The Longevity and Performance of Diamond-Ground Pavements

by Shreenath Rao, H. Thomas Yu, and Michael I. Darter

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KEYWORDS: concrete pavement rehabilitation, diamond grinding, faulting, pavement performance, roughness, smoothness, surface texture

ABSTRACT: Diamond grinding restores a smooth riding surface with desirable friction characteristics on concrete pavements. This technique was first used in 1965 on a 19-year-old section of I-10 in southern California to eliminate excessive faulting (Neal and Woodstrom 1976). Since then, diamond grinding has become an important element of concrete pavement restoration.

The study involved conducting a comprehensive review of existing information on diamond grinding, data collection, data analysis, and documentation of the study findings. Extensive field surveys were conducted to obtain the performance data needed for the analysis. In all, 60 pavement sections in 18 states were surveyed. In addition, performance data for 133 sections were obtained from an earlier study of the performance of diamond-ground pavements (Snyder et al. 1989). The data from the Long-Term Pavement Performance sections (concrete pavement rehabilitation) were also used to conduct direct side-by-side comparisons of the performance of diamond-ground pavement sections and other rehabilitation alternatives. Various analyses were conducted to document the performance of diamond-ground pavements, including an evaluation of faulting performance, longevity of diamond-ground texture, and the effects of diamond grinding on service life. Diamond-ground surfaces were demonstrated to provide several years of service. No evidence of any deleterious effects of diamond grinding was observed at any field site.

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MOTS CLÉS: meulage au diamant, mise en escalier, performance de chaussée, réhabilitation de pavage de béton, rugosité et uni de surface, texture de surface.

RÉSUMÉ: Le meulage au diamant permet de redonner aux pavages de béton une bonne qualité de roulement avec les caractéristiques de friction requises pour de tels pavages. Cette technique a été utilisée pour la première fois en 1965, sur une section âgée de 19 ans de la I-10 en Californie du sud, pour éliminer la mise en escalier excessive (Neal and Woodstrom 1976). Depuis, le meulage au diamant est devenu un élément important de la restauration des pavages de béton. Malgré une longue histoire de réussites avec cette méthode, il existe très peu de documentation valable sur la performance des pavages meulés au diamant. Le but de cette étude était de fournir une telle documentation. Cette étude comprenait la revue en profondeur de l'information existant sur le meulage au diamant, la collecte de données, l'analyse de donnée et la production des résultats.

Une vaste enquête a été menée sur le terrain afin d'obtenir les données de performance nécessaires à l'analyse. En tout, 60 sections de pavage dans 18 états ont été étudiées. De plus, les données de performance de 133 sections ont été obtenues à partir d'une étude antérieure sur la performance des pavages meulés au diamant (Snyder et al. 1989). Les données du programme "Performance à long terme des pavages" ayant trait à la réhabilitation des pavages de béton ont aussi été utilisées pour faire des comparaisons parallèles directes de la performance des sections meulées au diamant avec d'autres alternatives de réhabilitation. A l'aide des données recueillies, diverses analyses ont été faites pour documenter la performance de pavages meulés au diamant, incluant une évaluation de la mise en escalier, de la longévité de la texture du meulage au diamant et des effets du meulage au diamant sur la vie utile.

RÉFÉRENCE: Rao, Shreenath, Yu, H. Thomas, and Darter, Michael I., *La longévité et la performance des pavages meulés au diamant*, Bulletin de recherche et de développement RD118T, Association du Ciment Portland, Skokie, Illinois, Etats-Unis, 1999, 112 pages.

Cover photos: Top left: Diamond grinding of concrete pavement (S#61320)
Bottom left: Closeup of ground pavement (S#69075)
Right: Diamond grinding of concrete pavements (S#61262)

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by Shreenath Rao[1], H. Thomas Yu[2], and Michael I. Darter[3]

Executive Summary

Diamond grinding is a concrete pavement restoration (CPR) technique that provides a smooth riding surface with the desirable friction characteristics on concrete pavements that have developed excessive roughness. Diamond grinding offers numerous advantages over other rehabilitation alternatives, including the following:

- Costs substantially less than an overlay.
- Enhances surface friction and safety of an old pavement surface.
- Can be accomplished during off-peak hours with short lane closures and without encroaching into the adjacent lanes.
- Grinding of one lane does not require grinding of the adjacent lane, which may have perfectly acceptable surface characteristics.

For concrete pavements in good structural condition, diamond grinding can be a highly effective and economical rehabilitation alternative. The history of continuous diamond grinding for pavement restoration dates back to 1965, to a 19-year-old section of the San Bernardino Freeway (I-10) in California that was diamond-ground to eliminate excessive faulting. Since then, diamond grinding has become a major element of CPR projects. Despite the long and successful history, however, very little valid documentation of the performance of diamond-ground pavements exists. Recognizing the need for such documentation, the Portland Cement Association (PCA), in association with American Concrete Pavement Association (ACPA) and International Grooving and Grinding Association (IGGA), sponsored a study to evaluate the performance of diamond-ground pavements. The objective of this study was to provide the answers to frequently asked questions about diamond grinding, including the following:

- What is the immediate impact of diamond grinding on pavement performance?
- How long can diamond-ground surfaces provide acceptable ride quality?
- How long can diamond-ground surfaces provide acceptable surface texture?
- When is diamond grinding feasible and effective?
- Can diamond grinding be used more than once on a pavement?
- Are there any adverse effects of diamond grinding?

STUDY OF DIAMOND-GROUND PAVEMENTS

This study involved a comprehensive review of existing information on diamond grinding, data collection,

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data analysis, and documentation of the study findings. The performance data needed for the evaluation were obtained from existing databases and by conducting field surveys. The main source of existing data was the database created by Snyder et al. (1989) for the FHWA-sponsored study of concrete pavement rehabilitation. Another important source of existing data was the Long-Term Pavement Performance (LTPP) database. The LTPP SPS-6 sections (concrete pavement rehabilitation) provided the opportunity for direct, side-by-side comparison of the performance of diamond-ground pavement sections and other rehabilitation alternatives, including asphalt concrete (AC) overlays. The field survey for the current study included revisiting surviving sites from the 1989 FHWA study and additional sites in several states. In all, 60 sections at 54 sites were surveyed. In total, 193 pavement sections were available for data analysis.

The field survey consisted of a visual distress survey and faulting measurements. The macrotexture depth of diamond-ground surfaces was also measured at selected sites using the sand-patch method (ASTM E 965) to evaluate surface texture. Various analyses were conducted on the collected data to accomplish the study objectives, including the evaluation of service life, faulting performance, longevity of diamond-ground texture, and the effects of diamond grinding on slab cracking.

EFFECTS OF DIAMOND GRINDING

The immediate effect of diamond grinding is significant improvement in smoothness. The level of smoothness that can be achieved through diamond grinding is better than that of a new pavement. Another very important effect of diamond grinding is the significant increase in surface texture and consequent improvements in skid resistance and safety. Drakopoulos et al. (1998) showed that the accident rate of diamond-ground pavements is 58 percent of that of tined pavements (a 42-percent reduction) under both dry and wet conditions (Drakopoulos et al. 1998). The difference is less pronounced under snow and ice conditions, but the accident rate was 16 percent less on diamond-ground pavements.

Since slab thickness is one of the most sensitive factors affecting cracking performance of concrete pavements, any reduction can be a concern. However, structural analysis conducted for this study showed that the effects of the slight reduction in slab thickness are inconsequential when the long-term strength gain of concrete is considered. The fatigue analysis results showed that a typical concrete pavement may be ground up to three times without compromising its fatigue life. None of the pavement sections evaluated under this study exhibited an unusual level of slab cracking. The reduction in impact loads, resulting from the removal of roughness, may also be a significant factor contributing to the good cracking performance of diamond-ground pavements.

Field observations from this study did not indicate that diamond grinding causes an increased potential for durability problems. The excellent survival trends exhibited by diamond-ground pavements strongly suggest that any concern for increased risk of durability problems may be unwarranted. Survival analysis results show that the probability that a diamond-ground surface will last at least 10 years is 90 percent. This survival trend is consistent with the performance life based on faulting or surface texture, and no other cause of failure (such as material durability) is indicated.

PERFORMANCE OF DIAMOND-GROUND PAVEMENTS

The diamond-ground pavements evaluated under this study showed an excellent survival trend. The average age at overlaying or reconstruction was 29.1 years. The average age of the surviving sections in the database was 34.9 years. A survival analysis was conducted to quantify the effectiveness of diamond grinding in extending the service life of concrete pavements. The results showed that the probability that a diamond-ground pavement will have to be overlaid or reconstructed before the pavement reaches 30 years of age is less than 15 percent. The probability that the diamond-ground pavement will reach 40 years of age is 40 percent.

In terms of the longevity of the diamond-ground surfaces, the probability that a diamond-ground surface will last at least 8 years before requiring another rehabilitation (regrinding, overlay, or reconstruction) is 98 percent. The average service life of diamond-ground surfaces in the database was 11.4 years or 10.8 million ESALs. Thus, a diamond-ground surface may be expected to provide a minimum of 8 to 10 years of life with a high degree of reliability (90 to 98 percent). After the initial performance period, a diamond-ground pavement may be reground to further extend its service life.

Excessive faulting is perhaps the most common reason for grinding jointed concrete pavements. A mechanistic-empirical faulting model was developed for nondoweled jointed concrete pavements to evaluate faulting performance of diamond-ground pavements. The performance trends showed that there is a period of rapid redevelopment of faulting immediately following diamond grinding. The amount of faulting reaches 1.3 to 2.0 mm (0.05 to 0.08 in.) within 1 million ESALs; however, after this initial period, the rate of faulting drops to a much lower rate. The amount of precipitation is a significant factor affecting faulting performance. Pavements in wet climates develop faulting at a faster rate than those in dry climates. A 225-mm (9-in.) nondoweled jointed plain concrete pavement (JPCP) in a dry climate can sustain up to 20 million ESALs after grinding before the level of faulting becomes excessive. A similar pavement in a wet climate may require regrinding after 8 to 11 million ESALs.

Diamond grinding results in a significant increase in surface macrotexture and corresponding improvement in skid resistance. The improvement in skid resistance of a pavement immediately after diamond grinding is dramatic. Some studies have indicated that the improvement in skid numbers may be temporary, particularly if the pavement contains aggregate that is susceptible to polishing. Tyner (1981) observed that while the skid numbers will decrease over the first few years, an adequate macrotexture will normally be maintained for many years. Studies have shown that the longitudinal nature of diamond-ground texture may also be a significant factor contributing to the safety of diamond-ground surfaces (Horn; Drakopoulos et al. 1998).

The longevity of diamond-ground surface texture was evaluated using the macrotexture depths measured by sand-patch method (ASTM E 965) from in-service diamond-ground sections. The analysis showed that the texture life is most strongly correlated to the age since grinding. The climate (freeze vs. non-freeze) was also a significant factor. Surprisingly, however, the macrotexture life was not strongly correlated to traffic or aggregate hardness. It is possible that the effects of traffic are confounded with age, which is a surrogate for many parameters. The effects of aggregate hardness may have been lost because closer blade spacing is used on harder aggregates, resulting in smaller land area that would wear down faster. The analysis results showed that on average, the diamond-ground texture lasts about 8 years in the freeze region and about 12 years in the non-freeze region.

CONCLUSIONS

The diamond-ground pavements evaluated under this study showed excellent performance. The results of this study show that CPR with diamond grinding is an effective means of extending service life of concrete pavements. The key findings of this study include the following:

- The immediate effect of diamond grinding is a smooth pavement surface with the desirable surface texture. The level of smoothness that can be achieved through diamond grinding is often better than that of a new pavement.
- Diamond grinding results in significant increase in surface texture and corresponding improvements in skid resistance. Studies have shown that diamond-ground surface texture can lead to significant improvement in safety, in terms of reduced accident rates.
- The diamond-ground texture lasts about 8 years in the freeze region and about 12 years in the non-freeze region.
- Faulting redevelops at a relatively high rate initially on nondoweled diamond-ground pavements, but the rate of faulting levels off to a much lower rate after about 2 million ESALs.
- The amount of precipitation is a significant factor affecting faulting performance. In wet climates, faulting on diamond-ground pavements (nondoweled) becomes excessive after 8 to 11 million ESALs. A similar pavement in a dry climate may sustain up to 20 million ESALs before reaching the same level of faulting.
- The average service life of diamond ground pavements based on survival analysis is 37 years or 35 million ESALs since initial construction. Some of these pavements were diamond-ground two or more times, and many sections in the database have survived 40 or more years.
- The survival analysis results showed that the expected life of a diamond-ground surface (the time from grinding to regrinding, overlaying, or reconstruction) at different levels of reliability are as follows:
  - 50-percent reliability: 13.5 years or 12 million ESALs.
  - 75-percent reliability: 11 years or 8 million ESALs.
  - 85-percent reliability: 10.5 years or 7 million ESALs.
  - 90-percent reliability: 9.5 years or 6.5 million ESALs.
- Based on surface texture life, faulting performance, and survival trends, a diamond-ground surface may be expected to provide a minimum of 8 to 10 years of service with a high degree of reliability, depending on climatic conditions and traffic. At this time, the pavement may be reground to provide further extension to service life.
- Dowel bar retrofitting can be a highly effective method of extending service life of nondoweled PCC pavements with high traffic (more than 1,000,000 ESALs/year), especially for pavements in wet climates.
- A concrete pavement may be ground up to three times without significantly compromising its fatigue life.
- No evidence of any deleterious effects of diamond grinding was observed at any of the field sites.

The long-term effectiveness of a diamond-ground pavement depends on numerous factors, but the most significant factors are the condition of the existing pavement structure and level of CPR applied. It is important to recognize that diamond grinding addresses serviceability problems. If the existing pavement is structurally deficient, an overlay or reconstruction may be more appropriate. Pavements with a material problem such as D-cracking or reactive aggregate are also not good candidates for diamond grinding. Inappropriate use of diamond grinding is likely to lead to premature failures. However, even for pavements in poor condition, it may be appropriate to consider diamond grinding as an economical short-term (5 years) solution to a roughness problem until the pavement section can be overlaid or reconstructed.

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List of Abbreviations and Symbols

AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Concrete
ACPA	American Concrete Pavement Association
ADT	Average Daily Traffic
BPN	British Pendulum Number
CPR	Concrete Pavement Restoration
FHWA	Federal Highway Administration
GDOT	Georgia Department of Transportation
IGGA	International Grooving and Grinding Association
IRI	International Roughness Index
JCP	Jointed Concrete Pavement
JPCP	Jointed Plain Concrete Pavement
LTE	Load Transfer Efficiency
LTPP	Long-Term Pavement Performance
NCHRP	National Cooperative Highway Research Program
PCA	Portland Cement Association
PCC	Portland Cement Concrete
PSI	Present Serviceability Index
PSR	Present Serviceability Rating

Introduction

Diamond grinding is a concrete pavement restoration (CPR) technique that can be used to correct a variety of surface problems. Diamond grinding provides a smooth riding surface with the desirable friction and noise characteristics. The concrete pavement problems that can be addressed using diamond grinding include the following:

- Faulting at joints and cracks.
- Built-in or construction roughness.
- Polished concrete surface exhibiting inadequate macrotexture.
- Wheelpath rutting caused by studded tires.
- Unacceptable noise level.
- Permanent upward slab warping caused by moisture gradient and construction curling.
- Inadequate transverse slope.

The most common reason for diamond grinding has been to remove roughness caused by excessive faulting. Diamond grinding has also been used frequently in conjunction with other CPR techniques such as dowel bar retrofitting, full-depth repairs, partial-depth repairs, retrofit edgedrains, and slab stabilization as part of a comprehensive pavement restoration program.

Diamond grinding involves removing a thin layer at the surface of hardened portland cement concrete (PCC) using closely spaced diamond saw blades. The level surface is achieved by running the blade assembly at a predetermined level across the pavement surface, which produces saw-cut grooves. The uncut concrete between the saw cuts breaks off more or less at a constant level above the saw-cut grooves, leaving a level surface (at a macroscopic level) with longitudinal texture. Diamond grinding results in significant improvement in friction characteristics, and the longitudinal texture produced by diamond grinding has been shown to be highly effective in reducing accident rates in problem areas (Drakopoulos et al. 1998).

The immediate effect of grinding is usually a very smooth pavement surface (Mosher 1985). The ride quality after grinding is generally better than what can be achieved during new construction. However, if the cause of faulting is not treated prior to grinding, the faulting problem is likely to reoccur (ACPA 1990).

Not all concrete pavements that have developed excessive roughness are good candidates for diamond grinding, and grinding alone may not be enough to address the existing problems. Structural distresses, such as pumping, loss of support, corner breaks, working transverse cracks, and shattered slabs, require repair before grinding is conducted (Roman et al. 1985). Widespread material problems such as D-cracking, reactive aggregate, or freeze-thaw damage indicate that diamond grinding may not be a suitable restoration technique on those pavements and that a more comprehensive rehabilitation approach needs to be considered.

Diamond grinding offers numerous advantages over other rehabilitation alternatives, including the following:

- Costs substantially less than an overlay (Pierce 1995; McGovern 1995).
- Enhances surface friction and safety of an old pavement surface.
- Can be accomplished during off-peak hours with short lane closures and without having to close the adjacent lanes.
- Eliminates the need for taper, which is required with the overlay alternative at highway entrances, exits, and at side streets.
- Does not affect overhead clearances underneath bridges or hydraulic capacities of curbs and gutters on municipal streets.
- Grinding of one lane does not require grinding of the adjacent lane, which may have perfectly acceptable surface characteristics.

For concrete pavements where roughness problems predominate, diamond grinding can be a highly effective and economical rehabilitation alternative, especially if the pavement is in good structural condition. It may also be appropriate to consider diamond grinding as an economical short-term solution for pavements with structural problems or low levels of material durability problems to address roughness until a more comprehensive rehabilitation can be applied.

BACKGROUND

Diamond grinding was first used on a new concrete taxiway at Davis Monthan Air Base in Tucson, Arizona, in 1956 to correct localized profile problems. The history of continuous diamond grinding for pavement restoration dates back to 1965, when the technique was first used on a 19-year-old section of the San Bernardino Freeway (I-10) in California to eliminate excessive faulting (Neal and Woodstrom 1976). This section of I-10 had considerable spalling and faulted joints. Highway engineers were concerned that continuous pounding of heavy traffic would ultimately lead to total destruction of this otherwise structurally sound pavement. To improve smoothness, more than 93,000 m^2 (1,000,000 sq ft) of this pavement was ground on an experimental basis. In 1983, this same pavement section was reground as part of a CPR program to restore rideability and friction resistance. In 1997, this pavement was reground for the third time. This highway now has a two-way average daily traffic (ADT) of 158,000 with 14 percent trucks, which corresponds to 2.25 million equivalent single axle loads (ESALs) per year on the truck lane.

Since this beginning, diamond grinding has become a major element of PCC restoration; however, very little valid documentation of the performance of diamond-ground pavements exists. Various aspects of the performance of diamond-ground pavements are in question. A previous study of diamond-ground pavements showed that faulting after diamond grinding tends to develop at a faster rate than it did on the original pavement. The effects of diamond grinding on friction resistance of concrete pavements, as well as the longevity of the diamond-ground texture, are not well documented. Recognizing the need for such documentation, the Portland Cement Association (PCA), in association with American Concrete Pavement Association (ACPA) and International Grooving and Grinding Association (IGGA), sponsored a study to evaluate the performance of diamond-ground pavements.

The objectives of this study were to evaluate and document the performance of diamond-ground pavements and determine the effectiveness of diamond grinding as a CPR technique. The study was designed to provide answers to the questions frequently asked by highway agencies that are considering diamond grinding as a possible CPR option on a project. The key questions addressed under this project include the following:

- What is the immediate impact of diamond grinding on pavement performance?
- How long can diamond-ground surfaces provide acceptable riding quality?
- How long can diamond-ground surfaces provide acceptable surface texture?
- When is diamond grinding feasible and effective?
- Can diamond grinding be used more than once on a pavement?
- Are there any adverse effects of diamond grinding?

STUDY OF DIAMOND-GROUND PAVEMENTS

This study involved a comprehensive review of existing information on diamond grinding, data collection, data analysis, and documentation of the study findings. The performance data needed for the evaluation were obtained from existing databases and by conducting field surveys. The main source of existing data was the database created for the 1986 study of

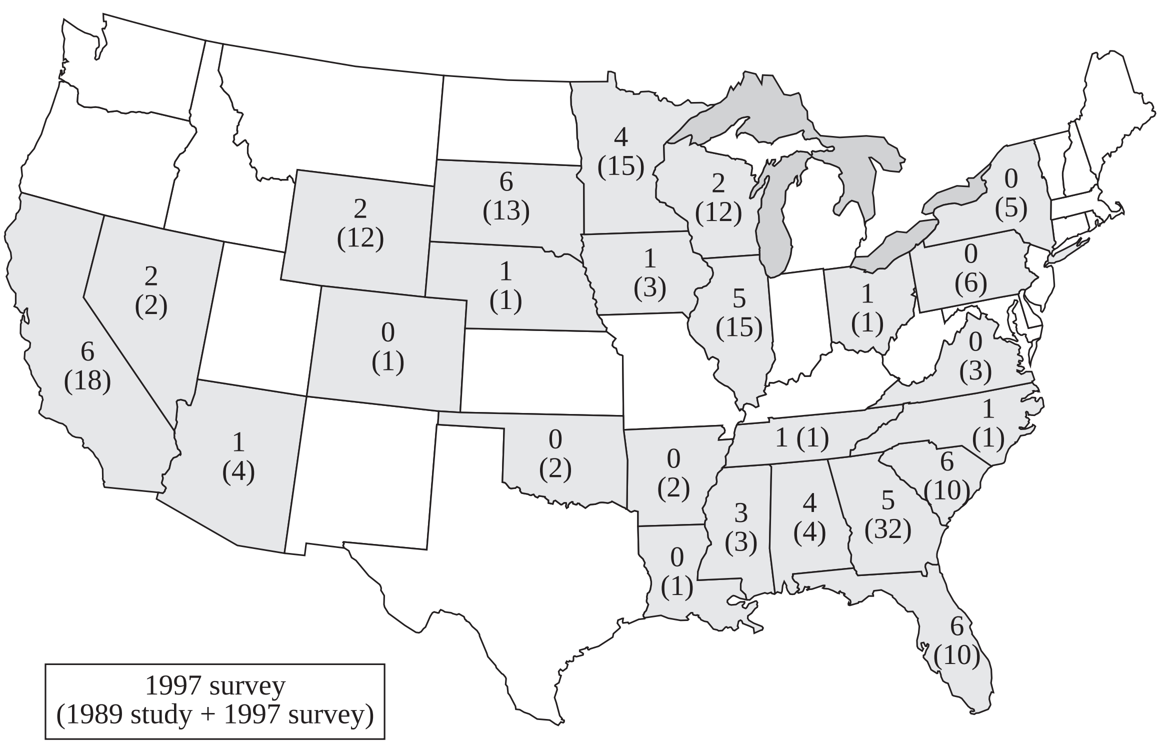

Figure 1. Geographical distribution of the pavement sections included in the study.

concrete pavement rehabilitation sponsored by the Federal Highway Administration (FHWA) (Snyder et al. 1989). The field survey for that project was performed in 1985 and 1986. The database contains 133 diamond-ground pavement sections at 76 sites. The number of sections is greater than the number of sites because replicate sections were surveyed at many of the sites.

Of the 133 sections in the database, 12 sections in Ohio were excluded from the present study because they had severe D-cracking, and one section in California was excluded because of insufficient information. In all, 120 sections at 69 sites from the FHWA database were used in the current analysis.

Another important source of existing data for diamond-ground pavements was the Long-Term Pavement Performance (LTPP) database. The LTPP SPS-6 sections (Specific Pavement Studies, concrete pavement rehabilitation) provide the opportunities for direct, side-by-side comparison of the performance of diamond-ground pavement sections and other rehabilitation alternatives, including asphalt concrete (AC) overlays.

The field survey for the current study focused on revisiting sites from the 1986 FHWA study. At the time of survey in fall 1997, 38 of the original 76 sites were surviving. The field survey also included additional sites located in several States. In all, 60 sections at 54 sites were surveyed; however, 3 of these sections could not be used because of insufficient information. In total, 177 sections were available for analysis. Figure 1 shows the geographical distribution of the pavement sections included in this study.

The field survey consisted of a visual distress survey and taking faulting measurements. The macrotexture depth of diamond-ground surfaces was also measured at selected sites for the evaluation of surface texture using the “sand-patch” method (ASTM E965). Various analyses were conducted on the collected data to accomplish the study objectives, including evaluation of service life, faulting performance, longevity of diamond-ground texture, and the effects of diamond grinding on slab cracking.

ABOUT THIS REPORT

This report was developed as a comprehensive guideline on diamond grinding, presenting both reference information and guidelines for implementation. The first three chapters of the report present the reference information, covering the results of the literature review and findings from this study. The reference information is organized under three headings: *Effects of Diamond Grinding*, *Performance of Diamond-Ground Pavements*, and *Case Studies*. The fourth chapter of the report presents the guidelines for implementation. The report also includes two appendices:

- Appendix A: Development of Faulting Model—A predictive model for faulting performance of diamond-ground, nondoweled jointed concrete pavement (JCP) was developed under this study. Appendix A presents the details of the development of this model.
- Appendix B: Summary Tables—The pavement design and performance data collected under this study are summarized in Appendix B.

CHAPTER 1
Effects of Diamond Grinding

Diamond grinding has both immediate and long-term effects on concrete pavement performance. The most direct and immediate result is a smooth pavement. The removal of a thin layer (typically 4 to 6 mm [0.10 to 0.25 in.]) from the concrete surface and the texture left by diamond grinding also affect various other aspects of concrete pavement performance, including safety, noise, structural capacity, and material durability. This section presents the direct and indirect effects of diamond grinding.

SMOOTHNESS

Excessive roughness is the most common reason for diamond grinding, and field data show that smoothness equal to, or better than, a new pavement can be achieved through diamond grinding. Diamond grinding removes faulting and surface irregularities and restores a smooth riding surface with desirable surface texture on concrete pavements that have developed excessive roughness. The significant improvement in ride quality provided by diamond grinding is demonstrated in Table 1, which lists the roughness measurements before and after grinding from several different projects.

The measured roughness of diamond-ground pavements shortly after grinding is often better than that of a new pavement. Note that the roughness after grinding is not directly correlated to the roughness before grinding. Numerous factors affect the roughness of JCP, including the initial roughness, number and severity of cracks, joint spacing, joint width, and settlements or heaves, but the most dominant factor is the amount of joint faulting. After diamond grinding, a pavement section with more joints or cracks will exhibit slightly more roughness than the sections with fewer discontinuities, but prior to grinding, the amount of faulting can completely overshadow any other

Table 1. Measured Roughness Before and After Grinding Using a Mays Ridemeter (Mosher 1985)

Location	Roughness, m/km (in./mi)		
	Before Grinding	After Grinding	Percent Decrease
I-65 Culman, AL, MP 308-316	1.86 (118)	0.92 (58)	51
I-17 Phoenix, AZ, MP 202-213	1.99 (126)	0.58 (37)	71
I-85 Atlanta, GA, MP 56-68	1.55 (98)	0.63 (40)	59
I-84 Newburg, NY, MP 42-46	2.35 (149)	0.60 (38)	74
I-90 Rapid City, SD, MP 50-66	1.45 (92)	0.58 (37)	60
Average of five projects	1.85 (117)	0.66 (42)	64

factors. The equipment and quality control during grinding can also significantly affect the level of smoothness achieved immediately after grinding.

The rate at which roughness redevelops on diamond-ground pavements depends on numerous factors, including pavement design and condition, the level of CPR applied prior to diamond grinding, the traffic level, and climatic conditions. The survival analysis results and the faulting model indicate that we can expect a diamond-ground surface to provide acceptable serviceability for up to 10 or more years. The details of long-term performance of diamond-ground pavements are presented in the next chapter.

SURFACE TEXTURE AND SAFETY

Another very important effect of diamond grinding is the significant increase in surface texture and consequent improvements in skid resistance and safety. Table 2 lists several grinding projects that demonstrate the effects of diamond grinding on skid resistance immediately after grinding.

The removal of hardened PCC using closely spaced diamond saw blades leaves a longitudinal groove-like texture on the pavement surface, which increases surface macrotexture. The increased macrotexture, in turn, improves friction characteristics of the pavement by providing for improved drainage of water at the tire–pavement interface. The effects are more pronounced for worn or smooth tires. Friction tests conducted using ribbed tires do not show the same sensitivity to macrotexture as those conducted using smooth tires. The grooves in the ribbed tire provide for effective drainage of water; thus, the tests conducted using smooth tires are more meaningful for correlating test results and accident rates.

Studies of longitudinally grooved pavements suggest that the direction of the texture may also have very significant effects on safety (Horne). Because of the longitudinal direction of the diamond-ground texture, diamond grinding may also be expected to have similar effects on safety. Other studies (Farnsworth 1971; Walters 1979; Drakopoulos et al. 1998) confirm the significant benefits of longitudinal grooving and diamond grinding in reducing accident rates, although the specific mechanism responsible for this effect was not investigated under these studies. A brief summary of the findings from these studies is presented in the following section.

Longitudinally Grooved Pavements

California. Farnsworth (1971) reported the accident rates before and after grooving at several locations in California, including those at 14 pavement sections in the Los Angeles area. During the 1- to 3-year period before longitudinal diamond grooving, there were an average of 16.8 dry-pavement accidents per year and 20.6 wet-pavement accidents per year at those sites. After grinding, the accident rates dropped to an average of 13.9 dry-pavement accidents and 1.9 wet-pavement accidents per year during the 1- to 7-year monitoring period. Thus, longitudinal grooving had a modest effect (17 percent drop) on the dry-pavement accident rate, but it had a drastic effect (91 percent drop) in reducing wet-pavement accidents. It must be noted that these pavement sections were grooved because they exhibited high accident rates. Such a dramatic improvement in accident rates may not occur on pavements that have lower initial accident rates.

Louisiana. Walters (1979) investigated the effects of longitudinal grooving on accident rates on an experimental section in Louisiana. In 1970, a section of I-10 / I-12 in Baton Rouge had 76 dry- and 31 wet-pavement accidents per 100 million vehicle-km (123 dry- and 50 wet-pavement accidents per 100 million vehicle-miles). The ADT was 21,700 vehicles per day. The pavement section was longitudinally diamond-grooved in 1973.

Table 2. Measured Friction Number Before and After Diamond Grinding Using a Saab Friction Tester With a Smooth Tire (ASTM E 524) (Mosher 1985)

Location	Friction Number		Percent Increase
	Before Grinding	After Grinding	
I-65 Culman, AL, MP 308-316	54	69	28
I-17 Phoenix, AZ, MP 202-213	42	64	52
I-85 Atlanta, GA, MP 56-68	45	95	111
I-84 Newburg, NY, MP 42-46	40	85	113
I-90 Rapid City, SD, MP 50-66	30	85	183
Average of five projects	42	80	90

During the first half of the study (1973 to 1975), the accident rate was down to 41 dry- and 4 wet-pavement accidents per 100 million vehicle-km (66 dry- and 7 wet-pavement accidents per 100 million vehicle-miles). This represents a 46 percent reduction in dry-pavement accidents and an 86 percent reduction in wet-pavement accidents. During the second half of the study (1976 to 1978), the ADT had jumped up to an average of 32,800 vehicles per day, primarily due to the opening of I-10, south of the subject section. The dry- and wet-pavement accident rates were now 53 and 11 per 100 million vehicle-km (85 and 18 per 100 million vehicle-miles), corresponding to 31 and 64 percent reductions in dry- and wet-pavement accident rates, respectively. This increase in accident rates may be related to the significant increase in traffic volume.

During the 5-year period following longitudinal grooving, the I-10 / I-12 section exhibited significantly lower accident rates (compared to pre-grooving) despite the increase in traffic volume. Again, as in the case of the California study, the accident rate on this section was relatively high prior to grooving.

Diamond-Ground Pavements

Drakopoulos et al. (1998) studied the effects of diamond grinding on accident rates on concrete pavements in Wisconsin over a 6-year period. The study included 30 diamond-ground pavement sections totaling 290 km (180 mi) and 21 tined pavement sections totaling 115 km (71 mi). Table 3 shows the accident rates for different pavement conditions for the ground and tined pavements evaluated.

The overall accident rates at diamond-ground sites were 60 percent of those at tined sites. For all types of pavement, diamond-ground pavements had significantly lower accident rates than tined pavements under all conditions. Figures 2 and 3 show all condition accident rates and wet-pavement accident rates for the 6-year period. No consistent trend in the deterioration of accident rates with time was observed. In contrast, the surface macrotexture of diamond-ground pavements changes considerably with time since grinding. Pavement macrotexture increases significantly immediately after grinding, along with a corresponding increase in skid numbers. During the first 2 years after grinding, the macrotexture wears off rapidly, but the rate of deterioration levels off after the rapid initial loss. A significant diamond-ground texture lasts approximately 8 years for pavements in freeze areas and 12 years for pavements in non-freeze areas based on results from this study, as measured by the sand-patch method (ASTM E965).

