

EFFECT OF DESIGN FEATURES ON CONCRETE PAVEMENT PERFORMANCE

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ABSTRACT

A major national field study has been conducted into the effect of various design features on the performance of jointed concrete pavement. Such variables as slab thickness, base type, joint spacing, transverse joint load transfer, shoulder type, and drainage were studied. These represent critical areas which current pavement design procedures do not satisfactorily address.

Eighty experimental rigid pavement sections representing the four main climatic zones were selected for inclusion in the study. A wide range of design variables were included to provide an indication of their relative impact on pavement performance. Visual condition surveys, roughness, drainage surveys, NDT deflection testing, and coring/boring testing were performed on every section to provide a complete indication of the performance of the pavement and its relevant properties and characteristics.

Preliminary findings on the effect of the design features on pavement performance are presented in this paper.

INTRODUCTION

The pavement design process is more than just the determination of slab thickness; other variables such as joint load transfer, joint spacing, base type, drainability, and shoulder type play a key role in the ultimate performance of the pavement. However, to date, very little long term field performance data has been available to verify the effect of these design features on pavement performance. Additionally, most pavement design methods do not attempt to directly incorporate these variables into the design procedure.

As part of a nationwide study funded by the Federal Highway Administration, the effect of different design features on concrete pavement performance is being examined by evaluating numerous experimental concrete pavements

throughout the country. Experimental projects were selected based on three criteria: 1) they incorporated many of the desired design features, 2) they sustained high volumes of traffic, and 3) they are well documented.

The experiments, however, were generally set up in such a way that only the various total pavement designs could be evaluated, and not the individual design features. Full factorial designs were the exception and not the rule; thus, there is frequently confounding of variable effects and the performance of the feature of interest cannot be isolated. Also, equal numbers of projects were not available from each climatic zone, and not all of the features were equally represented in each climatic zone. Therefore, the analysis that could be performed was not as complete as desired.

Sixteen experimental projects totaling 80 pavement sections were ultimately selected. In addition, several other single pavement sections not part of any experiment were included. Table 1 provides the location of each project, its date of construction, and the design features evaluated.

All of the projects were the subject of a visual condition survey and a visual drainage evaluation during the summer of 1987. Each section was also tested with a Falling Weight Deflectometer (FWD) in order to backcalculate material properties, to calculate load transfer efficiencies, and to identify loss of support beneath the joints. Center slab cores were taken for strength determination and joint cores were retrieved to provide an indication of deterioration beneath the joints. Borings were taken of the base, subbase, and subgrade for material identification and characterization. Finally, each section was evaluated for roughness using the Mays Ridemeter, and rated for rideability (PSR) by the survey crew. On selected sections, weigh-in-motion (WIM) traffic studies were performed to provide the actual truck axle weights and axle load distributions.

Nine design features are evaluated in this analysis: slab thickness, base type, joint spacing,

reinforcement, transverse joint load transfer, longitudinal joint design, joint sealant, shoulders, and drainage. The number of sections available for evaluating certain features is limited, so that their direct effect on performance cannot be readily determined.

All of the data presented here, except where noted, are from the outer traffic lane, which represents the heaviest travelled lane. Project designation numbers, defined in Table 1, are used for ease of presentation.

SLAB THICKNESS

Sections having slab thicknesses ranging from 7 to 15 in were included in the study. Although often confounded with other variables, the following comparisons can be made from the field data.

Minnesota 1 and Minnesota 2

Two experimental sections in Minnesota, Minnesota 1 on I-94 near Rothsay (1) and Minnesota 2 on I-90 near Albert Lea (2), showed mixed results with respect to slab thickness. For Minnesota 1, the 9-in jointed reinforced concrete pavement (JRCP) nondowelled slabs performed much better than the 8-in JRCP nondowelled slabs in every distress category. However, the dowelled 8-in JRCP sections performed slightly better than the 9-in JRCP sections. These results are shown in Table 2. For Minnesota 2, the 8- and 9-in JPCP slabs displayed nearly identical performance, except that the 9-in slab had more longitudinal cracking.

Arizona 1

On this experimental project on S.R. 360 in Phoenix (3), 11- and 13-in nondowelled, jointed plain concrete pavement (JPCP) sections placed directly on the subgrade are performing better than a nondowelled 9-in concrete pavement on a cement treated base in terms of faulting, longitudinal cracking, and joint spalling. No difference in performance was observed between the 11- and the 13-in slabs. However, the 9-in pavement was older and had sustained greater traffic loadings. No transverse cracks were observed for any pavement section. These results are summarized in Table 3.

California 1

This experimental project on I-5 near Tracy, California (4), included two slab thicknesses constructed over a cement-treated base course. The 8.4-in slabs had significantly more transverse and longitudinal cracking than the 11.4-in slabs. Other distresses were about the same for each section. However, the serviceability was lower for the thicker slab. This information is summarized in Table 4.

Ohio 2

On this experimental project on S.R. 2, near Vermilion, Ohio, two 15-in JPCP slabs were constructed on grade (5). One section contained a tied concrete shoulder. The pavement sections were

13 years old and had carried 3.3 million ESAL's. The slabs showed very little transverse cracking or transverse joint spalling. However, significant longitudinal cracking was observed. Faulting was 0.11 in for each of the two sections, which is fairly high. Increased slab thickness did not prevent joint faulting for these sections.

Ontario 1

On Highway 3N, near Ruthven, experimental 12-, 8-, and 7-in slabs were built (6). The 12-in slab on grade displayed no transverse cracking and only a small amount of longitudinal cracking. The 7- and 8-in slabs placed on lean concrete base courses showed much more longitudinal and transverse cracking. The amounts of spalling, faulting, and PSR were very similar for all of the sections. The transverse joints were not dowelled. Table 5 summarizes this information.

Summary of the Effects of Slab Thickness

In every case, regardless of the climatic zone, the thicker slabs reduced the amount of transverse and longitudinal cracking. Joint spalling and faulting, however, did not generally decrease with increased slab thickness. Thick slabs on grade, in the range of 11 to 15 in performed relatively well, although they did exhibit faulting in wet-freeze regions.

BASE TYPE

The base types included in this study were aggregate base courses (AGG), cement-treated base courses (CTB), asphalt-treated base courses (ATB), lean concrete base courses (LCB), soil-cement base courses (SC), and stabilized permeable base courses. In addition, some slabs were constructed directly on the subgrade without a base course.

Minnesota 1

This project on I-94 showed a large difference in performance between three different base types: aggregate, asphalt-treated, and cement-treated. The ATB sections displayed the most transverse cracking and by far the most longitudinal cracking. The CTB sections also exhibited a substantial amount of longitudinal cracking. The cause of the large amount of longitudinal cracking for the ATB and CTB sections is probably increased friction produced by those materials at the slab/base interface, coupled with the use of plastic tape inserts to form the longitudinal joint. Aggregate bases showed the best performance in terms of low longitudinal cracking and joint spalling. For both dowelled and nondowelled pavements, the CTB exhibited the most faulting. These results are displayed in Table 6.

Arizona 1

This project demonstrated that 11- and 13-in thick slabs placed directly on the subgrade had the same low amount of faulting as 9-in slabs placed on LCB. They also had less faulting than a pavement slab on a CTB base which was subjected to more traffic.

California 1

This project on Interstate 5 near Tracy included a direct comparison between CTB and LCB pavement sections with all other factors held constant. The LCB section displayed more than twice as much transverse cracking as the CTB section. The LCB section also exhibited a large amount of base pumping and a lower PSR than the CTB, probably due to the extensive transverse cracking. However, the CTB section exhibited more longitudinal cracking and more faulting than the LCB. These results are tabulated in Table 7.

California 2

Two experimental sections on Interstate 210 in Los Angeles (7) compared a conventional CTB with a two-layered base consisting of a dense AC layer over a permeable CTB. Both pavements exhibited a large amount of faulting, averaging 0.11 in. The CTB section had a large amount of transverse cracking, while the dense AC/permeable CTB had no cracking. This may be due to the fact that the 2.5-in AC layer served as a bondbreaker for that section. However, it is believed that the AC layer completely altered the drainage and erosion characteristics of the permeable base, essentially rendering it ineffective. The performance results for these sections are illustrated in Table 8.

