

Performance of Arizona's SPS-4 Joint Sealing Experiment

Kathleen T. Hall, ProTech Engineering
Lynn D. Evans, ERES Consultants, A Division of ARA, Inc.
James A. Crovetti, Marquette University
Carlos E. Correa, ProTech Engineering
Larry Scofield, Arizona DOT

Abstract

Currently, nearly all state highway agencies require sealing of transverse joints in jointed plain concrete pavement (JPCP). Joint sealing is estimated to increase initial construction costs by approximately 2 to 7 percent. Periodic resealing of joints also increases the costs of these pavements. Sealed joints are commonly believed to be beneficial to concrete pavement performance in two ways: in reducing infiltration of water into the pavement structure, and in reducing infiltration of incompressibles into the joints. However, despite the conventional wisdom concerning the need to keep transverse joints in concrete pavements 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, roughness, or deflections.

Arizona is one of four states that has built and monitored experimental concrete pavement test sections with sealed and unsealed joints as supplements to the LTPP SPS-4 experiment. The supplemental experiments in three of these states differ from the other SPS-4 sites in that the pavements were constructed for the experiment, and incorporate a variety of joint sealant types and joint configurations, as well as narrow unsealed joints.

The performance of the concrete pavement test sections and the joint sealants at the Arizona SPS-4 site has been monitored since the pavement's construction in 1991. This monitoring has included deflection testing, distress surveying, profile measurement, test section length measurement, joint width measurement, and joint sealant condition evaluation. This paper presents the findings from nine years of monitoring of the performance of the pavements and joint sealants at the Arizona SPS-4 site.

Introduction

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.

Currently, nearly all state highway agencies require sealing of transverse joints immediately after construction for all new jointed concrete pavements (JPCP). 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.

However, a review of previous studies on the subject reveals that JPCP with sealed joints and JPCP with unsealed joints have not been shown to perform differently in terms of spalling, faulting, International Roughness Index (IRI), or deflections.^{1,2,3,4,5,6}

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 certain rigid pavement maintenance treatments (joint sealing and undersealing) on performance, relative to the performance of untreated control sections. 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 were not designed to remain unsealed and are, in fact, sections in which the joints were previously sealed or filled, and the sealant or filler was later either removed or destroyed.

A joint that is truly designed and constructed to remain unsealed is formed by a single narrow sawcut (e.g., 3 mm, or approximately 1/8 inch, wide). The initial sawcut is not subsequently widened, as would normally be done if a sealant were to be installed. There are just five SPS-4 sites in which the pavements were newly constructed for the experiment and in which the joints in the control sections were designed and constructed to remain unsealed: one site in Arizona, one in Colorado, and three in Utah.

This paper presents the findings from nine years of monitoring of the performance of the pavement sections and joint sealant installations at the Arizona SPS-4 supplemental study site. These findings are drawn from three studies: NCHRP Project 20-50(2), on the relative performance of jointed plain concrete pavement with sealed and unsealed joints at the SPS-4 supplemental study sites,⁷ an FHWA study on the performance of joint seals at the SPS-4 supplemental sites,⁸ and on-going monitoring of the sites by the Arizona DOT.^{9,11}

Project Description

The Arizona SPS-4 supplemental experiment is located in the eastbound lanes of the Superstition Freeway (U.S. 60, formerly numbered 360), between Power and Ellsworth Roads, in Mesa, Arizona, an eastern suburb of Phoenix. The site receives an average of 215 mm (8.5 inches) of precipitation annually, and the temperatures range from an average low of 3.4°C (38°F) in January to an average high of 40.5°C (105°F) in July.

This section of the freeway is a six-lane divided highway with concrete shoulders. The pavement is a 330-mm (13-inch) JPCP on 10 cm (4 inches) of aggregate base. The joints are undowelled, skewed, and spaced at 3.9, 4.6, 5.2, and 4.6 m (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 3 meters (10 feet). The site received an estimated 190,000 equivalent single-axle loads (ESALs) in 1991, and an estimated 3 million accumulated ESALs through 2000.

The concrete pavement was placed on February 13, 1991. Primary sawcutting for all joints, to a depth of one third of the slab thickness, was done on February 13 and 14. All joint sawing was done with conventional riding saws. For those joints which were to be sealed, secondary sawcutting of sealant reservoirs was done on February 15. Prior to installation of the sealants, the wet-sawed joints were rigorously cleaned by dry sawing, sandblasting, waterblasting, and three airblasting operations. Backer rod was used with all hot-applied and silicone sealants.

⁷ International Conference on Concrete Pavements – Orlando, Florida, USA – September 9-13, 2001

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. The construction report¹⁰ indicates that the contractor took action to correct this erosion. Nonetheless, it is conceivable that the base and subgrade moisture content were unusually high at the time of paving, and that the quality of support along the longitudinal paving joint may have been nonuniform.

The positions of the test sections in the matrix of joint sealants and reservoir widths used at the Mesa site are illustrated in Table 1. The test sections are grouped according to sealant type and reservoir width in Table 2.

Table 1. Positions of test sections in the Mesa, Arizona SPS-4 experimental matrix.

Sealant		Joint Width		
Type	Name	3 mm	6 mm	9 mm
Unsealed		04A430 04A455		
Asphalt	Crafco RS 221			04A449 04A459
	Crafco SS 444			04A446 04A461
Silicone	Crafco 903-SL			04A410 04A458
	Dow 888			04A445 04A453
	Dow 888-SL			04A444 04A457
	Dow 890-SL	04A447 04A451	04A450 04A462	04A442 04A456
	Mobay 960-SL			04A448 04A454
Neoprene	DS Brown E-437H			04A441 04A452
	Watson Bowman 687			04A443
	Watson Bowman 812			04A460

Table 2. Definitions of groups in the Mesa, Arizona SPS-4 experimental matrix.