The fact that the accident rates on diamond-ground pavements do not correspond directly to macrotexture depth suggests that at least two factors contribute to the accident rate performance of diamond-ground surfaces: the macrotexture depth and the type of surface texture. The increased macrotexture depth provides for improved drainage of water at the tire–pavement interface, especially for worn tires. The longitudinal texture of grinding may also play a significant role in providing directional stability and reducing hydroplaning, neither of which can be quantified using skid number or macrotexture measurements. Reductions in accident rates on diamond-ground and longitudinally grooved pavements are caused not only by the increase in surface texture, but also by the longitudinal direction of the texture.

STRUCTURAL EFFECTS

Diamond grinding reduces slab thickness by approximately 4 to 6 mm (0.10 to 0.25 in.). Since slab thickness is one of the most sensitive factors affecting cracking performance of concrete pavements, any reduction can be a concern. The effects of diamond grinding on cracking performance of concrete pavements were evaluated based on both analytical results and field observations.

For the analytical evaluation, a fatigue analysis was conducted to examine the sensitivity of fatigue life to slab thickness and concrete strength. The results

Table 3. Accident Rates For Different Pavement Conditions (Drakopoulos et al. 1998)

Pavement Conditions	Accident Rate (accidents per 100 million vehicle-km [100 million vehicle-miles])		Accident Rate Reduction on Diamond-Ground Pavements
	Ground	Tined	
Dry	65 (105)	112 (180)	42%
Wet	99 (159)	170 (274)	42%
Snow/Ice	173 (278)	205 (330)	16%

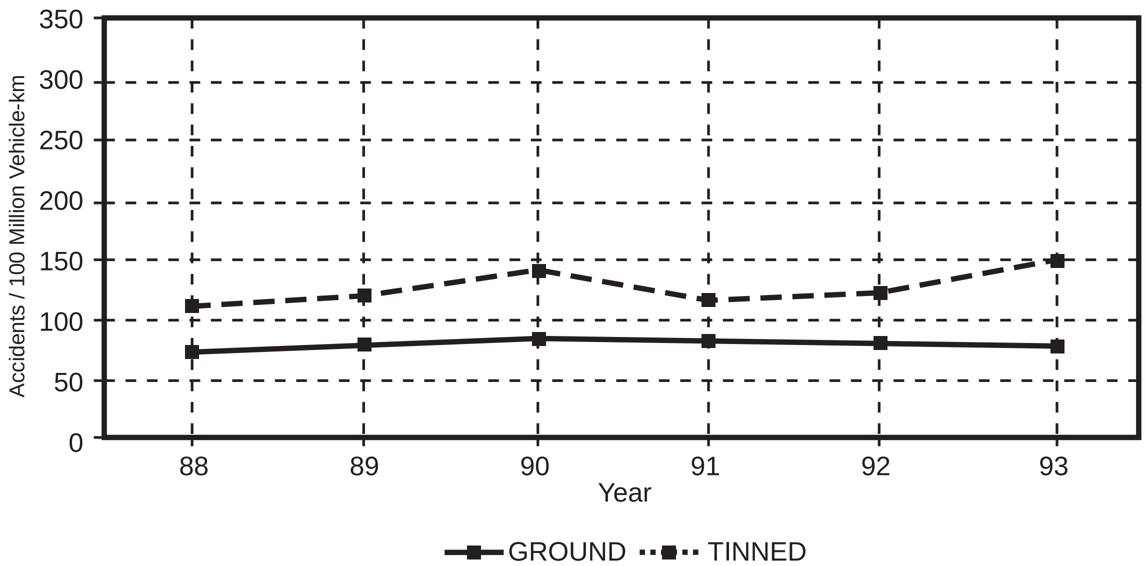

Figure 2. Accident rates between 1988 and 1993 for diamond-ground and tined pavements (Drakopoulos et al. 1998)

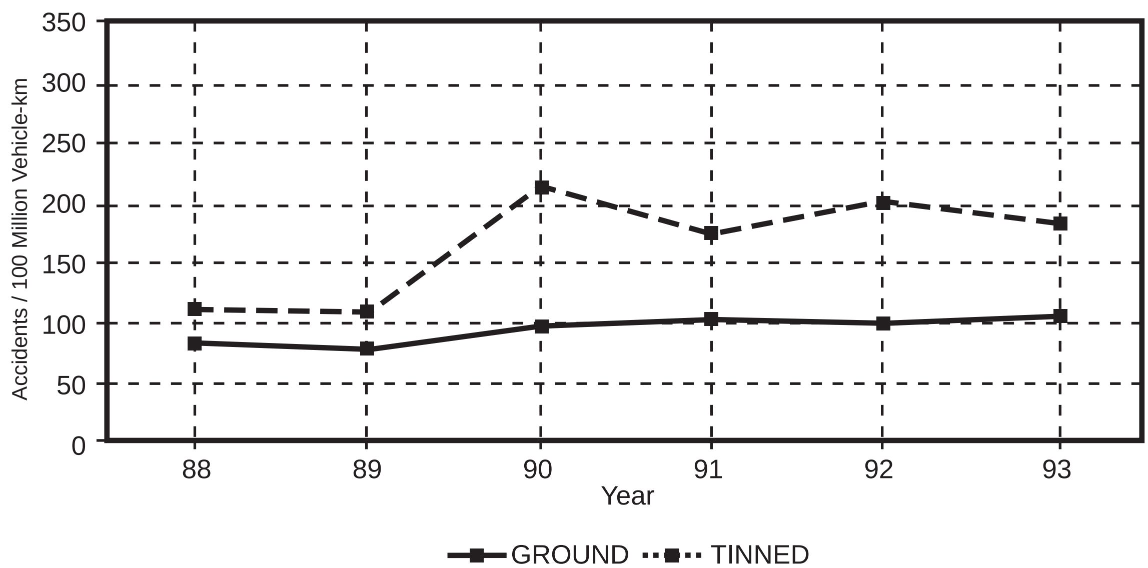

Figure 3. Wet pavement accident rates between 1988 and 1993 for diamond-ground and tined pavements (Drakopoulos et al. 1998)

are shown in figure 4. The predicted fatigue life was determined using the jointed plain concrete pavement (JPCP) cracking model developed by Yu et al. (1997). Fatigue life of a pavement slab is extremely sensitive to slab thickness. A 5-mm (0.2-in.) reduction in slab thickness results in about 30 percent reduction in fatigue life if the concrete strength remains constant. However, long-term strength of concrete is significantly higher than the design strength, which is typically the 28-day strength. The strength of conventional concrete (non-fast-track) after 1 year can be up to 20 percent higher than the 28-day strength (Mindess and Young 1981). If the increase in concrete strength is considered, the small reduction in slab thickness has negligible effect on service life.

Figure 4 shows that with 10 percent increase in concrete strength, up to 13 mm (0.5 in.) of thickness can be removed and still achieve the design life predicted based on design strength. If the long-term strength is 15 percent higher than the design strength, up to 18 mm (0.7 in.) may be removed without compromising the projected performance. These results suggest that a typical concrete pavement may be ground up to three times without compromising its fatigue life.

These predictions based on analytical evaluation are consistent with the field observations presented in the next chapter. None of the pavement sections evaluated under this study exhibited an unusual level of slab cracking. This result is supported by the fact that several concrete pavements have been ground three times and are still performing well after 20 years. The reduction in impact loads, resulting from the removal of roughness, may also be a significant factor contributing to the good cracking performance of diamond-ground pavements.

REDUCTION IN NOISE LEVEL

The noise level on a concrete pavement increases with increasing roughness. Roughness caused by abrasion (e.g., “rutting” caused by studded tires or chains) results in increased tire–pavement interaction noise. Pavement roughness caused by faulted joints and cracks add to the noise generated by the tire–pavement interaction. Transversely tined pavements can also produce a whistling and whining sound (discrete frequencies) that can be annoying to the driver and nearby residents. A multi-State study on noise and texture on PCC pavements, sponsored by Wisconsin DOT and the FHWA (Marquette University 1998), concluded that longitudinally textured concrete pavements are among the quietest pavements for interior and exterior noise.

Diamond grinding retextures worn surfaces with a longitudinal texture and provides a quieter surface. Diamond grinding also removes faults by leveling the pavement surface, thus eliminating the thumping and slapping sound created by faulted joints. Noise level measurements on highways in Belgium indicate a 5 to 8 dbA reduction in pavement noise after diamond grinding. Although the actual drop in total noise resulting from grinding is relatively minor (only a few decibels), there is a considerable difference in the frequency of noise produced, resulting in a more pleasant ride.

Michigan DOT (DeFrain 1989) measured noise generated from a road surface before and after a tined pavement had been diamond ground. The noise level measured inside a vehicle showed that the overall noise level is reduced only slightly after grinding. However, there was a 5dbA reduction in noise at the peak frequencies of 500 Hz and the first harmonic of 1000 Hz. Grinding of PCC pavements reduces noise peaks or spikes in the noise spectrum, thus providing significant reduction in objectionable noise.

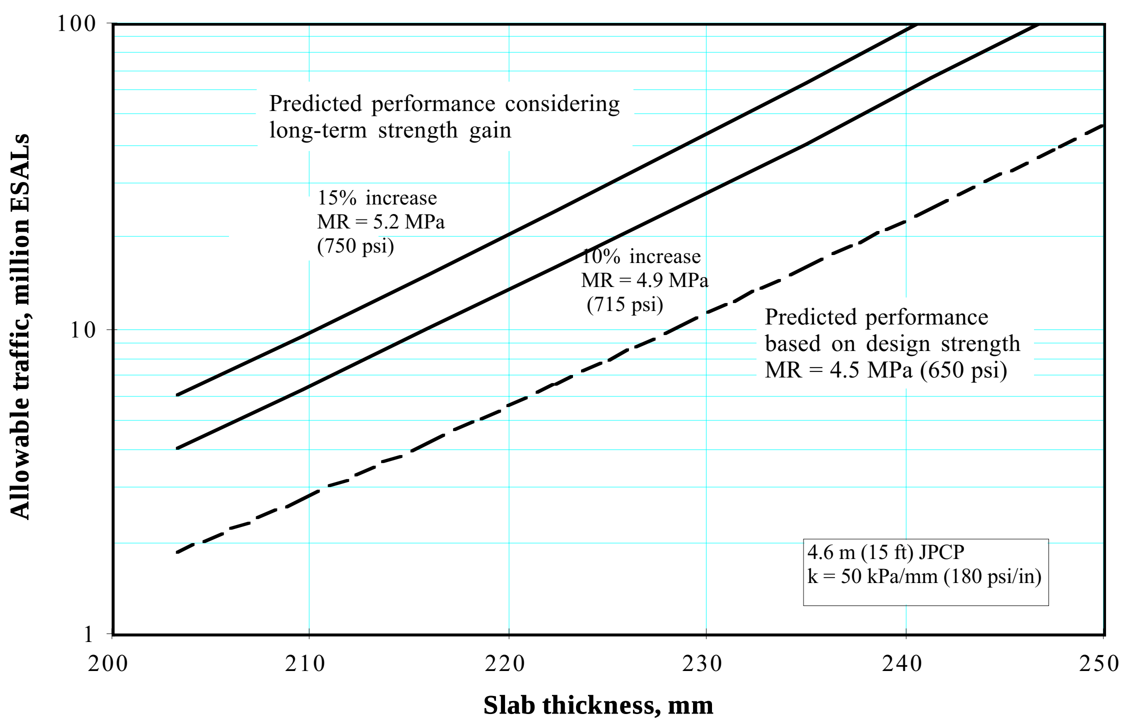

Figure 4. Effects of slab thickness and concrete strength on fatigue life of JPCP.

MATERIAL DURABILITY

Diamond grinding was not found to introduce any unusual conditions that would lead to poorer surface durability. The fact that the ground surface is nearly always dry (except during storms) reduces any freeze-thaw problems. A laboratory study conducted at PCA (Perenchio 1964) indicated that there might be an increased potential for durability problems for concrete made with nondurable aggregate when large areas of coarse aggregates are exposed by grinding or sawing. Although the durability of concrete made with durable aggregate was unaffected, concrete made with nondurable aggregate exhibited more scaling compared to the uncut control specimens when subject to immersion. The loss of the protective cover provided by the air-entrained mortar was thought to be the cause for the increased scaling. The study concluded, however, that removal of thin layers from the concrete surface does not affect the durability. The durability was only affected when the depth of removal was sufficient to expose large areas of susceptible aggregate and the exposed areas were immersed in water.

Field observations from the diamond grinding study do not indicate that diamond grinding causes an increased potential for durability problems. The excellent survival trends exhibited by diamond-ground pavements strongly suggest that any concern for increased risk of durability problems is most likely unwarranted. Durability problems on fresh cut surfaces of concrete containing susceptible aggregate can develop in about 5 years (Janssen and Snyder 1994). Survival analysis results (see the section entitled *Service Life* in the next chapter) show that the probability that a diamond-ground surface will last at least 10 years is almost 90 percent. At about 10 to 15 years of age, depending on the level of traffic, faulting on diamond-ground sections reaches a point where another cycle of rehabilitation is needed. Thus, the survival trend is consistent with the performance life based on faulting or surface texture, and no other cause of failure (such as material durability) is indicated.

SERVICE LIFE EXTENSION

One important benefit of the significant reduction in pavement roughness provided by diamond grinding is service life extension. Pavements exhibiting excessive roughness require corrective measures to provide a safe riding surface with acceptable ride quality. For concrete pavements that have developed excessive roughness, diamond grinding can be an effective means of restoring smoothness and extending service life, especially if the pavement is structurally sound. The survival analysis results show that a diamond-ground surface can provide 8 to 10 years of service or more. A diamond-ground pavement can also be reground to further extend the service life. Detailed information on these effects is presented in the following chapter.

COST CONSIDERATIONS

Diamond grinding can be a highly cost-effective means of rehabilitating concrete pavements that have developed surface problems. Grinding can be accomplished during off-peak hours with short lane closures, and grinding of one lane does not require grinding of the adjacent lane, which may have perfectly acceptable surface characteristics. Georgia DOT has used CPR with grinding for more than 20 years, and they find CPR with grinding to be 3 to 4 times as cost-effective as a 150-mm (6-in.) AC overlay. CPR projects in Georgia typically last between 7 and 10 years, and some have performed for more than 17 years before a second CPR (Gulden 1996).

State of Washington conducted a cost comparison of retrofit dowel bars with grinding, 1.2-m (4-ft) tied shoulders with grinding, and 110-mm (4-in.) AC overlay (Pierce 1995). The estimated costs of the rehabilitation alternatives, based on material costs, are as follows:

- Retrofit dowel bars with diamond grinding (truck lane only): $73,000 per lane-km ($117,450 per lane-mile).
- Tied PCC shoulders with diamond grinding in the truck lane: $69,100 per lane-km ($111,200 per lane-mile).
- AC overlay: $118,300 per lane-km ($190,300 per lane-mile).

The comparison was based on the rehabilitation of two 3.7-m (12-ft) lanes and 4.3 m (14 ft) of total shoulder width for the 53.1-km (33-mile) project. The retrofit dowel bar and grinding option can be applied on truck lane only, whereas the AC overlay has to be placed on both lanes. Therefore, on a four-lane highway, dowel bar retrofitting with grinding can be up to three times more cost-effective than an AC overlay.

CPR with grinding was performed on 11.3 km (7 miles) of I-26 in North Carolina. The project cost $3.5 million, or about $311,000 per four-lane km ($500,000 per four-lane mile) and took 1 year to complete. A comparable 4.2-km (2.6-mile) AC overlay project cost $3.9 million, or about $930,000 per four-lane km ($1.5 million per four-lane mile). Again the CPR with grinding option was three times as cost-effective as an AC overlay (McGovern 1995).

CHAPTER 2

Performance of Diamond-Ground Pavements

SERVICE LIFE

The 76 individual projects included in the 1986 FHWA study were used to evaluate the effects of diamond grinding on pavement service life. Because replicate sections are subjected to the same rehabilitation treatments, they could not be considered independently for this analysis. Of the 76 projects included in the 1986 FHWA study, 38 (50 percent) were surviving (had not been overlaid or reconstructed) at the time of the 1997 field surveys for the current study.

PAVEMENT AGE OR TRAFFIC SINCE INITIAL CONSTRUCTION

The age distributions of overlaid (or reconstructed) and surviving projects are shown in Figure 5. Diamond-ground pavements have shown good long-term performance:

- Only 14 percent of the projects were overlaid at an age less than 25 years.
- 59 percent of the projects lasted 30 years or more, and 60 percent of the projects that have lasted 30 years or more are still surviving.
- 37 percent of the projects have lasted 35 years or more (68 percent of these projects are still surviving).
- 11 percent of the projects have lasted more than 40 years and all of them are still surviving.
- The average age of the overlaid or reconstructed projects was 29.1 years.
- The average age of the surviving projects was 34.9 years.
- The average age for all 76 projects was 32.0 years. Therefore, the average age at overlay (or reconstruction) for all 76 projects will be more than 32.0 years because of the remaining life of the 38 projects surviving in 1997.

The distribution of traffic placed on these projects is shown in Figure 6. Again, good long-term performance is observed.

- 53 percent of the sections have carried more than 20 million ESALs since initial construction.
- 50 percent (20 of 40) of the projects with cumulative traffic greater than 20 million ESALs were still surviving in 1997.
- 24 percent of the projects have carried more than 30 million ESALs.
- The average cumulative traffic on the overlaid or reconstructed projects was 25.1 million ESALs.
- The surviving projects have carried an average cumulative traffic of 20.6 million ESALs through 1997.

A survival analysis was conducted to quantify the effectiveness of diamond grinding in extending service life of concrete pavements. Figures 7 and 8 show survival curves based on age and cumulative 80-kN (18-kip) ESALs since construction. These curves were generated using SAS statistical software. Three California projects whose reconstruction year was not verifiable and the six Ohio projects with D-cracking, spalling, and cracking problems that were not addressed prior to diamond grinding were not included in the survival analysis.

Figure 7 shows that the probability that a diamond-ground pavement will have to be overlaid or reconstructed by age 30 years is less than 15 percent.

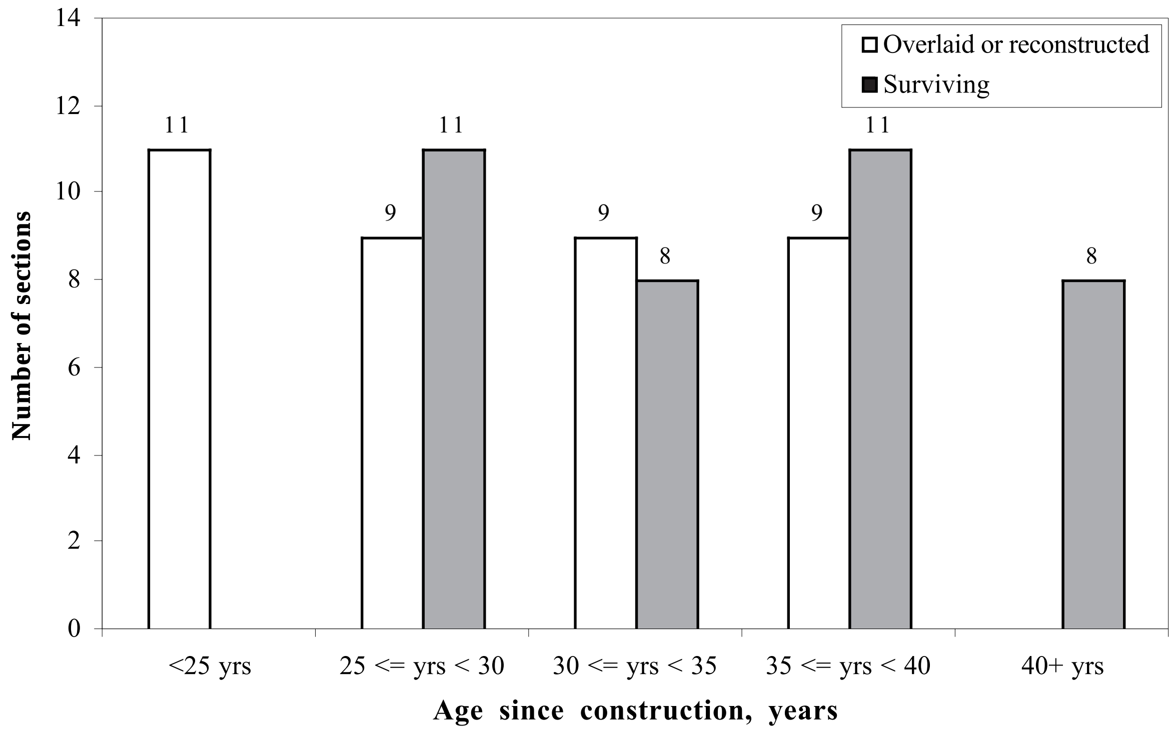

Figure 5. Age distribution of diamond-ground pavement projects for service life evaluation.

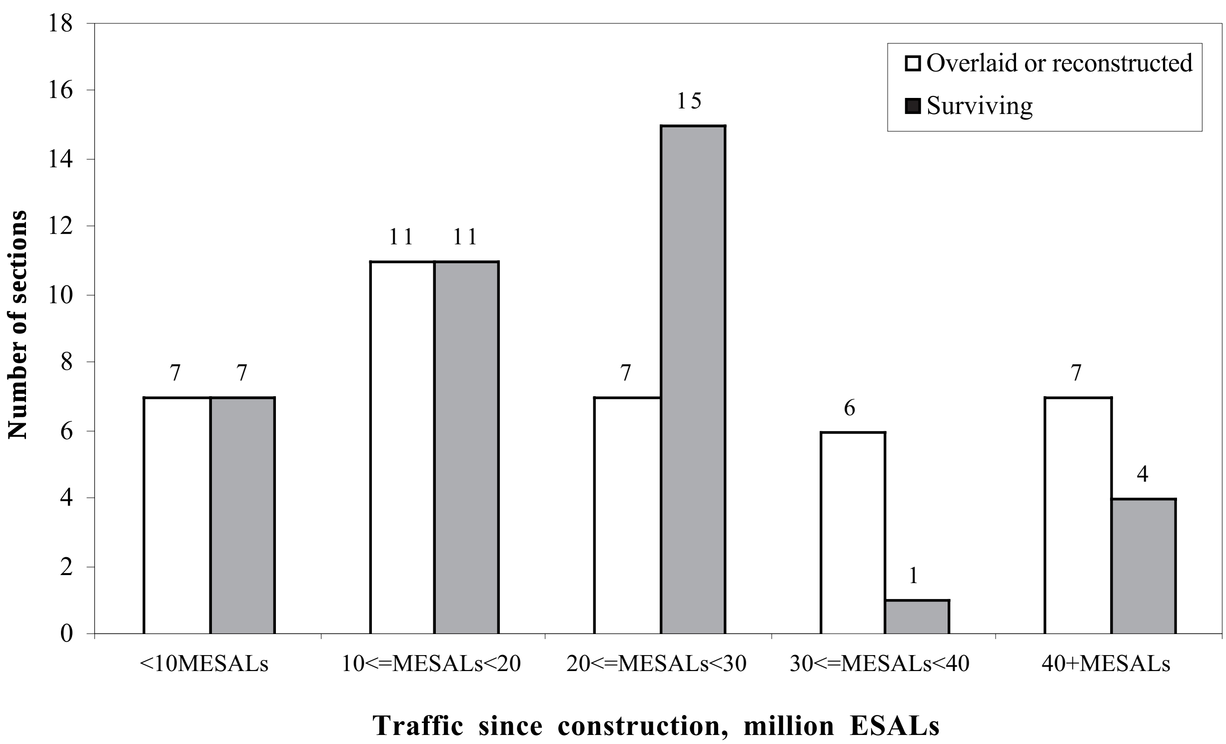

Figure 6. Distribution of traffic levels of the pavement projects for service life evaluation.

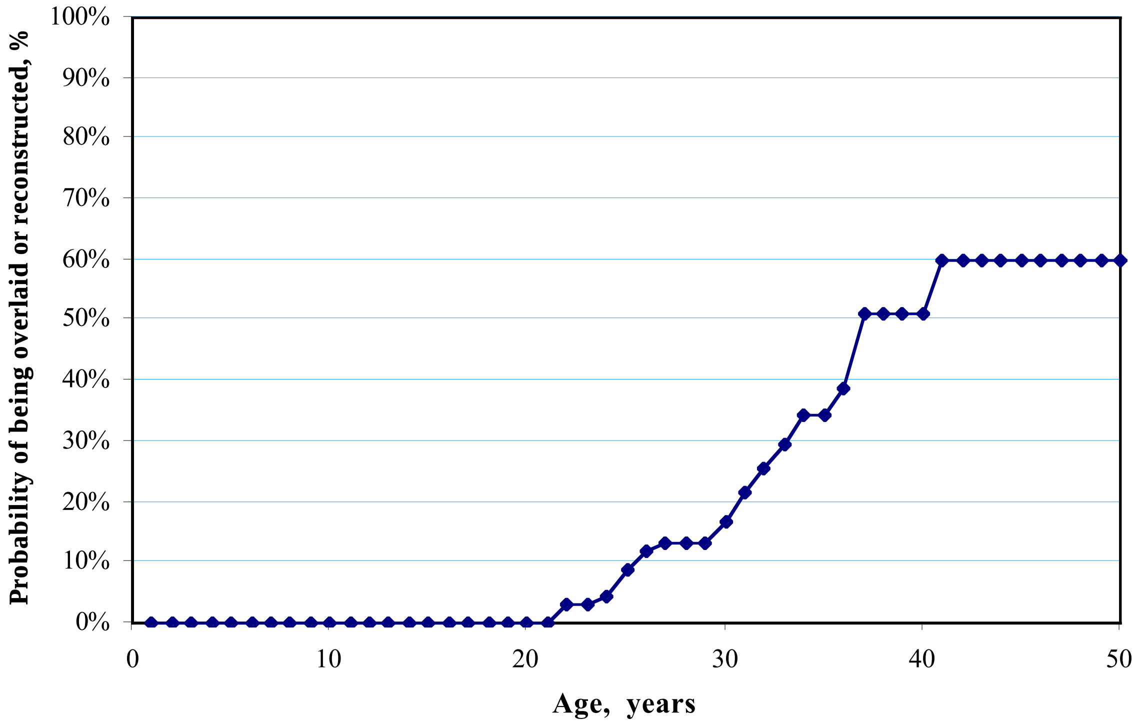

Figure 7. Survival curve showing probability of being overlaid or reconstructed vs. age since initial construction for diamond-ground pavements.

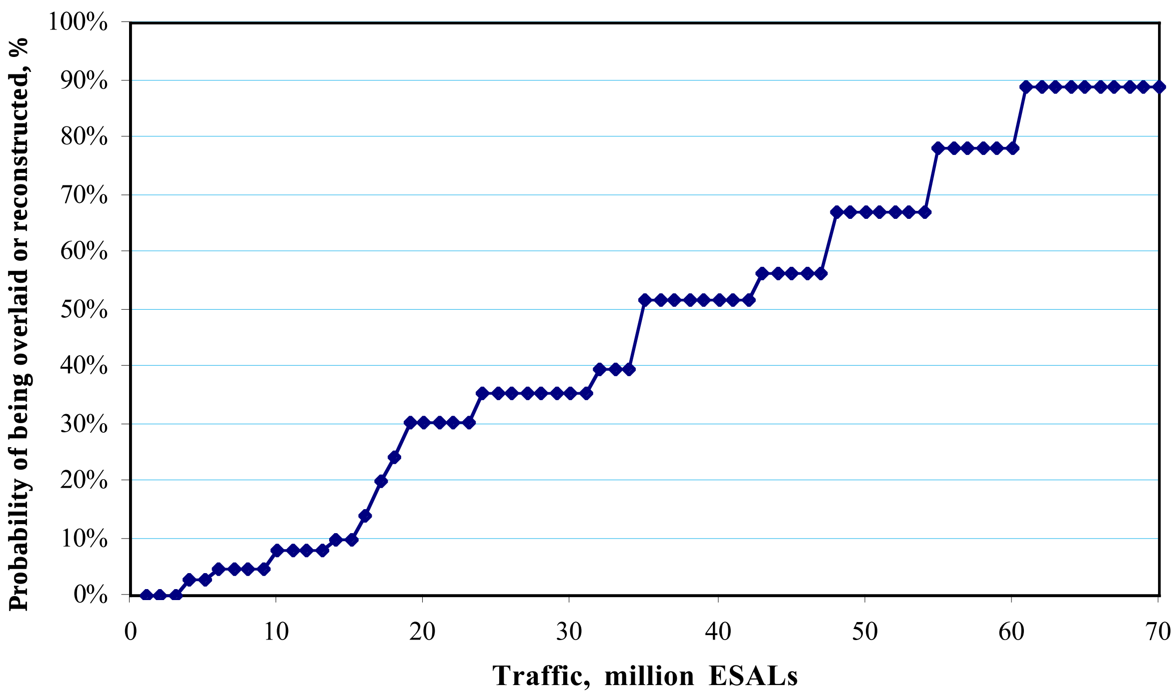

Figure 8. Survival curve showing probability of being overlaid or reconstructed vs. cumulative traffic since initial construction for diamond-ground pavements.

The probability that the pavement will be overlaid after age 40 years is 60 percent. This relatively low probability for older pavements is reflected in the high average age (34.9 years) of the 38 surviving projects and the high number of surviving projects in the 35+ to 40+-year range.

Figure 8 shows the probability of overlaying (or reconstruction) of diamond-ground pavements in terms of total applied traffic since construction. The probability that a diamond-ground pavement will be overlaid before carrying at least 15 million ESALs is less than 10 percent. The probability that a diamond-ground pavement will carry at least 30 million total cumulative ESALs since construction is 65 percent. Thus, diamond-ground pavements exhibit excellent survival trends with respect to both age and traffic.

Service Life Extension Due to Diamond Grinding

Assuming that CPR with diamond grinding was applied to the study sections as an alternative to overlaying or reconstruction, the time or applied traffic to overlaying (or reconstruction) after grinding is the service life extension provided by diamond grinding. Figure 9 shows the service life extension provided by CPR and diamond grinding. The figure shows the time since the first grinding until overlaying or reconstruction. Some of the projects in the database have been diamond ground more than once.

The following can be observed from Figure 9:

- Of the 76 projects in the database, 82 percent lasted more than 10 years since the first grind.
- 42 percent of the projects lasted more than 15 years after they were first diamond-ground. Thirty pavements whose service life had been extended by more than 15 years are still surviving, indicating that the service life extension for many projects will be more than 20 years.
- Diamond grinding has extended the life of overlaid projects by 9.8 years and the life of surviving projects by 16.0 years.
- Average extension in life for all 76 projects was 12.9 years. Therefore, the average life extension of a concrete pavement due to diamond grinding is greater than 12.9 years because of the remaining life of the surviving pavements.

Figure 10 shows the service life extension in terms of traffic distribution for the projects included in the database.

- Sixty-three percent of the 76 projects carried more than 10 million ESALs since they were first diamond ground.
- Twenty-one percent of the projects carried more than 20 million ESALs since they were first diamond ground.

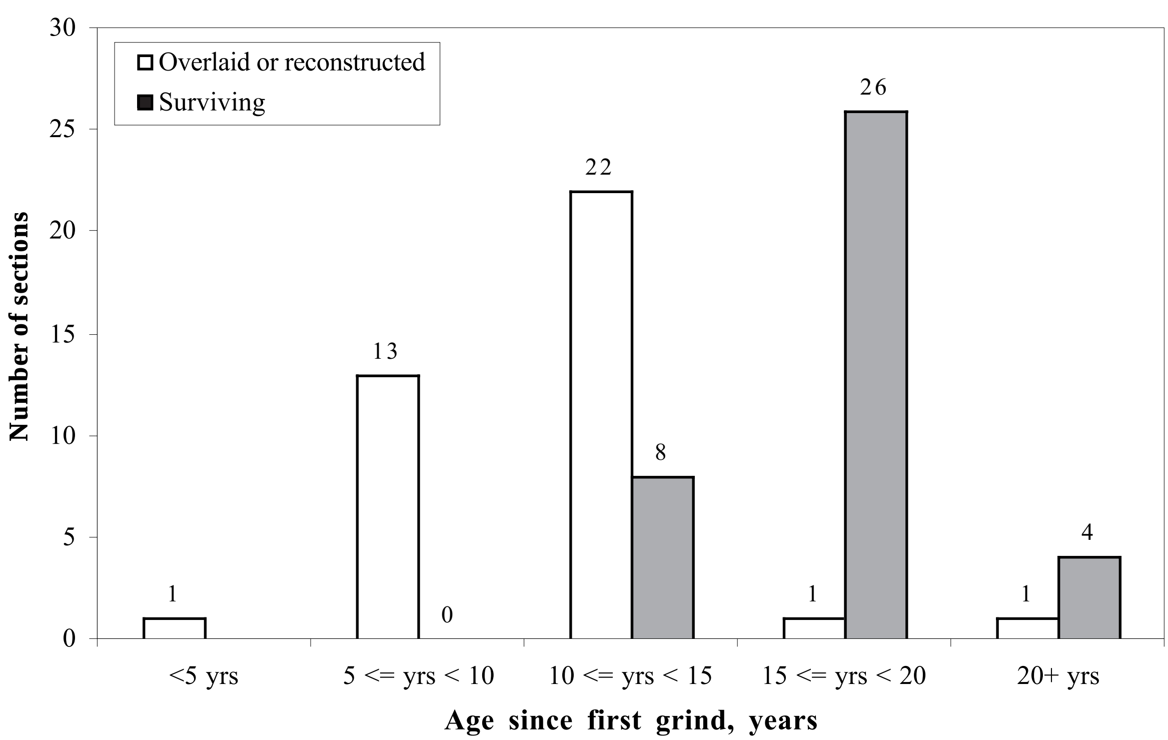

Figure 9. Service life extension provided by diamond grinding.