Michigan 1

Experimental sections on U.S. 10 near Clare, Michigan (8), had a nondowelled permeable ATB and a dowelled aggregate base course with far better performance characteristics than pavement sections constructed over a dense-graded asphalt base course. The permeable ATB section had very little faulting and joint spalling, no cracking, and a high PSR. The aggregate base course sections had a fair amount of drainability that resulted in far better performance than the sections constructed over the dense-graded asphalt-treated base. The dense-graded ATB sections with no dowels were the poorest performers, since they formed a bathtub design that held water and contributed to a large amount of erosion and spalling from "D" cracking. They also had a lot of cracking and faulting and a low PSR. Table 9 includes these results. It should be noted that the pavement sections are under relatively light traffic (less than one million ESAL's after twelve years).

New York 1

Experimental sections on Route 23 near Catskill (9) included ATB and aggregate base courses. This experiment showed that for JPCP, the sections built over ATB performed better than those constructed over aggregate base courses in terms of transverse and longitudinal cracking and joint spalling. However, for JRCP, the asphalt-treated base course sections had more faulting and spalling. Neither base type had any cracking. It should be noted that the load transfer devices used in this project were two-part malleable iron (ACME) devices. The performance data for this section is summarized in Table 10.

Ohio 1

This experimental project on U.S. 23 near Chillicothe, Ohio (10), had ATB and aggregate base courses. Sections with ATB performed better than the sections with aggregate bases. The aggregate bases had more faulting and much more cracking, and also displayed many more voids beneath the slab corners. The sections with the ATB did have more spalling on the under side of the joint than did the aggregate base sections, probably due to its poorer drainability. Table 11 provides the pertinent performance information.

Ontario 1

These pavement sections on Highway 3N near Ruthven compared a permeable ATB, an LCB, and a thick slab placed directly on subgrade. The section with the LCB displayed far more longitudinal cracking and more transverse cracking than the other two base types. This is probably due to either reflective cracking from the base or to the stiffness of the base when the slab is undergoing curling effects. The section with the permeable ATB had very little distress, although it did display a lot of visible pumping. This may be attributed to the fact that the outlets for the drainage layer were clogged, resulting in excess water remaining in the pavement system. The 12-in slab placed directly on grade performed relatively well. The performance data for these sections is shown in Table 12.

Pennsylvania 1

The pavement sections on Rt. 66 near Kittanning (11) exhibited no significant differences in performance between five different base sections: CTB, permeable ATB, permeable aggregate base of relatively uniform gradation, a more well-graded, high permeability aggregate base, and a conventional dense-graded aggregate base. Essentially no deterioration had taken place and the PSR of all of the sections was very high. This is probably due to the very low traffic level which, after seven years of service, was only 0.27 million ESAL's. All five sections have longitudinal underdrains, which may have provided adequate drainage of free water from each of the base layers and prevented erosion and pumping for the low level of truck traffic.

New Jersey 3

This project on Interstate 676 near Camden (12) was eight years old and had carried 4.2 million ESAL's at the time of survey. The project included two permeable bases: aggregate and ATB. Each of these performed very well and exhibited only minimal faulting. No cracking or pumping occurred. The permeable ATB did exhibit more joint spalling than the permeable aggregate sections, but this is may be construction-related since the joints are all expansion joints spaced at 78.5-ft intervals. The wide expansion joints may have caused the roughness that resulted in a PSR of 3.5.

North Carolina 1

This older, heavily-loaded project on I-95 near Rocky Mount, North Carolina (13), had four dense graded base types: CTB, ATB, SC, and aggregate. The sections with the ATB displayed superior performance in every category over the sections constructed on the other base courses. The sections constructed on CTB, SC, and aggregate base courses all had a substantial amount of faulting and lower PSR. The sections constructed on the SC base course displayed a large amount of longitudinal cracking. This information is shown in Table 13.

Summary of the Effects of Base Type

The type of base course material usually had a large impact on the visual and measured distresses observed on the pavement surface, particularly for heavily trafficked pavements. Key variables appear to be base/slab interface friction, stiffness, erodibility, and permeability. These had a large effect on transverse and longitudinal cracking, joint spalling, and faulting. Table 14 provides a subjective ranking of the relative performance of the bases at each project site. The following summarizes the overall results for each base type, starting with the poorest performing base:

- Very Poor:** Cement-treated or soil cement bases. Practically every site showed very poor performance in terms of pumping, faulting, and cracking.
- Poor:** Lean concrete bases. This base was associated with considerable pumping, faulting and transverse and longitudinal cracking. This is particularly true when located in a "bathtub" situation with poor subdrainage.
- Fair-Poor:** Asphalt-treated, dense-graded bases. Most sections performed poorly, but there were some that showed good performance. Severe longitudinal cracking and faulting existed at several locations. A bathtub section at Clare, for example, resulted in severe faulting and "D" cracking at joints.
- Fair:** No base. Only two sites were included in the study, with one showing good performance and one showing fair performance. However, both sites had thicker slabs and were subjected to relatively low traffic.
- Good:** Dense-graded aggregate bases. Performance ranged from poor to very good, but was generally fair to very good. The more open the gradation, the better the performance.
- Very Good:** Permeable bases. Virtually no faulting, cracking or joint spalling was present on sections with permeable bases. At one

site pumping was observed, but this is believed to be caused by clogged drainage outlets.

JOINT SPACING

There was a wide variation of joint spacings included in the study. Joint spacing consisted of both uniform and random lengths. A major confounding factor in the analysis of joint spacing was pavement type (JRCP vs. JPCP).

Minnesota 1

The experimental sections on I-94 near Rothsay all were 27-ft JRCP. However, an adjacent section of the Minnesota DOT's old 40-ft JRCP design was also tested and included in the study. The age and traffic are similar to that of the experimental sections. Results show that faulting and PSR are about the same while the amount of deteriorated transverse cracks and spalled joints are much higher for the section with 40 ft joint spacing. These results are shown in Table 15. However, the amount of reinforcement (0.085%) present in both JRCP sections was small and believed to be inadequate for the large temperature extremes encountered in this climate. Thus, the steel was unable to hold the cracks tight, which resulted in their further breakdown and deterioration.

Minnesota 2

These sections on I-90 near Albert Lea consist of long-jointed JRCP and short-jointed JPCP and show essentially equal performance. Faulting, joint spalling, transverse cracking, and PSR were very similar for these sections.

California 1

This project on I-5 allows a comparison of a pavement with an extremely short, random joint spacing of 5-8 11-7 ft and California's conventional, short, random joint spacing of 13-19-18-12 ft. The pavements with the shorter joint spacing had less faulting and less transverse cracking than the pavements with conventional spacing. However, overall roughness was about the same for each section. Most of the transverse cracks that occurred on the longer slab section were in the longer slab segments (i.e., in the 18- and 19-ft slabs). This comparison is illustrated in Table 16.

Michigan 1

The sections on U.S. 10 included long-jointed JRCP and short-jointed dowelled JPCP placed on a granular base. The long-jointed pavements, with a joint spacing of 71 ft, performed similarly to the short, random-jointed pavements which had a joint spacing pattern of 13-17-16-12 ft. Faulting was about the same for each section. The long-jointed JRCP sections had preformed joint seals and 0.15% reinforcement, a combination that resulted in no spalling and very little deteriorated transverse cracking. The shorter jointed sections had more joint spalling than the longer sections, while the longer jointed sections had

a lower PSR. This information is summarized in Table 17.

New York 1

These sections on Route 23 had 20-ft dowelled JPCP and 60-ft JRCP. The long-jointed JRCP sections had no transverse crack deterioration, perhaps because they contained 0.2% steel. However, the JRCP on ATB did have much more joint spalling and much higher joint faulting than the short-jointed sections. The performance data results are shown in Table 18.

New York 2

Interstate 88 near Otego includes a range of joint spacings from 20 ft to 63 ft (14). As illustrated in Table 19, the JRCP section with 63-ft joint spacing exhibited excellent performance. It did not have any deteriorated cracks, although it did have some low severity transverse cracks that were held tight by the reinforcement (0.2%). The 20- and 27-ft JPCP showed some transverse cracking. The faulting was low for all of the sections and the PSR was very high and about the same for all of the sections.

Ohio 1

U.S. 23 near Chillicothe had JRCP sections with 21-ft and 40-ft joint spacing. Sections with shorter 21-ft joint spacing consistently performed better than the long-jointed pavements. The 40-ft slabs displayed higher faulting and much more deteriorated transverse cracking. However, the overall PSR was about the same. Both slab lengths had the same amount of reinforcement, 0.09 percent. The performance data for this project is shown in Table 20.