Sealant type	Joint width		
	3 mm	6 mm	9 mm
Unsealed	Group 1		
Asphalt			Group 2
Silicone	Group 3	Group 4	Group 5
Neoprene			Group 6

The joint widths are expressed in millimeters, but the actual joint sawcuts were nominally 1/8 inch (this width corresponds to that labelled in Tables 1 and 2 as “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 (labelled as “6 mm” and “9 mm” respectively.)

The monitoring data from the Mesa site that were available for analysis are summarized in Table 3 below.

Table 3. Monitoring data available for Mesa, Arizona SPS-4 supplemental test sections.

Data type	Years with data available									
	'91	'92	'93	'94	'95	'96	'97	'98	'99	'00
Spalling					✓		✓		✓	✓
Sealant Damage					✓		✓		✓	✓
Faulting							✓		✓	✓
IRI		✓	✓			✓	✓	✓	✓	
Deflections	✓			✓	✓		✓		✓	✓

Statistical Analysis Methods

Because the experimental matrix shown in Table 2 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 like a feasible alternative. An ANOVA tests whether the experimental factor or factors being tested give 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 is that an ANOVA will determine whether at least one significant difference exists somewhere among the treatment means being compared, but it will not identify which treatments are significantly different.

A similar statistical analysis approach is the use of a multiple comparison test method. Multiple comparison tests are designed to identify significant differences among treatment means. These tests should only be applied when the ANOVA has detected that at least one significant difference exists. Among the multiple comparison methods available (in approximate order of decreasing sophistication and robustness) are those associated with the names Newman-Keuls, Tukey's honestly significant difference (HSD), Fisher's least significant difference (LSD), Scheffé, Bonferoni, and Duncan. These tests differ in their details and their appropriateness to different applications, but in general, they all (with the exception of Bonferoni) seek to detect significant differences among all possible combinations of pairs of treatment means. In general, multiple comparison tests are appropriate when analyzing a set of treatments among which all possible comparisons are of potentially equal 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.

The objective of the analysis of performance and deflection data from the Mesa site conducted for the NCHRP study⁷ was to compare the performance of the test sections with unsealed joints (Group 1, as shown in Table 2) to that of the test sections with sealed joints (Groups 2 through 6). Comparisons among the sealed-joint groups were outside the scope of the study. Therefore, for each of the distress, roughness, and deflection parameters considered, the Group 1 to the other five groups was made using a set of five pairwise *t*-tests.

For a given performance indicator, such as joint faulting, at an experimental site with $k = 6$ nonempty test section groups, there are $k-1 = 5$ possible pairwise comparisons of Group 1 performance with the performance of the other five groups. Pairwise *t*-tests are preferred over multiple comparison tests in this situation because the conclusions that may be drawn from these five specific comparisons can be associated with a narrower overall confidence interval than that which would encompass all possible comparisons, i.e., among groups 2 through 6 as well. In other words, using the data for five comparisons (Group 1 versus Groups 2 through 6), rather than all possible (six factorial) comparisons, makes the tests more powerful.

It is important to note that, for specific pairwise comparisons and also for some multiple comparison tests, 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, 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 $1 - \alpha$ such that $(1 - \alpha)^n = 0.95$. For $n = 5$ comparisons, the individual confidence level $1 - \alpha$ must be 0.99.

The pairwise *t*-tests were conducted by first calculating, for each test section and for each parameter of interest (e.g., joint faulting) determined from several individual measurements, the sample mean and sample variance of the measurements. For each of the six groups, the group mean was calculated as the mean of the test section sample means within the group, and the group, or pooled, variance was calculated. For each of the five possible pairs of Group 1 with another group, the pooled variance of the two groups was then calculated. For each of the five possible pairs, a *t*-test was conducted, at the preselected individual confidence level $1 - \alpha = 0.99$, to determine whether the difference between the Group 1 mean and the other group mean was significantly different than zero.

In contrast to the NCHRP study, the objective of the study of joint sealant performance data from the Mesa site¹¹ was to group the test sections in three categories: unsealed, partially sealed, and poorly sealed, and to compare the performance of the test sections in these three categories. For this purpose, all possible comparisons (unsealed versus partially sealed, unsealed versus poorly sealed, and partially sealed versus poorly sealed) are of equal interest. Thus, the statistical analysis method employed was a multiple comparison test, described earlier.

Center-Slab Deflection Analysis

The deflections measured at the slab centers were used to determine the dynamic *k*-value of the foundation and the effective thickness of the concrete slabs, based on an assumed modulus of elasticity of 34500 MPa (5 million psi). For each deflection basin, initial estimates of the *k*-value and radius of relative stiffness were obtained based on infinite slab size assumptions, and then adjusted using correction factors calculated as a function of slab length and width.

⁷ International Conference on Concrete Pavements – Orlando, Florida, USA – September 9-13, 2001

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. Although concrete slabs are commonly thought of and analyzed as plates that experience only bending, in fact some compression does occur under load. This effect is greater for thicker slabs, lower concrete elastic moduli, and/or stiffer foundations. The backcalculation results referred to here as those from the AREA4 analysis are those obtained using the deflection at the center of the load plate, as well as the deflections at approximately 30, 60, and 90 cm (12, 24, and 36 inches) away. The backcalculation results referred to here as those from the AREA5 analysis are those obtained using the deflections at approximately 30, 45, 60, 90, and 150 cm (12, 18, 24, 36, and 60 inches), without using the deflection at the center of the load plate.

Deflections measured at multiple load levels were analyzed to differentiate slab curling from poor foundation support. If the slab temperature is high and the foundation is stiff, the center of the slab may be lifted off the foundation at some times during daytime testing. 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 are 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, compared to values backcalculated from individual load/deflection pairs.

The overall project average effective slab thickness values (calculated assuming a concrete elastic modulus of 34500 MPa, or 5 million psi) obtained by the AREA4 and AREA5 methods are summarized by year in Table 4.