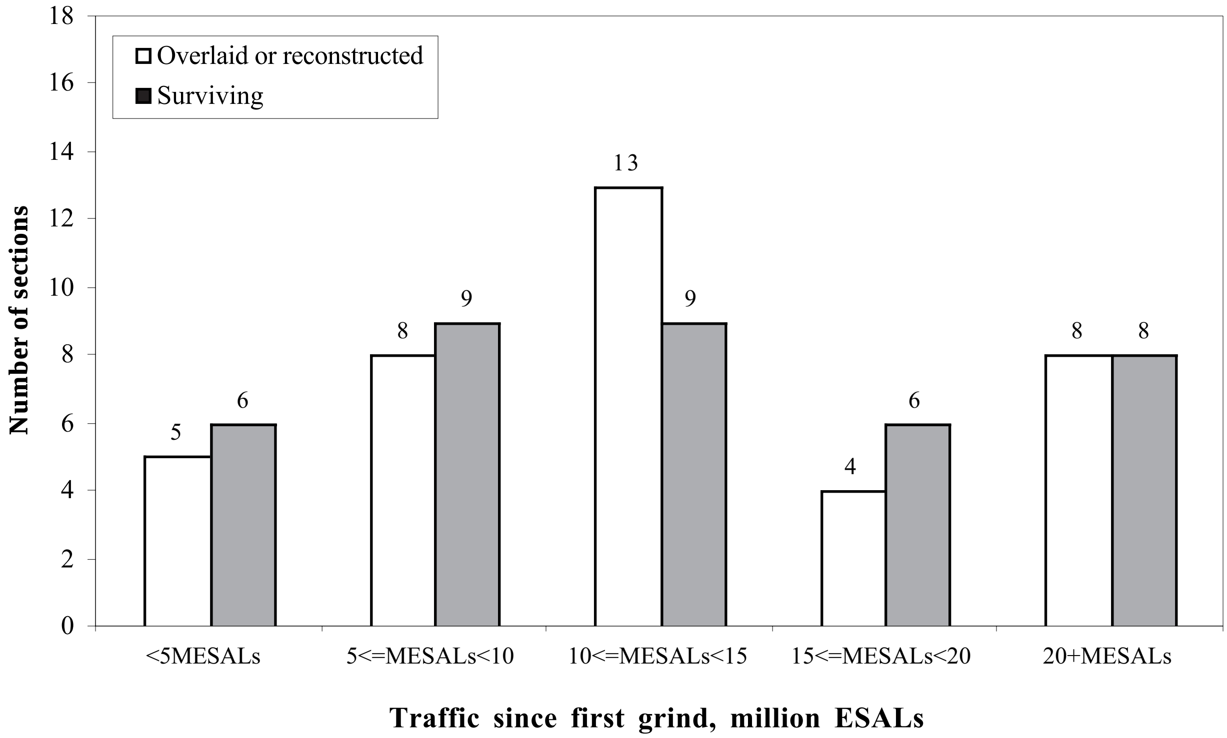

Figure 10. Traffic life extension provided by diamond grinding.

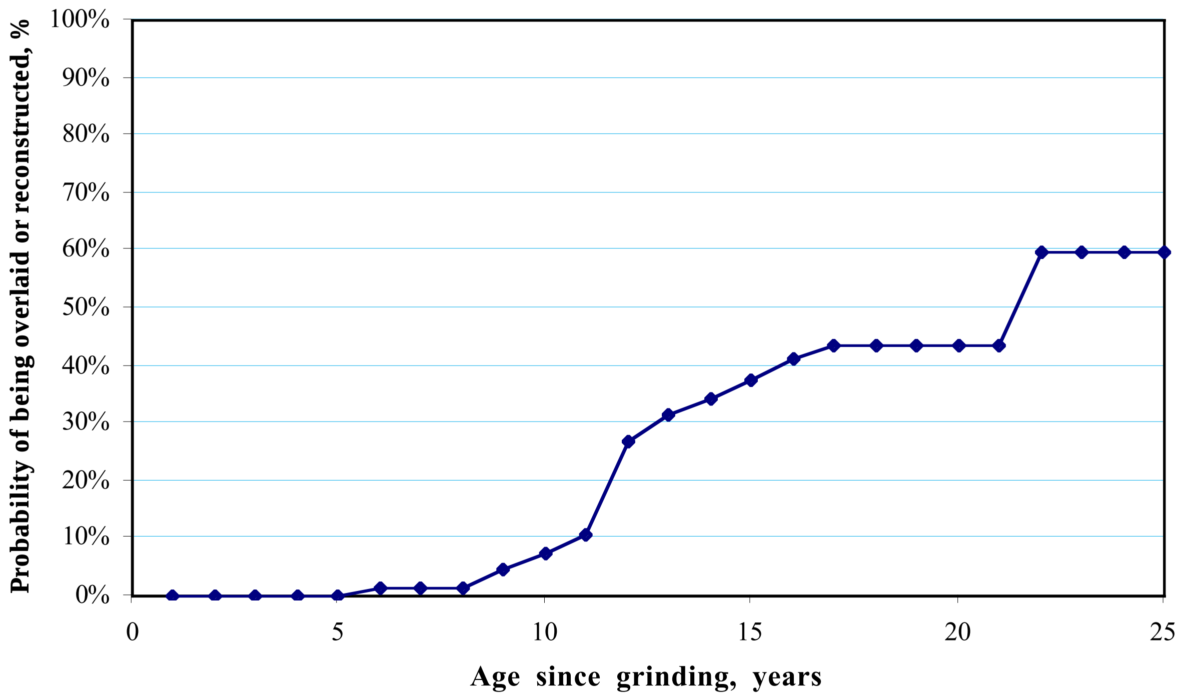

Figure 11. Survival curve showing probability of being overlaid or reconstructed vs. age since first grind for diamond-ground pavements.

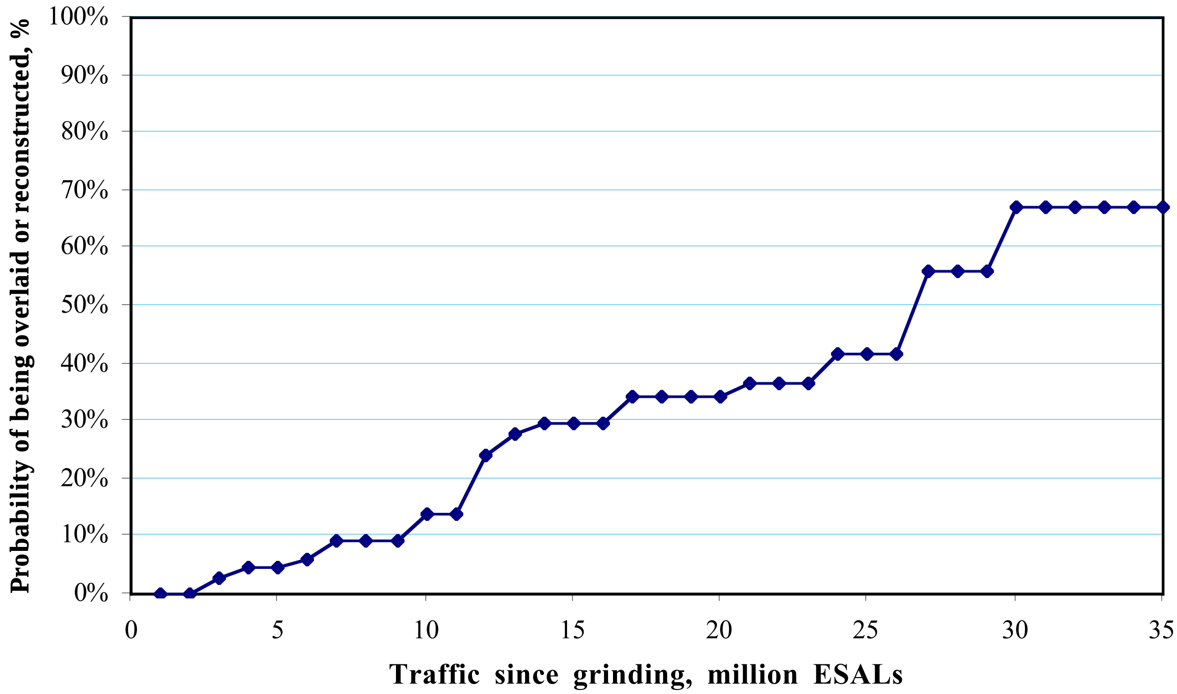

Figure 12. Survival curve showing probability of being overlaid or reconstructed vs. cumulative traffic since first grind for diamond-ground pavements.

- The average service life extension of a concrete pavement due to diamond grinding was 13.5 million ESALs. This figure will increase because of the remaining life of the surviving projects.

Figures 11 and 12 show the probability of overlaying (or reconstructing) diamond-ground pavements after the first diamond grinding in terms of age and traffic. The probability that a diamond-ground pavement will be overlaid or reconstructed without providing at least 10 years of service or carrying 9 million ESALs is less than 10 percent.

Survival of Diamond-Ground Surface

Figures 13 and 14 show the survival trends of diamond-ground surfaces to regrinding, overlaying, or reconstruction.

- Of the 76 pavement projects in the database, only 28 percent required resurfacing less than 10 years after grinding.
- Twenty-one percent of the projects did not have to be resurfaced until at least 15 years after diamond grinding.
- The average age at resurfacing was 11.4 years, or 10.8 million ESALs. Therefore, the average service life of diamond-ground surfaces is greater than 11.4 years and 10.8 million ESALs (factoring in the remaining life of the 17 projects that have not been reground, overlaid, or reconstructed to date).

Figures 15 and 16 show the survival curves for diamond-ground resurfacing. Figure 15 shows that the probability that a diamond-ground pavement will require resurfacing before providing at least 10 years of service is about 12 percent. In other words, the probability that a diamond-ground surface will last at least 10 years is almost 90 percent. The probability that it will last fewer than 8 years is less than 2 percent. The survival analysis results show that an agency can expect, with a high degree of reliability, a minimum life of 8 to 10 years for diamond-ground surfaces. A diamond-ground pavement may be reground to further extend its service life.

Figure 16 shows that there is a 95 percent probability that a diamond-ground pavement will carry more than 5 million ESALs and a 50 percent probability that it will carry at least 12 million ESALs after grinding. At this time, the pavement may be resurfaced by overlaying, reconstruction, or regrinding.

Predicted Traffic Based on AASHTO Design Procedure

The AASHTO procedure for design of new pavements (AASHTO 1993) was used to estimate traffic

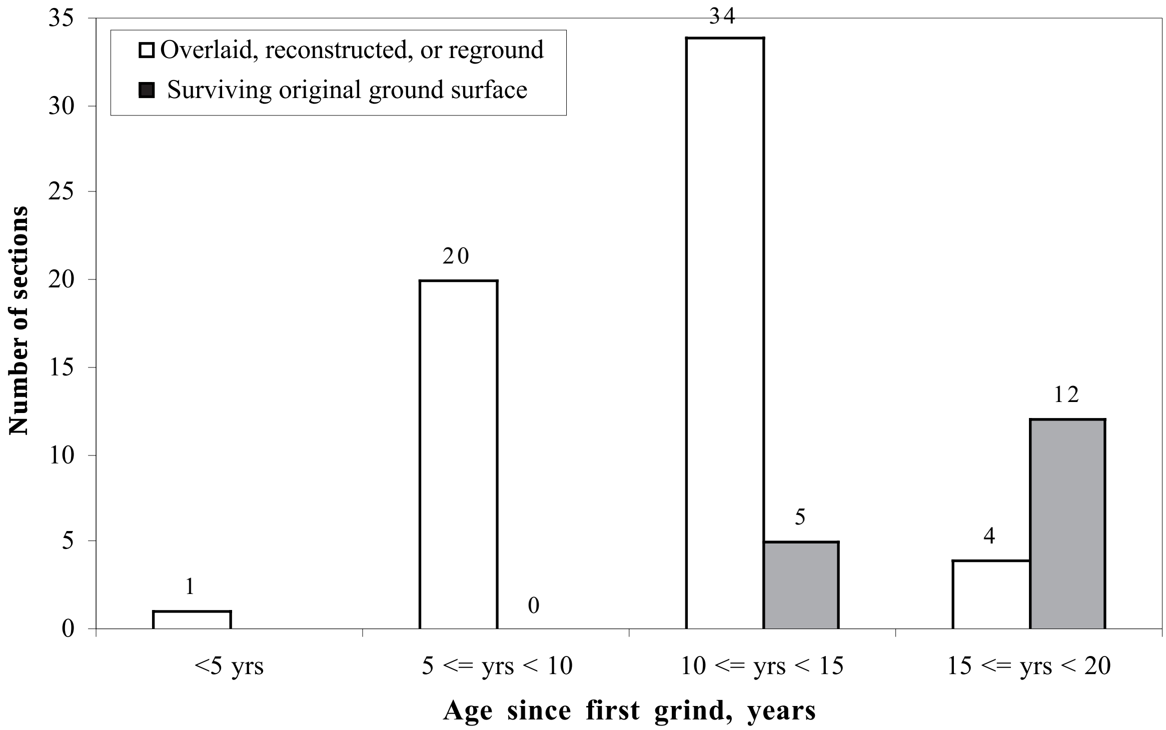

Figure 13. Distribution of pavement age since grinding to resurfacing (overlaying, reconstruction, or regrinding).

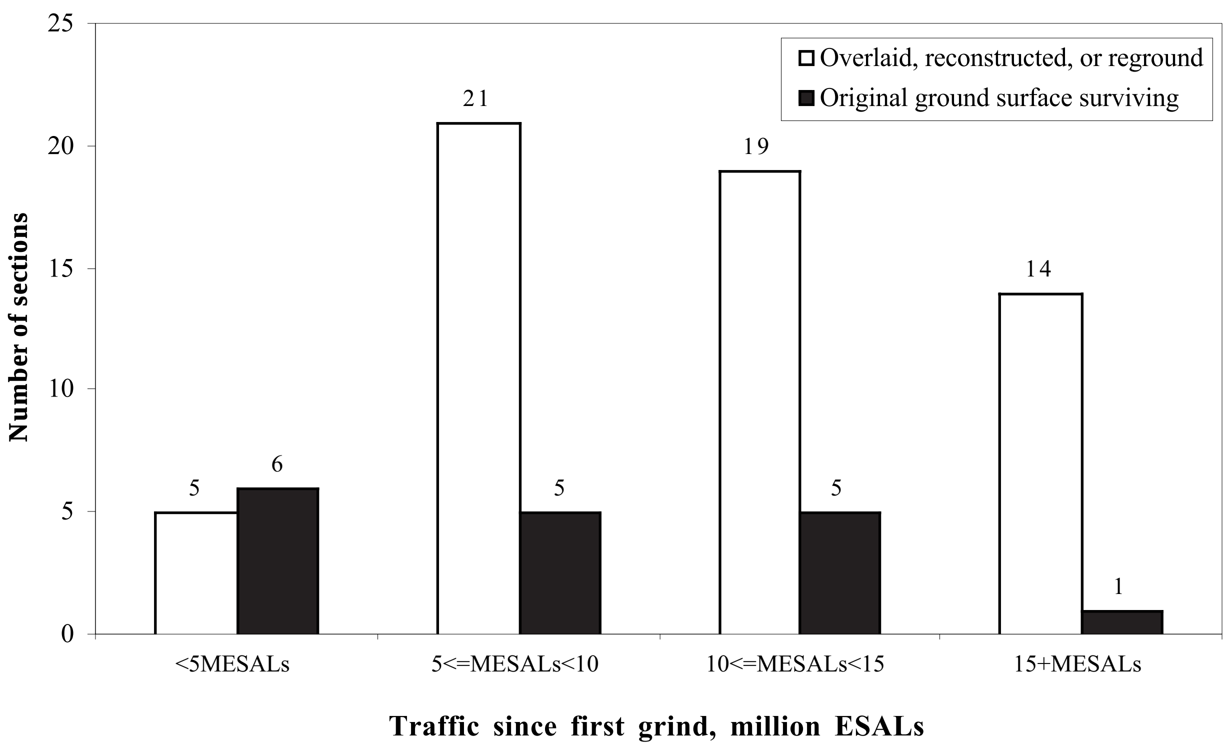

Figure 14. Distribution of traffic since grinding to resurfacing (overlaying, reconstruction, or regrinding).

Figure 15. Survival curve showing probability of resurfacing (overlaying, reconstruction, or regrinding) vs. age since first grind for diamond-ground pavements.

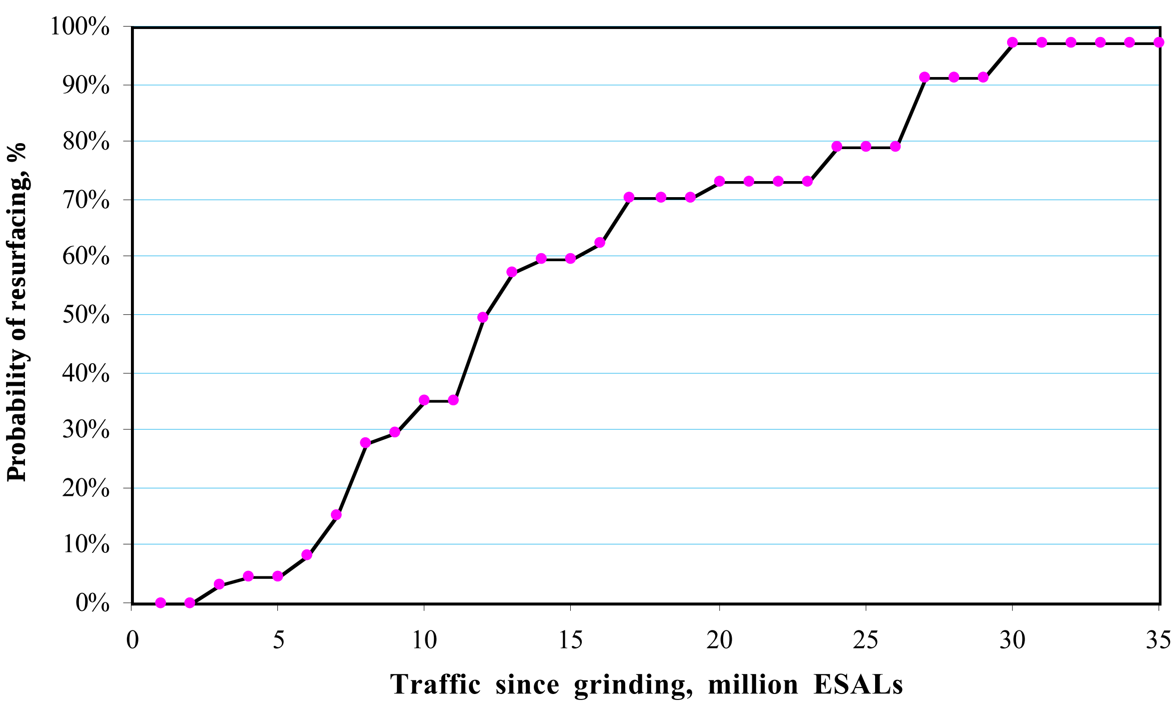

Figure 16. Survival curve showing probability of resurfacing (overlaying, reconstruction, or regrinding) vs. traffic since first grind for diamond-ground pavements.

capacity for all pavements included in this study. Figure 17 shows a plot of predicted traffic based on the AASHTO procedure versus actual traffic carried by the diamond-ground pavements since initial construction. The 45-degree line shown in the graph corresponds to when the actual traffic carried by a pavement is equal to traffic predicted using the AASHTO design equations.

At 50 percent reliability one would expect half the projects to be above the 45-degree line, indicating that these pavements carried more traffic than predicted using the AASHTO design equations. Likewise half the projects are expected to be below the 45-degree line. However, the figure shows that a majority of both the surviving sections and the overlaid or reconstructed sections are above the 45-degree line, indicating that the diamond-ground pavements have carried more traffic than predicted by the AASHTO design equations. As seen in Figure 17, many of the diamond-ground pavements have carried significantly more traffic (3 to 10 times more traffic) than expected based on the AASHTO model predictions.

Figures 18 through 22 show a comparison of predicted traffic based on the AASHTO design procedure, actual cumulative traffic from construction to CPR with grinding, and total traffic since initial construction. The figures represent projects in 24 states from each of the 4 climatic regions. The difference between the total traffic since initial construction and cumulative traffic from construction to CPR represents the extension in service life due to diamond grinding. The significant difference between total traffic since initial construction and predicted traffic based on the AASHTO design procedure indicates the significant benefits of grinding with respect to increase in traffic-carrying capacity of a pavement. The figures also show that diamond grinding has provided a significant increase in service life for pavements located in all four climatic regions.

SMOOTHNESS

The immediate effect of diamond grinding is a significant improvement in smoothness. Figure 23 shows a

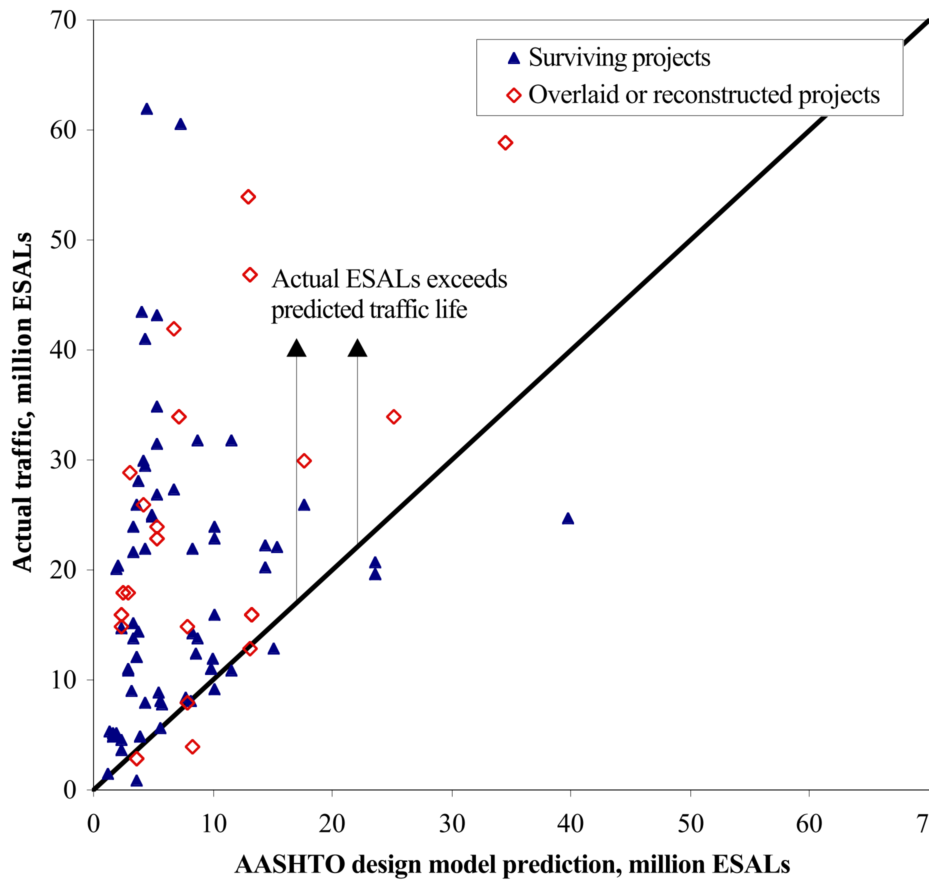

Figure 17. Actual cumulative traffic since initial construction vs. predicted traffic based on the AASHTO design procedure (50% reliability) for diamond grinding projects.

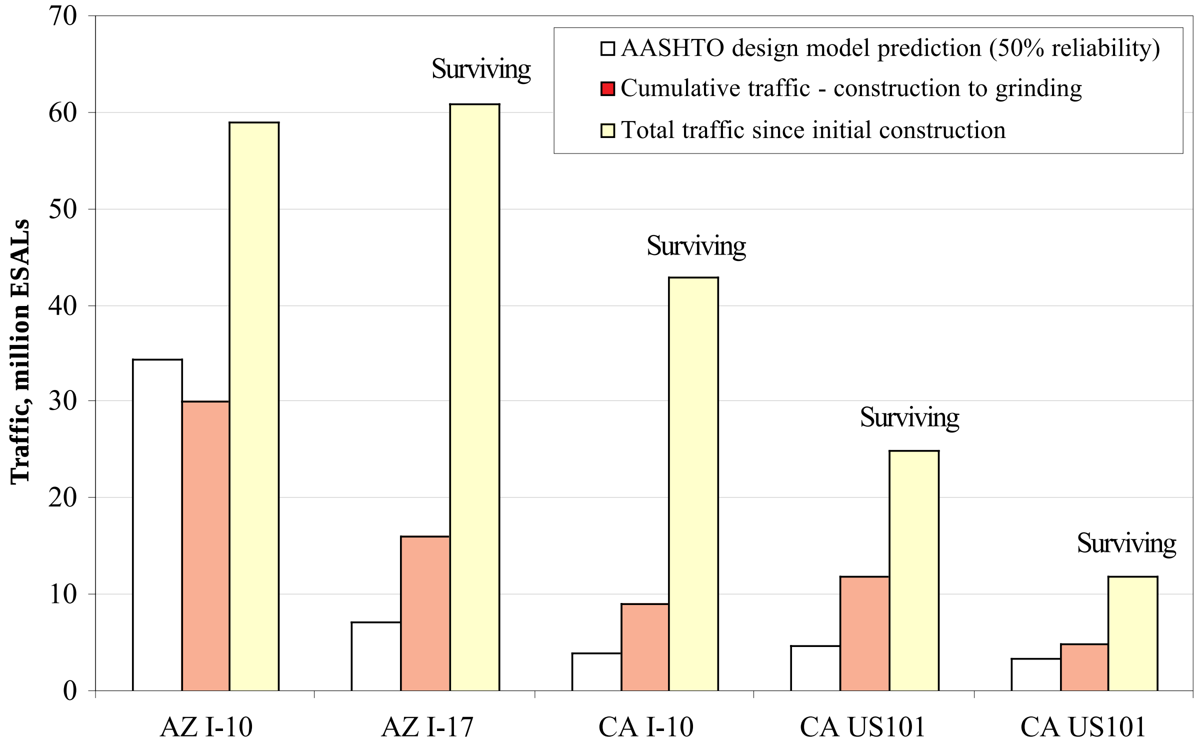

Figure 18. Predicted traffic based on the AASHTO design procedure and actual traffic since initial construction on diamond-ground projects in dry non-freeze climatic region.

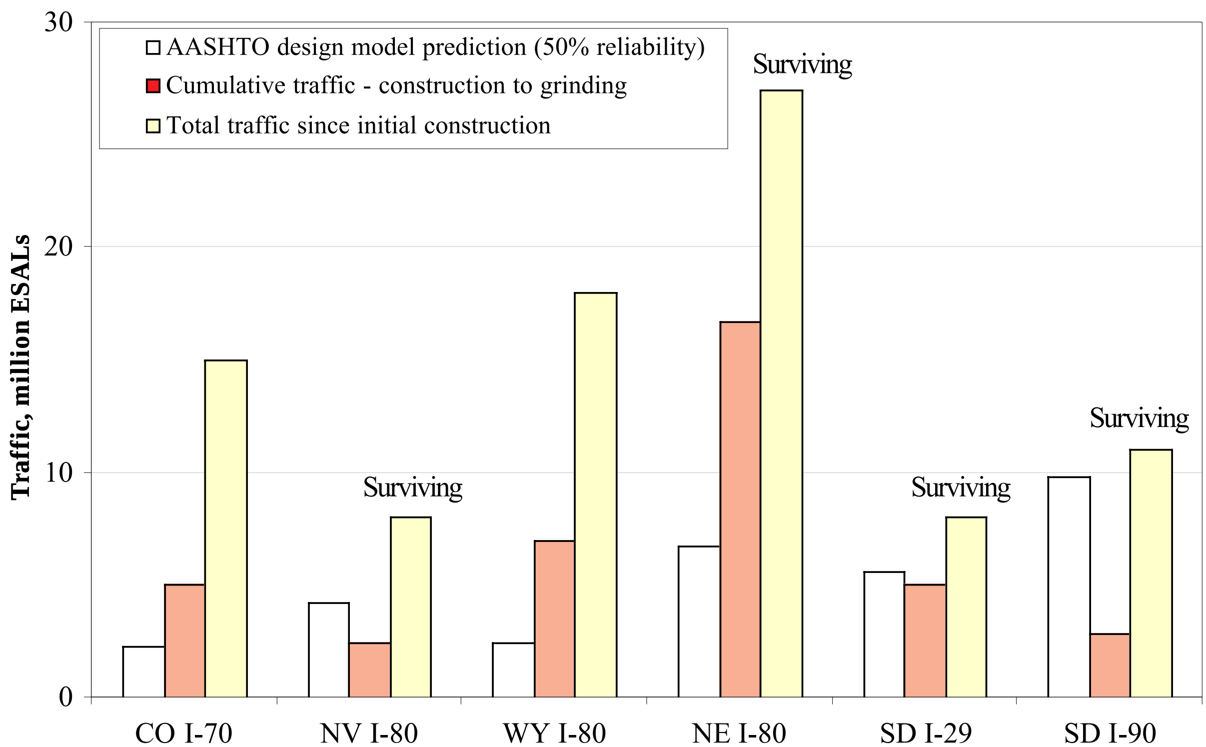

Figure 19. Predicted traffic based on the AASHTO design procedure and actual traffic since initial construction on diamond-ground projects in dry freeze climatic region.

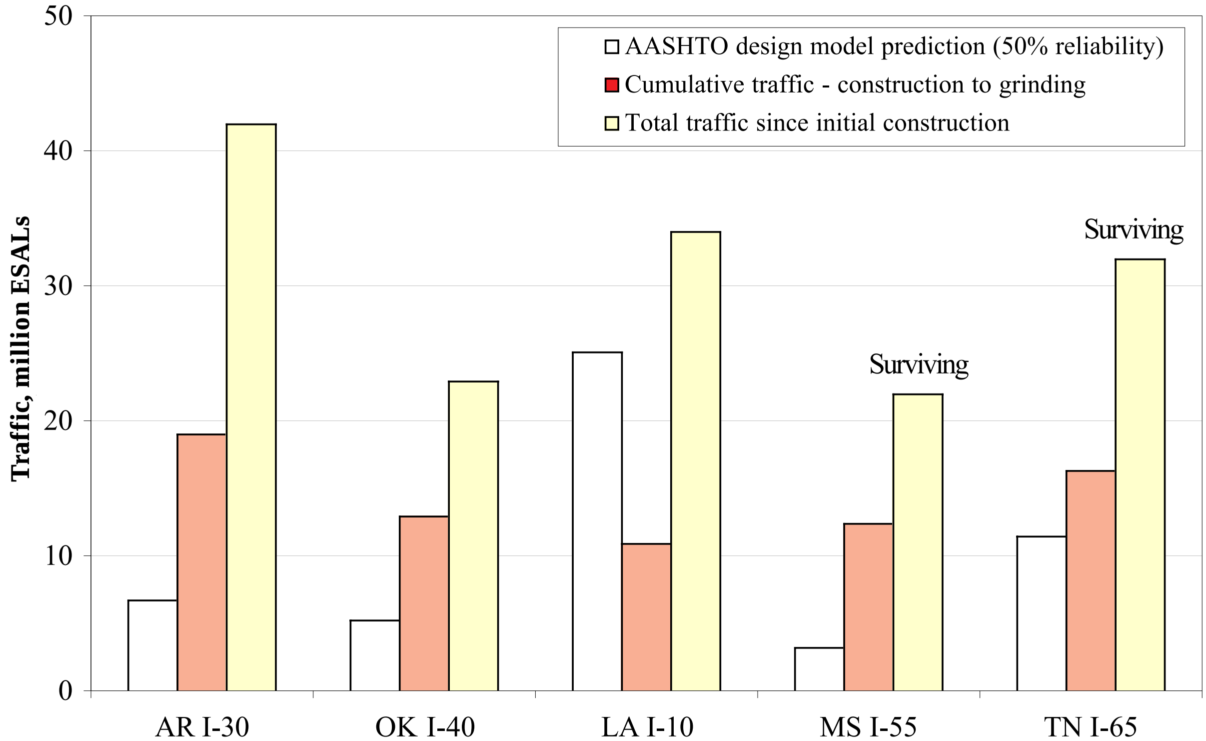

Figure 20. Predicted traffic based on the AASHTO design procedure and actual traffic since initial construction on diamond-ground projects in wet non-freeze climatic region.