North Carolina 1

These sections on I-95 near Rocky Mount had 30-ft JPCP and 60-ft JRCP, but the results are confounded by slab thickness. The long-jointed slabs displayed no deteriorated transverse cracking, but a substantial amount of longitudinal cracking. Short jointed sections had no cracking at all. Faulting was observed to be higher for the longer slabs. Spalling was not observed on any of the pavement sections.

Summary of the Effects of Joint Spacing

Joint spacing was observed to have a large effect on the performance of both JRCP and JPCP for most of the projects. For JPCP, the California 1 sections showed that reducing joint spacing from an average of 15 ft to an average of 7.75 ft reduced faulting and transverse cracking significantly. This effect was observed on many of the random-jointed JPCP sections where the 18- and 19-ft slabs always exhibited cracking and the 12- and 13-ft slabs rarely displayed cracking.

For JRCP, the shorter joint spacings (e.g. 20- to 27-ft) usually provided better performance in terms of fewer deteriorated transverse cracks, less transverse joint faulting, and a reduced percent of joints spalled. However, several long-jointed reinforced sections in Michigan, New York, and North Carolina displayed surprisingly good performance in terms of minimal

faulting, little or no transverse cracking, and little joint deterioration. Some of these JRCP sections had a high percentage of reinforcement coupled with a good joint sealant maintenance program, which probably contributed to their enhanced performance.

REINFORCEMENT DESIGN

Reinforcement is placed in JRCP to keep transverse cracks from opening and deteriorating. Little guidance has been provided on the amount of reinforcement needed to hold cracks tight. This is reflected by the fact that several JRCP sections were observed which displayed a considerable amount of deteriorated transverse cracks while other sections showed none.

Minnesota 1

The transverse cracking data from this project on I-94 near Rothsay clearly suggests that dowels had locked-up due to corrosion. Although this result occurred for every base type, there were differences among base types. There were more deteriorated transverse cracks in the dowelled sections, which implies that the reinforcement design was unable to accommodate the increased stresses of immobile joints. The slabs contained only 0.085% steel, which is believed to be inadequate for the temperature extremes of the region.

The aggregate base sections had the least transverse cracking, followed by the CTB and the ATB sections. The longer 40-ft JRCP slabs had more transverse cracking than the shorter 27-ft JRCP slabs, and also had less reinforcing steel.

New Jersey 2 and New Jersey 3

These heavily trafficked pavements both contain a high percentage (0.14% and 0.16%) of reinforcement steel and 1.25-in diameter dowels with a stainless steel cladding. Since both of these factors are expected to contribute to the reduction of transverse cracking, it is not possible to further differentiate between them. The transverse cracking was very low for both of these sections, considering the number of ESAL's they have both experienced.

Other Sections

MI 1 and MI 4, NY 1 and NY 2, and NC 1 all have fairly high percentages of reinforcement steel, ranging from 0.15% to 0.20%. Traffic on these pavements ranges from 0.9 million ESAL's to 9.1 million ESAL's, and joint spacings are all long, ranging from 60 ft to 71 ft. These are constructed over a variety of base types. With the exception of MI 4, which had 205 transverse cracks/mile, none of these pavements had a significant amount of deteriorated transverse cracks.

Summary of the Effects of Reinforcement

The isolated effect of reinforcement content on transverse crack deterioration and pavement performance cannot be directly determined because there were no direct comparisons available. However,

an examination of Table 21, which gives a summary of performance data related to reinforcement design, indicates that the sections with more than 0.10% reinforcement appear to have performed very well regardless of the joint spacing, base type, or dowel coating. At lower levels of reinforcement, it appears that the shorter joint spacings performed better than the longer joint spacings and that bases with higher friction coefficients had more cracking than those with lower friction coefficients.

TRANSVERSE JOINT LOAD TRANSFER

Transverse joint load transfer, the mechanism by which wheel loads are transferred from one side of the joint to the other, is a critical design element for jointed concrete pavements. The following discussion addresses the pavement sections included in the study which evaluate the effectiveness of the two primary means of load transfer, dowel bars and aggregate interlock.

Minnesota 1

This project on I-94 clearly demonstrates that dowel bars are very effective in reducing faulting in cold climates subject to moderate-heavy traffic levels. The summary results in Table 22 show that dowelled joints had greater measured load transfer, lower corner deflections, and a lower percentage of corners with voids (loss of support). As a result, the faulting was reduced by 87% on average. However, the 1-in diameter dowel bars used on the project may not have been large enough, as indicated by the relatively low load transfer. It should be noted that the sections with dowels displayed higher levels of joint spalling and more deteriorated transverse cracks.

North Carolina 1

The results for this section are shown in Table 23. One-inch diameter dowel bars were fairly effective in reducing faulting on aggregate base courses under heavy traffic as compared to no dowel bars. Dowelled joints showed far higher load transfer, lower corner deflections, and one-third fewer corners with loss of support after the very heavy traffic loadings on this project. However, due to the magnitude of faulting for the dowelled joints and the loss of load transfer, it appears that the dowels were not of sufficient diameter for the traffic loadings and joint spacing. The one-inch dowels were completely ineffective for the soil cement stabilized bases, which displayed large faulting and loss of support at the corners.

Nondowelled Sections

In addition to the above directly comparable results, there existed many sections of JPCP without dowels under medium to heavy traffic that provide informative results. This data is presented in Table 24. The following results are suggested from the data shown in Table 24.

1. Nondowelled joints, irrespective of other design features or climate, will generally develop

significant faulting under heavy traffic loadings (over 1 million ESAL's). For short-jointed JPCP, a mean faulting of 0.13-in is considered significant.

2. Although significant faulting developed in all climates, colder and wetter climates develop more faulting than warmer and dryer climates.
3. In general, the higher the joint load transfer, the lower the corner deflection, the fewer the corners with loss of support, and the lower the joint faulting.
4. Longer joint spacing, in the range of 18- to 30-ft, has a much greater amount of faulting than joint spacings in the range of 5- to 13-ft.
5. There were sections with ATB, CTB, LCB, and dense-graded aggregate base courses that showed significant faulting. The permeable bases showed low faulting, although they were not heavily loaded.

Other Dowelled Sections

There were also several individual sections of JPCP and JRCP with dowels that generally showed lower faulting. However, dowel bar diameter also appears to be important in controlling faulting. The following results are obtained from an examination of Table 25.

1. Load transfer is consistently higher for dowelled joints than for nondowelled joints.
2. The longer jointed pavements tend to have higher faulting. Acceptable faulting for longer jointed pavements is about 0.20 in before rideability becomes a problem.
3. A one-inch dowel bar diameter may be too small for heavy traffic. For example, the NC 1-4 and 1-7 sections showed considerable faulting after 9 million ESAL's. However, the shorter joints of MN 1 and ONT 2 containing 1-in diameter dowels showed very little faulting after 5.5 million and 35 million ESAL's, respectively.
4. Several sections with 1.25-in dowel bars showed low faulting after heavy ESAL loadings.

Summary Of the Effects Of Load Transfer

Dowel bars were found to substantially reduce the amount of joint faulting when compared directly with nondowelled sections of otherwise similar design. The diameter of the bar was a factor, with 1.25-in diameter bars providing good load transfer and faulting control under heavy traffic, while a 1-in diameter dowel was sometimes ineffective.

Nondowelled JPCP generally developed significant faulting, regardless of pavement design or climate. Sections with dowels displayed higher levels of joint spalling and more deteriorated transverse cracks. This is believed to be the result of joint lockup caused by corrosion of the dowel bars.

Several other observations can be made concerning load transfer. Although significant faulting developed in all climates, colder and wetter climates develop more faulting than warmer and dryer climates. Also, the longer the joint spacing the greater the faulting, both in sections with and without dowel bars. Finally, base type seemed to have an effect on faulting. Nondowelled pavement designs included sections with dense-graded aggregate bases, ATB or CTB, and LCB that showed significant faulting. The permeable bases showed lower faulting, although they were not heavily loaded.

