administration, "Report on the 1992 U. S. Tour of European Concrete Highways," Report FHWA-SA-93-012, 1993.

[4] Smith, K. D. and Hall, K. T., "Concrete Pavement Design and Construction Practices – State of the Art Technical Digest," Federal Highway Administration/National Highway Institute training course, 1999.

[5] American Concrete Pavement Association, "Design and Construction of Joints for Concrete Highways," Technical Bulletin TB010.0D, 1991.

[6] Smith, R. E., Freeman, T. J., and Pendleton, O., "Pavement Maintenance Effectiveness," Report SHRP-H-358, 1993.

[7] Morian, D. A., Epps, J. A., and Gibson, S. D., "Pavement Treatment Effectiveness, 1995 SPS-3 and SPS-4 Site Evaluations, National Report," Report No. FHWA-RD-96-208, 1997.

[8] Morian, D. A., Gibson, S. D., and Epps, J. A., "Concrete Pavement Maintenance Treatment Performance Review: SPS-4 5-Year Data Analysis," Report No. FHWA-RD-97-155, 1998.

[9] Owusu-Antwi, E. B., Titus-Glover, and Darter, M. I., "Design and Construction of PCC Pavements, Volume 1: Summary of Design Features and Construction Practices that Influence the Performance of Pavements," Report No. FHWA-RD-98-052, 1998.

[10] Khazanovich, L., Darter, M. I., Bartlett, R. J., and McPeak, T., "Common Characteristics of Good and Poorly Performing Pavements," Report No. FHWA-RD-97-131, 1998.

[11] Perera, R. W., Byrum, C., and Kohn, S. D., "Investigation of Development of Pavement Roughness," Report No. FHWA-RD-97-147, 1998.

[12] Byrum, C., “Analysis of Slab Curvatures Using High-Speed Profiles,” Annual Meeting of the Transportation Research Board, Washington, D.C., 2000, in press.

[13] Smith, K. L., Pozsgay, M. A., Evans, L. D., and Romine, A. R.,”LTPP Pavement Maintenance Materials: SPS-4 Supplemental Joint Seal Experiment,” Report FHWA-RD-99-151, 1999.

[14] Evans, L. D., “SHRP Joint Seal Study: A Seven-Year Look,” Annual Meeting of the Transportation Research Board, Washington, D.C., 1999, in press.

[15] Meier, W. R., Jr. and Elnicky, E. J., “Construction Report for Arizona’s SHRP SPS-4 Experiment,” Arizona Department of Transportation Report AZ92-377-1, 1992.