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

Year	Effective slab thickness, mm (inches)	
	AREA4 method	AREA5 method
1991	305 (12.0)	361 (14.2)
1994	322 (12.7)	356 (14.0)
1995	297 (11.7)	350 (13.8)
1997	318 (12.5)	353 (13.9)
1999	318 (12.5)	353 (13.9)

According to the AREA4 results, the 1991 average effective thickness is less than the design thickness of 330 mm (13 inches). This suggests that the concrete's elastic modulus was less than 34500 MPa (5 million psi) in 1991. This seems reasonable, since the state's past experience suggests that 28-day compressive strengths of about 34 to 42 MPa (about 5000 to 6000 psi) are typical, which correspond to elastic modulus values in the range of about 27600 to 31000 MPa (about 4 to 4.4 million psi). According to the AREA4 results, an average effective thickness equal to the design thickness of 330 mm (13 inches) would correspond to an elastic modulus of about 27100 MPa (about 3.9 million psi) and a compressive strength of about 33 MPa (about 4800 psi). These values would be reasonably consistent with the state's experience.

According to the AREA5 results, the average effective thickness in 1991 was considerably greater than the design thickness. This implies that the concrete modulus was considerably greater than 34500 MPa (5 million psi) in 1991, and/or that the constructed slab thickness is considerably greater than the design thickness. The state's past experience suggests that constructed thicknesses tend to exceed design thicknesses by about 6 to 12 mm ($\frac{1}{4}$ to $\frac{1}{2}$ inch). If the constructed thickness actually averaged 343 mm (13.5 inches), for example, according to the AREA5 results, the concrete's elastic modulus in the year of construction would have had to be about 40000 MPa (about 5.8 million psi), which corresponds to a compressive strength of 72 MPa (about 10400 psi). These values do not seem reasonable, based on the state's experience.

The AREA4 results also appear to be more realistic in that they show an increase over time in overall average effective slab thickness. An increase in backcalculated effective slab thickness for an assumed constant elastic modulus implies that the elastic modulus is increasing over time, as one would expect. This trend and the above observations concerning the concrete modulus in the year of construction all suggest that for this particular project, the AREA4 results are more realistic than the AREA5 results.

The results of the backcalculation of dynamic k values revealed a surprising trend: a decrease in k values over time. The group mean k value results from the AREA4 solution method are shown in Figure 1. 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. These apparent declines in k-value over time cannot be attributed to slab curling during testing, because the incremental analysis of the deflections adjusts for variation in contact due to slab curling.

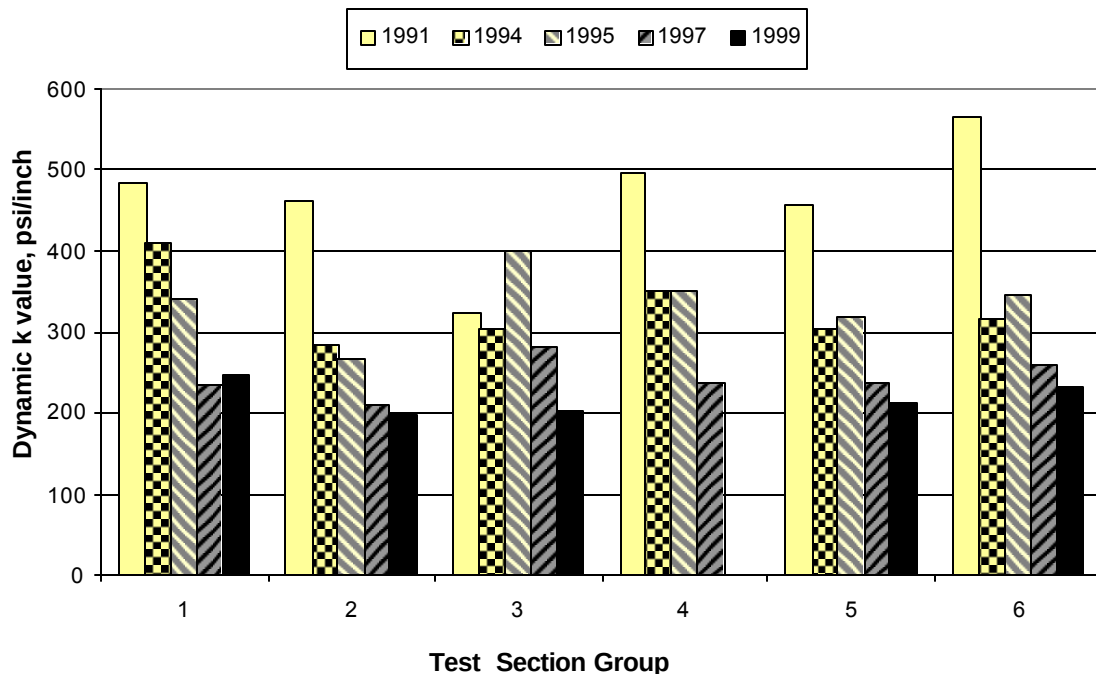


Figure 1. Change in dynamic k-value over time, Mesa, Arizona SPS-4 supplemental test section group means.

The trend in decreasing k-values over time is consistent with an increasing trend in the maximum deflections (those measured at the center of the load plate) over time, as shown in Figure 2. The values shown are the group means of the maximum center-slab deflection, adjusted for slab size and normalized to 40 kN (9000 lbs). For Group 3, the apparent decrease in maximum deflection in 1999 is due to the fact that one of the Group 3 test sections (04A447), which had the higher deflections among the sections in that group in previous years, has no deflection data available for 1999.