LONGITUDINAL JOINT DESIGN

There existed a substantial amount of longitudinal cracking on some sections and none on many others. It is believed that the design and construction of the longitudinal joint and the base/slab friction may greatly influence the occurrence of longitudinal cracking, which can lead to serious pavement deterioration, increased maintenance efforts, and reduced pavement life. The results shown in Table 26 were obtained from the field data that has been averaged over similar sections.

The results show that base type is a very large factor in the occurrence of longitudinal cracking. The average longitudinal cracking for each base type is summarized as follows:

No Base	=	86 ft/mile
Lean Concrete Base	=	77
Aggregate Base	=	99
Cement Treated Base	=	729
Asphalt Treated Base	=	664

There is likely a great difference between slab/base friction for these base types. A slab-on-grade design or a section with LCB (typically constructed with a bond-breaker layer) both show very little longitudinal cracking. Both probably have low friction. The CTB and ATB are known to bond very tightly to the slab and, due to their stiffness, produce a large amount of interlayer friction. An aggregate base ranks between the two extremes. This ranking of base types corresponds to friction factors commonly used for pavement design.

The depth of saw cut ranged from 22 to 34% of the slab depth. Within each base type, there does not appear to be any obvious relation between sawcut/slab thickness ratio and longitudinal cracking.

The type of joint forming technique may have an effect on longitudinal cracking. Two types were used: plastic inserts and sawing.

1. The MN 1 sections used inserts and developed extensive longitudinal cracking. All other sections in Minnesota used sawing, and experienced very little longitudinal cracking.

2. Of all the sections with ATB, the section with the most longitudinal cracking was MN 1, which used plastic inserts. The rest of the ATB sections, with one exception, had very little longitudinal cracking.

Summary of Effects of Longitudinal Joint Design

Two conclusions may be reached from examining Table 26. The first is that the type of base has a profound effect on the amount of longitudinal cracking observed. The second conclusion is that the establishment of the longitudinal joint is critical to minimize longitudinal cracking. It is not believed that the inserts were effective in creating the longitudinal joint. For example, it was observed that the average longitudinal cracking on all sections with inserts was 772 ft/mile compared to 309 ft/mile for the sections with the sawcut longitudinal joint. Every effort must be made to ensure the formation of the longitudinal joint. For inserts it would need to be established that the insert was actually placed to the specified depth. For sawcuts, the important factor is the time of sawing. If the sawcut is not made in sufficient time, a longitudinal crack will not form beneath the cut and there will be an increased tendency for longitudinal cracking in the pavement.

TRANSVERSE JOINT SEALANT

Data from the field sections were analyzed to determine if joint sealant type had any effect on joint spalling. The types of sealants included asphalt cement, rubberized asphalt, silicone, and preformed compression seals. Sections were also included which contained no joint sealant. Three test locations were studied where joint sealant type was varied and other factors held constant. In addition, several other observations on the effect of joint sealants can be made from other sites. It should be noted that joint sealing maintenance could also have an effect on joint spalling as well as the original sealant type, but this information was typically not available.

Michigan 4

These two experimental sections, located on I-69 near Charlotte, evaluated different shoulder and sealant types (15). One section had a preformed joint sealant and exhibited no spalling. The other section had a rubberized asphalt joint sealant and displayed joint spalling at 6% of the joints.

California 3

Three experimental pavement sections on U.S. 101 near Geyserville, California evaluated tied and nontied concrete shoulders and sealed and nonsealed joints (16). The section with a rubber asphalt sealant exhibited joint spalling at 2% of the joints whereas the nonsealed sections displayed joint spalling at 3% and 6% of the joints, respectively.

Minnesota 2

These four sections on I-90 were constructed in 1977. Two sections were sealed with a hot-poured sealant and two sections were sealed with preformed

compression sealants. Spalling was minimal for both sealant types, averaging about 6% of all joints for both. However, the preformed sealants were in better condition than the hot poured, rating good as opposed to fair. Also, the preformed sealant was used on the section with 27-ft joints, whereas the hot poured was used on the shorter, random joint spacing section. Thus, the preformed seals were able to perform as well as the hot poured while accommodating larger joint movements.

California Sections

Joint sealants are typically not used in concrete pavement construction in California. Therefore, it is interesting to note that the sections included in the study did not display significant spalling, despite the fact that the oldest section in California was 16 years old. Typical spalling amounts ranged from 0% to 10% of the joints spalled. The climate of California would certainly contribute to the performance of nonsealed joints. Low average rainfall, moderate temperature extremes, and the absence of deicing materials (salt, sand) applied to the roadway, apparently assist in the good performance of nonsealed joints.

Preformed Compression Seals

Preformed compression seals were used on several sections. The following summarizes some of the observed results on these sections.

1. Joints that used preformed compression seals included the MI 1 sections. Even though "D" cracking existed, a long jointed JRCP section showed no joint spalling after 12 years.
2. NY 1 had two sections that were 22 years old, had no load transfer devices, and had preformed compression sealants. None of the joints exhibited any spalling.
3. The OH 1 sections were 14 years old and included preformed compression sealants. They had virtually no joint spalling.
4. The ONT 2 section was 16 years old at the time of the survey and had sustained approximately 35 million ESAL's. It contained preformed compression seals and did not display any joint spalling.

Summary of Effects of Joint Sealing

From the limited data, it appears that there may be some benefit gained from sealing joints in harsh climates. Preformed joint sealants were used quite extensively on the projects evaluated and have performed very well. They appear to be able to offer long-term service lives on the order of 10 to 15 years or more. However, California's mild climates, absence of deicing materials, and short joint spacing appears to allow for their nonsealed joints to perform well.

SHOULDER TYPE

Several sections included in the study provided a direct comparison between a tied PCC shoulder and an AC shoulder. Tied PCC shoulders were also included on several other sections. Observations pertaining to these features are discussed below.

Minnesota 2

Four sections in all were constructed, two with tied PCC shoulders and two with AC shoulders. In terms of shoulder condition, PCC shoulders constructed on I-90 near Albert Lea are performing much better than AC shoulders. The PCC shoulders are rated as excellent with minor deterioration, compared to the poor rating of the AC shoulders, which is due to alligator cracking and lane shoulder dropoff.

The effect of the shoulders on traffic lane performance was unclear. Although very high load transfer efficiencies were measured across the longitudinal lane-shoulder joint, much higher corner deflections were exhibited in the traffic lanes with the PCC shoulders than in the traffic lanes with the AC shoulders. This could perhaps be due to excessive spacing of the tiebars. It was noted that joint faulting was similar for both sections.

The sections with PCC shoulders averaged 448 ft/mi of longitudinal cracking, while the AC shoulder sections had no longitudinal cracking. The tied shoulders may have increased the thermal shrinkage stresses in the traffic lane slabs, resulting in increased longitudinal cracking. The full tied width of a 15-ft inner traffic lane, a 12 ft outer traffic lane, and a 10 ft outer shoulder may have resulted in large tensile stresses, which subsequently caused longitudinal cracking.

Arizona 1

The tied PCC shoulders on S.R. 360 are in better overall condition than the AC shoulders. Corner deflections have been reduced by 25% by the PCC shoulders. No apparent influence on the performance of the traffic lane pavement was noted.

Michigan 1

One section of this project on U.S. 10 near Clare contained a tied PCC acceleration lane. The acceleration lane apparently had no effect on the mainline pavement, as this section is displaying faulting, cracking, and pumping. The acceleration lane itself is separated noticeably from the mainline pavement. It is believed that poor drainage conditions and inadequate tiebar installation are the reasons for the poor performance of the acceleration lane.

Michigan 4

Two adjacent sections, one with tied PCC shoulders and the other with AC shoulders, were constructed on I-69 near Charlotte. Load transfer measured across the lane/shoulder joint was only

35%, which may have decreased the beneficial effect of the PCC shoulder edge support.

The section with tied PCC shoulders exhibited increased joint faulting, decreased crack faulting and transverse cracking compared to the section with AC shoulders. The tied PCC shoulders had lower corner deflections (reduced by nearly 60%) and far fewer corners with loss of support (22% vs. 95%). Both shoulders were rated in fair condition. Mainline performance data as influenced by the shoulders is summarized in Table 27.

New York 2

Both PCC and AC shoulders were constructed on I-88 near Otego. Very little faulting occurred for either section. However, the PCC shoulders were only 6-in thick which, coupled with the bathtub design, made them susceptible to frost heave. The frost heave which occurred was believed responsible for the longitudinal cracking that occurred on each of the concrete shoulder sections.