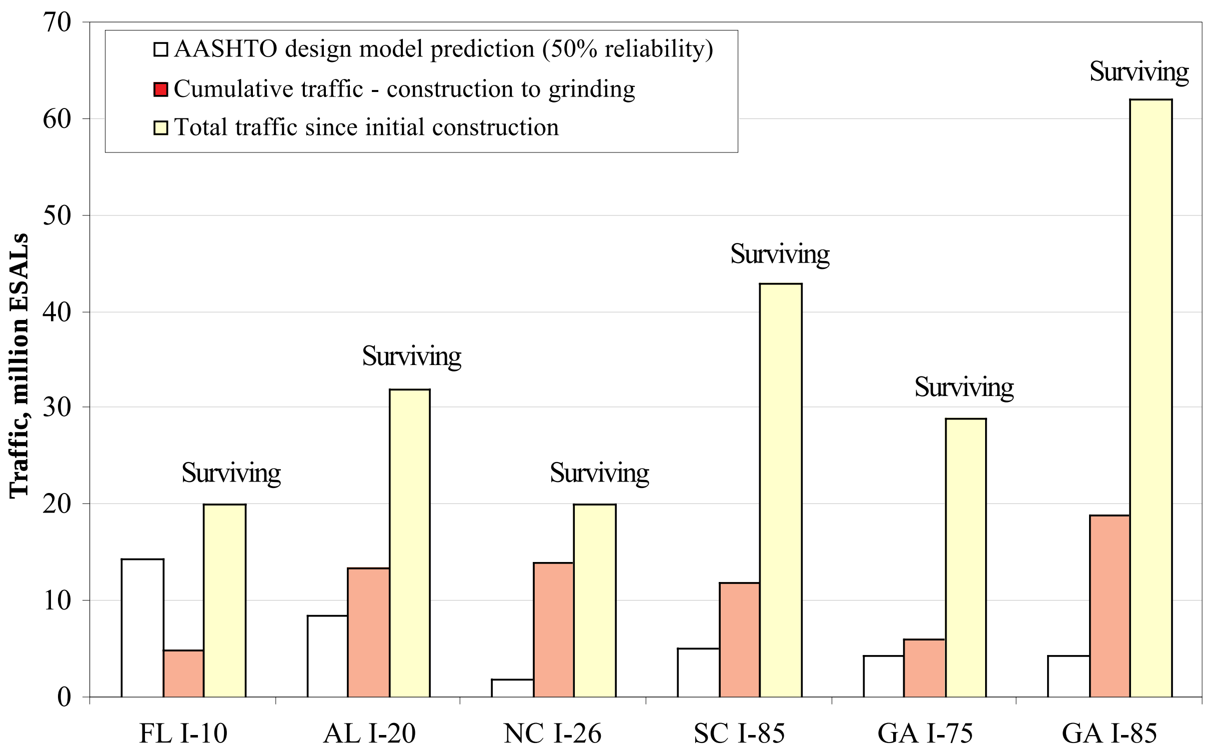

Figure 21. Predicted traffic based on the AASHTO design procedure and actual traffic since initial construction on diamond-ground projects in wet non-freeze climatic region.

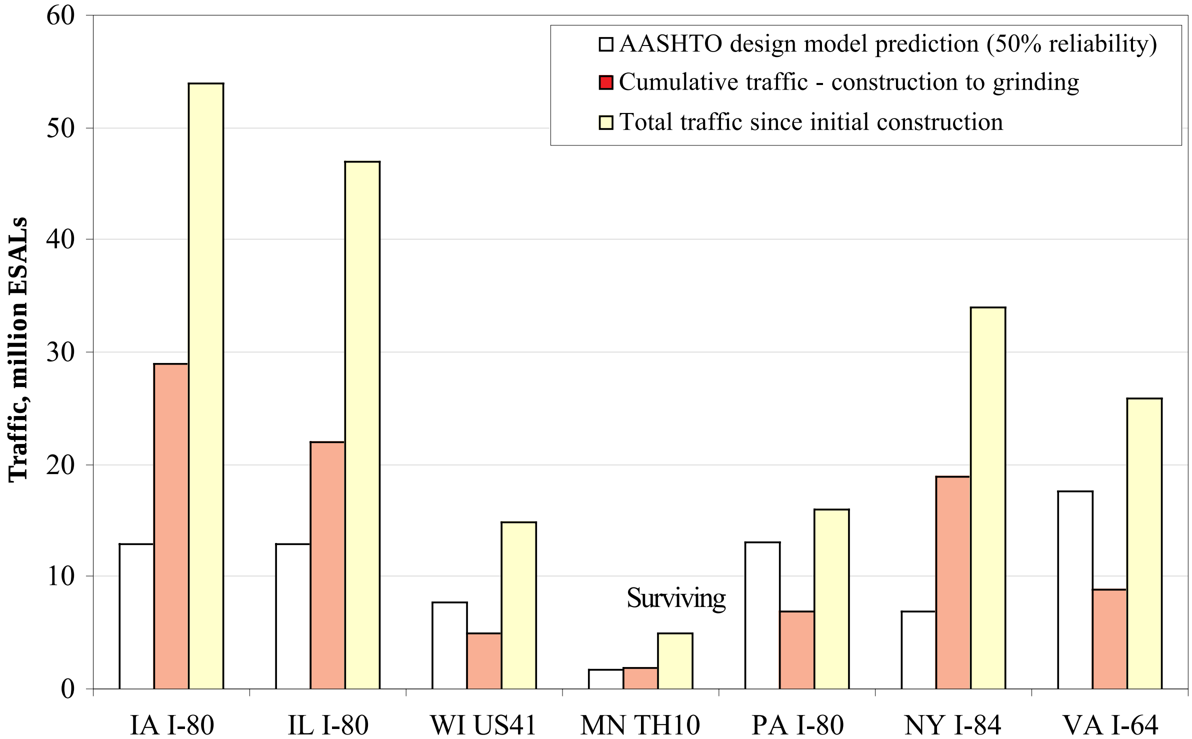

Figure 22. Predicted traffic based on the AASHTO design procedure and actual traffic since initial construction on diamond-ground projects in wet freeze climatic region.

comparison of International Roughness Index (IRI) values before and shortly after rehabilitation of SPS-6 sections. *Min Prep* and *Max Prep* refer to the following levels of CPR:

- Min Prep—includes some full-depth, doweled concrete repairs of severely distressed areas, sealing of transverse and longitudinal joints, sealing of mid-slab breaks, bituminous shoulder removal and replacement, and diamond grinding (in most cases).
- Max Prep—similar to Min Prep, but all working cracks are full-depth repaired and retrofit edgedrains are added. Again, diamond grinding was performed in most cases.

The Indiana sections were the only sections that were not diamond ground, and the CPR activities actually caused a slight increase in roughness. Other sections show a significant drop in IRI after diamond grinding. Figure 24 shows that the level of smoothness that can be achieved through diamond grinding is comparable to that of a new pavement or an AC overlay.

Long-term smoothness of both diamond-ground pavements and AC overlays depends on numerous factors, but CPR with diamond grinding can provide short-term smoothness that is as good as an AC overlay. Figure 24 shows no significant difference in IRI between diamond-ground and AC-overlaid sections, after 4 years of service. It is important to recognize that diamond grinding addresses smoothness only; if the cause of roughness is not treated prior to grinding, redevelopment of roughness over the long term is likely (Snyder et al. 1989). Also, not all JCP that have developed excessive roughness are good candidates for diamond grinding, and grinding alone may not be enough to address existing problems. Structural distresses, such as pumping, loss of support, corner breaks, working transverse cracks, and shattered slabs, require repair before grinding (Roman et al. 1985).

If the existing pavement is structurally deficient, an overlay or reconstruction may be more appropriate. Diamond grinding a structurally deficient pavement can result in rapid redevelopment of roughness; however, even in such cases, diamond grinding could be considered as an economical short-term (i.e., 5-year) solution to a roughness problem until the pavement section can be overlaid or reconstructed.

FAULTING

Excessive faulting of joints and transverse cracks is perhaps the most common reason for grinding JCP. Faulting at joints and cracks is the single most important factor that affects ride quality on JCP. *Transverse*

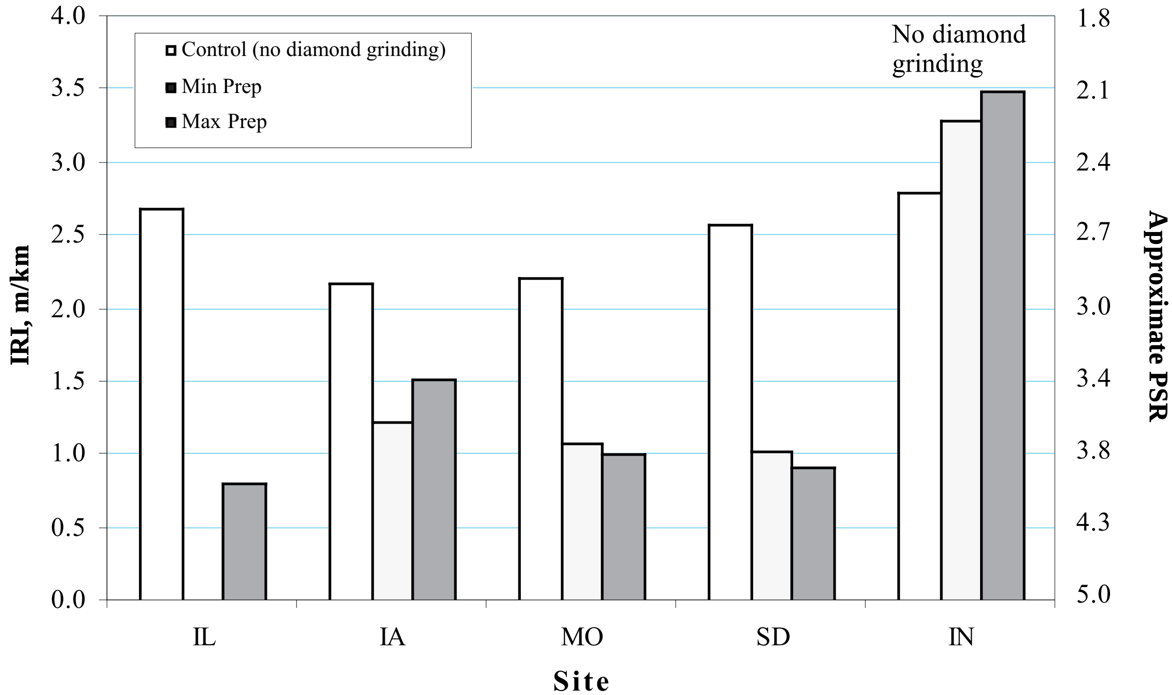

Figure 23. Immediate effects of diamond grinding on pavement smoothness.

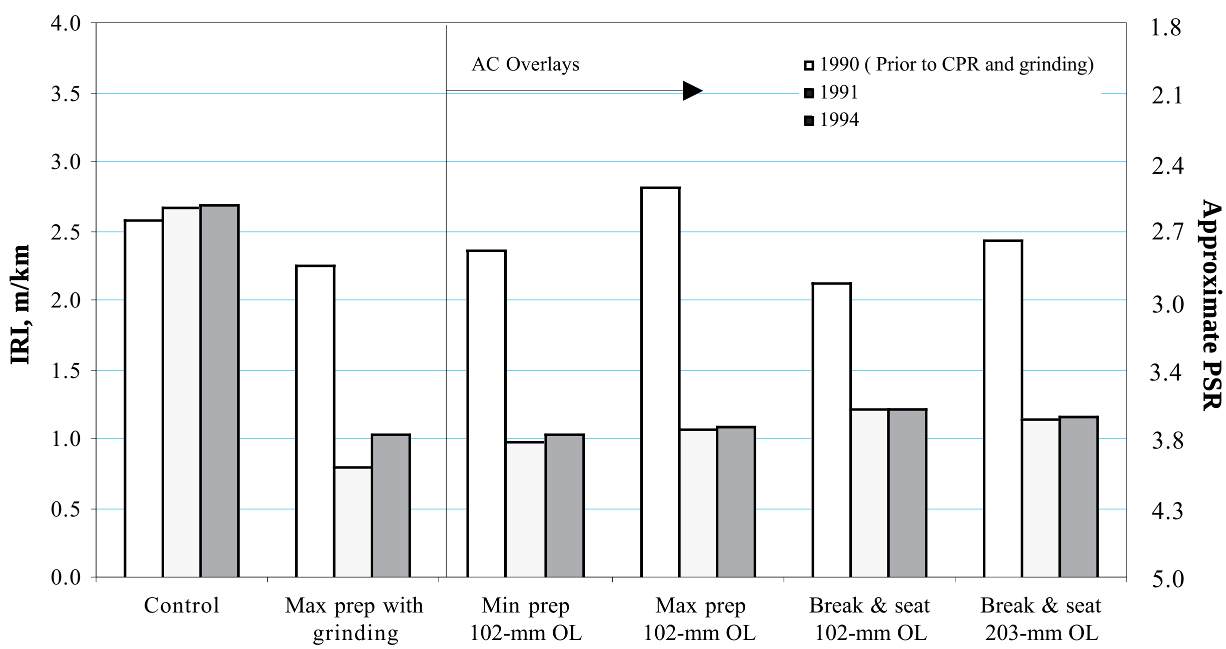

Figure 24. Smoothness performance of diamond-ground and AC overlay sections at the Illinois SPS-6 site.

joint faulting refers to the difference in elevation across the joint. Faulting is caused by a combination of repeated heavy axle loads, insufficient load transfer between adjacent slabs, free moisture in the pavement structure, and erodible base or subgrade material. Erosion occurs when excess moisture is ejected from beneath the leave slab corner under the influence of a moving vehicle load (typically heavy trucks). The moisture that is ejected carries base and subgrade fines with it, resulting in a void. In addition, there is a corresponding deposit of fines under the approach slab corner, causing it to lift slightly. Faulting and corner cracking can develop due to the build-up of material beneath the approach corner and the loss of support under the leave corner.

Faulting is noticeable when the average faulting in the pavement section reaches about 2.5 mm (0.1 in.). When the average faulting reaches 3.8 mm (0.15 in.), diamond grinding or other rehabilitation measures should be considered. Figure 25 shows the historical faulting data for some of the sections that were surveyed for the 1986 FHWA study and were resurveyed in 1997 for the current study. Immediately after grinding, the faulting on a pavement is zero. Average faulting on a diamond-ground pavement increases with time under the influence of traffic loading. Faulting is a function of loss of aggregate interlock (poor load transfer), loss of support, voids, dowel bar failure, and other factors. It is important to note that many of these problems are preexisting on diamond-ground pavements, and diamond-ground pavements will continue to fault unless load transfer efficiency is improved.

Faulting performance of diamond-ground pavements is shown in Figure 26 for nondoweled sections and in Figure 27 for doweled sections. Both figures show significant scatter in faulting performance, especially on the nondoweled sections. Diamond-ground pavements are typically (but not always) those that had developed significant faulting prior to grinding, with accompanying void and loss of support problems at slab corners. Hence, the high degree of variability in faulting performance for these sections is not surprising.

Concurrent rehabilitation is another source of variability in faulting performance after diamond grinding. The amount, type, and performance of rehabilitation performed concurrent with diamond grinding can all affect the faulting performance. Full-depth repairs are typically doweled and reduce average faulting by replacing existing original faulted joints. Slab stabilization and retrofit edgedrains also affect average faulting by reducing pumping. However, the performance of stabilized slabs and retrofitted edgedrains can be highly variable, resulting in high variability in faulting performance (Snyder et al. 1989).

Nondoweled JCP

Figure 26 shows the faulting data from the study sections, along with a trend line, and predicted faulting for a new pavement determined using the model developed under a recent National Cooperative Highway Research Program (NCHRP) project (Yu et al. 1998). The faulting performance of new pavements is characterized by rapid initial development of faulting, followed by a more gradual increase. The model shows that, on average, faulting in new nondoweled JCP reaches 3 mm (0.12 in.) within the first 4 million ESAL applications. Figure 26 shows that for the first 4 million ESALs after grinding, most diamond-ground pavements have lower faulting than new pavements with similar traffic levels. After 4 million ESALs, diamond-ground pavements are faulted an average of 2.3 mm (0.09 in.). After approximately 8 to 10 million ESALs, diamond-ground pavements have the same amount of faulting as new nondoweled pavements. At this point, faulting has reached an average value of 4 mm (0.15 in.), and regrinding or overlaying is recommended. The trend line representing average faulting of diamond-ground pavements is always lower than the prediction model for new nondoweled JCP for the life of the diamond-ground surface. Therefore, diamond-ground pavements provide a smoother ride (with respect to joint faulting-related roughness for the first 10 million ESALs) than a new (nonground) pavement.

A JCP model was developed for faulting in nondoweled pavements after grinding using the data collected under this study and the data obtained from the 1986 FHWA study. The database assembled for the faulting model contains 89 nondoweled JCP sections, and it includes time series data for some of the sections from the FHWA database. This section of the report presents the final model and examples of performance trends predicted by the model. Details of model development are presented in appendix A.

Nondoweled JCP faulting model for diamond-ground pavements. A mechanistic-empirical faulting model was developed using the energy of deformation in the subgrade at the slab corner as the main component of faulting. The final faulting model is given in equation 1:

$$FAULT = DAMAGE^{0.35} (7.874 + 0.00474 \cdot PRECIP) \quad (1)$$

R^2 = 53%
SEE = 1.07 mm (0.042 in.)
N = 89

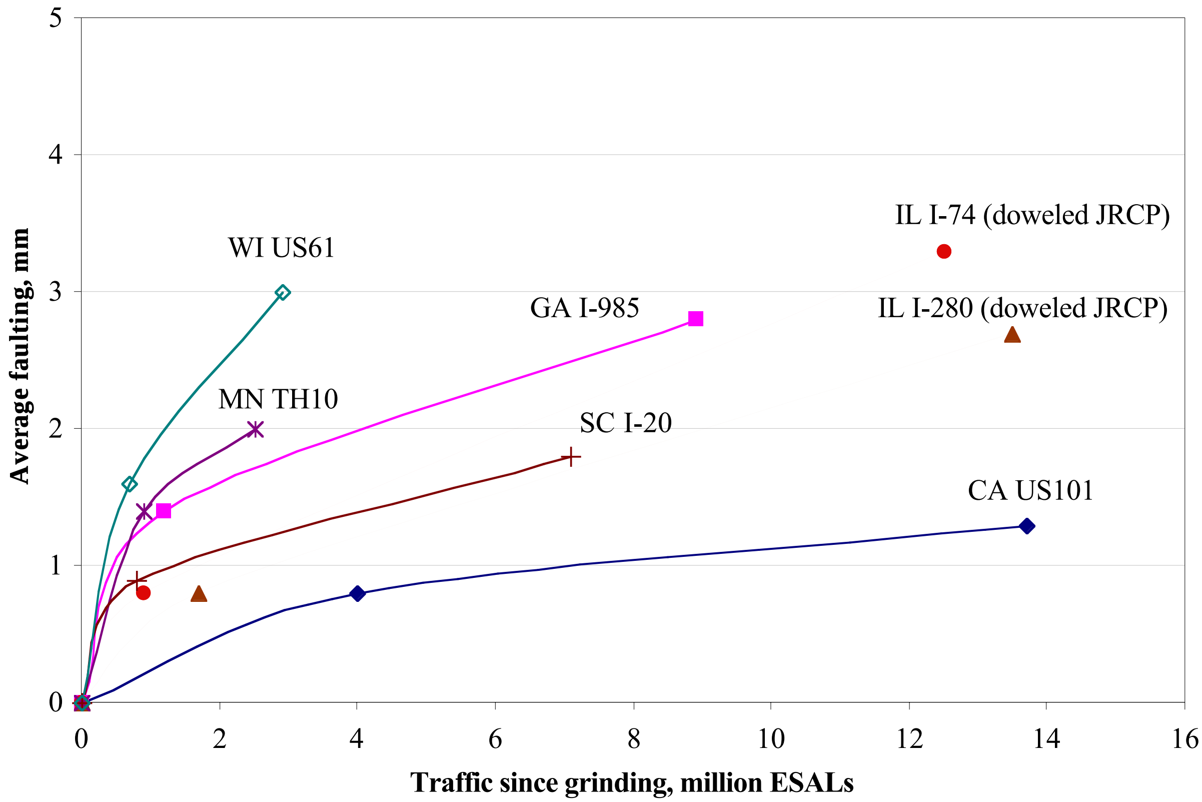

Figure 25. Time history faulting data (since diamond grinding) for projects surveyed for the 1986 FHWA study and resurveyed in 1997.

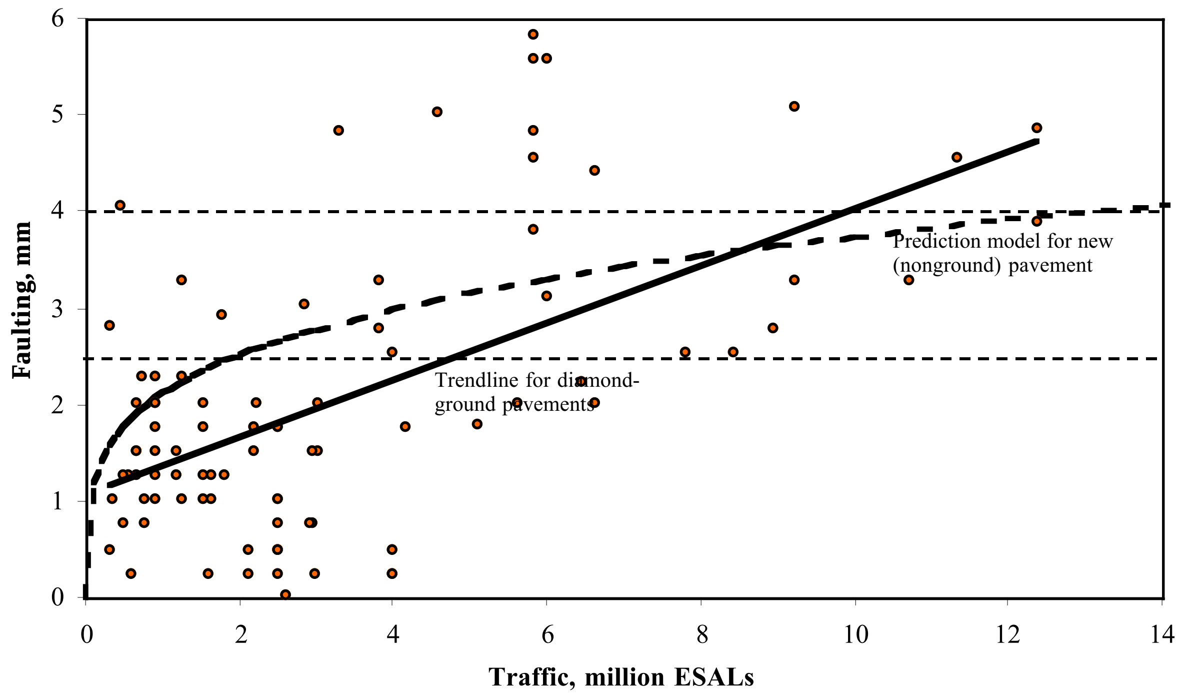

Figure 26. Faulting performance of nondoweled diamond-ground pavements.

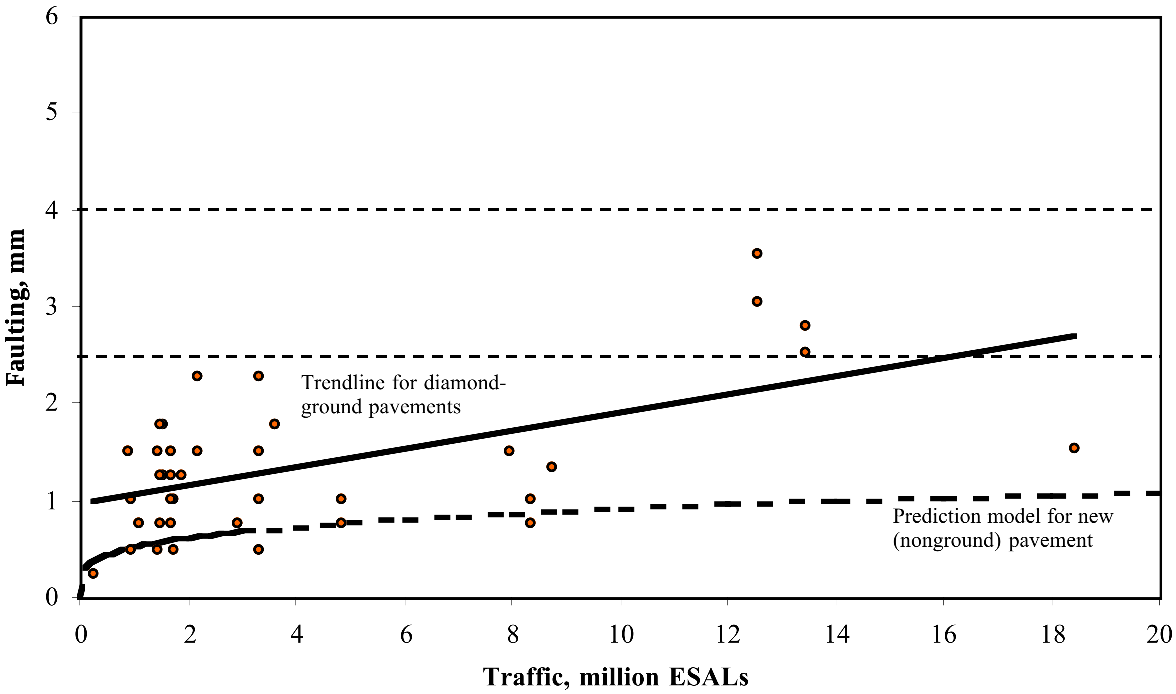

Figure 27. Faulting performance of doweled diamond-ground pavements.

where:

FAULT = Mean transverse joint faulting, mm.

DAMAGE = n/N.

n = Cumulative number of applied traffic, million ESALs.

N = Allowable number of 80-kN (18-kip) load applications, million ESALs (equation 2).

PRECIP = Average annual precipitation, mm.

$$\log N = 2.87028 - 0.92991 \cdot \log(DE) \qquad (2)$$

where:

DE = Differential energy density, kPa-mm (equation 3).

$$DE = \frac{k}{2} w_{fc}^2 \frac{1 - LTE}{1 + LTE} \qquad (3)$$

where:

k = Modulus of subgrade reaction, kPa/mm.

w_{fc} = Free corner deflection (no load transfer between adjacent slabs) with the load placed in the mean wheelpath, mm. This can be determined using the finite element program ILLI-SLAB or using simplified regression equations developed by Yu et al. (1998).

LTE = Effective load transfer efficiency over time determined using equation 4.

$$LTE = LTE_0 + (LTE_{36} - LTE_0)\,\frac{x}{0.9144} \qquad (4)$$

where:

LTE_0 = Effective LTE with load placed at corner (equation 5).

LTE_{36} = Effective LTE with load placed 0.9144 m (36 in.) from corner (equation 6).

x = Distance from mean wheelpath to slab corner, m.

$$LTE_0 = \frac{1}{(0.01 + 0.012AGG^{*-0.849})} \qquad (5)$$

$$LTE_{36} = \frac{1}{(0.01 + 0.003483AGG^{*-1.13677})} \qquad (6)$$

where:

*AGG** = Nondimensional aggregate load transfer factor determined using equation 7.

$$AGG^* = 2.3 \cdot \mathrm{Exp}(1 - 0.165583\ JTSPACE/I) \qquad (7)$$

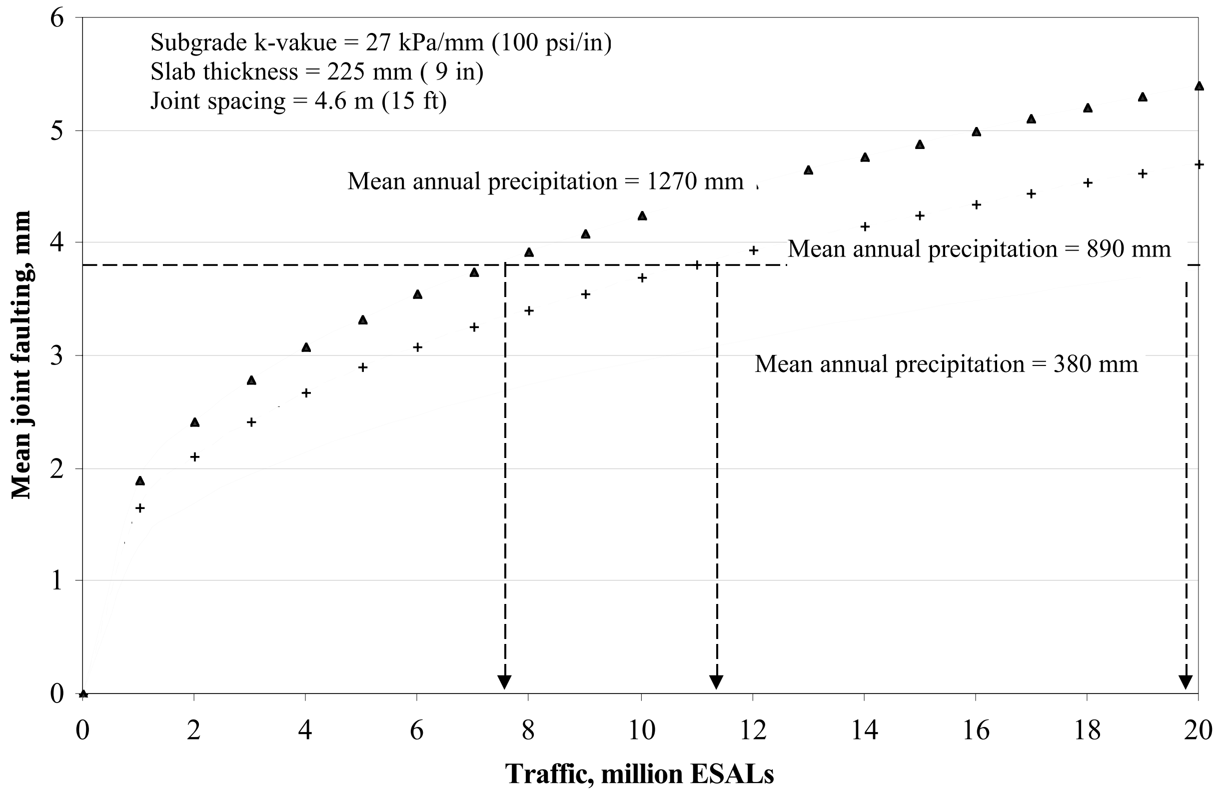

Figure 28. Effects of precipitation on nondoweled JCP faulting after diamond grinding.

where:

JTSPACE = Joint spacing, m.

l = Radius of relative stiffness, m (equation 8).

$$l = 0.001 \cdot \sqrt[4]{\frac{E_{PCC} h^3_{PCC}}{12(1-\mu^2)k}} \tag{8}$$

where:

E_{PCC} = PCC modulus of elasticity, kPa.

h_{PCC} = PCC slab thickness, mm.

μ = PCC Poisson's ratio.

Factors affecting faulting performance of diamond-ground pavements. The effect of precipitation on faulting performance of nondoweled JCP is shown in Figure 28. Immediately after grinding, nondoweled JCP shows a rapid increase in faulting and reaches 1.3 to 2.0 mm (0.05 to 0.08 in.) of faulting within 1 million ESALs. The rate of faulting decreases after 2 million ESALs. Faulting performance after diamond grinding varies significantly depending on the amount of precipitation. Pavements in dry regions (precipitation < 380 mm [15 in.]) fault to 3.8 mm (0.15 in.) after 20 million ESALs. Pavements in wet regions (precipitation > 1270 mm [50 in.]) reach this level of faulting after about 7 million ESALs.

Figures 29, 30, and 31 show the effects of joint spacing, subgrade support, and slab thickness on faulting of nondoweled pavements after diamond grinding. The critical faulting level of 3.8 mm (0.15 in.) is reached after 7 to 11 million ESALs for pavements ranging in joint spacing from 9.1 to 4.6 m (30 to 15 ft). Pavements constructed on a soft subgrade (k = 27 kPa/mm [100 psi/in.]) reach the critical level of faulting after 11 million ESALs. Pavements constructed on a stiff subgrade (k = 109 kPa/mm [400 psi/in.]) reach the critical level of faulting only after 15 million ESALs. In addition, pavements with thicker slabs fault at a slower rate than those with thin slabs.