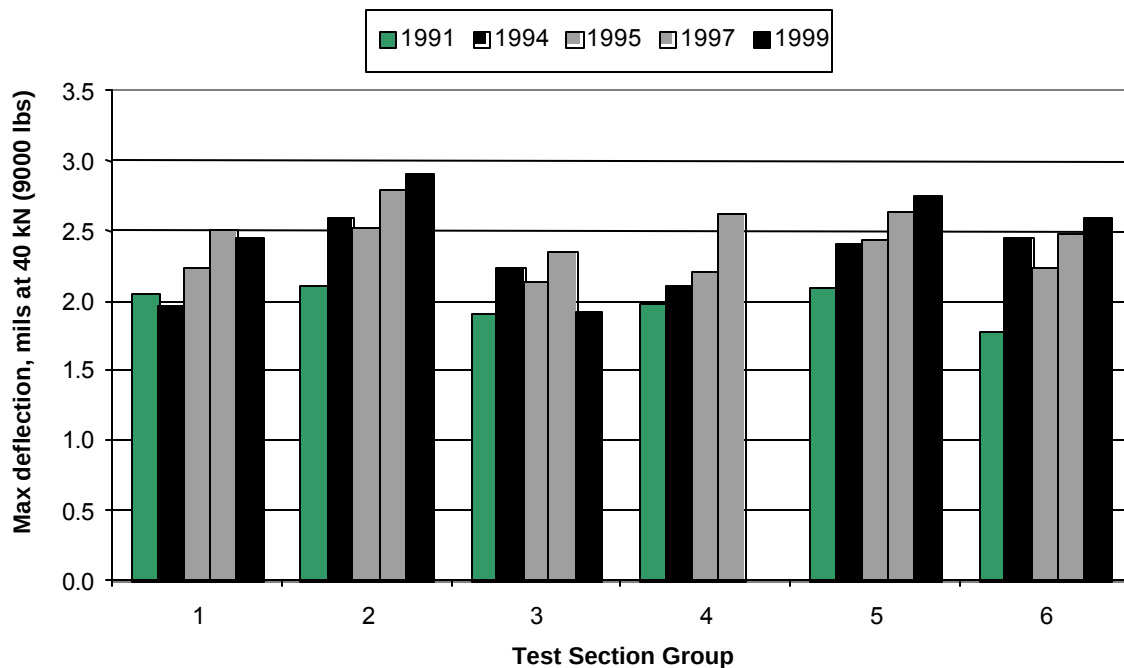


Figure 2. Change in incremental normalized maximum deflection over time, Mesa, Arizona SPS-4 supplemental test section group means.

Perhaps it should not be surprising that the k-values of the test section groups decreased in the first several years after construction. The placement of a pavement in a dry climate could be expected to inhibit evaporation of moisture from the underlying soil, which could cause a gradual increase in soil moisture content over time. The same trend was observed in the backcalculated k-values at every one of the five SPS-4 supplemental study sites evaluated, all five of which are in dry climates. It may be that this phenomenon has not been noted previously because, until the construction of the SPS-4 supplemental study sites and the SPS-2 experimental sites, few if any examples existed of concrete pavement experiments that had been deflection tested in the year of construction and for several years thereafter. Analysis of the SPS-2 deflection data may reveal whether k values tend to increase in all or only some climates, and for all or only some types of soil.

Joint Spalling

In the FHWA study⁸ on the joint sealant performance at the SPS-4 supplemental sites, the term “sliver spall” is used 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 recent research suggests that inadequate curing plays a large role). The occurrence of sliver spalling is not believed to be related to the presence or type of sealant. The occurrence of sliver spalling in the unsealed-joint test sections at the Mesa site has been no different than in the sealed-joint test sections. None of the test sections have exhibited other spalling.

Joint Sealant Damage

For all of the sealed-joint sections for which sealant condition data are available in the LTPP database, a weighted seal damage index was calculated, by weighting the proportions of low-, medium-, and high-severity joints within each test section by factors of 1, 2, and 4 respectively. These weighting factors were selected based on an analysis of the influence of joint sealant damage severities on pavement condition ratings calculated using the Pavement Condition Index (PCI) procedure.

The weighted seal damage index 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. For the most recent LTPP 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:

- Group 2 (9-mm asphalt-sealed) = 3.93
- Group 3 (3-mm silicone-sealed) = 2.93
- Group 6 (9-mm neoprene-sealed) = 2.06
- Group 4 (6-mm silicone-sealed) = 1.88
- Group 5 (9-mm silicone-sealed) = 1.53

The performance of the joint seals in most of the test section groups was evaluated in more detail in 2000.¹¹ Sealant distress modes noted for the hot-applied silicone sealants included full-depth adhesion loss, full-depth cohesion loss, and full-depth sliver spalls. Figure 3 summarizes the failure types and amounts noted for each sealant/width combination, after nine years of service (1991 to 2000).

For the neoprene sealants, failure modes included compression set, gaps between the sealant and the reservoir sidewall, and full-depth sliver spalls. On the basis of the year 2000 sealant evaluation, Groups 3, 4, and 5 were considered to be still well sealed, while Groups 2 and 6 were considered to be only partially sealed.