Ontario 1

PCC shoulders constructed on Highway 3N near Ruthven, Ontario were in much better overall condition than AC shoulders. The AC shoulders were cracked and badly deteriorated. The effect of the shoulders on traffic lane performance is confounded, however, by other pavement design variables.

California 3

The pavement sections on U.S. 101 near Geyserville had both tied and nontied PCC shoulders. The tied PCC shoulders exhibited higher longitudinal lane-shoulder joint load transfer efficiencies than the nontied pavement sections. The tied PCC shoulders were also shown to reduce corner deflections by 50% and appeared to reduce the amount of transverse cracking occurring in the mainline pavement. Joint faulting was about the same for tied and nontied PCC shoulders.

Summary of the Effects of Shoulders

The effect of a tied PCC shoulder on the adjacent traffic lane was mixed. Tied concrete shoulders appear to improve performance of the adjacent traffic lane pavement, provided that an adequate tie is established, subdrainage is provided, and that too wide of a section is not tied together. Corner deflections and the percent of corners with voids was usually less for pavements having tied PCC shoulders. In frost areas, the PCC shoulder should be the same thickness as the traffic lane to avoid differential frost heave and corresponding cracking.

The shoulder pavement condition exhibited by the concrete shoulders was always better than that of the AC shoulders in terms of lane-shoulder dropoff, separation, and surface condition.

DRAINAGE

Very few projects directly compared drained and nondrained pavement sections. Those that did are included for discussion. The subdrainage provided to a pavement was accomplished either by the installation of longitudinal edge drains or by the inclusion of a drainable base layer and edge drains. Many of the projects that did not directly include drainage as a variable did include sections with quite different drainabilities. This was quantified through the use of the AASHTO drainage coefficient, (C_d), which assesses the drainability of the section on a scale of 0.7 (very poor) to 1.25 (excellent). The determination of the AASHTO drainage coefficient is a rational approach that considers the pavement cross-section; the permeabilities of the base, subbase, and subgrade layers; and the amount of available moisture.

Michigan 1

This experimental project allows a determination of the effect of edge drains and a permeable ATB. Sections with edge drains had 21% less faulting on average. Edge drains also reduced the number of slab corners exhibiting loss of support and, for dense-graded asphalt-treated base courses, reduced the amount of spalling from 61% to 41%, due to better drainage and less "D" cracking.

The performance of the pavement section constructed with a permeable ATB material was far superior to that of sections with dense-graded ATB course in terms of faulting and joint spalling. The dense-graded aggregate section with dowels performed similarly to the nondowelled, permeable ATB section in terms of faulting and spalling. Most notably, faulting in the permeable ATB section was held to 0.03 in without the use of dowel bars versus 0.20 in faulting for the ATB. Performance data for this project is summarized in Table 28.

The high percentage of corners indicating a loss of support for the JPCP with either a granular base and dowelled joints (95%) or a permeable ATB without dowels (75%), and high corner deflections, causes concern for the long term performance of the sections under heavy traffic. Actually, traffic has been relatively light, only 0.9 million ESAL's over a 12-year period.

Arizona 1

On S.R. 360 in Phoenix, no difference in performance was observed between a section with edge drains and a section without. However, the sections are only five years old and are placed in a mild, dry climate where edge drains may not be appropriate.

California 2

Section CA 2, on I-210 near Los Angeles showed that the construction of a strong permeable CTB can be accomplished ($k=1400$ pci). However, the pavement consisted of a dense-graded AC layer

directly beneath the PCC slab and on top of the permeable CTB. This apparently resulted in extensive erosion of the AC and caused faulting of 0.11 in after only 4.4 million ESAL's.

Pennsylvania 1

On Rt. 66 near Kittanning, it appears that any positive means of removing water, either in the form of edge drains or a permeable base layer, is enough to provide good performance. However, the sections are only seven years old and under relatively light traffic. The composite k-value for the permeable ATB was 1000 pci, much higher than that of either the CTB or aggregate bases.

New Jersey 3

Permeable base courses constructed on I-676 near Camden demonstrated good performance under heavy traffic (4.4 million ESAL's). No significant distress was observed. Composite k-values of 350 pci for the permeable aggregate base course and 210 pci for the permeable asphalt-treated base course were obtained.

Summary of the Effects of Drainage

Positive drainage, in the form of either edge drains or permeable base layers with edge drains, generally showed a marked effect in reducing faulting and other distresses, particularly "D" cracking spalling. Several sections having a permeable base are performing very well. Most of these are under lighter traffic, however. The structural support provided by a permeable base is excellent at the slab center. However, the corner deflections are quite high and there is indication of loss of support at the corners also. Thus, it may be necessary to dowel these pavements when they are to be loaded with very heavy traffic.

Placement of the permeable drainage layer directly beneath the slab is critical, as shown by the results of CA 2. No intermediate layer of AC should be placed between the drainage layer and the slab, or erosion will occur.

Another way of looking at the data is to organize the projects with different base types by AASHTO drainage coefficient, C_d . The projects with different base types are organized by drainage coefficient along with several key distresses in Table 29. For the pavements which have sustained enough traffic loads for the trends to become clear, it can generally be stated that as the drainage coefficient improves or increases, the load-related distresses decrease. This appears to hold true for joint faulting and transverse cracks with very few exceptions.

SUMMARY AND CONCLUSIONS

Performance results obtained from a number of experimental projects constructed over the past 20 years were studied to assess the effect of a large number of pavement variables on pavement performance. While there is some confounding of variables, some general conclusions can still be made

relative to the design and performance of jointed plain and reinforced concrete pavements.

The effect of slab thickness on pavement performance was unambiguous. It was found that increasing slab thickness helped to reduce transverse and longitudinal slab cracking. This effect was much more noticeable when slab thicknesses were increased an inch on thinner pavements, say from 8 in to 9 in, then when they were increased several inches, from 9-in to 11-in or from 11-in to 13-in. Increasing the thickness of the slab did not appear to reduce joint spalling or joint faulting. Thick slabs on grade, especially in wet climates and exposed to heavy traffic, faulted as much as thinner pavements constructed on a base course.

It was not possible to directly compare the performance of the thinner slabs and the thicker slabs, as the thick slabs were all constructed directly on the subgrade and the thinner slabs were all constructed on base courses.

The base type seemed to have a very large effect on the performance of jointed concrete pavements. The aspects of base type which affected pavement performance include: base/slab interface friction; base stiffness; base erodibility; and base permeability or drainability. The major performance indicators which were affected by variations in base type were transverse and longitudinal cracking, joint spalling, and faulting.

The worst-performing base type was the cement-treated or soil cement bases. These tended to exhibit excessive pumping, faulting, and cracking. This is most likely a result of the use of an impermeable layer that traps moisture and yet can break down and contribute to the movement of fines beneath the slab.

The use of lean concrete bases produced generally poor performance. Large curling and warping stresses have been associated with slabs constructed over lean concrete bases. These stresses result in considerable transverse and longitudinal cracking of the slab. Lean concrete bases have performed well in areas exposed to very little moisture.

The performance of the asphalt-treated base courses ranged from poor to good. The asphalt-treated base courses have the potential to contribute to a bathtub section in which moisture is trapped within the pavement cross-section. This will cause severe cracking, faulting, and possibly pumping.

Pavements constructed over aggregate bases had varied performance, but were generally in the fair to very good category. In general, the more open the gradation of the aggregate in the base, the better the pavement performed. An advantage of the aggregate bases is that they contribute the least to the high curling and warping stresses in the slab.

The best bases in terms of pavement performance were those designed to be permeable. Typical base courses have permeabilities ranging from 0 to less than 1 ft/day; good permeable bases have permeabilities upwards of 1000 ft/day. The highly permeable bases typically performed very well, except in one case where the drain outlets were clogged.

Several experimental projects evaluated the effects of different joint spacings on pavement performance. The average range of joint spacings for JPCP pavements was from 7.5 ft to 30 ft. For JPCP pavements, reducing the joint spacing reduces the magnitude of joint faulting and also reduces the amount of transverse cracking. On sections with random joint spacing, it was found that if the longer slabs were greater than 18 ft long, they experienced more transverse cracking than the shorter slabs.