Dowel-bar retrofitting. The nondoweled concrete pavements constructed in the U.S. during the 1960s and 1970s have served well, carrying significantly more traffic than they were designed for; however, excessive faulting has been a problem for many of those pavements. Inadequate load transfer capacity of the nondoweled joints is the main reason for the faulting problem. Since diamond grinding has no effect on load transfer efficiency, a diamond-ground pavement will eventually redevelop faulting unless the load transfer efficiency is improved. Dowel bar retrofitting offer a practical solution to the poor load transfer efficiency problem in existing nondoweled pavements (Mack 1995).

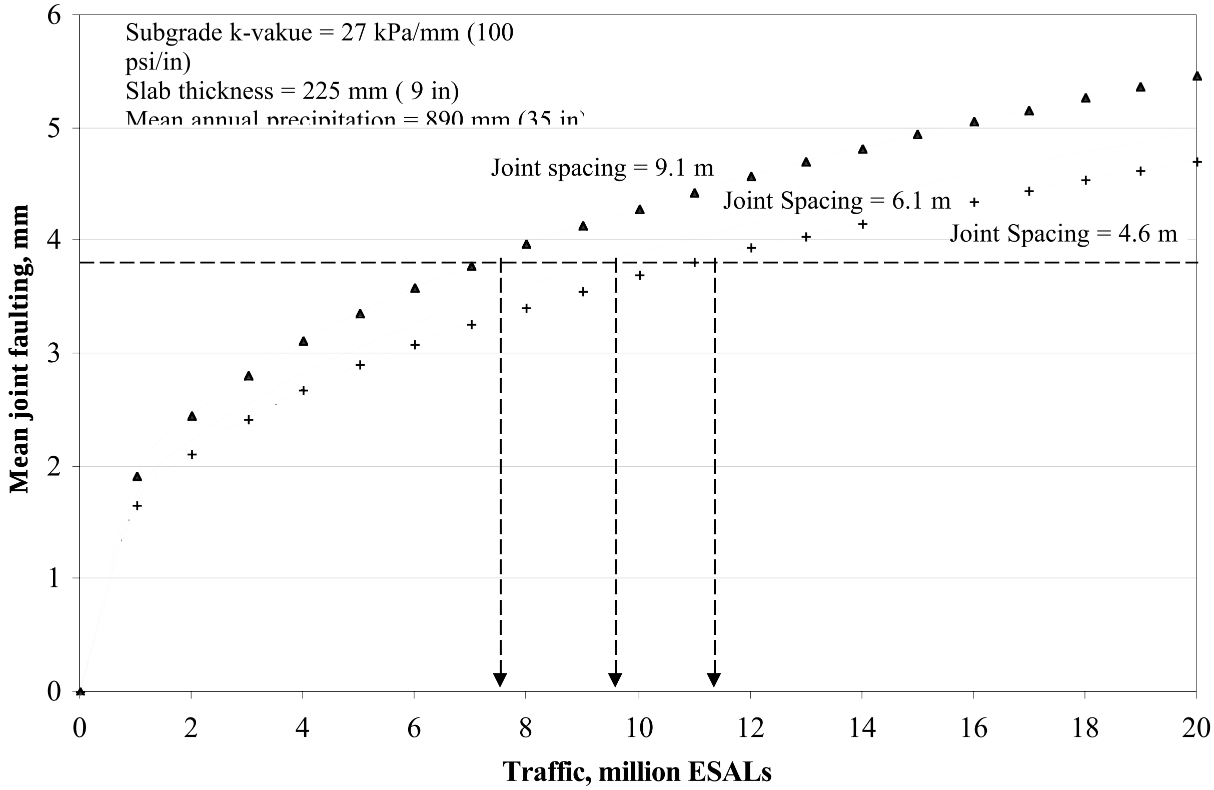

Figure 29. Effect of joint spacing on nondoweled JCP faulting after diamond grinding.

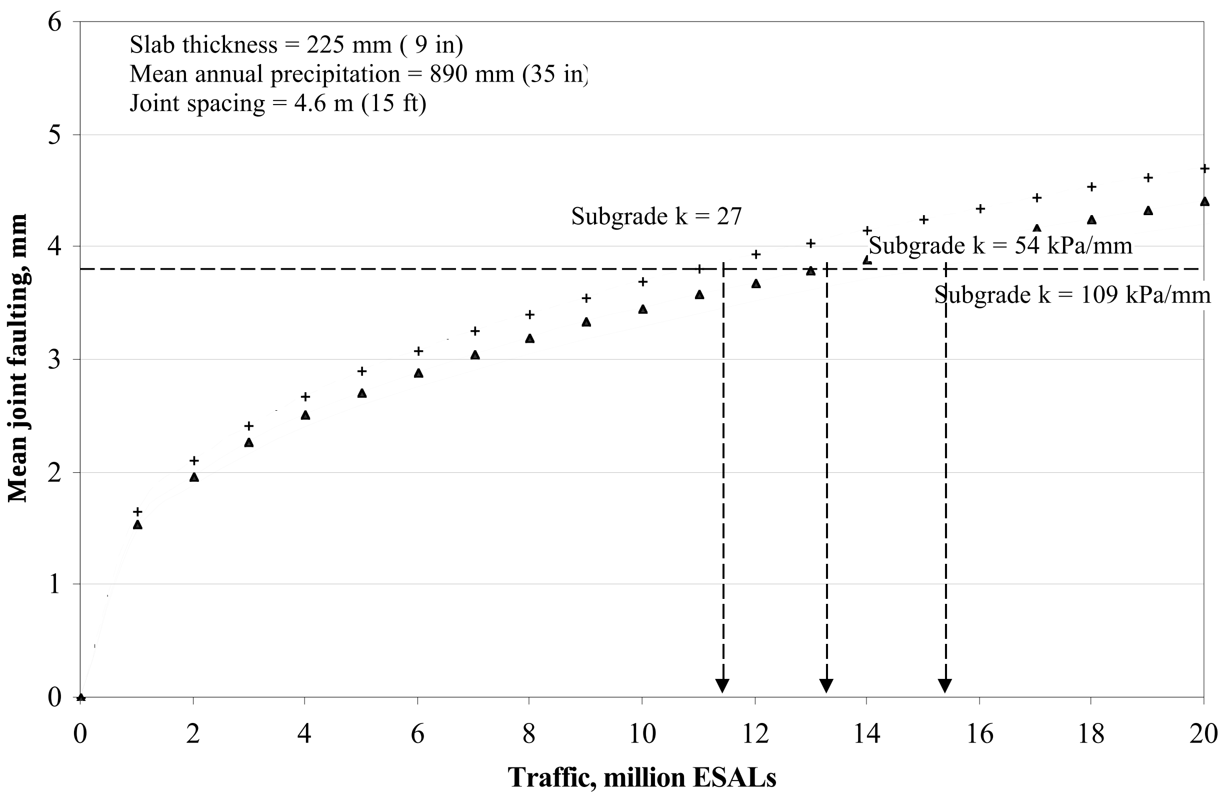

Figure 30. Effect of subgrade k-value on nondoweled JCP faulting after diamond grinding.

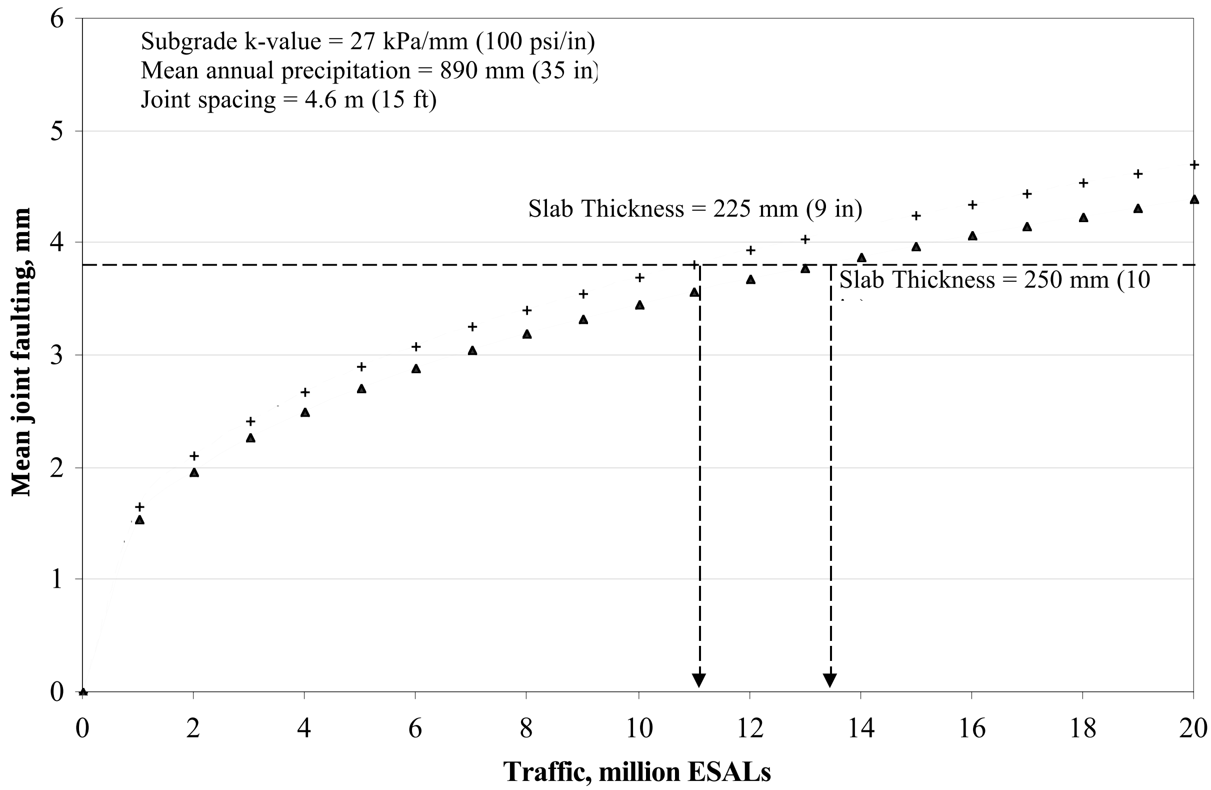

Figure 31. Effect of slab thickness on nondoweled JCP faulting after diamond grinding.

Dowel bar retrofitting involves cutting slots into the concrete across the joints or cracks, placing dowel bars in the slots, and backfilling with a patching material (usually concrete). Retrofitted dowel bars improve faulting performance by providing significantly better load transfer capacity at transverse joints and cracks. Dowel bar retrofitting followed by diamond grinding results in a smooth pavement with good load transfer capabilities and good faulting performance. As with diamond grinding, the placement of retrofit dowels in one lane does not require retrofitting the adjacent lane, offering a significant cost advantage over an overlay option.

Pierce (1995) studied the effectiveness of retrofit dowel bars in the State of Washington. Different rehabilitation strategies were performed on four pavement sections on I-90 (230-mm [9-in.] JPCP). Prior to rehabilitation, the average faulting in the four sections was 8 mm (0.30 in.). Seven months and approximately 700,000 ESALs after the rehabilitation, the diamond-ground section (without any other concurrent CPR) had an average faulting of 1.8 mm (0.07 in.). The section provided with retrofitted tied shoulders and grinding had an average faulting of 0.6 mm (0.02 in.). The two sections that were retrofitted with dowel bars and ground had an average faulting of 0.1 mm (<0.01 in.). Thus, the retrofitted dowel bars were highly effective in minimizing redevelopment of faulting on this project.

Dowel bar retrofitting was also found very cost-effective in Washington State. The cost effectiveness of dowel bar retrofitting was evaluated by comparing the costs of the following rehabilitation alternatives (Pierce 1995):

- Dowel bar retrofitting with grinding.
- 1.2-m (4-ft) tied shoulders with grinding.
- 110-mm (4-in.) AC overlay.

The comparison was based on the rehabilitation of two 3.7-m (12-ft) lanes and 4.3 m (14 ft) of total shoulder width for the 53.1-km (33-mile) project. The comparison showed that the dowel bar retrofit with grinding alternative was up to three times as cost-effective as the AC overlay option, if the dowel bar retrofitting is applied only on the truck lane. Even if both lanes are retrofitted with dowels, the retrofit-dowel option is about 40 percent cheaper than the AC overlay option.

Doweled JCP

Figure 27 shows that doweled diamond-ground pavements have higher faulting than predicted for new doweled pavements. The reason for higher faulting may be that dowels had become loose from repeated traffic loading. However, it is important to note that the amount of faulting did not reach critical levels in any of the sections. Most doweled diamond-ground pavements surveyed had average faulting less than 2.5 mm (0.1 in.) and all pavements had average faulting less than 4 mm (0.15 in.).

A faulting model could not be developed for doweled diamond-ground pavements because of high variability in faulting performance. There may be numerous reasons for the high variability. Some of the sections may have developed excessive faulting because of inadequately sized dowels, loose dowels, or excessive joint opening. It is also possible that excessive faulting was not the reason for the grinding. The pavement may have been ground to improve smoothness after full-depth repairs, to level out faulted cracks, or to improve friction. Despite the variable performance, none of the doweled diamond-ground pavements developed the level of faulting that requires a corrective action. The equation of the trend line shown in Figure 27 is given by:

$$FAULT = 0.094 \cdot n + 0.97 \qquad (9)$$

$R^2 = 31\%$
$N = 43$

where:

FAULT = Mean transverse joint faulting, mm.
n = Cumulative number of applied traffic, million ESALs.

The trend line indicates that faulting in diamond-ground doweled JCP is only 2.9 mm (0.11 in.) after 20 million ESALs, well below the level of faulting that requires corrective action (typically 3.8 mm [0.15 in.]).

Effect of Edgedrains on Faulting Performance

The faulting model developed for the 1986 FHWA study showed a significant benefit of edgedrains on faulting of nondoweled pavements. The field survey for that study was conducted in 1985 and 1986, an average of 3 years after the pavements were retrofitted with edgedrains. On average, the ground sections with edgedrains were faulted about 50 percent less than the sections without edgedrains.

When the sections were visited in 1997, no significant difference in faulting performance could be observed between the edgedrained and nondrained sections. The edgedrained sections were faulted about the same as or slightly more than the nondrained sections. The analysis conducted for the development of the faulting model (for nondoweled JCP) also did not show the presence of edgedrains to be a significant factor affecting faulting performance of diamond-ground pavements. The suspected cause of this discrepancy (between the 1986 and 1997 observations) is clogged edgedrains.

Proper construction and maintenance are critical for obtaining the benefits associated with retrofitted edgedrains. Nonfunctional edgedrains can adversely affect pavement performance, rather than providing any benefit. An edgedrain that is practically never maintained becomes almost totally inoperative after about 10 years (Christory 1990). The average age of the edgedrains in the pavement sections included in the 1997 survey was more than 7 years. Video inspections show that only about 30 percent of edgedrains in in-service pavements are fully functional (Sawyer 1995; Daleiden 1998). The leading cause of this problem is poor maintenance, but improper design and construction can also be contributing factors (ERES 1998).

SURFACE TEXTURE

Although the primary reason for diamond grinding is to reduce roughness, grinding also improves skid resistance by increasing surface macrotexture. Pavement macrotexture is defined as the amplitude of deviations of a pavement surface with wavelengths from 0.5 to 50 mm (0.02 to 2.00 in.)(Wu and Nagi 1995). The increased macrotexture provides for improved drainage of water at the tire–pavement interface, especially for worn tires, thus reducing the potential for wet weather accidents.

A consequence of increased macrotexture is improved friction characteristics of the pavement surface. Immediately after grinding, the locked-wheel longitudinal friction number (skid number) of a pavement measured using ASTM E274 increases dramatically. One study showed an increase in average friction number from 42 before grinding to 80 after the grinding at 5 projects in the United States (Mosher 1985). Some studies have indicated that the improvement in skid numbers may be temporary, particularly if the pavement contains aggregate susceptible to polishing. In Georgia, Tyner (1981) observed that although these values decrease over the first few years, an adequate macrotexture would normally be maintained for many years.

Many studies have indicated a strong relationship and excellent correlation between pavement surface texture measured using the sand patch test and skid numbers measured using smooth tires (Henry and Saito 1983; Ardani 1996). The sand-patch test involves spreading a known volume of material (uniformly graded round glass beads) on a clean and dry pavement surface and measuring the area covered. The material spread on the surface completely fills the surface voids to the tips of the surface aggregate particles. The area covered by the material is related to the surface macrotexture, and the average depth between the bottom of the pavement surface and the tops of surface aggregate particles (the average macrotexture depth) can be determined from the known volume and the measured area. Figures 32 and 33 show the sand patch for typical ground and nonground pavements.

Mean texture depths using the sand-patch test were measured for several diamond-ground pavements across the United States. For consistency, one operator obtained all the measurements, and ASTM E965 procedures were followed meticulously to achieve reliable results. All data were collected in the fall of 1997.

Macrotexture Database

Sand-patch tests were conducted in the wheelpaths of 41 in-service diamond-ground pavement sections in 15 states. Five of these sections were not included in the analysis due to scaling and map cracking. Figure 34 shows the climatic region distribution for the 36 remaining sections. These 36 sections are distributed equally between the freeze and the non-freeze climatic region. Figure 35 shows the number of sections in each state where macrotexture data were collected. Table 4 shows the minimum, maximum, and average values of traffic and climatic parameters for these 36 sections. The age distribution, accumulated vehicle-pass distribution, and two-way ADT distribution are shown in Figures 36, 37, and 38, respectively. Representative sections from different climatic regions and different levels of traffic were included in the analysis.

Historical information regarding aggregate hardness or source of aggregate could not be obtained for many of the sections. However, diamond grinding of hard aggregates is typically performed with blade spacings between 180 and 197 blades per m (55 and 60 blades per ft). Diamond grinding of soft aggregates is typically performed with blade spacings between 171 and 184 blades per m (52 and 56 blades per ft) (Mosher 1985). ACPA (1990) recommends using 174 to 187 blades per m (53 to 57 blades per ft) for typical hard aggregates and 164 to 177 blades per m (50 to 54 blades per ft) for typical soft aggregates. Therefore, blade spacing was used to represent aggregate hardness in this analysis. It is important to note that the correlation of blade spacing with aggregate hardness is a very general approximation. Many agencies specify tighter blade spacing primarily to reduce "steering sensation" in light vehicles. Figure 39 shows the distribution of blade spacing for the 36 sections.

The mean texture depth for the 36 sections varied from a low value of 0.31 mm (0.012 in.) to a high value of 1.67 mm (0.066 in.) for a newly ground pavement. The average macrotexture value for the 36 pavements was 0.62 mm (0.024 in.), with an average age since grinding of 7.36 years. Figures 40, 41, and 42 show the measured mean texture depth versus age since grinding, ESALs since grinding, and cumulative vehicle passes since grinding, respectively, sorted by climatic

Table 4. Range of Factors for Macrotexture Survey Sections

	Minimum	Maximum	Average
Pavement age (yrs)	16	51	31.3
Age since grinding (yrs)	0	16	7.36
Two-way average daily traffic (ADT)	2,450	158,000	24,900
ESALs since grinding (million)	0	24.3	6.6
Vehicle passes since grinding (million)	0	165	21.5
Truck passes since grinding (million)	0	23.2	4.4
Slab thickness, mm (in.)	178 (7)	254 (10)	231 (9.1)
Latitude, degree	30	46	37.8
Freezing index, ^{0}C-days (^{0}F-days)	0 (0)	1167 (2100)	236 (425)
Average annual minimum temperature, ^{0}C (^{0}F)	-2.1 (28.2)	13.7 (56.7)	6.2 (43.1)
Average annual maximum temperature, ^{0}C (^{0}F)	10.2 (50.4)	31.3 (88.4)	19.8 (67.6)
Average annual temperature, ^{0}C (^{0}F)	4.1 (39.3)	22.2 (72.0)	13.0 (55.4)
Average annual precipitation, mm (in.)	69 (2.71)	1609 (63.35)	904 (35.61)
Blade spacing, blades per m (blades per ft)	161 (49)	200 (61)	178 (54.2)

Figure 32. Sand-patch measurement for nonground pavement.

Figure 33. Sand-patch measurement for newly ground pavement.

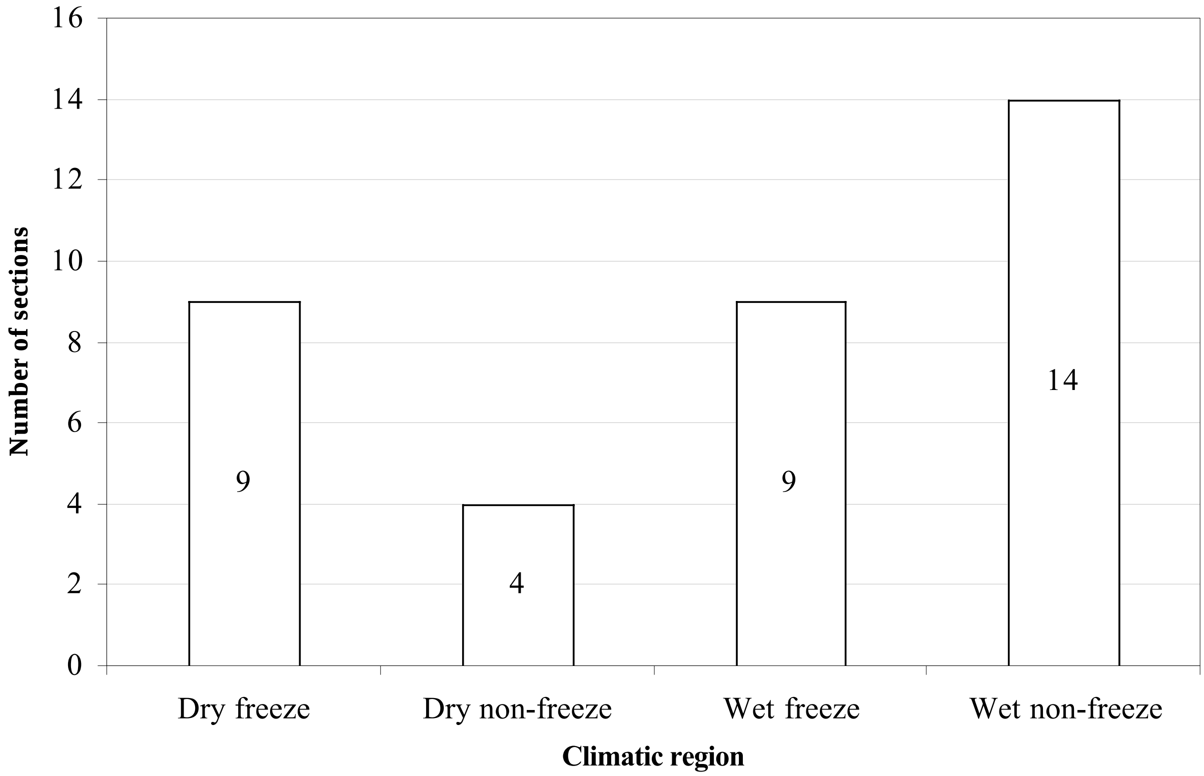

Figure 34. Climatic region distribution for macrotexture survey sections.

Figure 35. Distribution of diamond-ground macrotexture survey sections across the United States.

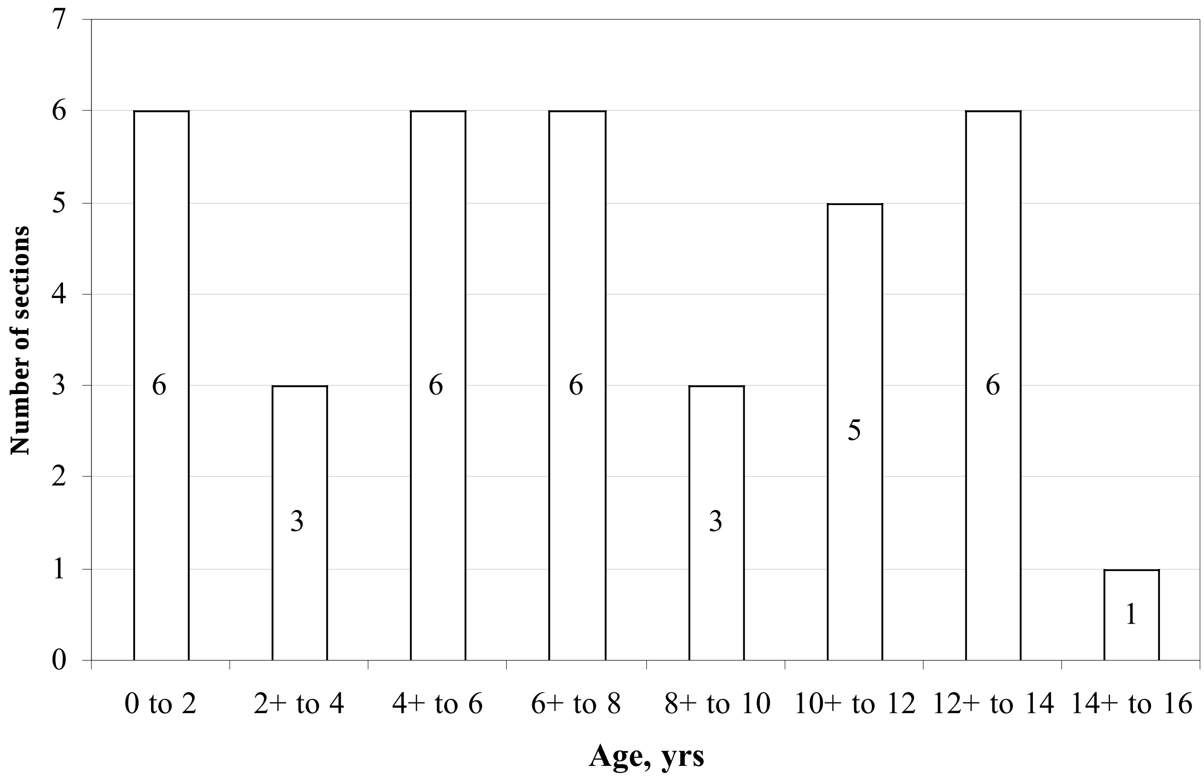

Figure 36. Distribution of age since grinding for macrotexture survey sections.

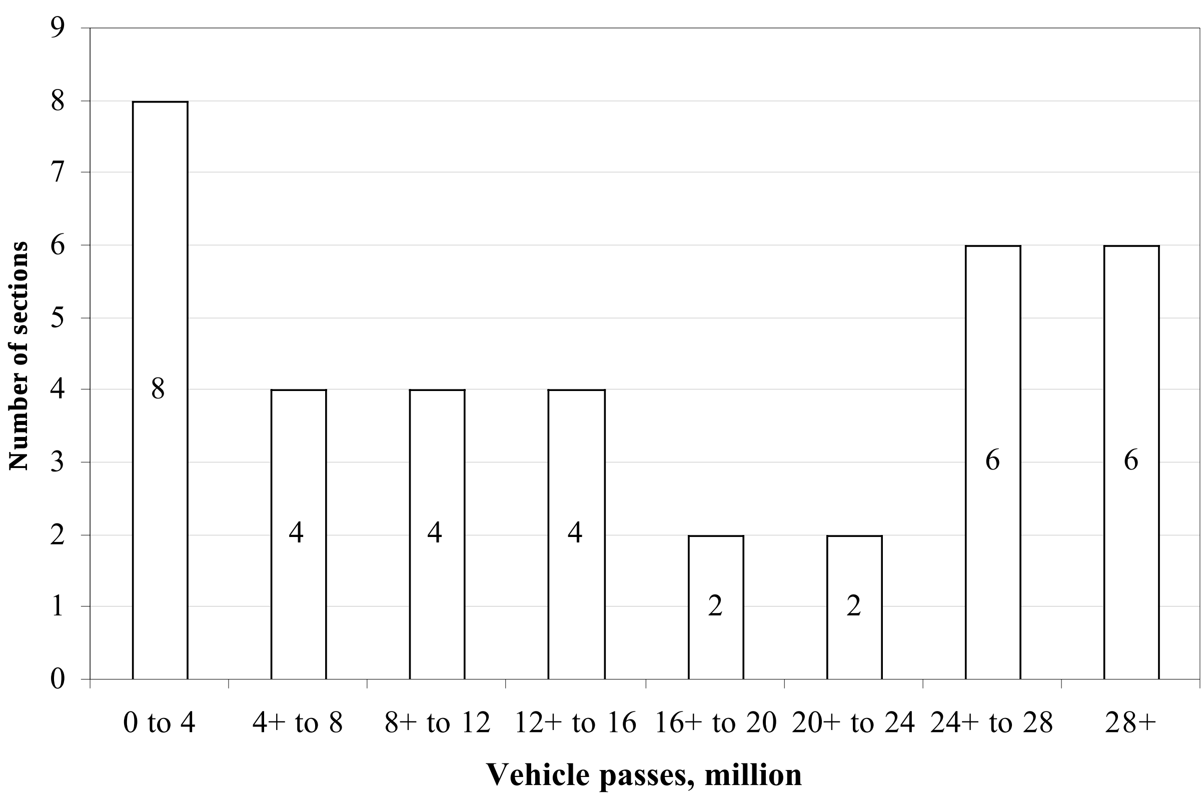

Figure 37. Distribution of accumulated vehicle passes since grinding for macrotexture survey sections.

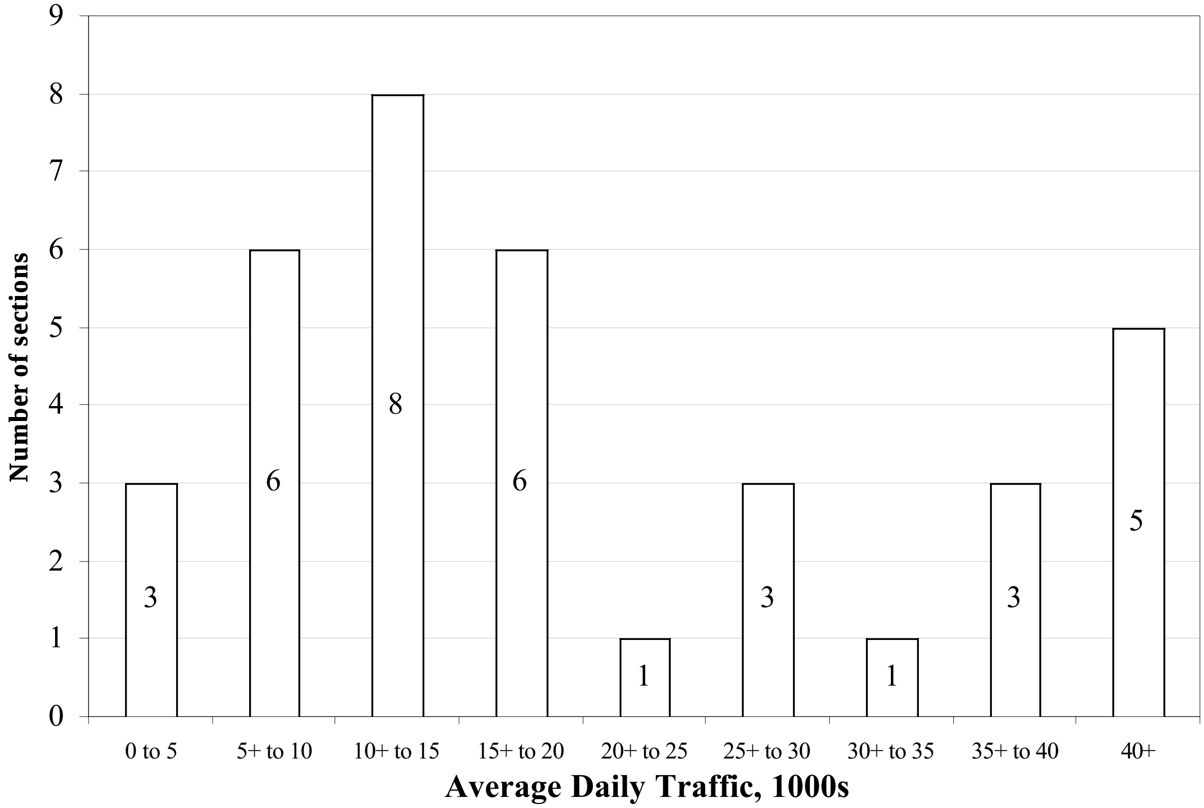

Figure 38. Two-way average daily traffic distribution for macrotexture survey sections.