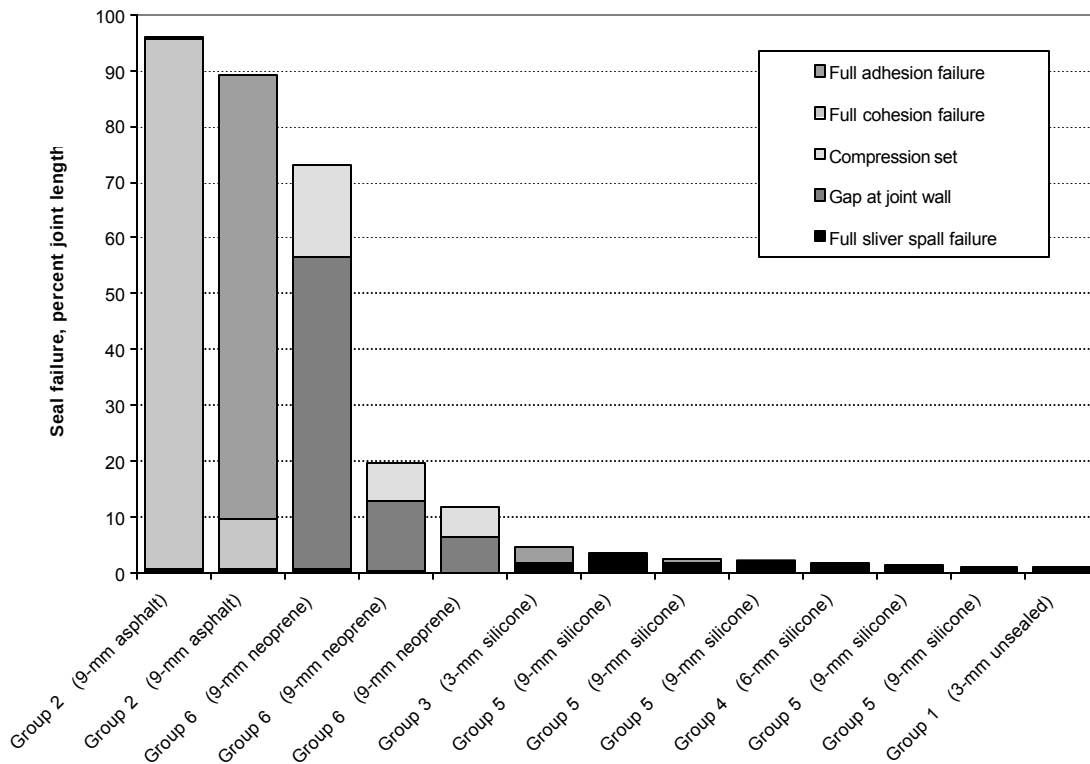


Figure 3. Sealant failure levels by type, after 9 years.

Joint Faulting

The LTPP faulting data used in this study were used to calculate a parameter called average absolute faulting, which is the arithmetic average of the absolute values of the individual joint faulting measurements. The average absolute faulting group means calculated from the 1997 and 1999 LTPP data are shown in Figure 4. As one would expect, average absolute faulting increased in every test section group between 1997 and 1999.

The 1999 data show no statistically significant differences between the average absolute faulting in Group 1 and that in Groups 2, 3, and 4. The differences between the 1999 average absolute faulting in Group 1 and that in Groups 5 and 6 were statistically significant.

The year 2000 faulting data were analyzed using Duncan's multiple comparison test method. The mean faulting in the unsealed group (Group 1) was judged to be significantly different, at a 95 percent confidence level, than that in the partially sealed groups (Groups 2 and 6) and well sealed groups (Groups 3, 4, and 5). This was true for faulting measured at 30 cm (about 1 ft) and 75 cm (about 2.5 ft) from the slab edge. The differences between faulting in the partially sealed and well sealed groups were not statistically significant.

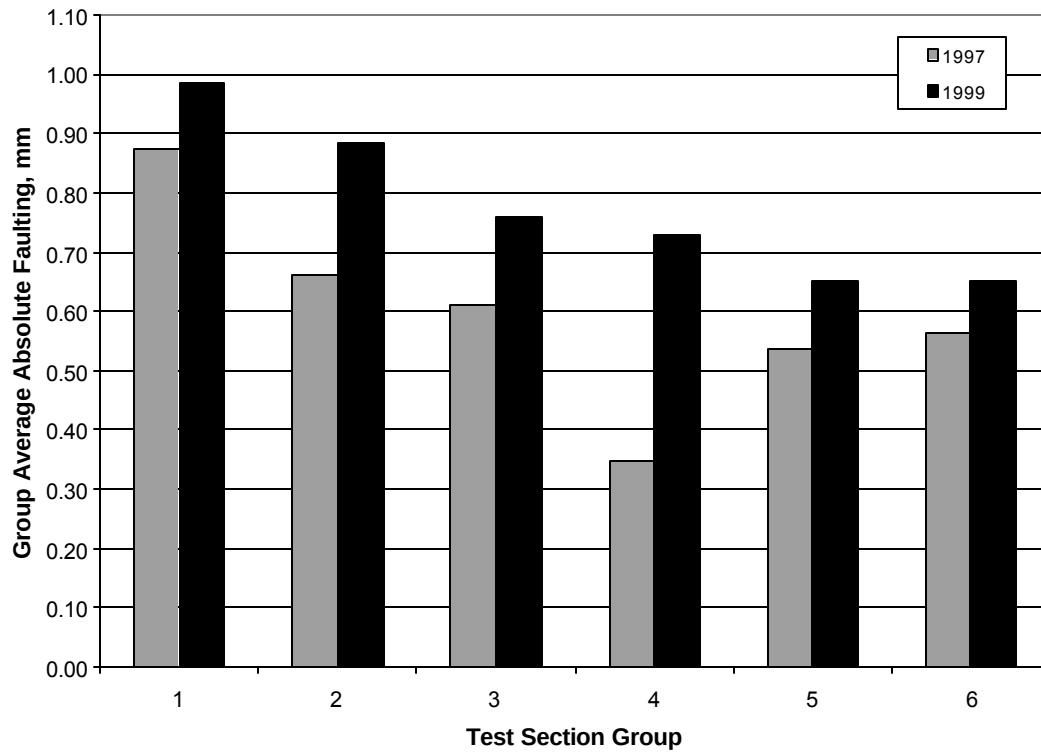


Figure 4. Average absolute faulting in 1997 and 1999, Mesa, Arizona SPS-4 supplemental test section groups.

International Roughness Index

The trends in group mean IRI are illustrated in Figure 5. No significant differences in IRI were detected between Group 1 and any of the other five groups in any year, except for a significant difference, in the 1998 data only, with respect to the Group 4 (6-mm silicone sealed) 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 increases in group mean IRI between 1992 and 1999 are listed in Table 5. The increase in group mean IRI for the Group 1(3-mm unsealed) test sections was greater than or equal to the increase for all but one of the sealed-joint test sections groups.

Table 5. Increase in IRI between 1992 and 1999, Mesa, Arizona SPS-4 supplemental test section groups.