The range of joint spacings for JRCP pavements included in this study was from 21 ft to 78 ft. Generally, the shorter joint spacings performed better, as measured by deteriorated transverse cracks, joint faulting, and joint spalling. However, several long-jointed JRCP slabs performed quite well, probably due to a combination of adequate reinforcement and good joint seal maintenance.

There are many sections included in the study that allow for the direct comparison of the two methods of transverse joint load transfer. Dowel bars were found to substantially reduce the amount of joint faulting when compared directly with nondowelled sections of otherwise similar design. The diameter of the dowel bar had an effect on performance, as larger diameter bars provided better load transfer and control of faulting under heavy traffic than did smaller dowels.

Nondowelled JPCP slabs generally developed significant faulting, regardless of pavement design or climate. This effect was somewhat mitigated by the use of permeable bases.

Some sections with dowels displayed higher levels of joint spalling and more deteriorated cracks than sections of similar design without load transfer devices. It is strongly suspected that the load transfer devices in these sections had an effect on performance, as larger diameter bars provided better load transfer and control of faulting under heavy traffic than did smaller bars.

Nondowelled JPCP slabs generally developed significant faulting, regardless of pavement design or climate. This effect was somewhat mitigated by the use of permeable bases.

Some sections with dowels displayed higher levels of joint spalling and more deteriorated cracks than sections of similar design without load transfer devices. It is strongly suspected that the load transfer devices in these sections have corroded and locked up, or were misaligned at the time of placement.

It is a widespread belief that the construction of tied shoulders on jointed concrete pavements increases pavement life by reducing edge stresses and edge and corner deflections in the mainline pavement. The use of tied shoulders on concrete pavements also allows for the establishment of an easily maintained joint reservoir between the mainline pavement and the shoulder. This study showed that Portland cement concrete shoulders performed better than asphalt concrete shoulders, although many of the tied shoulder designs were deficient and did not assist in the performance of the mainline pavement. Several factors are critically important if the benefits of tied concrete shoulders are to be realized. Ties must be strategically located near slab corners to provide support. Also, in areas susceptible to frost heave action, it is recommended that the shoulder have the same thickness as the slab to avoid the effects of differential frost heave.

One area of pavement design for which little guidance exists is drainage. The provision of positive drainage, either in the form of longitudinal edge drains or the combination of a drainage layer and edge drains, generally resulted in a reduction in faulting and spalling. This study evaluated drainage indirectly, by the rational calculation of the AASHTO drainage coefficient (C_d) for every pavement section.

The results showed that, with very few exceptions, as the drainage characteristics of the pavement sections improved, the load-related distresses decreased. This was especially true for faulting and transverse cracking. However, it appears that overall pavement performance can be improved by the construction of a base layer with a high drainage coefficient. Two major components of such a base are an open-graded material with a high permeability and the restriction of the percent fines.

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Table 1. List of Experimental Projects Included in Study.

DRY-FREEZE CLIMATIC ZONE

State	Designation/ Location	Year Constructed	Number of Sections	Design Features Evaluated
MN	Minnesota 1 I-94, Rothsay	1970	12	Base Type Slab Thickness Load Transfer
MN	Minnesota 2 I-90, Albert Lea	1977	4	Pavement Type Slab Thickness Joint Spacing PCC Shoulders

DRY-NONFREEZE CLIMATIC ZONE

State	Designation/ Location	Year Constructed	Number of Sections	Design Features Evaluated
AZ	Arizona 1 S.R. 360, Phoenix	1972- 1982	6	Slab Thickness Base Type PCC Shoulders Drainage
CA	California 1 I-5, Tracy	1971	5	Slab Thickness Base Type Joint Spacing
CA	California 2 I-210, Los Angeles	1980	2	Base Type Drainage

WET-NONFREEZE CLIMATIC ZONE

State	Designation/ Location	Year Constructed	Number of Sections	Design Features Evaluated
NC	North Carolina 1 I-95, Rocky Mount	1967	8	Pavement Type Slab Thickness Base Type Joint Spacing Load Transfer PCC Shoulders
CA	California 3 U.S. 101, Geyserville	1975	3	PCC Shoulders

WET-FREEZE CLIMATIC ZONE

State	Designation/ Location	Year Constructed	Number of Sections	Design Features Evaluated
MI	Michigan 1 U.S. 10, Clare	1975	8	Pavement Type Base Type Joint Spacing Load Transfer Drainage
MI	Michigan 4 I-69, Charlotte	1972	2	PCC Shoulders
NY	New York 1 Route 23, Catskill	1965	6	Pavement Type Base Type Joint Spacing Load Transfer
NY	New York 2 I-88, Otego	1975	4	Pavement Type Joint Spacing PCC Shoulders
OH	Ohio 1 U.S. 23, Chillicothe	1973	7	Pavement Type Base Type Joint Spacing Load Transfer
OH	Ohio 2 Route 2, Vermilion	1974	2	PCC Shoulders
ONT	Ontario 1 Hwy 3N, Ruthven, Ontario	1982	4	Slab Thickness Base Type PCC Shoulder
ONT	Ontario 2 Hwy 427, Toronto, Ontario	1971	1	Control
PA	Pennsylvania 1 Route 66, Kittanning	1980	5	Base Type Drainage
NJ	New Jersey 2 Route 130, Yardville	1951	1	Control
NJ	New Jersey 3 I-676, Camden	1979	2	Base Type Drainage

Table 2. Outer lane performance data relevant to slab thickness for Minnesota 1 (Age = 17 years, ESAL's = 5.5 million).

Slab Thick, in	Dowels	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	% Joint Spalling	PSR
8	No	0.39	25	2406	23	3.5
9	No	0.33	0	968	8	3.7
8	Yes	0.04	47	1464	25	3.4
9	Yes	0.08	55	1744	30	3.3

Longitudinal cracking includes inner and outer lane.

Table 3. Outer lane performance data relevant to slab thickness for Arizona 1.

Slab Thick, in	ESAL's, millions	Age	Joint Faulting, in	Longitudinal Cracks, ft/mi	% Joint Spalling	PSR
9	4.0	15	0.08	233	22	3.4
11	2.8	8	0.03	0	0	3.8
13	3.4	12	0.01	0	0	3.8

Longitudinal cracking includes inner and outer lane.

Table 4. Outer lane performance data relevant to slab thickness for California 1 (Age = 16 years, ESAL's = 7.6 million).

Slab Thick, in	Dowels	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	% Joint Spalling	PSR
8.4	No	0.10	30	940	3	3.0
11.4	No	0.11	0	0	3	2.7

Longitudinal cracking includes inner and outer lane.

Table 5. Outer lane performance data relevant to slab thickness for Ontario 1 (Age = 5 years, ESAL's = 0.84 million).

Slab Thick, in	Dowels	Base Type	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	PSR
7	No	LCB	0.05	48	345	3.8
8	No	LCB	0.06	27	480	3.8
12	No	None	0.06	0	40	3.8

Longitudinal cracking includes inner and outer lane.

Table 6. Outer lane performance data relevant to base type for Minnesota 1 (Age = 17 years, ESAL's = 5.5 million).

Base Type	Dowels	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	% Joint Spalling
AGG	No	0.31	17	150	6
ATB	No	0.34	21	2807	20
CTB	No	0.44	0	2160	20
AGG	Yes	0.06	35	0	15
ATB	Yes	0.03	51	4302	50
CTB	Yes	0.10	44	510	18

Longitudinal cracking includes inner and outer lane.

Table 7. Outer lane performance data relevant to base type for California 1 (Age = 16 years, ESAL's = 7.6 million).

Base Type	Dowels	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	Pumping	PSR
CTB	No	0.10	30	940	None	3.0
LCB	No	0.06	75	230	High	2.7

Longitudinal cracking includes inner and outer lane.

Table 8. Outer lane performance data relevant to base type for California 2 (Age = 7 years, ESAL's = 4.4 million).

Base Type	Dowels	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	Pumping	PSR
AC/PCTB	No	0.11	0	0	None	3.8
CTB	No	0.11	290	0	None	4.1

Longitudinal cracking includes inner and outer lane.

Table 9. Outer lane performance data relevant to base type for Michigan 1 (Age = 12 years, ESAL's = 0.9 million).

Type	Base Type	Dowels	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	% Joint Spalling	PSR
JPCP	PATB	No	0.03	0	0	9	3.9
JPCP	ATB	No	0.18	24	0	69	2.9
JPCP	AGG	Yes	0.04	0	0	12	3.6
JRCP	AGG	Yes	0.07	2	0	0	3.4

Longitudinal cracking includes inner and outer lane.