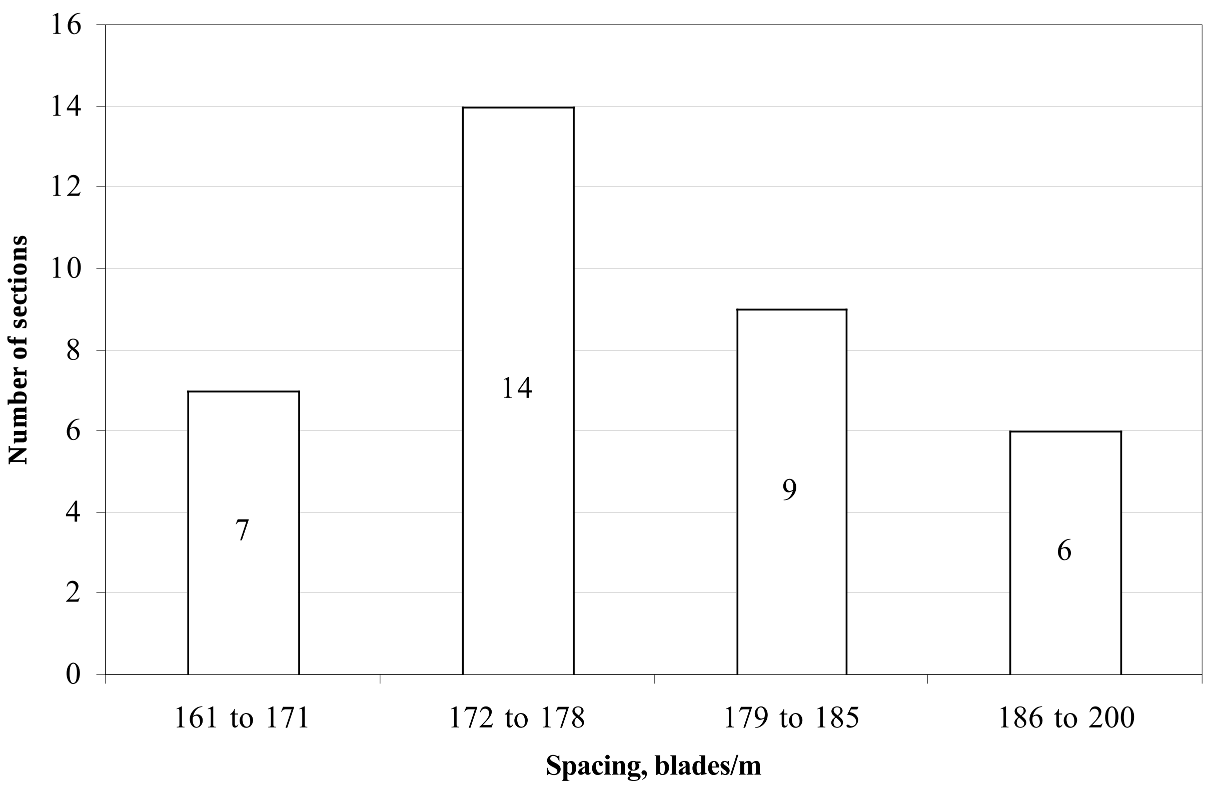

Figure 39. Blade spacing distribution for macrotexture survey sections.

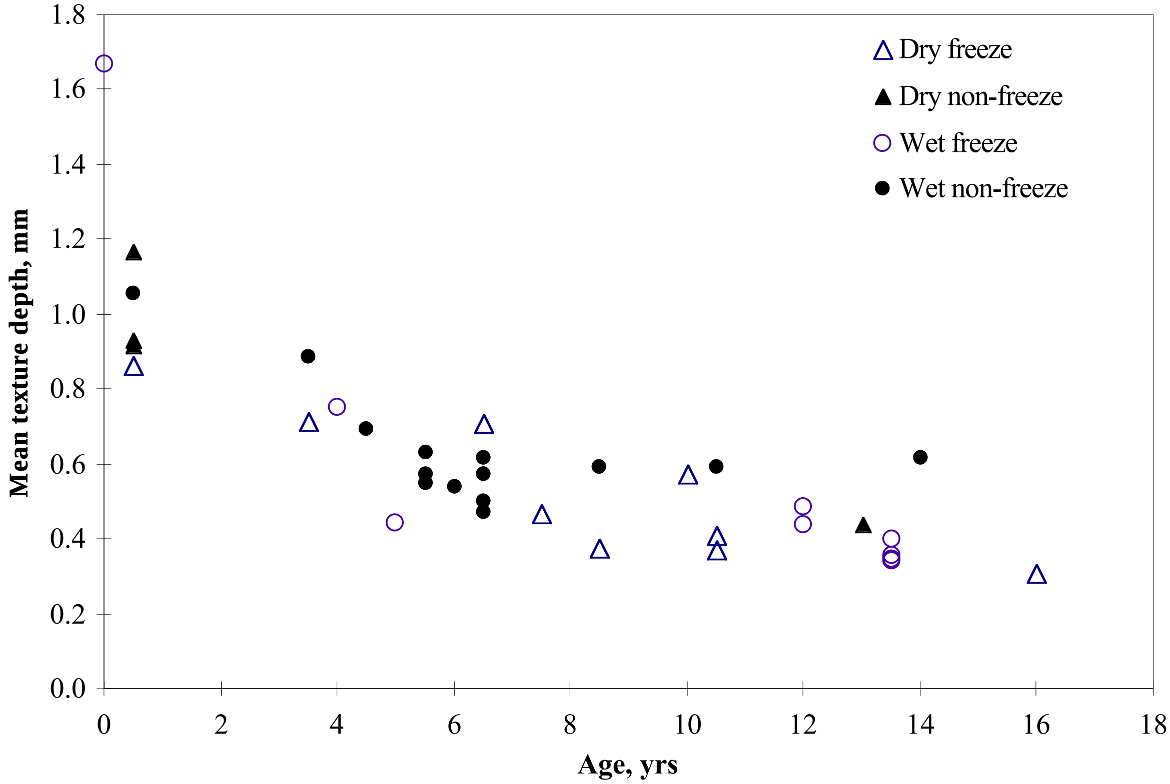

Figure 40. Mean texture depth vs. age since grinding, sorted by climatic region.

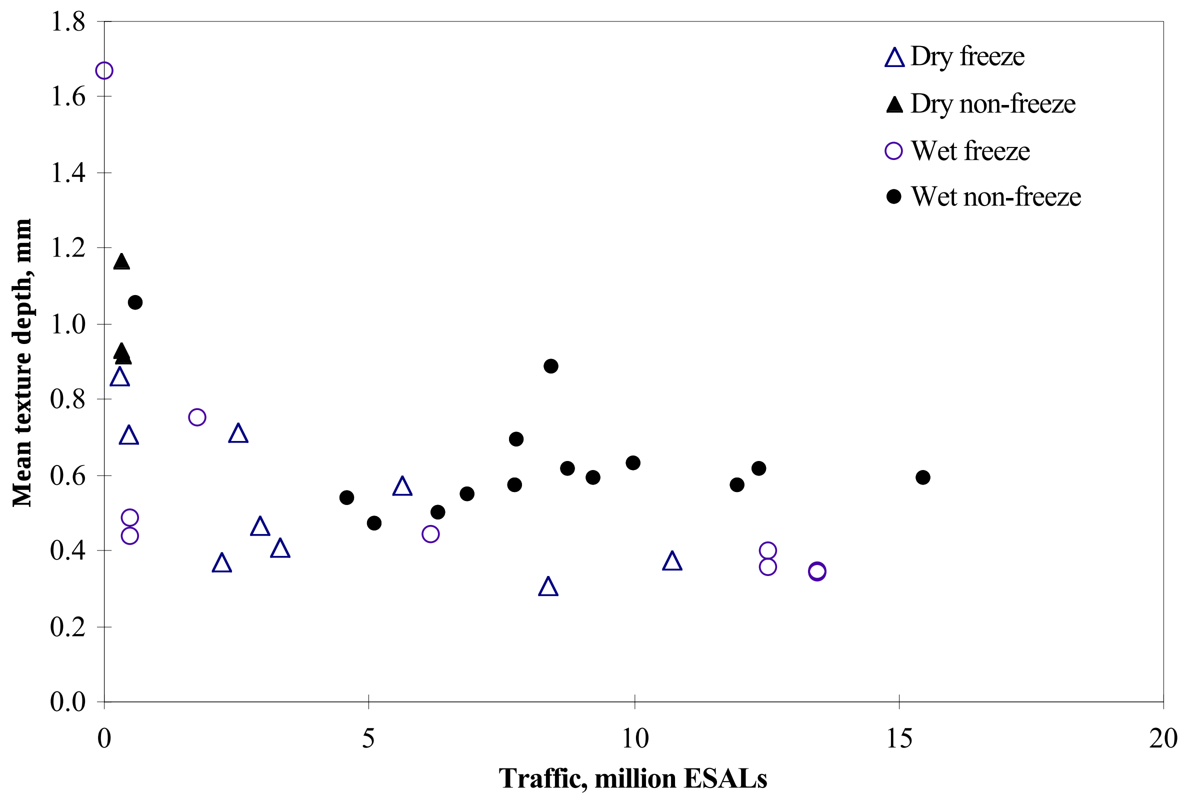

Figure 41. Mean texture depth vs. traffic since grinding, sorted by climatic region.

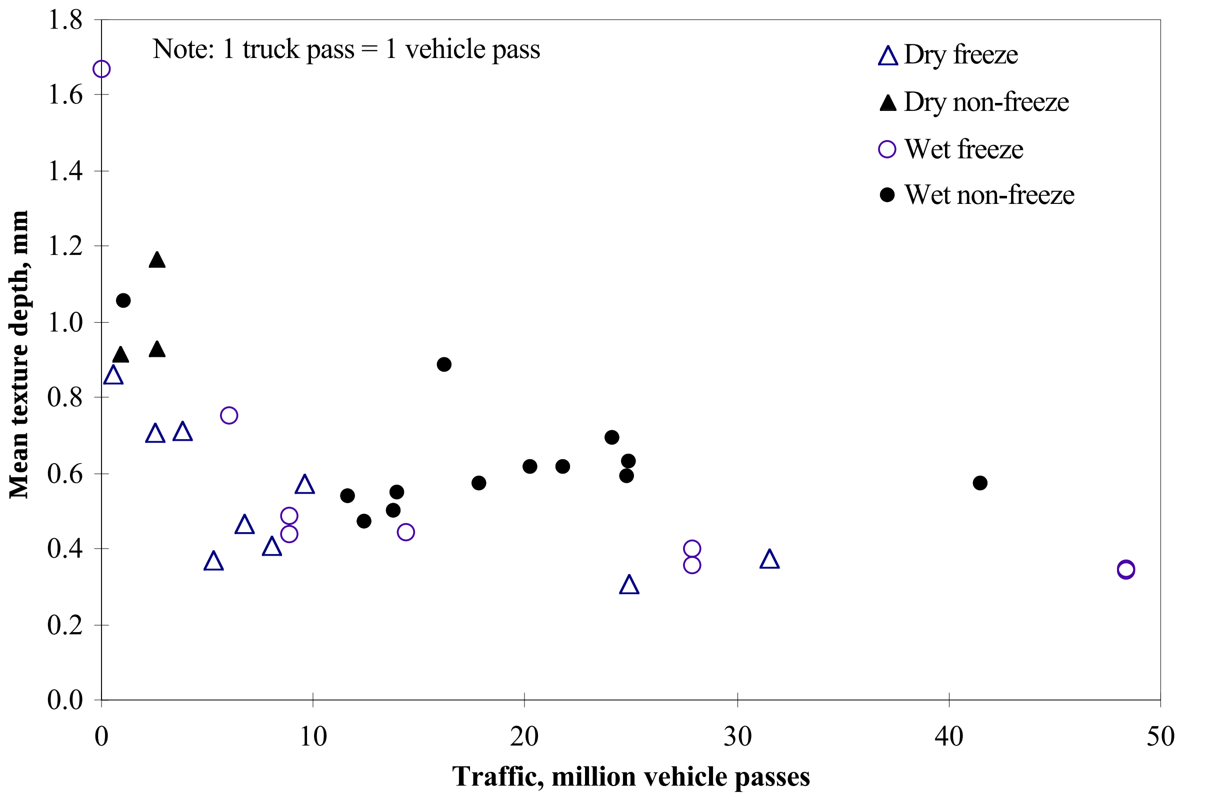

Figure 42. Mean texture depth vs. accumulated vehicle passes since grinding, sorted by climatic region.

region. In a study conducted by Georgia DOT, Gulden and Thornton (1985) considered one truck pass to be equivalent to eight vehicle passes with respect to macrotexture wear. Figure 43 shows the measured mean texture depth versus cumulative vehicle passes since grinding where one truck pass is equivalent to eight vehicle passes. The measured mean texture depth, sorted by aggregate hardness, is shown in Figure 44. For this analysis, it was assumed that blade spacing less than or equal to 177 blades per m (54 blades per ft) corresponds to soft aggregates and blade spacing greater than or equal to 180 blades per m (55 blades per ft) corresponds to hard aggregates.

Macrotexture Model

Figures 40 through 44 suggest that mean texture depth is strongly dependent on age since grinding (and, to a lesser extent, traffic since grinding). The mean texture depth for diamond-ground pavements is also evenly distributed between wet and dry climatic regions. However, many diamond-ground pavements in both the wet freeze and dry freeze climatic regions seem to have lower macrotexture than those in the non-freeze climatic regions, especially for pavements with grinding more than 8 years old. The use of winter maintenance equipment such as snowplows could be responsible for the increased rate of wear in surface texture in the freeze regions.

The following factors were considered in developing the macrotexture model:

- Age since grinding.
- ESALs since grinding.
- Vehicle passes since grinding.
- Truck passes since grinding.
- Geographical latitude of pavement section.
- Average annual temperature.
- Average annual maximum temperature.
- Average annual minimum temperature.
- Average annual precipitation.
- Freezing index.
- Dummy variable for freeze climatic region (0 = wet non-freeze or dry non-freeze, 1 = wet-freeze or dry-freeze).
- Blade spacing.
- Dummy variable for blade spacing (0 = blade spacing £178 blades/m [54 blades/ft], 1 = blade spacing ≥179 blades/m [55 blades/ft]).

The traffic variables are correlated with each other and with age since grinding; therefore, only one of these variables was required for the regression model. The age since grinding was most strongly correlated

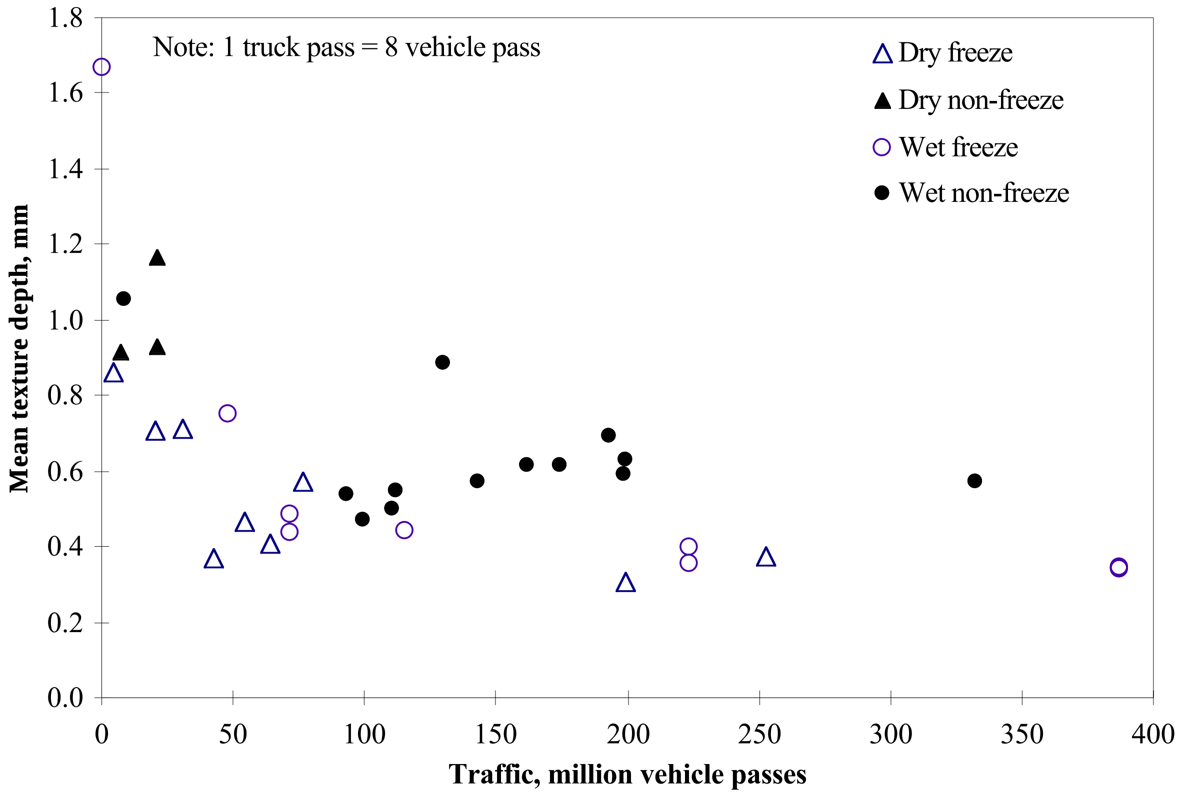

Figure 43. Mean texture depth vs. accumulated vehicle passes since grinding, sorted by climatic region (one truck pass = eight vehicle passes).

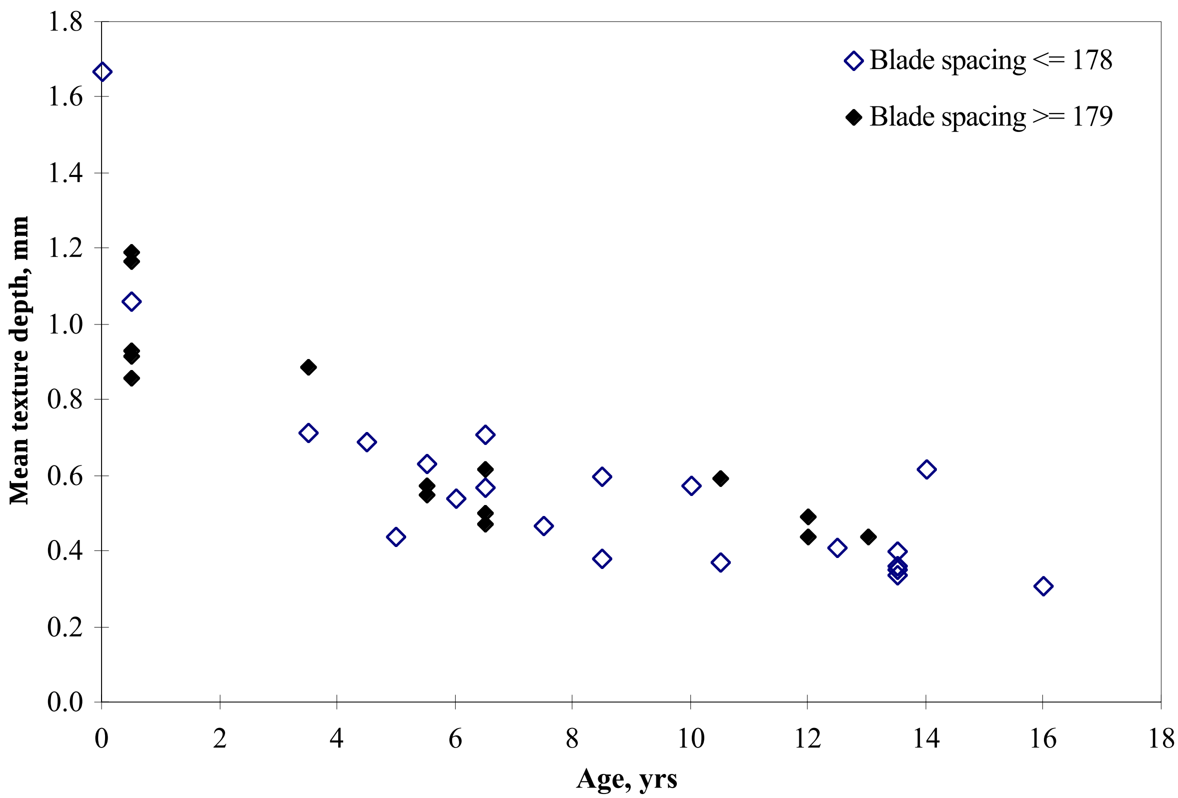

Figure 44. Mean texture depth vs. age since grinding, sorted by blade spacing.

with the mean texture depth and was therefore chosen to develop the model. Addition of traffic variables to the macrotexture model was not found to be significant, so these variables were not included in the final model. The correlation factors of most climatic variables and blade spacing variables were small, so these variables also were not included in the final model. The dummy variable for wet freeze and dry freeze climatic regions was found to improve the model goodness-of-fit. The final regression model is given by:

MTD = -0.152 (1 = 0.233 FREEZE) in (AGE) + 0.887 (10)

R^2 = 0.833

SEE = 0.098 mm (0.0038 in.)

N = 35 (the freshly ground pavement with zero age was not used in developing the regression equation)

where:

MTD = Mean texture depth, mm.

AGE = Age since grinding, years (0.5 to 16 years).

FREEZE = Dummy variable for freeze climatic region. (0 = wet non-freeze or dry non-freeze, 1 = wet freeze or dry freeze)

The strong correlation with age rather than traffic may be because factors other than traffic have a strong influence on deterioration of the grinding texture. The other factors may include the use of snowplows, abrasion from ice in the freeze regions, and erosion by surface runoff of water in the non-freeze regions. Water can be shot out at very high velocities by moving traffic, resulting in "water blasting" type erosion. During field surveys, only about one in four sections exhibited any observable difference in macrotexture between wheelpath and non-wheelpath, suggesting that the influence of the factors other than traffic may be significant.

Because the database included sections with traffic levels ranging from two-way ADT less than 5,000 to ADT greater than 40,000 vehicles per day, the above regression equation is valid at all levels of traffic. No correlation was observed between blade spacing and mean texture depths. If, as assumed, blade spacing is related to aggregate hardness, then blade spacing is expected to be a factor affecting mean texture depth. The following factors may contribute to the poor correlation:

- The lower blade spacing (£178 blades/m [54 blades/ft]) used to grind pavements with soft aggregates increases the land area and reduces wear, but softer aggregates are inherently more susceptible to wear. The reduced blade spacing compensates for the wear of soft aggregate pavement macrotexture.
- Macrotexture wear on a concrete pavement occurs on both the aggregate and the mortar. Although the rate of wear on aggregates is a function of aggregate hardness and blade spacing, the same is not true for the exposed mortar.
- Correlation of blade spacing with aggregate hardness is a very general approximation. Detailed analysis may show a greater effect of aggregate hardness on macrotexture survival.

Figure 45 shows a plot of predicted versus actual mean texture depth for the 35 pavements used to develop the model. Figure 46 shows a plot of the residuals for the regression equation. The plot is evenly distributed above and below the zero residual line, which implies that the regression equation is not biased. Figure 47 shows the variation of predicted macrotexture with age since grinding. A newly ground pavement has high macrotexture, which is typically greater than 1.65 mm (0.065 in.). Within the first 6 months after the ground pavement is opened to traffic, most of the ridges created by grinding are broken and the macrotexture falls to approximately 1.02 mm (0.04 in.). This is the starting point for the regression model. Within the first 2 to 2.5 years, the pavement macrotexture falls to 0.76 mm (0.03 in.). The rate of change of macrotexture decreases with time, and 8 years after grinding, the pavement in the freeze regions reaches a macrotexture of 0.51 mm (0.02 in.). This level is reached in the non-freeze climatic regions after 12 years. After 12 years, pavement surfaces in the freeze climatic regions have a mean texture depth of less than 0.4 mm (0.016 in.).

The mean texture depth of pavements in the freeze regions appears to be less than that of non-ground pavements. The diamond blades of the grinding machine tend to polish and level out the original macrotexture in the grooves, and the ridges eventually wear off with age, traffic, and winter maintenance equipment. This is especially true for soft aggregates that are susceptible to polishing. However, it is also possible for the original macrotexture to wear off after construction and reach low levels. Mean texture depth measured on non-textured PCC pavements in Pennsylvania varied from 0.33 to 0.51 mm (0.013 to 0.020 in.) 5 to 28 years after construction (Henry 1980).

Skid Number

A relationship between skid number and mean texture depth developed by Henry and Wambold (1992) was used to develop plots correlating macrotexture depth and skid number. Skid number, as measured with the blank tire, is a function of both microtexture

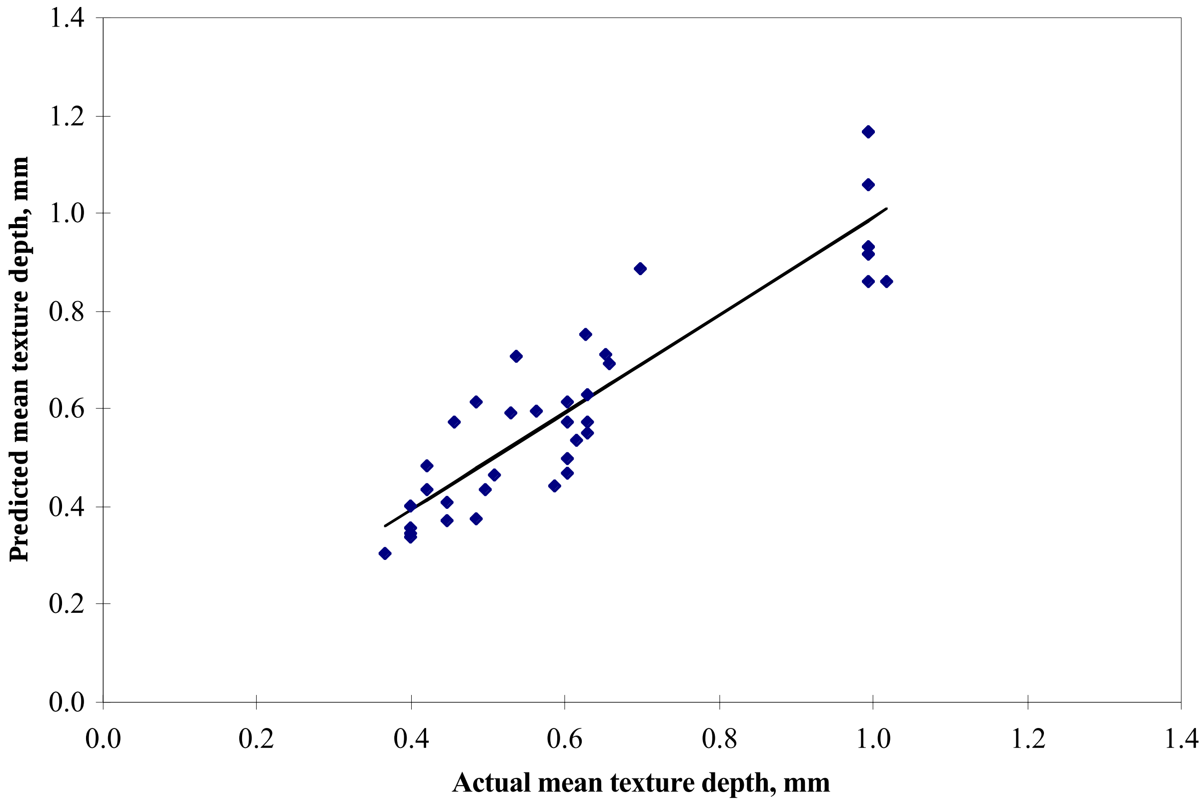

Figure 45. Plot of predicted vs. actual mean texture depth for the macrotexture model.

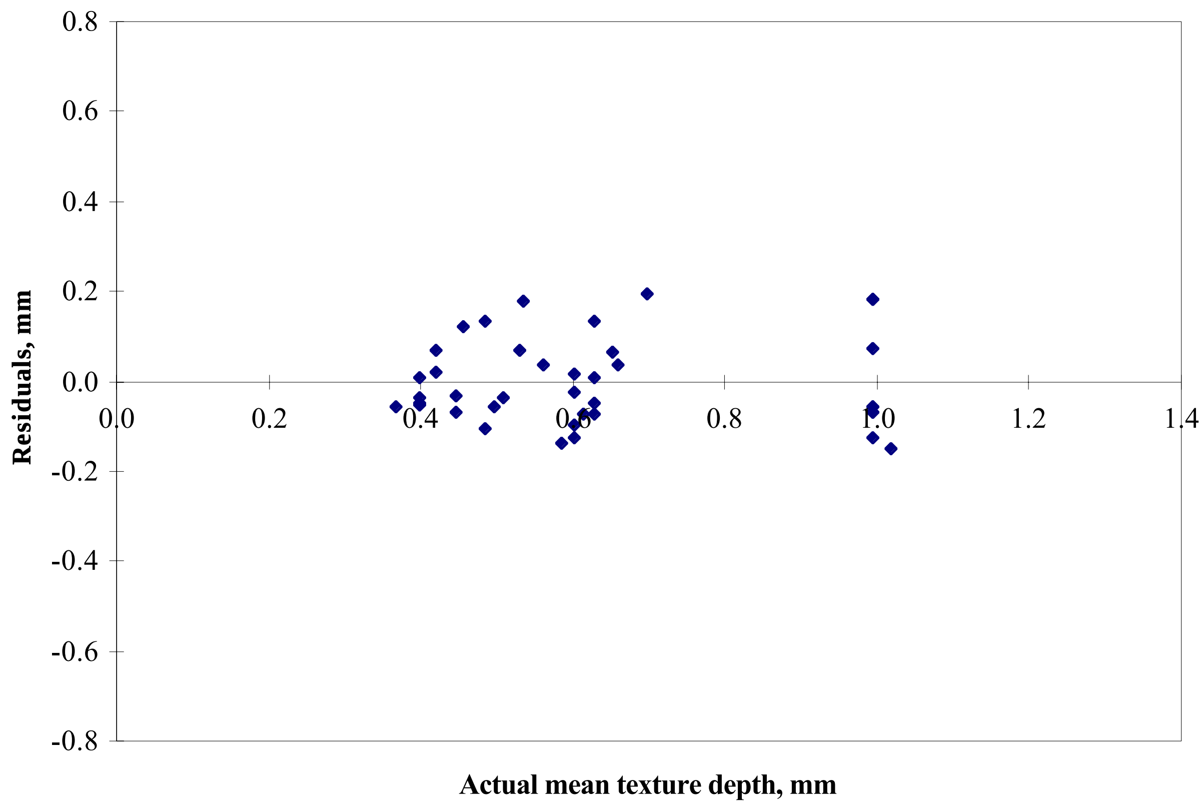

Figure 46. Residual plot for the macrotexture model.

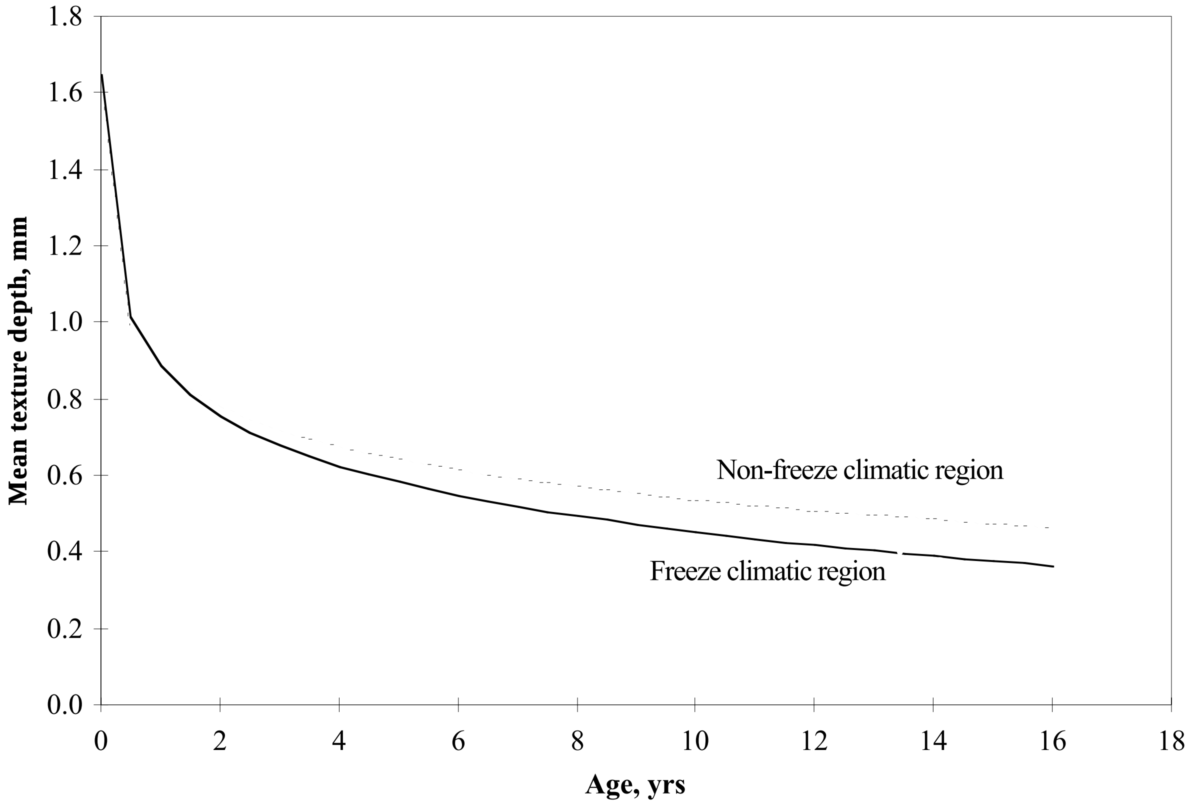

Figure 47. Change in PCC surface macrotexture after diamond grinding for pavements in freeze and non-freeze climatic regions.

and macrotexture. Microtexture is defined as the amplitudes of deviations from the plane with wavelengths less than or equal to 0.5 mm (0.02 in.) (Wu and Nagi 1995).