Group	Description	1992 IRI, m/km	1999 IRI, m/km	IRI increase, m/km
1	3-mm unsealed	0.700	0.958	0.258
2	9-mm asphalt sealed	0.700	0.960	0.260
3	3-mm silicone sealed	0.709	0.906	0.197
4	6-mm silicone sealed	0.738	0.858	0.120
5	9-mm silicone sealed	0.760	0.905	0.225
6	9-mm neoprene sealed	0.718	0.926	0.208

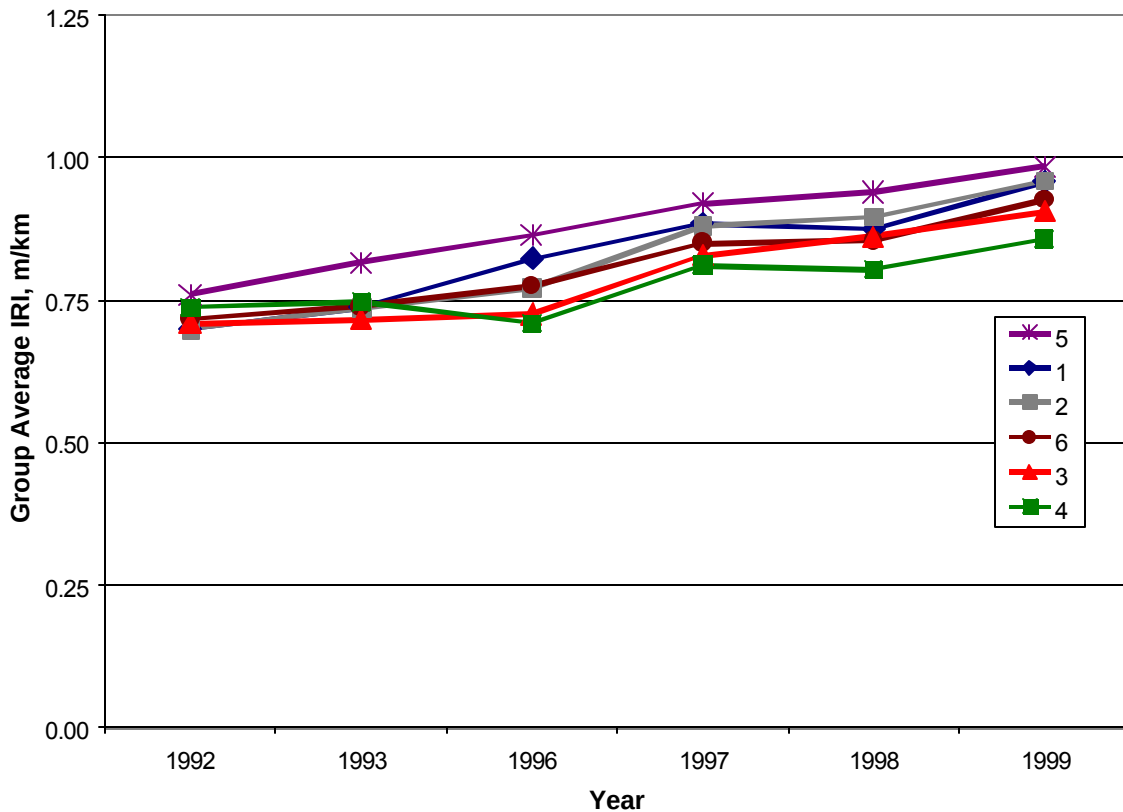


Figure 5. Trends in group mean IRI,
Mesa, Arizona SPS-4 supplemental test section groups.

Deflection Load Transfer

Deflection load transfer is 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 said to be 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 normalized 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.

Pavement temperatures during deflection testing should be taken into consideration when analyzing deflection load transfer, so that differences in deflection load transfer that may be partially due to temperature differences are not erroneously attributed entirely to treatment differences. The minimum and maximum pavement temperatures recorded during each of the five deflection surveys are summarized in Table 6.

Table 6. Pavement temperatures during deflection testing, Mesa, Arizona SPS-4 supplemental test sections.

Month and Year	Minimum, °C (°F)	Average, °C (°F)	Maximum, °C (°F)
Apr 1991	21 (70)	33 (91)	40 (104)
Sep 1994	31 (88)	41 (106)	53 (127)
Jul 1995	38 (100)	53 (127)	65 (149)
Dec 1997	10 (50)	21 (70)	38 (100)
Feb 1999	13 (55)	24 (75)	34 (93)

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:

$$LT\% = \frac{1}{a_1 + a_2 * a_3^{(-\text{temperature}, ^\circ\text{C})}}$$

The a_1 , a_2 , and a_3 coefficients determined for each treatment group, and the square of the correlation coefficient (R^2) between estimated and observed deflection load transfer values for each group, are summarized in Table 7. The curves are shown in Figure 6.

Table 7. Coefficients of equations for deflection load transfer versus temperature, Mesa, Arizona SPS-4 supplemental test section groups.

	Test Section Group					
	1	2	3	4	5	6
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 6 illustrates, the two groups with the highest deflection load transfer over the widest range of temperatures are Groups 1 (3-mm unsealed) and 3 (3-mm silicone-sealed). The next-best load transfer is exhibited by Group 2 (9-mm asphalt-sealed), and the remaining three groups (4, 5, and 6; 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.

It is cautioned that the extrapolation of these curves below a level of about 20 to 25 percent is only shown for illustrative purposes. There are no data points below this level, and in the authors' experience, deflection load transfer values below this level are not obtained from field measurements.