Table 10. Outer lane performance data relevant to base type for New York 1
(Age = 22 years, ESAL's = 2.0 million).

Type	Base Type	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	% Joint Spalling	PSR
JPCP	ATB	0.02	0	0	6	4.0
JPCP	AGG	0.03	35	264	13	3.9
JRCP	ATB	0.14	0	0	73	3.6
JRCP	AGG	0.09	0	0	9	3.4

Longitudinal cracking includes inner and outer lane.

Table 11. Outer lane performance data relevant to base type for Ohio 1
(Age = 14 years, ESAL's = 3.4 million).

Joint Spacing, ft	Base Type	Dowels	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	% Joint Spalling	PSR
21	ATB	Yes	0.06	0	0	13	4.2
21	AGG	Yes	0.07	15	0	0	4.2
40	ATB	Yes	0.07	29	0	0	4.1
40	AGG	Yes	0.10	118	0	0	4.2

Longitudinal cracking includes inner and outer lane.

Table 12. Average performance data relative to base type for Ontario 1
(Age = 5 years, ESAL's = 0.84 million).

Slab Thick, in	Base Type	Dowels	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	Pumping	PSR
8	PATB	No	0.06	0	0	Medium	3.8
8	LCB	No	0.07	28	480	Low	3.8
12	None	No	0.06	0	40	Low	3.8

Longitudinal cracking includes inner and outer lane.

Table 13. Outer lane performance data relative to base type for North Carolina 1 (Age = 20 years, ESAL's = 9.1 million).

Base Type	Dowels	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	PSR
AGG	Yes	0.13	0	0	3.4
AGG	No	0.17	32	0	3.5
ATB	No	0.05	0	0	3.8
CTB	No	0.16	0	179	3.2
SC	No	0.13	5	3068	3.6
SC	Yes	0.16	10	1451	3.5

Longitudinal cracking includes inner and outer lane.

Joint spacing = 30 ft, Dowel diameter = 1 in.

Table 14. Overall relative summary of the performance of base types.

PROJECT ID	B A S E T Y P E					
	CTB/SC	ATB	LCB	NONE	AGG	PERM
MN 1	VP	VP	---	---	G	---
MI 1	---	VP	---	---	VG	VG
NY 1	---	G	---	---	F	---
OH 1	---	G	---	---	F	---
ONT 1	---	---	VP	F	---	F
PA 1	(VG)	---	---	---	(VG)	(VG)
NJ 1	---	---	---	---	---	VG
NC 1	VP	VG	---	---	P	---
CA 1	VP	---	VP	---	---	---
CA 2	VP	VP	---	---	---	---
AZ 1	P	---	VG	VG	---	---

KEY: CTB: Cement treated base SC: Soil cement
 ATB: Asphalt treated base AGG: Dense graded aggregate base
 LCB: Lean concrete base NONE: No base course
 PERM: Permeable, highly drainable base

RATINGS: VG: Very good base performance (no distress)
 G: Good base performance
 F: Fair base performance
 P: Poor base performance
 VP: Very poor base performance (extensive distress)

(): Indication that pavement was subjected to less than one-half million ESAL's in the outer traffic lane.

Table 15. Outer lane performance data relevant to joint spacing for Minnesota 1 (Age = 17 years, ESAL's = 5.5 million).

Joint Spacing, ft	Dowels	Reinf, %	Joint Faulting, in	Transverse Cracks/mi	% Joint Spalling	PSR
27	Yes	0.08	0.10	23	14	3.3
40	Yes	0.04	0.09	53	36	3.3

Sections are 9-in JRCF with aggregate base.

Table 16. Outer lane performance data relevant to joint spacing for California 1 (Age = 16 years, ESAL's = 7.6 million).

Joint Spacing, ft	Dowels	Joint Faulting, in	Transverse Cracks/mi	% Joint Spalling	Pumping	PSR
5-8-11-7	No	0.06	5	2	Medium	2.9
13-19-18-12	No	0.10	30	3	None	3.0

Table 17. Outer lane performance data relevant to joint spacing for Michigan 1
(Age = 12 years, ESAL's = 0.9 million).

Joint Spacing, ft	Dowels	Joint Faulting, in	Transverse Cracks/mi	% Joint Spalling	Pumping	PSR
13-17-16-12	Yes	0.04	0	11	None	3.6
71	Yes	0.06	2	0	None	3.2

The short jointed JPCP cracking includes all severities. The long jointed JRCP includes only deteriorated cracks.

Table 18. Outer lane performance data relevant to joint spacing for New York 1
(Age = 22 years, ESAL's = 2.0 million).

Joint Spacing, ft	Base Type	Dowels	Joint Faulting, in	Transverse Cracks/mi	% Joint Spalling	PSR
20	ATB	Yes	0.02	0	6	4.0
60	ATB	Yes	0.14	0	73	3.6
20	AGG	Yes	0.03	35	13	3.9
60	AGG	Yes	0.09	0	9	3.4

Dowels are two-part malleable iron load transfer devices.

Table 19. Outer lane performance data relevant to joint spacing for New York 2
(Age = 12 years, ESAL's = 1.43 million).

Joint Spacing, ft	Joint Dowels	Joint Faulting, in	Transverse Cracks/mi	% Joint Spalling	PSR
20	Yes	0.01	30	0	4.1
27	Yes	0.01	26	0	4.1
63	Yes	0.02	0	0	4.0

Dowels are 1-in epoxy-coated I-beams.

Table 20. Outer lane performance data relevant to joint spacing for Ohio 1
(Age = 14 years, ESAL's = 3.4 million).

Joint Spacing, ft	Base Type	Dowels	Joint Faulting, in	Transverse Cracks/mi	% Joint Spalling	PSR
21	ATB	Yes	0.06	0	13	4.2
40	ATB	Yes	0.07	29	0	4.1
21	AGG	Yes	0.06	15	0	4.2
40	AGG	Yes	0.11	114	0	4.2

All pavements are 9-in JRCP.

Table 21. Summary of performance data related to reinforcement design.

Project ID	ESAL's (millions)	Base Type	Joint Spacing, ft	Percent Reinf.	Dowel Dia., in	Transverse Cracks/mile
DRY-FREEZE						
MN 1	5.5	AGG	27	0.085	0.00	16
		AGG	27	0.085	1.00 NC	35
		ATB	27	0.085	0.00	21
		ATB	27	0.085	1.00 NC	51
		CTB	27	0.085	0.00	0
		CTB	27	0.085	1.00 NC	44
		AGG	40	0.040	1.00 NC	53
MN 2	2.8	AGG	27	0.090	1.00 NC	3
WET-FREEZE						
MI 1	0.9	AGG	71	0.150	1.25 EC	3
MI 4	4.3	AGG	71	0.150	1.25 NC	205
NY 1	2.0	AGG	61	0.200	ACME	0
		ATB	61	0.200	ACME	0
NY 2	1.1	AGG	63	0.200	I-BEAM	0
OH 1	3.4	AGG	21	0.090	1.25 NC	0
		AGG	21	0.090	1.25 PC	31
		AGG	40	0.090	1.25 NC	0
		AGG	40	0.090	1.25 PC	235
		ATB	21	0.090	1.25 NC	0
		ATB	40	0.090	1.25 NC	29
NJ 2	35.0	AGG	78	0.140	1.25 SSC	24
NJ 3	4.2	PAGG PATB	78	0.160	1.25 SSC	0
WET-NONFREEZE						
NC 1	9.1	AGG	60	0.170	1.25 NC	0

Key: NC = No Coating (of Dowels)
 EC = Epoxy-Coated
 PC = Plastic-Coated
 SSC = Stainless-Steel Coating

Table 22. Outer lane performance data relevant to joint load transfer for Minnesota 1 (Age = 17 years, ESAL's = 5.5 million).

Slab,in	Dowel Dia,in	Joint Faulting,in	Transverse Cracks/mi	% Joint Spalling	L.T. Eff.%	Corner Defl,mils	% Corner Voids
8	0.00	0.39	25	23	53	12.3	12
8	1.00	0.04	32	25	62	11.1	0
9	0.00	0.33	0	8	38	13.1	5
9	1.00	0.08	55	30	67	10.5	0

Data is averaged across all base types.