The microtexture of pavement surfaces is high immediately after initial construction. Over time, the microtexture wears off and the rate of wear of microtexture decreases. Grady and Chamberlin (1981) reported that new pavements wore rapidly but stabilized after about 2 million passes of traffic. Diamond grinding is typically performed on pavements that have aged and, consequently, have lower microtexture. The grinding process increases pavement microtexture, particularly on the land area, by weakening and dislodging sand particles in the mortar matrix, which are then vacuumed along with the slurry by the grinder. The high microtexture and high macrotexture immediately after grinding result in high skid numbers (Mosher 1985). Under the influence of weather and traffic, the increased microtexture may be expected to decrease rapidly and return to the levels exhibited prior to grinding within 1 or 2 years, as is the case of new pavements.

In the following analysis, to a first order of approximation, it is assumed that the pavement microtexture as characterized by the British Pendulum Number (BPN) does not change after grinding. For the first few years after grinding, this is a conservative assumption because the BPN can be expected to be higher after grinding than before grinding. The increased BPN is assumed to decrease rapidly and stabilize after the first few years. The skid number measured in the fall season at 64 km / hr (40 mph) using a smooth tire before grinding is given by:

$$SN40S_i = -20.5 + 0.65\ BPN + 16.4\ MTD_i \qquad (11)$$

$$SN40S_f = -20.5 + 0.65\ BPN + 16.4\ MTD_f \qquad (12)$$

where:

MTD	=	Mean texture depth, mm.
BPN	=	British Pendulum Number.
SN40S	=	Skid number measured at 64 km / hr (40 mph) using smooth tire.
Suffix i	=	Before grinding.
Suffix f	=	After grinding.

Therefore, the change in skid number after diamond grinding can be expressed as follows:

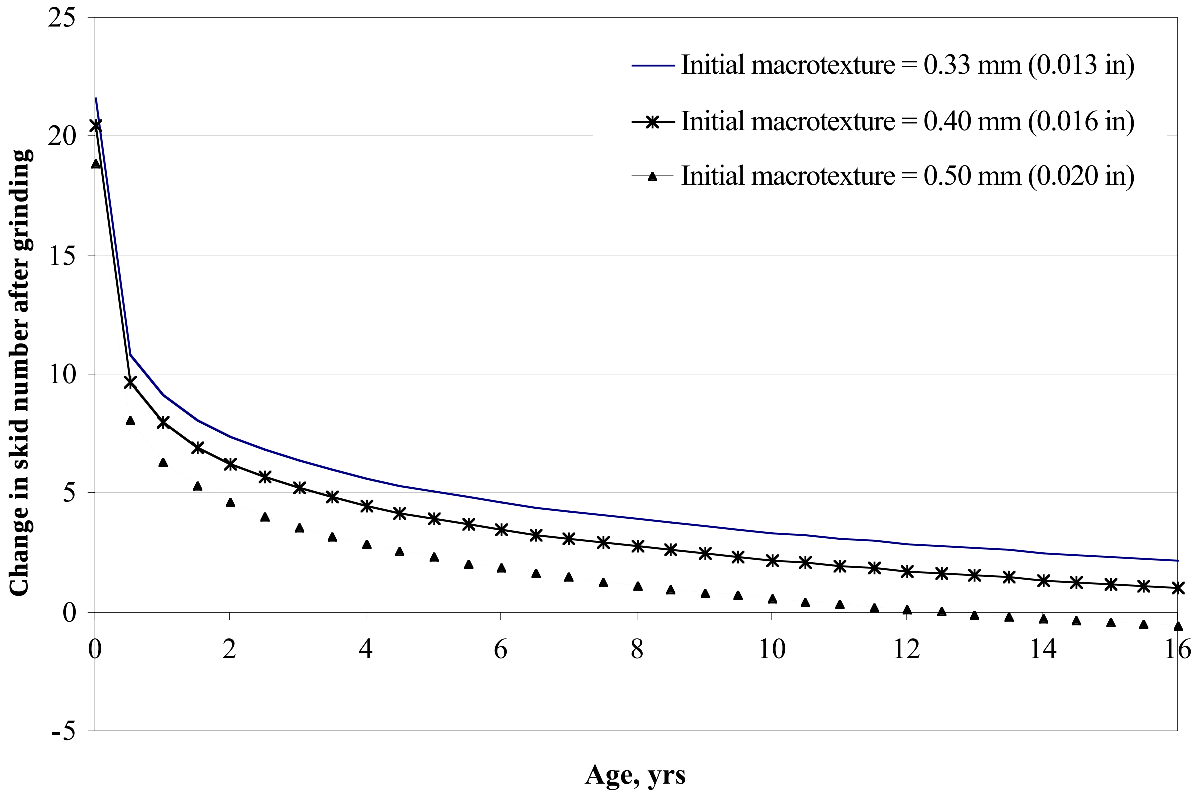

Figure 48. Change in estimated SN40 after grinding for pavements in the non-freeze region.

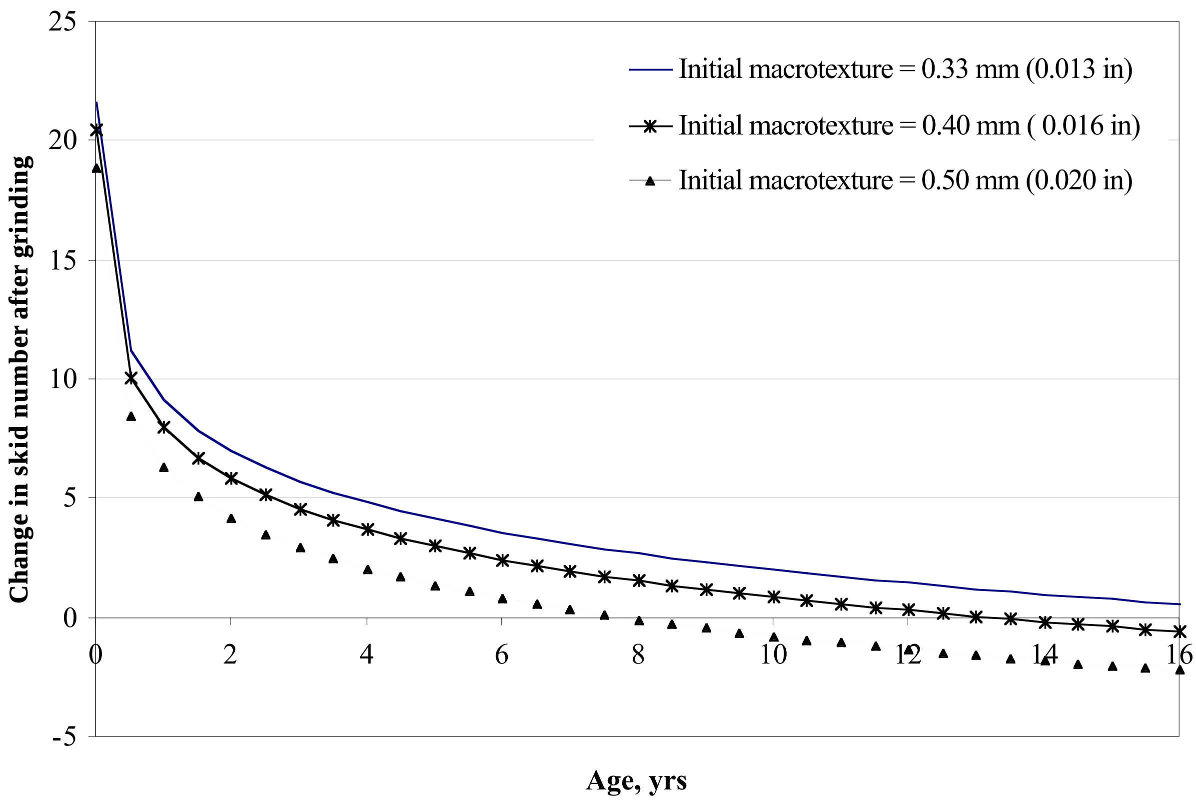

Figure 49. Change in estimated SN40 after grinding for pavements in the freeze region.

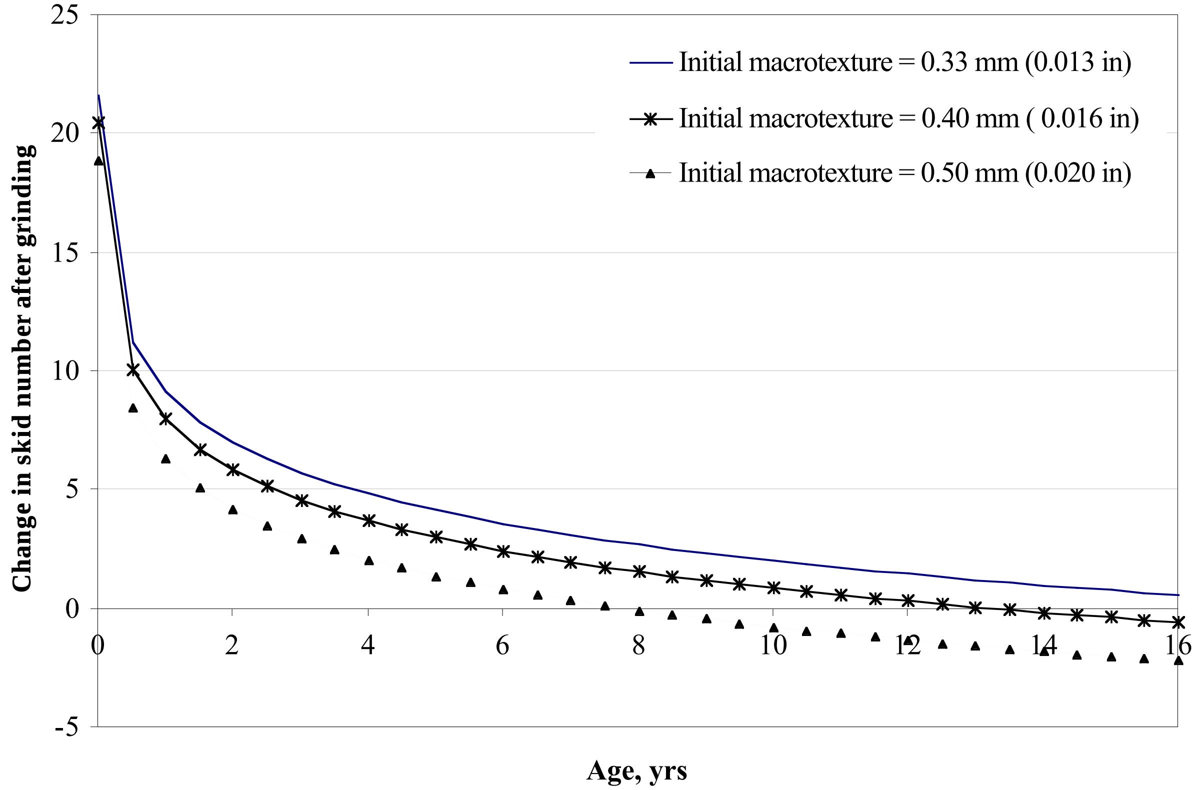

Figure 50. Change in SN50 after grinding for Highbury Avenue in City of London, Canada (Bradbury and Kazmierowski 1997).

$$\Delta SN40S = SN40S_f - SN40S_i = 16.4\ (MTD_f - MTD_i) \quad (13)$$

$$\Delta SN40S = 16.4\ \{[-0.152\ (1 + 0.233\ FREEZE)\ \ln(AGE) + 0.887] - MTD_i\} \quad (14)$$

where:

MTD_i = Mean texture depth before grinding, mm.
AGE = Age since grinding, years (0.5 to 16 years).
FREEZE = Dummy variable for freeze climatic region (0 = wet non-freeze or dry non-freeze, 1 = wet freeze or dry freeze).

Note that the equation 14 does not consider the change in BPN over time because it is assumed to stabilize and remain fairly constant 2 to 16 years after grinding. Between 0 to 2 years, the equation and figures are conservative because BPN values are higher after grinding than before grinding. Figure 48 shows the change in skid number after diamond grinding for three levels of macrotexture before grinding for the wet freeze and dry freeze climatic regions. Figure 49 shows the same for the wet non-freeze and the dry non-freeze climatic regions.

Using the above model, the improvement in skid number immediately after grinding is approximately 20. This is lower than the improvement reported by Mosher (1985). However, it should be noted that the equation and figures are conservative for the first 2 years because they do not account for the higher microtexture. The high BPN numbers immediately after grinding may be partially responsible for the higher friction numbers. Also, the Henry and Wambold equation (used to calculate skid numbers using mean texture depths) may not be appropriate for pavements with rough textures, such as those of a newly ground pavement. The surface texture is high (typically around 1.65 mm [0.065 in]) on newly ground pavement because of unbroken ridges. These sharp, unbroken ridges may be responsible for an increase in coefficient of friction and, consequently, high friction numbers. The relationship may be more accurate after the ridges are broken and the texture becomes more uniform.

The improvement in skid numbers following grinding is consistent with observations made by Bradbury and Kazmierowski (1997) in Canada. Skid numbers were measured at 80 km / hr (50 mph) on a diamond-ground section of the northbound lane on Highbury Avenue in the City of London. Prior to grinding, the average skid number measured was 25. Following grinding, the average skid number increased to 48. Data were collected each year from November 1989 to July 1996. Figure 50 shows the difference in skid numbers following grinding for this section. It should be noted that the skid numbers were measured at 80

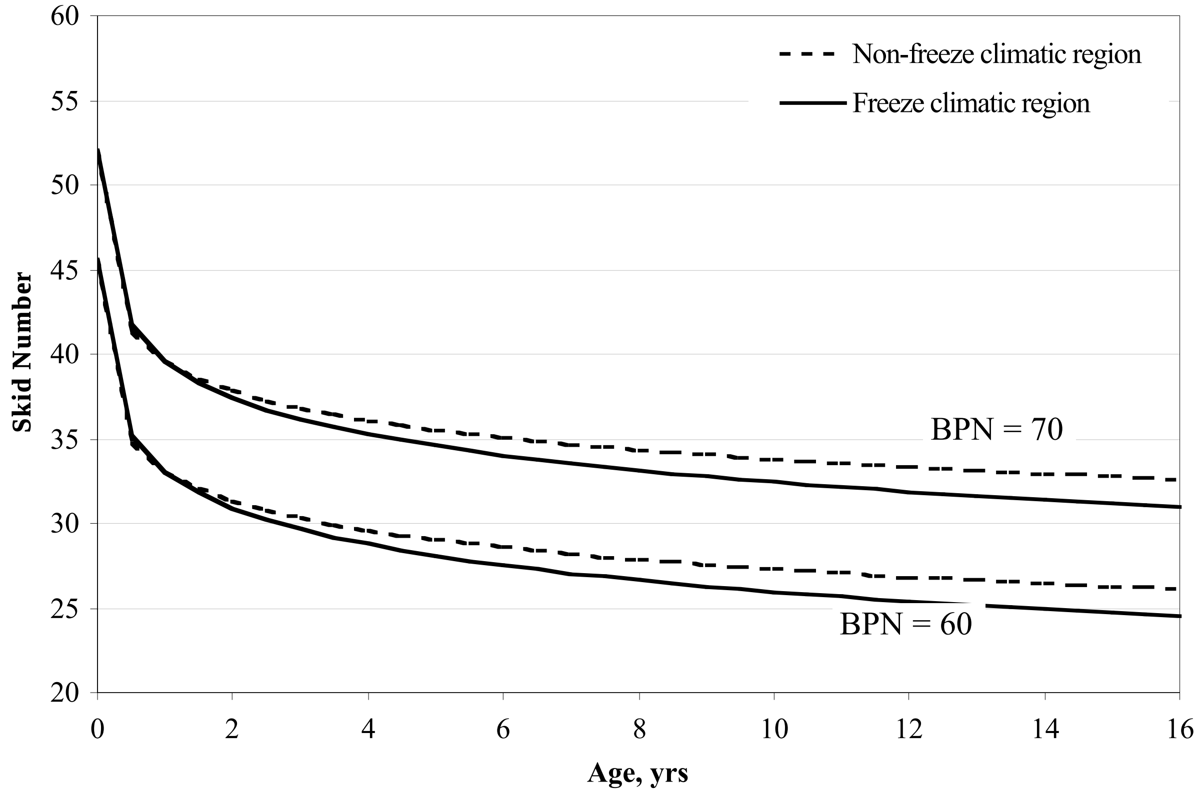

Figure 51. Predicted SN40 after grinding for two values of BPN.

km / hr (50 mph), whereas equations 13 and 14 and figures 48 and 49 are valid for skid numbers measured at 64 km / hr (40 mph). Nevertheless, the trend in figure 50 is consistent with the trend in Figure 49.

Knowing the BPN for a given pavement, the change in skid number of diamond-ground pavements over time can be calculated using equations 15 and 16. Since diamond grinding is primarily performed to reduce roughness, knowledge of the change in skid number over time for a given pavement can be used to decide whether concurrent grooving is required. This is particularly true for pavements with low microtexture or polished pavements in freeze climatic regions that exhibit faulting. For example, I-80 in Rapid City, South Dakota, was diamond ground in 1981 and transversely grooved in 1986. When this section was surveyed in 1997, the grooves had held up very well, even in the wheelpath. Figure 51 shows skid numbers as a function of age since grinding for BPN of 60 and 70 for both the freeze and the non-freeze climatic regions.

$$SN40S_f = -20.5 + 0.65\ BPN + 16.4\ MTD_f \qquad (15)$$

$$SN40S_f = -20.5 + 0.65\ BPN + 16.4[-0.152\ (1 + 0.233\ FREEZE)\ \ln(AGE) + 0.887] \qquad (16)$$

where:

AGE = Age since grinding, years (0.5 to 16 years).

FREEZE = Dummy variable for freeze climatic region (0 = wet non-freeze or dry non-freeze, 1 = wet freeze or dry freeze).

Note: For AGE = 0,use MTD_f = 1.65 mm (0.065 in) in equation 15.

Diamond Grinding and Accident Rates

The increased macrotexture of a diamond-ground pavement surface provides for improved drainage of water at the tire–pavement interface (especially for worn tires), thus reducing the potential for wet weather hydroplaning-related accidents. This is especially true for the first few years after grinding. The macrotexture wears off rapidly in the beginning, but significant texture remains for approximately 8 years for pavements in freeze areas and 12 years for pavements in non-freeze areas.

The longitudinal texture of grinding also helps to provide directional stability and reduce hydroplaning. This may be a very important factor controlling the accident rates, particularly 2 or more years after

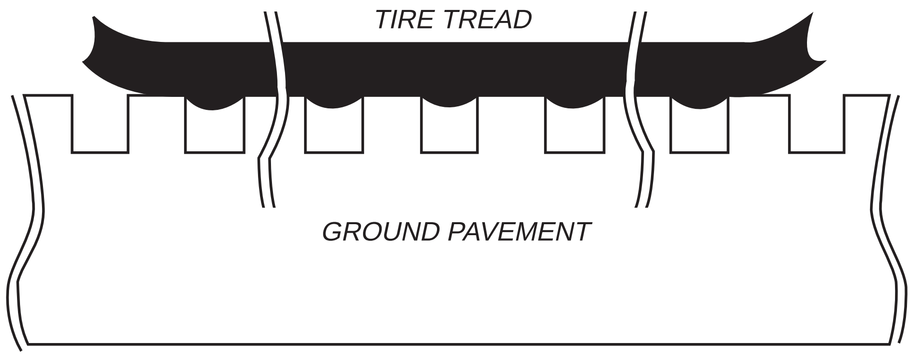

Figure 52. Mechanical interlock between tread rubber and grooves in a diamond-ground pavement.

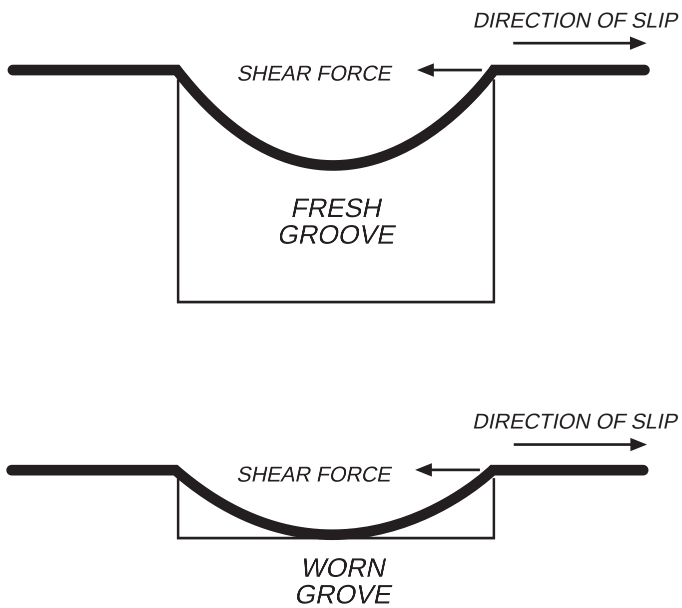

Figure 53. Shear force opposite to the direction of slip produced by tread rubber–pavement groove interaction.

grinding, but neither of these effects can be measured using skid number or macrotexture measurements. Studies have shown the benefits of longitudinal grooving on accident rates (Farnsworth 1971; Walters 1979). However, it is not known whether the increased macrotexture due to grooving or the longitudinal direction of grooving (causing an increase in lateral stability) is responsible for the reduction in accident rates on these pavements.

In Wisconsin, Drakopoulos et al. (1998) found that the overall accident rate for diamond-ground surfaces was only 60 percent of the rate for the nonground surfaces. The diamond-ground pavements provided significantly reduced accident rates up to 6 years after grinding, although a major portion of the diamond-ground texture wears off within the first 2 years of grinding. This suggests that, in addition to macrotexture depth, the direction of texture may be a significant factor affecting accident rate on diamond-ground pavements.

Tire tread–pavement groove interlocking. Under the action of a vehicle weight on a pneumatic rubber tire that is in contact with grooves in a pavement (such as those produced by diamond grinding), the tread rubber is extruded into the pavement grooves, as illustrated in Figure 52. A mechanical interlock is formed between the tread rubber extruded into the groove and the pavement (Horne). When the wheel starts to slip with respect to the pavement, during braking or yawed rolling, the tread rubber in the groove is deformed, producing a braking or cornering force, as illustrated in Figure 53. These shear forces that oppose the slip of the tire in the radial direction are independent of the adhesion-type shear forces

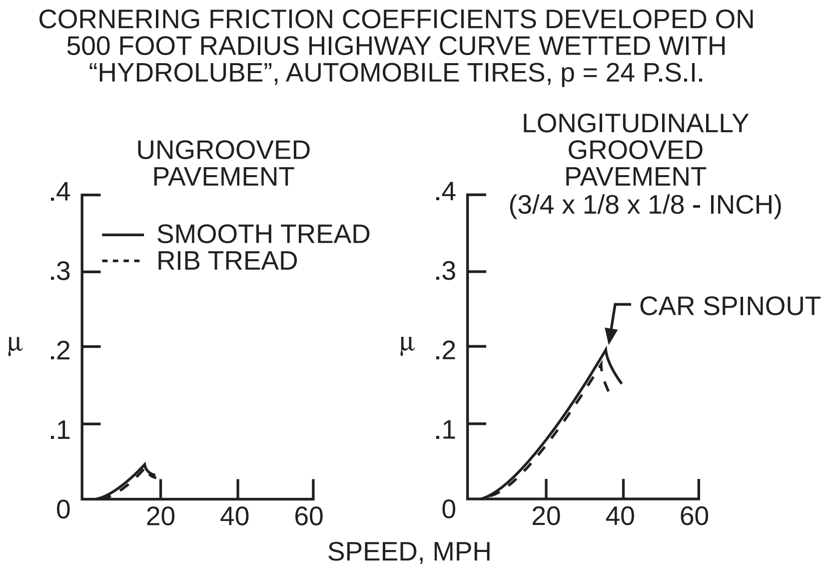

Figure 54. Effect of pavement grooves on cornering friction (Horne 1969).

measured during friction coefficient measurements in the direction of travel.

Tire tread extrusion into pavement grooves is a function of tire size, inflation pressure, and groove dimensions and can vary from 0.40 to 1.60 mm (1/64 to 1/16 in.) (Horne). The cumulative shear force due to tread rubber extrusion into the grooves can be a significant anti-skid component on curved roads and during side-slip, particularly on diamond-ground pavements, which typically have 160 to 200 grooves per m (50 to 60 grooves per ft). Although tire tread extrusions into grooves may decrease with macrotexture wear, it is likely that adequate shear forces are still produced at the groove–tire contact point.

To study the improvement in friction due to mechanical interlock, automobile spinout tests were performed on a 152.4-m (500-ft) radius highway curve (Horne 1969). To reduce the adhesion component of friction to very low values and make the pavement “very slippery,” the longitudinal grooved and nongrooved highway curves were covered with “hydralube” mixed with water. Under these conditions, the tire tread design had very little effect on improving the friction coefficient on either the grooved or the nongrooved pavement, as shown in Figure 54. However, the grooved pavement had significantly increased vehicle spinout speed (from 26 km/hr nongrooved to 39 to 58 km/hr grooved [16 mph nongrooved to 24 to 36 mph grooved]). The cornering friction number for the grooved pavement was around 17 to 19, as compared to 4.5 for the nongrooved pavement. Thus the improvement in cornering friction number due to tire tread–pavement groove interlock was approximately 14. The tire pressure in this study was 165 kPa (24 psi), and the grooves were 3.2 mm ¥ 3.2 mm (1/8 in. ¥ 1/8 in.) spaced at 19 mm (3/4 in.).

Diamond-ground pavements have grooves that are typically spaced at 5 mm (0.2 in.), resulting in four times as many grooves in contact with the tire as longitudinally grooved pavements. The greater number of grooves in ground pavements may compensate for any decrease in tire extrusion due to shallower grooves. Therefore, a similar increase in cornering friction number can be expected on diamond-ground pavements as on longitudinally grooved pavements.

Conventional methods of measuring skid numbers using ASTM E274 specifications do not measure cornering or side-slip friction numbers. The improvement in cornering friction numbers caused by the grooves on a diamond-ground pavement may be responsible for the consistently lower accident rate. The cornering friction numbers may not be substantially affected by the decrease in surface texture of the pavement; hence, no significantly consistent increase in accident rate was observed at the diamond-ground pavements in Wisconsin.

CRACKING

The effects of diamond grinding on subsequent cracking performance of concrete pavements were evaluated based on both analytical results and field observations. Since slab thickness is one of the most sensitive factors affecting cracking performance of concrete

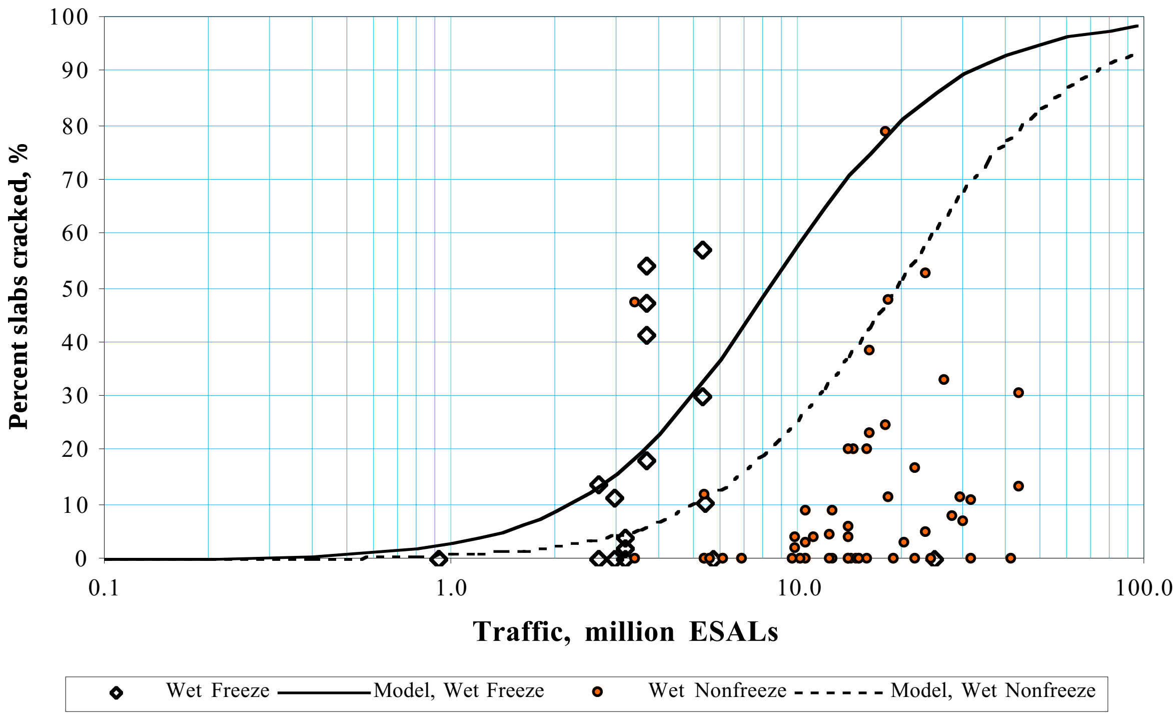

Figure 55. Comparison of predicted and observed cracking for JPCP sections in wet freeze and wet non-freeze climatic regions.

pavements, any reduction in slab thickness can be a concern. However, as discussed in the previous chapter, the analytical results showed that the removal of a thin layer from the pavement surface would not adversely affect cracking performance of concrete pavements because concrete strength increases with age and offsets the slight loss of slab thickness.

The field observations are consistent with the analytical evaluation results. None of the pavement sections evaluated exhibited an unusual level of cracking. Figure 55 shows the cracking data for the sections in wet freeze and wet non-freeze climatic regions, along with the predicted performance for typical sections in each climatic region. This figure shows significant scatter because the plot includes pavement sections with significantly different design features. For the wet freeze region, the model predictions represent the average case, which means the unusually high levels of cracking are not observed in the field data. For the wet non-freeze region, the predicted performance represents the upper bound of the cracking data, meaning that actual performance was better than predicted.

The results of the cracking evaluation clearly show that diamond grinding is not expected to cause increased slab cracking. These findings are consistent with the findings from a California study (Wells and Marsh 1993).

CONCLUSIONS

The diamond-ground pavements evaluated under this study showed excellent performance. The results of this study show that CPR with diamond grinding is an effective means of extending service life of concrete pavements. The key findings of this study include the following:

- The immediate effect of diamond grinding is a smooth pavement surface with the desirable surface texture. The level of smoothness that can be achieved through diamond grinding is often better than that of a new pavement.
- Diamond grinding results in significant increase in surface texture and corresponding improvements in skid resistance. Studies have shown that diamond-ground surface texture can lead to significant improvement in safety, in terms of reduced accident rates.
- The diamond-ground texture lasts about 8 years in the freeze region and about 12 years in the nonfreeze region.
- Faulting redevelops at a relatively high rate initially on nondoweled diamond-ground pavements, but the rate of faulting levels off to a much lower rate after about 2 million ESALs.
- The amount of precipitation is a significant factor affecting faulting performance. In wet climates, faulting on diamond-ground pavements (nondoweled) becomes

excessive after 8 to 11 million ESALs. A similar pavement in a dry climate may sustain up to 20 million ESALs before reaching the same level of faulting.

- The average service life of diamond ground pavements based on survival analysis is 37 years or 35 million ESALs since initial construction. Some of these pavements were diamond ground two or more times, and many sections in the database have survived 40 or more years.
- The survival analysis results showed that the expected life of a diamond-ground surface (the time from grinding to regrinding, overlaying, or reconstruction) at different levels of reliability are as follows:
  - 50-percent reliability: 13.5 years or 12 million ESALs.
  - 75-percent reliability: 11 years or 8 million ESALs.
  - 85-percent reliability: 10.5 years or 7 million ESALs.
  - 90-percent reliability: 9.5 years or 6.5 million ESALs.
- Based on surface texture life, faulting performance, and survival trends, a diamond-ground surface may be expected to provide a minimum of 8 to 10 years of service with a high degree of reliability, depending on climatic conditions and traffic. At this time, the pavement may be reground to provide further extension to service life.
- Dowel bar retrofitting can be a highly effective method of extending service life of nondoweled PCC pavements with high traffic (more than 1,000,000 ESALs/year), especially for pavements in wet climate.
- A concrete pavement may be ground up to three times without significantly compromising its fatigue life.
- No evidence of any deleterious effects of diamond grinding was observed at any of the field sites.

The long-term effectiveness of a diamond-ground pavement depends on numerous factors, but the most significant factors are the condition of the existing pavement structure and level of CPR applied. It is important to recognize that diamond grinding addresses serviceability problems. If the existing pavement is structurally deficient, an overlay or reconstruction may be more appropriate. Pavements with a material problem such as D-cracking or reactive aggregate are also not good candidates for diamond grinding. Inappropriate use of diamond grinding is likely to lead to premature failures. However, even for pavements in poor condition, it may be appropriate to consider diamond grinding as an economical short-term (5 years) solution to a roughness problem until the pavement section can be overlaid or reconstructed.