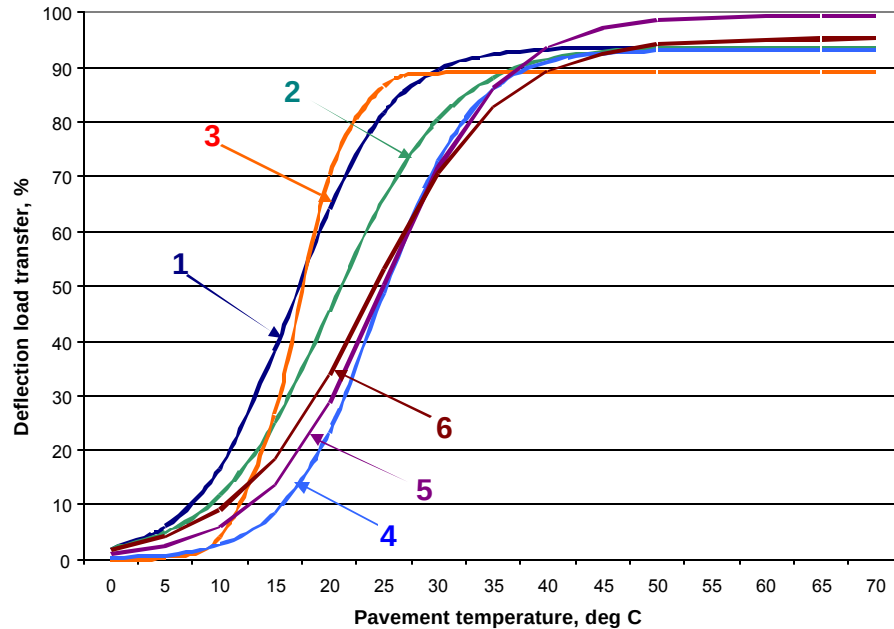


Figure 6. Deflection load transfer versus temperature, Mesa, Arizona SPS-4 supplemental test section groups.

Total Edge Deflection

Total deflection is the sum of the deflections of the loaded and unloaded sides of the joint, normalized to a uniform load level. 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. The total edge deflection may 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 total edge deflection should remain relatively constant regardless of available deflection load transfer, provided that slab thickness, 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 1999 deflection results indicate that the total edge deflection in Group 1 (3-mm unsealed) was not significantly different than that of groups 2 and 3 (9-mm asphalt-sealed and 3-mm silicone sealed) but was significantly lower than that of groups 5 and 6 (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, as shown in Figure 7. This suggests similar net decreases in edge structural capacity over this time period.

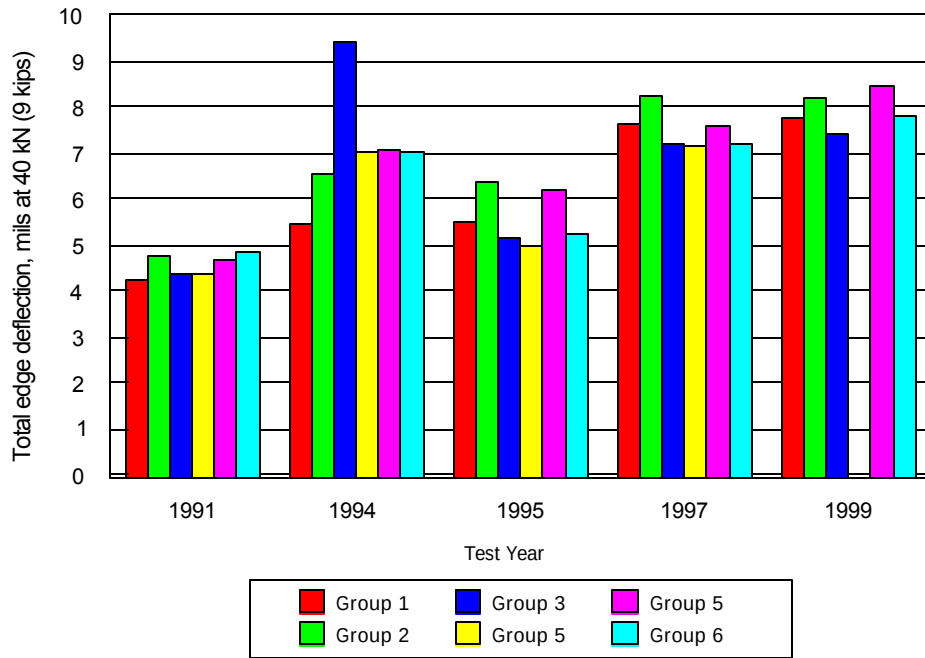


Figure 7. Trends in total edge deflection, Mesa, Arizona SPS-4 supplemental test section groups.

Differential Deflection

Differential deflection is the difference in deflections of the loaded and unloaded sides of the joint, normalized to a uniform load level. The magnitude of differential deflection is a measure of the deficiency in load transfer capability of the joint, and may indicate the potential for the development of spalling, faulting, and corner breaks. 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 mean differential deflections are shown in Figure 8. In most of the years for which deflection data are available, including the most recent year, the normalized differential edge deflection of Group 1 was significantly lower than the differential edge deflection for the sealed-joint groups. This is consistent with the results presented earlier for deflection load transfer as a function of temperature.

Transverse Edge Slab Support Ratio

The edge foundation support (edge k-value) is backcalculated on the assumption that each tested slab is of uniform thickness and elastic modulus. The uniformity of support under the transverse slab edge is expressed by the ratio of the backcalculated edge k-value to the backcalculated interior k-value. In general, slab edge support ratios less than approximately 0.75 are indicative of slabs with poor edge support due to foundation densification, pumping, and/or slab curling. Incremental analysis of the transverse edge deflections was conducted to differentiate slab curling effects from erosion of edge support. For those cases where poor edge support is solely the result of upward temperature curling, incremental slab support ratios should increase over those computed based on individual load levels, provided that at least two load levels produced sufficient total edge deflection to close any curl-induced voids.