Table 23. Outer lane performance data relevant to joint load transfer for North Carolina 1 (Age = 20 years, ESAL's = 9.1 million).

Base Type	Dowel Dia,in	Joint Faulting,in	Transverse Cracks/mi	% Joint Spalling	L.T. Eff,%	Corner Defl,mils	% Corner Voids
AGG	0.00	0.22	64	0	33	9.7	20
AGG	1.00	0.13	0	0	68	8.6	7
SC	0.00	0.13	5	0	100	4.0	0
SC	1.00	0.16	10	0	59	8.8	15

Sections are 9-in JPCP with 30-ft joint spacing.

Table 24. Load transfer performance data for nondowelled pavement sections.

Project ID	ESAL's (mill)	Base Type	Joint Spacing,ft	L.T. Eff,%	Corner Defl,mils	% Corner Voids	Pumping	Joint Fault,in
WET-NONFREEZE								
CA 3	3.6	CTB	12-19	80	4.3	4	None	0.10
WET-FREEZE								
MI 1								
4a	0.9	PATB	12-19	19	21.8	75	None	0.03
10ab	0.9	ATB	12-19	42	17.1	85	Low	0.17
OH 2	3.3	NONE	20	--	----	--	Medium	0.11
ONT 1								
1	1.0	NONE	12-19	--	----	--	Low	0.05
2	1.0	PATB	12-19	--	----	--	Medium	0.05
3,4	1.0	LCB	12-19	--	----	--	Low	0.04
NY 1								
8ab	3.1	ATB	20	100	10.9	22	None	0.02
DRY-NONFREEZE								
CA 1								
1	7.6	CTB	5-11	85	15.9	60	Medium	0.06
3,5,9	7.6	CTB	12-19	87	14.0	55	Low	0.11
7	7.6	LCB	12-19	88	8.2	10	High	0.06
CA 2								
2	4.4	AC/PCTB	12-19	10	14.9	65	None	0.11
3	4.4	CTB	12-19	19	24.4	90	None	0.11
AZ 1								
1	4.0	CTB	13-17	94	8.6	30	None	0.08
2,4,5	2.9	NONE	13-17	100	7.8	12	None	0.02
6,7	1.8	LCB	13-17	97	6.0	12	None	0.01
DRY-FREEZE								
MN 1								
1,3,5,7,9,11	5.5	AGG ATB CTB	27	46	12.7	8	N-Med	0.36

Note: Some of the data from similar sections are averages.

Table 25. Load transfer performance data for dowelled sections.

Project ID	ESAL's (mill)	Joint Spacing, ft	Dowel Dia, in	L.T. Eff, %	Corner Defl, mils	% Corner Voids	Joint Fault, in
WET-FREEZE							
OH 1	3.4	40	1.25	88	13.2	63	0.10
MI 4	4.4	71	1.25	59	21.3	58	0.09
OH 1	3.4	21	1.25	83	12.6	31	0.06
WET-NONFREEZE							
NC 1-7	9.1	60	1.00	77	8.3	0	0.15
NC 1-4	9.1	30	1.00	59	8.8	15	0.13
DRY-FREEZE							
MN 1 2,4,6, 8,10,12	5.5	27	1.00	64	10.8	0	0.06

Table 26. Summary of longitudinal cracking and related design data.

Project ID	Age	Base	Joint Forming	Joint Depth, in	Depth/Thick	Long. Cracks ft/mile
OH 2	13	None	Saw	3.75	0.25	258
ONT 1-1	5	None	Saw	3.00	0.25	0
AZ 1-2,3	10	None	Saw	3.25	0.25	0
NO BASE MEAN = 86						
MN 2	10	AGG	Saw	2.75	0.32	224
NY 2	12	AGG	Saw	2.25	0.25	194
NY 1	22	AGG	Saw	2.00	0.22	132
MI 4	15	AGG	Saw	2.75	0.30	91
MN 1	17	AGG	Insert	2.75	0.32	75
NC 1						
1,4,7,8	20	AGG	Saw	2.75	0.30	74
OH 1	14	AGG	Saw	2.25	0.25	0
MI 1	12	AGG	Saw	2.75	0.30	0
AGG BASE MEAN = 99						
CA 1-5	16	LCB	Insert	2.00	0.24	230
AZ 1-6,7	5	LCB	Saw	2.25	0.25	0
CA 6	7	LCB	Insert	2.00	0.24	0
LCB MEAN = 77						
NC 1-2,3	20	SC	Saw	2.75	0.30	2260
CA 7	7	CTB	Saw	3.00	0.29	2060
MN 1	17	CTB	Insert	2.75	0.32	1320
CA 1-1,3,9	16	CTB	Insert	2.00	0.24	500
AZ 1-1	15	CTB	Saw	2.25	0.25	233
NC 1-5	20	CTB	Saw	2.75	0.30	179
CA 3	12	CTB	Insert	2.00	0.22	11
CA 1-5	16	CTB	Insert	2.00	0.26	0
CA 2-3	7	CTB	Saw	2.00	0.24	0
CTB MEAN = 729						
MN 1	17	ATB	Insert	2.75	0.32	3550
CA 8	7	AC	Saw	3.00	0.29	1026
NY 1	22	ATB	Saw	2.00	0.22	73
OH 1	14	ATB	Saw	2.25	0.25	0
NC 1-6	20	ATB	Saw	2.75	0.30	0
CA 2-2	7	AC/PCTB	Saw	2.00	0.24	0
ONT 1-2	5	PATB	Saw	2.20	0.27	0
ATB MEAN = 664						

Table 27. Influence of shoulder on performance of mainline pavement for Michigan 4 (Age = 15 years, ESAL's = 4.4 million).

Section	Shoulder Type	L.T. Eff, %	Corner Defl, mils	% Corner Voids	Joint Faulting, in	Crack Faulting, in
MI 4-1	PCC	51	15.6	22	0.12	0.20
MI 4-2	AC	67	27.1	95	0.06	0.31

Table 28. Performance data relevant to drainage for Michigan 1
(Age = 12 years, ESAL's = 0.9 million).

Pavement Type	Base Type	Edge Drains	Joint Faulting, in	% Joint Spalling	% Corner Voids	Corner Defl, mils
JPCP	ATB	Yes	0.14	41	70	13.7
JPCP	ATB	No	0.19	62	100	20.4
JPCP	PATB	Yes	0.03	9	75	21.8
JRCP	AGG	Yes	0.05	0	0	6.7
JRCP	AGG	No	0.08	0	15	10.3
JPCP	AGG	No	0.04	12	100	31.8
JPCP	AGG	Yes	0.04	11	94	26.3

Table 29. Summary of selected performance data related to C_d .

Project ID	ESAL's (millions)	Base Type	Joint Faulting, in	% Joint Spalling	Transverse Cracks/mi	Average C _d
DRY-FREEZE						
MN 1	5.5	CTB	0.27	19.0	22	0.80
		ATB	0.19	34.8	36	0.85
		AGG	0.19	10.3	28	1.05
MN 2	2.8	AGG	0.06	5.8	1	0.84
WET-FREEZE						
MI 1	0.9	ATB	0.18	59.7	16	0.80
		AGG	0.05	5.8	1	0.90
		PATB	0.03	9.0	0	1.10
MI 4	4.3	AGG	0.09	3.0	205	0.73
NJ 2	35.0	AGG	0.06	20.0	24	1.05
NJ 3	4.2	PAGG	0.05	0.0	0	1.10
		PATB	0.06	43.0	0	1.10
NY 1	2.0	AGG	0.06	11.0	18	0.78
		ATB	0.05	19.8	2	0.93
OH 1	3.4	ATB	0.07	6.5	74	0.90
		AGG	0.09	0.0	15	0.96
ONT 1	1.0	NONE	0.05	0.0	0	0.95
		PATB	0.05	0.0	0	1.00
		LCB	0.04	0.0	38	1.00
PA 1	0.3	CTB	0.03	0.0	0	0.90
		AGG	0.03	0.0	0	1.02
		PATB	0.02	0.0	0	1.05
WET-NONFREEZE						
NC 1	9.1	CTB	0.16	0.0	0	0.75
		AGG	0.16	0.0	17	0.80
		SC	0.15	0.0	8	0.89
		ATB	0.05	0.0	0	0.90

Note: All values presented are averages by base type.