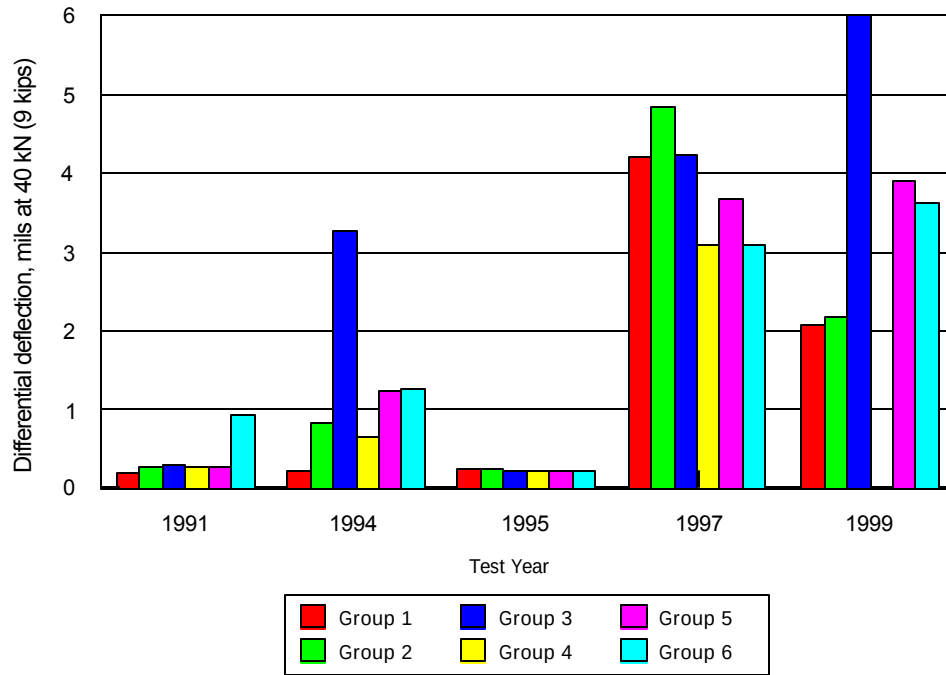


Figure 8. Trends in differential deflection, Mesa, Arizona SPS-4 supplemental test section groups.

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, as shown in Figure 9. Group 6 (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.

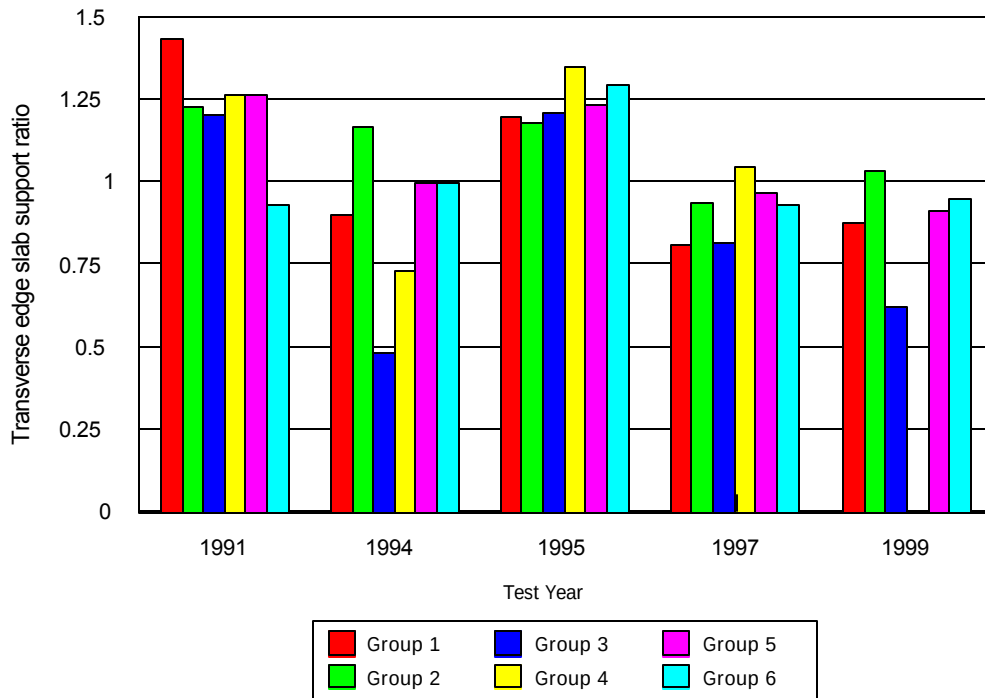


Figure 9. Trends in transverse edge slab support ratio, Mesa, Arizona SPS-4 supplemental test section groups.

Summary Observations

After nine years of monitoring of the Arizona SPS-4 joint sealing experiment, the following observations may be made concerning the performance of the sealed-joint and unsealed-joint test sections:

- **Joint seal damage:** The sealed-joint test sections exhibit, to different degrees, joint seal damage such as adhesive and cohesive failure.
- **Spalling:** Neither the unsealed-joint test sections nor the sealed-joint test sections in the Mesa, Arizona SPS-4 supplemental experiment exhibit any joint spalling.
- **Joint faulting:** Although average absolute joint faulting is low overall, as of the most recent survey it was significantly higher in Group 1 (3-mm unsealed joints) than in some of the sealed-joint groups.
- **IRI:** The Mesa test sections had impressively low and uniform IRI values in 1991. Although IRI levels remain low, Group 1 (3-mm unsealed joints) has exhibited a greater increase in IRI than most other groups.

The analysis of deflections measured on the Arizona SPS-4 test sections yielded the following observations:

- **Decreases in k-value** have occurred in every test section group over time. This is surprising, considering that the project is located in one of the country's driest climates. The tendency of k-values to decline in the first several years after construction, at least in some climates and/or for certain soil types, may be a more common occurrence than we realize, simply because deflection data have not been available before to demonstrate it.
- **Slab edge support ratios** have decreased over time for nearly all of the Mesa test section groups. 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.
- **Deflection load transfer** was the best in Groups 1 and 3 (the 3-mm unsealed and 3-mm silicone-sealed test section groups), over the range of temperatures at which load transfer has been measured.
- **Differential deflections at transverse joints** were also lowest in Group 1 (the 3-mm unsealed test section group).

The narrow unsealed-joint 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 less faulting than in the other groups. In fact, the narrow unsealed test sections have more faulting than some other groups.

The findings presented here apply only to this particular project, located in a hot, dry climate. It would be unwise to extrapolate these findings to other regions of the United States. Nonetheless, the analysis methods used in this study may serve as useful examples for analyses of other sealed-versus-unsealed joint experiments in other climates.

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