CHAPTER 3

Case Studies

GEORGIA

The Georgia Department of Transportation (GDOT) has been monitoring the conditions of its roads since its first comprehensive Interstate condition survey in 1971 (Banasiak 1996). By the mid-1970s, there was a growing concern over the rate of deterioration of the Interstate system in Georgia. In particular, Georgia had approximately 1,200 km (750 miles) of JCP in its Interstate system, much of which was paved in the 1960s.

In 1974, to counter the increasing rate of deterioration, GDOT began researching maintenance procedures for PCC pavements. Although many pavement repair methods were used in other states, Georgia was the first to combine multiple rehabilitation techniques into a comprehensive system of repairs. In 1976, the first CPR workshop with diamond grinding was held on a section of I-20 near Augusta, Georgia. Several member companies of ACPA, including IGGA, provided labor, material, and equipment for the demonstration. CPR involves the use of various combinations of one or more rehabilitation techniques, depending on the repair needs. The main philosophy of CPR is to attack the cause of the loss in serviceability of a pavement. Existing problems are repaired or their recurrence is prevented (ERES 1993; Snyder et al. 1989; IGGA 1989). Because CPR specifically addresses loss in serviceability and is performed to increase the service life of the pavement, it is usually more effective and economical than an overlay or reconstruction.

In 1977, distress survey procedures were modified by the GDOT so that observations and a record of distress were made for every slab and faulting measurements were made on every fourth joint (Thornton and Gulden 1980). These data were used for detailed planning of CPR and maintenance activities of individual projects and for evaluating the performance of rehabilitation projects. The data were then reduced to record faulting, pavement roughness, number of cracked slabs, and number of replaced slabs every year for each mile of concrete pavement in Georgia. The idea was to pinpoint damage in the early stages so that it could be repaired before the problem became too large. Based on this information, a priority ranking of restoration needs for projects was assigned.

Georgia has had excellent success with its CPR and diamond grinding programs. Most of the Interstate system in Georgia that was constructed in the 1960s has been rehabilitated with CPR and diamond grinding at least once in the last 30 years. Many of these pavements have been ground two or three times, thus maintaining a high level of serviceability through many years. In addition, the service life of many of the high-volume interstate roads in Georgia has been extended significantly by CPR and diamond grinding. Some Georgia roads have exceeded five times their design life through CPR maintenance (Banasiak 1996). Since the inception of CPR in 1976, Georgia has tightened the riding quality specification for diamond grinding five times. In the 1990s, Georgia was cited by the FHWA as having the smoothest concrete roads in the United States (Banasiak 1996).

Interstate 75, Monroe County, GA1

Based on the success of the CPR experiment, GDOT began equipping its maintenance force with the equipment necessary to perform CPR. In 1977, the very first CPR project was performed on I-75 between mileposts 205 and 188, near Forsyth, Georgia. Four sections between these mileposts in the southbound direction were surveyed for the 1986 FHWA project. The design information for GA1 is summarized in Table 5.

Traffic, age, and rehabilitation history. The first major rehabilitation on this pavement constructed in 1968 was performed in 1977 after 6 million ESALs in the truck lane. Some parts of the project were retrofitted with dowel bars and shear devices in 1981 on an experimental basis. When this pavement was surveyed in 1985, it had carried 8 million ESALs in the truck lane since rehabilitation. In 1986, one lane was added on the inside of each direction to extend the 4-lane highway to 6 lanes. The truck lane was ground for the second time in 1986. In 1993, this project was ground for the third time. This pavement has carried heavy traffic volumes, which have grown significantly over the past 10 years. The ADT in 1997 was 51,000, with roughly 24 percent heavy trucks. This represents more than 3 million ESALs per year in the truck lane and 10 million ESALs since being rehabilitated in 1993. By 1997, the truck lane had carried 35 million ESALs.

Pavement condition. In 1985, the average faulting for the four sections in this project was 1.3 mm (0.05 in.). All distresses had been addressed during CPR, and no cracks or spalls were observed.

Distress records obtained from GDOT of surveys conducted in 1996 indicate the average faulting over the 22-km (14-mile) range equals 1.8 mm (0.07 in.) in the southbound direction, 3 years and 8 million ESALs after grinding. Roughly one out of every three slabs had been replaced or had full-depth repairs, and 20 percent of the slabs were broken or cracked. Seven percent of the joints exhibited spalling, and 4 percent of the slabs exhibited longitudinal cracking at various levels of severity.

Conclusion. Twenty years after this first CPR project, the original concrete pavement surface still exists. Because this pavement has been diamond ground three times, faulting is low, and high ride quality has been maintained throughout the pavement's life. CPR and maintenance on this pavement have resulted in good service life (30 years and 35 million ESALs). Although this pavement has failed structurally, with many cracked and replaced slabs, further extension of service life is expected due to additional CPR using slab replacement. At the current high rate of traffic, this pavement could very well carry more than 50 million ESALs before the truck lane is reconstructed, which is many times its original design life.

Interstate 85, Fulton County, GA2

GA2 is a project included in the 1986 FHWA study. This portion of Interstate 85 is a nondoweled JPCP in Fulton County, southwest of Atlanta. The design information for GA2 is summarized in Table 6.

Traffic, age, and rehabilitation history. This project was constructed in 1968. Edgedrains were installed in 1978 and in 1982, after 20 million ESALs since construction on the truck lane, CPR with diamond grinding was performed. Other CPR activities included partial-depth repairs, full-depth repairs, slab stabilization, and joint resealing. When this project was surveyed in 1985, it had carried an additional 7 million ESALs. The ADT was approximately 43,000 with 16 percent trucks, corresponding to 2.25 million ESALs on the truck lane in 1985. By 1997, this pavement had carried 42 million ESALs since it was first ground in 1982, and has carried 62 million ESALs since construction.

Pavement condition. In 1985, average faulting for this section was 1.8 mm (0.07 in.) after 7 million ESALs and 3 years. One out of every three slabs surveyed exhibited transverse cracks at various levels of severity. No other distresses were observed.

Table 5. Design Summary for GA1

Section ID	Basic Design	Climatic Information	Activity
GA1 MP 205 to 188 (SB)	229-mm (9-in.) JPCP 76-mm (3-in.) soil cement base 127-mm (5-in.) gravel subbase 305-mm (12-in.) cement-stabilized soil Nondoweled joints 9.1-m (30-ft) joint spacing 3-m (10-ft) AC shoulder 4-lane highway extended to 6 lanes in 1986	Wet non-freeze Latitude = 33 degrees Freezing index = 0^0C-days (0^0F-days) Mean annual temperature = 18.2^0C (64.7^0F) Mean annual precipitation = 1139 mm (44.9 in.)	1968 – Construction 1977 – 6 million ESALs – CPR with grinding 1986 – 16 million ESALs – CPR with grinding 1993 – 25 million ESALs – CPR with grinding 1997 – 35 million ESALs

Table 6. Design Summary for GA2

Section ID	Basic Design	Climatic Information	Activity
GA2 MP 58 to 69 (NB)	229-mm (9-in.) JPCP 76-mm (3-in.) soil cement base 127-mm (5-in.) gravel subbase Nondoweled joints 9.1-m (30-ft) joint spacing AC shoulder 6-lane highway	Wet non-freeze Latitude = 34 degrees Freezing index = 0^0C-days (0^0F-days) Mean annual temperature = 16.2^0C (61.2^0F) Mean annual precipitation = 1235 mm (48.6 in.)	1968 – Construction 1982 – 20 million ESALs – CPR with grinding 1985 to 1995 – 2 more CPR with grinding 1997 – 62 million ESALs

Table 7. Design Summary for AZ1-1

Section ID	Basic Design	Climatic Information	Activity
AZ1-1 MP 199.3 to MP 199.2 (SB)	229-mm (9-in.) JPCP 229-mm (9-in.) soil cement base Nondoweled joints 4.3-5.2-4.3-4.9-m (14-17-14-16-ft) joint spacing 3-m (10-ft) AC shoulder Longitudinal pipe edgedrains 8-m (25-ft) cut section	Dry non-freeze Latitude = 33 degrees Freezing index = 0^0C-days (0^0F-days) Mean annual temperature = 21.8^0C (71.2^0F) Mean annual precipitation = 181 mm (7.11 in.)	1961 – Construction 1976 – 15 million ESALs – CPR with grinding 1991 – 45 million ESALs – CPR with grinding 1997 – 60 million ESALs

Distress records obtained from GDOT of surveys conducted in 1996 indicate the average faulting over the 16-km (10-mile) range equals 1.3 mm (0.05 in.) in the northbound direction. Roughly 37 percent of the original slabs had been replaced or had full-depth repairs, and 6 percent of the slabs were broken or cracked. Only 1 percent of the joints exhibited spalling.

Conclusion. This pavement was first diamond ground in 1982, at which point it had already carried 20 million ESALs. CPR and grinding have significantly increased the life of this pavement, which has carried and additional 42 million ESALs since 1982. Replacing failed slabs and diamond grinding have resulted in good ride quality, with average faulting of 1.3 mm (0.05 in.). This project has already carried 62 million ESALs over the 29 years since initial construction, which is many times more traffic than that for which it was originally designed.

ARIZONA

Interstate 17, SB, Maricopa County, AZ1-1

AZ1-1 is a project included in the 1986 FHWA study. This portion of Interstate 17 is a nondoweled JPCP in Maricopa County in downtown Phoenix, Arizona. The design information for AZ1-1 is summarized in Table 7.

Traffic, age, and rehabilitation history. This pavement section was constructed in 1961. It was rehabilitated and diamond ground in 1976 after approximately 15 million ESALs on the truck lane. The primary rehabilitation performed on this pavement in 1976 included partial-depth (spall) repairs, full-depth repairs, diamond grinding, and joint sealing. The grinding contractor completed the job in approximately 60 days. In order to prevent disruption to traffic, the grinding was done between 9:00 p.m. and 6:00 a.m. When this pavement was surveyed in 1986, the truck lane had carried an additional 15 million ESALs.

The pavement was rehabilitated and ground for a second time in 1991. The second rehabilitation also consisted of partial-depth repairs, full-depth repairs, diamond grinding, and joint sealing. The 1997 2-way ADT for this 6-lane highway was 130,000, with roughly 10 percent trucks. This corresponds to more than 2.5 million ESALs per year in the driving lane and more than 15 million ESALs since being restored and ground in 1991.

The total traffic on this pavement in the 36 years since construction is approximately 60 million ESALs. It should be noted that only some sections within this project have been overlaid or reconstructed. One of the advantages of grinding is that only critical areas have to be treated, and most often no complete road closings are required. The alternative is to overlay all the lanes

Table 8. Distress Summary for AZ1-1

Year constructed	1961
Year ground	1976, 1991
Faulting, mm (in.) Average Range	 1.3 (0.05) -0.5 to 3.0 (-0.02 to 0.12)
Corner breaks, % slabs	0.0
Transverse cracking, % slabs Low-severity Moderate-severity High-severity	 0.0 0.0 0.0
Transverse joint spalling	Many post- and pre-ground partial-depth repairs
PSR	3.4

and the shoulder, which is far more expensive, more time-consuming, and also requires increased lane closures. Grinding the pavement also avoided the need to remove and replace the pavement beneath the overpasses to provide sufficient clearance for trucks.

Pavement condition. In 1986, 9 years and 15 million ESALs after diamond grinding, this pavement had an average faulting of 0.5 mm (0.02 in.). No other distresses were observed. The distress data for AZ1-1 is summarized in Table 8.

This pavement was rehabilitated and ground for the second time in 1991. In this time, the pavement has faulted to an average value of 1.3 mm (0.05 in.). In a 1997 survey, many longitudinal cracks were observed on this section. Although specific spalls were not observed, this section had partial- and full-depth repairs at roughly one out of every two joints. These have apparently been placed to rehabilitate the spalled joints. Ride quality was fair, with a Present Serviceability Rating (PSR) of 3.4 measured at a speed of 88 km / hr (55 mph). The pavement appears to be in fair to poor condition, and many of the partial-depth repairs are beginning to spall.

Conclusions. Diamond grinding with partial- and full-depth repairs was first performed on this section in 1976. This was done again in 1991 to increase the service life of the pavement and provide a smoother ride. This pavement section is scheduled for reconstruction in 1998. The second grinding with partial- and full-depth repairs seems to have added roughly 7 years to the service life of the pavement. The pavement has faulted to only an average value of 1.3 mm (0.05 in.) 6 years after the second grinding. Overall, this pavement has carried approximately 60 million ESALs and has had a service life of more than 36 years, many of which have been added by CPR and grinding. This pavement is a good example of the extension of service life due to CPR and diamond grinding.

CALIFORNIA

Interstate 10, EB, San Bernardino County, CA6-1

CA6-1 is a diamond-ground JPCP section in San Bernardino County, near Fontana. The pavement structure consists of a 203-mm (8-in.) slab on a 76-mm (3-in.) asphalt-treated base with a 4.6-m (15-ft) constant joint spacing. This is the historic project where continuous diamond grinding was used for the first time on a concrete pavement. CA6-1 is also part of historic Route 66 (now designated as I-10). The design information for CA6-1 is summarized in Table 9.

Traffic, age, and rehabilitation history. This is a 51-year-old concrete pavement that was constructed in 1946. In 1965, after 9 million ESALs on the truck lane, two lanes in each direction were added on the inside to widen the 4-lane highway to an 8-lane highway. The original truck lane was diamond ground as part of the very first CPR grinding project ever performed. By 1984, the 2-way ADT on this pavement was 84,000; the truck lane had carried more than 19 million ESALs since construction. The pavement was ground for a second time in 1984 and retrofit edgedrains were installed. Fifty-one years since it was constructed, this pavement was ground for the third time in 1997. At that time, the ADT on this freeway was 158,000, which corresponds to more than 2.25 million ESALs per year on the truck lane. The truck lane had carried more than 24 million ESALs since rehabilitation in 1984 and more than 43 million ESALs since construction.

Pavement condition. The part of the pavement surveyed was last ground in 1984. Average faulting on this section in 1997 (before grinding) was 2.5 mm (0.10 in.), which varied from 0.0 to 5.3 mm (0.00 to 0.21 in.). This section had low-severity spalling at many of the

Table 9. Design Summary for CA6-1

Section ID	Basic Design	Climatic Information	Activity
CA6-1 MP 17.4 to MP 17.5 (EB)	203-mm (8-in.) JPCP 76-mm (3-in.) asphalt-treated base Nondoweled joints 4.6-m (15-ft) joint spacing 3-m (10-ft) AC shoulder Longitudinal pipe edgedrains Retrofitted in 1984	Dry non-freeze Latitude = 34 degrees Freezing index = 0^0C-days (0^0F-days) Mean annual temperature = 19.0^0C (66.2^0F) Mean annual precipitation = 397 mm (15.63 in.)	1946 – Construction 1965 – 9 million ESALs – CPR with grinding 1984 – 19 million ESALs – CPR with grinding 1997 – 43 million ESALs – CPR with grinding

Table 10. Distress Summary for CA6-1

Year constructed	1946
Year ground	1965, 1984
Faulting, mm (in.)	
Average	2.5 (0.10)
Range	0.0 to 5.3 (0.00 to 0.21)
Corner breaks, % slabs	0.0
Transverse cracking, % slabs	
Low-severity	88.2
Moderate-severity	0.0
High-severity	0.0
Transverse joint spalling, % joints	
Low-severity	41.2
Moderate-severity	0.0
High-severity	0.0

joints, and most of the slabs exhibited low-severity transverse cracking. The distress summary for this project is shown in Table 10.

Conclusions. CA6-1 is an excellent showcase of the endurance and durability of concrete pavements and the effectiveness of CPR with diamond grinding. In 1997, this section had low-severity cracks on almost every slab and spalling at many joints. However, 14 years after the second grinding, this nondoweled 203-mm (8-in.) JPCP had average faulting of only 2.5 mm (0.10 in.) after 24 million ESALs. To date, CA6-1 has carried in excess of 43 million ESALs since construction and more than 34 million ESALs since it was first ground in 1965. The 1997 rehabilitation provided further extension of service life for this 51-year-old pavement. Ride quality on the newly ground pavement was very good.

MINNESOTA

Trunk Highway 10, EB and WB, Elk River, MN1-1 and MN1-2

MN1 and MN2 are also projects included in the 1986 FHWA study and were surveyed in 1985. This portion of TH10 is a nondoweled JPCP located near Elk River in Sherburne County, approximately 40 km (25 mi) northwest of Minneapolis. The design information for MN1-1 and MN1-2 is summarized in Table 11.

Traffic, age, and rehabilitation history. This pavement, constructed in 1946, was rehabilitated and diamond ground in 1982. The rehabilitation consisted of diamond grinding and joint sealing in the eastbound truck lane and grinding, joint sealing, and edge support installation in the westbound truck lane. This project had been subjected to relatively low levels of traffic, with ADT of 11,500 and 10 percent trucks. In 1982, this pavement had carried 2.7 million ESALs in the truck lane. When this pavement was surveyed for the FHWA study in 1985, it had carried an additional 900,000 ESALs since rehabilitation. In 1997, the truck lanes had carried approximately 2.5 million ESALs since rehabilitation and 5.2 million ESALs since construction.

Pavement condition. Approximately 3 years and 900,000 ESALs since grinding, MN1-1 and MN1-2 had average faulting of 1.3 mm (0.05 in.) and 1.8 mm (0.07 in.), respectively. Twenty-three percent of the slabs in MN1-1 and 36 percent of the slabs in MN1-2 had transverse cracks. Longitudinal cracking for MN1-1

Table 11. Design Summary for MN1-1 and MN1-2

Section ID	Basic Design	Climatic Information	Activity
MN1-1 MP 208.0 to MP 208.1 (EB) MN1-2 MP 205.5 to MP 205.4 (WB)	203-mm (8-in.) JPCP 76-mm (3-in.) aggregate base Nondoweled joints 4.6-m (15-ft) joint spacing 3-m (10-ft) AC shoulder for MN1-1 0.5-m (1.5-ft) PCC edge support + 2.4-m (8-ft) AC shoulder for MN1-2	Wet freeze Latitude = 45 degrees Freezing index = 972^{0}C-days (1750^{0}F-days) Mean annual temperature = 6.4^{0}C (43.6^{0}F) Mean annual precipitation = 716 mm (28.20 in.)	1946 – Construction 1982 – 2.7 million ESALs – CPR with grinding 1997 – 5.2 million ESALs

Table 12. Distress Summary for MN1-1 and MN1-2

	MN1-1	MN1-2
Year constructed	1946	1946
Year ground	1982	1982
Faulting, mm (in.) Average Range	 1.8 (0.07) 0.5 to 3.3 (0.02 to 0.13)	 1.8 (0.07) 0.3 to 3.3 (0.01 to 0.13)
Corner breaks, % slabs	0.0	0.0
Transverse cracking, % slabs Low-severity Moderate-severity High-severity	 0.0 54.0 3.0	 0.0 27.0 3.0
Transverse joint spalling, % joints Low-severity Moderate-severity High-severity	0.0 0.0 0.0	0.0 0.0 0.0
PSR	3.30	3.35

and MN1-2 was 14 percent and 23 percent, respectively, and many of these cracks may have existed prior to rehabilitation. The higher cracking and faulting in MN1-2 suggest differential directional distribution of traffic. Note that only MN1-2 was retrofitted with 0.45-m (1.5-ft) PCC edge support in 1982.

In 1985, two replicate sections were surveyed for MN1-1. One of these two sections had been overlaid. The surviving section had average faulting of 1.8 mm (0.07 in.), 57 percent of the slabs had transverse cracks, and 57 percent of the slabs had longitudinal cracks. Two replicate sections on MN1-2 were also surveyed in 1985 for the FHWA study. Both of these sections were still surviving; however, because of geometric constraints (side streets and traffic sight distance), a different section within the same project was surveyed. The section surveyed in the westbound direction with the retrofitted edge support had average faulting of 1.8 mm (0.07 in.), 30 percent of the slabs had transverse cracks, and none of the slabs had longitudinal cracks. The distress data for MN1-1 and MN1-2 are summarized in Table 12.

Conclusion. This is an example of a low-volume rural highway that has lasted more than 50 years since construction and more than 15 years since rehabilitation. Concrete pavements and rehabilitation with diamond grinding are as useful and effective on low-volume roadways as on high-volume roadways. Other examples of 40+-year-old diamond ground pavements in Minnesota include TH10 near Wadena and TH23 in St. Cloud. TH10 is a rural low-volume two-way roadway near Wadena, constructed in 1948 and diamond ground in 1985. The diamond-ground TH23 section in St. Cloud is in an urban area with a high volume of passenger cars and low truck traffic. This project was constructed in 1958 and diamond ground in 1982.

SOUTH CAROLINA

Interstate 85, NB, Anderson, SC1-1

SC1 is also a project that was included in the 1986 FHWA study and was surveyed in 1985. This portion of I-85 in Anderson County is approximately 50 km (30 miles) from the Georgia border. The pavement is a nondoweled JPCP placed on a permeable asphalt-treated base and a soil cement subbase. The design information for SC1-1 is summarized in Table 13.

Traffic, age, and rehabilitation history. This project was constructed in 1963. The first major rehabilitation on this pavement was performed in 1977 after 12 million ESALs in the truck lane. The 1977 rehabilitation of this project consisted of full-depth repairs, joint undersealing, diamond grinding, and joint resealing. Some parts of the project were also retrofitted with edgedrains. When this pavement was surveyed in 1985, it had carried 11 million ESALs since rehabilitation. In 1991, this project was rehabilitated again with full-depth repairs and was diamond ground for the second time to eliminate faulting. The ADT in 1997 was 26,000, with roughly 48 percent heavy trucks. This represents more than 2 million ESALs per year in the truck lane and more than 12 million ESALs since the 1991 rehabilitation. In 1997, the truck lane had carried more than 43 million ESALs since construction.

Pavement condition. This pavement was severely distressed in 1985, with average faulting values for the two replicate sections being 4.6 mm (0.18 in.) and 8.4 mm (0.33 in.), respectively. Roughly 29 percent of the slabs exhibited transverse cracking. The ride quality was very poor, with a PSR of 2.5.

In 1997, the average faulting at SC1-1a and SC1-1b was 3.8 mm (0.15 in.) and 4.8 mm (0.19 in.), respectively. The slabs at SC1-1b had been saw-cut at the

Table 13. Design Summary for SC1-1

Section ID	Basic Design	Climatic Information	Activity
SC1-1a MP 27.0 to 27.1(NB) SC1-1b MP 31.0 to 31.1(NB)	229-mm (9-in.) JPCP 102-mm (4-in.) permeable asphalt-treated base 203-mm (8-in.) soil cement Nondoweled joints 7.6-m (25-ft) joint spacing 3-m (10-ft) AC shoulder	Wet non-freeze Latitude = 35 degrees Freezing index = 0^0C-days (0^0F-days) Mean annual temperature = 16.3^0C (61.3^0F) Mean annual precipitation = 1306 mm (51.40 in.)	1963 – Construction 1977 – 12 million ESALs – CPR with grinding 1991 – 31 million ESALs – CPR with grinding 1997 – 43 million ESALs

Table 14. Distress Summary for SC1-1a and SC1-1b

	SC1-1a	SC1-1b
Year constructed	1963	1963
Year ground	1977, 1991	1977, 1991
Faulting, mm (in.)		
Average	3.8 (0.15)	4.8 (0.19)
Range	0.0 to 9.1 (0.00 to 0.36)	0.0 to 11.9 (0.00 to 0.47)
Corner breaks, % slabs	8.7	0.0
Transverse cracking, % slabs		
Low-severity	0.0	0.0
Moderate-severity	4.3	0.0
High-severity	26.1	13.6
Transverse joint spalling, % joints		
Low-severity	0.0	0.0
Moderate-severity	0.0	0.0
High-severity	0.0	0.0
PSR	3.75	3.45

Table 15. Characteristics of Pavement Section 1 in Brussels Before and After Diamond Grinding

	Before	After	Note
APL (evenness)			
10 m	163	66	
2.5 m	75	24	
Friction	0.56-0.61	0.89-0.90	Side Force Coefficients (unknown speed)
Noise	99.4 dB(A)	94.8 dB(A)	

Table 16. Characteristics of Pavement Section 2 in Brussels Before and After Diamond Grinding

	Before	After	Note
APL (evenness)			
10 m	179	107	
2.5 m	77	37	
Friction	0.56-0.61	0.87-0.91	Side Force Coefficients (unknown speed)
Noise	99.4 dB(A)	94.8 dB(A)	

Table 17. Friction and Noise Characteristics for Bekkevoort Pavement Section Diamond Ground to 75 Percent of Transverse Groove Depth

	Before	After	Note
Friction	-	1.1	Side Force Coefficients (unknown speed)
Noise	101 dB(A)	94 dB(A)	

Table 18. Friction and Noise Characteristics for Bekkevoort Pavement Section Diamond Ground to 50 Percent of Transverse Groove Depth

	Before	After	Note
Friction	0.63	0.98	Side Force Coefficients (unknown speed)
Noise	102 dB(A)	94 dB(A)	

center of the slab, presumably to decrease transverse cracking. Although none of the joints had any spalling, there was visible seal damage at roughly 20 percent of the transverse joints. Many of the original transverse cracks had been repaired when the pavement was rehabilitated in 1991. Thirty percent of the slabs in SC1-1a and 13 percent of the slabs in SC1-1b had medium- to high-severity transverse cracks. The ride quality was fair to good, with PSR of 3.75 and 3.45 for SC1-1a and SC1-1b, respectively. The southbound portion of this project had less faulting, repairs, and distresses than the northbound portion, the direction in which the survey was conducted. The distress data for SC1-1 are summarized in Table 14.

Conclusion. This is a good example of a section that was diamond ground for a second time to extend service life. CPR was first performed on this pavement in 1977, after 12 million ESALs in the truck lane. When this project was surveyed in 1985, after an additional 11 million ESALs, it had severe faulting, many transverse cracks, and a very poor ride. This pavement was rehabilitated in 1991 with full-depth repairs and diamond grinding, resulting in a second extension of life. This project is scheduled for reconstruction in 1998, 35 years after initial construction, at which point the pavement would have carried 30 million ESALs since the first grind, 12 million ESALs since the second grind, and more than 43 million ESALs since initial construction.

Table 19. Friction Characteristics Before and After Diamond Grinding on E40 in Bierbeek

	Before	After	Note
Direction Liège			Side Force Coefficients at 50
Slow lane	0.54	0.98	km/hr (31 mph) and 20^{0}C (68^{0}F)
central lane	0.58	1.04	Yaw angle-20 degrees
fast lane	0.70	1.00	PIARC smooth tire
Direction Brussels			Side Force Coefficients at 50
slow lane	0.54	0.84	km/hr (31 mph) and 20^{0}C (68^{0}F)
central lane	0.64	1.01	Yaw angle-20 degrees
fast lane	0.72	1.04	PIARC smooth tire

BELGIUM (EUROPE)

Brussels (A12)

Two test sections were prepared in 1990 with different texture depths. Tables 15 and 16 summarize the measurements carried out before and after grinding on these sections. Both roughness and noise decreased greatly while friction increased.

Bekkevoort (A2)

The pavement had been previously grooved in the transverse direction. In 1991, the existing grooves were ground away for 75 percent of the groove depth in one test section and for 50 percent of the groove depth in the other test section. Tables 17 and 18 summarize the friction and noise measurements before and after grinding for the two sections. Noise level decreased greatly after diamond grinding.

Bierbeek (E40)

This pavement had been transversely grooved previously. These grooves were completely ground off when this pavement section was diamond ground around 1992. Table 19 shows friction characteristics measured before and after diamond grinding (Caesteker and Heleven 1993). The Belgium experience shows significant improvements in both ride quality and side force friction numbers immediately after diamond grinding. However, no time history data for these sections are available. Significant reduction in noise was also documented.

CHAPTER 4
Guidelines

NEED FOR GRINDING

Diamond grinding is a viable option for the following distress types:

- Removal of transverse joint and crack faulting.
- Smoothing out built-in or construction roughness.
- Texturing of a polished concrete surface exhibiting inadequate macrotexture.
- Removal of wheelpath rutting caused by studded tire wear.
- Reducing noise level caused by tire-pavement interaction.
- Removal of permanent upward slab warping at joints.
- Improvement of transverse slope to improve surface drainage.

Removal of Transverse Faulting

The most common use of diamond grinding is to improve pavement ride quality through the removal of faulting. A typical vehicle operator can discern faulting of 5.1 mm (0.2 in.) and feels a level of discomfort as faulting approaches 6.4 mm (0.25 in.)(ACPA 1990). Grinding should be conducted prior to reaching these critical levels of vehicle ride discomfort.

Several approaches have been used to quantify a ride quality performance threshold to indicate the most beneficial time to perform diamond grinding. The present serviceability index (PSI) of a pavement, used in the American Association of State Highway and Transportation Officials (AASHTO) design approach, is typically used as a trigger for restoration work (AASHTO 1993). Roughness measurements using pavement profile data can also be used to set trigger values for diamond grinding projects. A faulting index, similar to that developed by Georgia, or average faulting data can be useful in establishing threshold values for diamond grinding (Snyder et al. 1989). The following guidelines are recommended to determine the need for diamond grinding to remove joint and crack faulting (ACPA 1997):

- If the PSR drops within a range of 3.4 to 3.8, a thorough evaluation of the cause of this loss in serviceability should be conducted. After addressing any functional deficiencies, diamond grinding should be conducted to restore the serviceability to a higher level. The following trigger values can be used to determine need for grinding:
  - High traffic volume (ADT > 10,000) – PSR = 3.8, IRI = 1.0 m/km (63 in./mi)
  - Medium traffic volume (3,000 < ADT < 10,000) – PSR = 3.6, IRI = 1.2 m/km (76 in./mi)
  - Low traffic volume (ADT < 3,000) – PSR = 3.4, IRI = 1.4 m/km (89 in./mi)
- Grinding should be conducted before faulting reaches critical levels. Such levels are dependent upon many factors. For example, less faulting is tolerable for pavements with short joint spacing. Highway agencies should establish threshold values of faulting for various pavement configurations.

The faulting index developed by Georgia has been proven a reliable indicator for scheduled diamond grinding. Each faulting index value of 5 represents 0.8 mm (1/32 in.) of faulting (ACPA 1990). For example, a faulting index of 25 represents an average fault of 4.0 mm (5/32 in.), as shown in Table 20. The representative numbers of joints that need to be measured to accurately characterize the degree of faulting are shown in Table 21. The average of all fault measurements is used to develop a faulting index value. Diamond grinding is performed at a faulting index of 15 based on the average fault measurements. At a faulting index of 15, which represents average faulting of 2.4 mm (3/32 in.), typically some joints are faulted to the critical level of a 6.4 mm (1/4 in.). Most of the joints were spaced at 9.1 m (30 ft). Shorter joint spacing would have a lower criterion.

Removal of Wheelpath Rutting Caused by Studded Tire Wear

Studded tires and tire chains, typically used in northern and mountainous areas with high snowfall, can wear down the surface of a concrete pavement. This rutting increases the depth of water in the wheelpaths during rainy weather, thereby creating more hazardous conditions that involve larger splash volume and a greater possibility of hydroplaning. The severity of studded tire wear is a function of the hardness of the mortar and the coarse aggregate used in the concrete (ACPA 1990).

In flexible pavements, 12.7 mm (0.5 in.) is considered a tolerable amount of rutting. The same criterion can be used as a trigger value to consider diamond grinding on concrete pavements that have rutted due to studded tire wear. Diamond grinding removes wheelpath ruts caused by studded tires, improves drainage in wet weather by eliminating pooling of water in the ruts, and reduces the possibility of hydroplaning. Removal of ruts also reduces splash volume, thereby creating safer driving conditions. The longitudinal direction of grooves in a diamond-ground pavement also enhances braking and vehicle directional stability, particularly on highway curves. The alternative to diamond grinding is an overlay. An overlay may be the appropriate choice if the existing pavement is structurally deficient. On structurally sound pavements, the overlay option will likely be less economical.

In Sweden, where extensive use of studded tires is common during winters, the increase in surface texture measured using sand-patch tests and surface friction as measured using the SAAB friction tester at 70 km/hr (44 mph) was short-lived. However, the smoothness and rut reduction convinced Sweden to construct new PCC pavements 10 to 20 mm (0.4 to 0.8 in.) thicker to allow for future grinding (Hultqvist and Carlsson 1994).

Texturing of Polished Concrete Surface

Pavement skid resistance is improved through diamond grinding by increasing the macrotexture of the

Table 20. Faulting Index

Average Fault, mm (in.)	Faulting Index	Comments
0.8 (1/32)	5	No roughness
1.6 (1/16)	10	Minor faulting
2.4 (3/32)	15	Grinding project
3.2 (1/8)	20	Expedite project
4.0 (5/32)	25	
4.8 (3/16)	30	Discomfort begins
5.6 (7/32)	35	
6.4 (1/4)	40	Grind immediately

Table 21. Recommended Number of Fault Measurements Needed (ACPA 1990)

Joint Spacing, m (ft)	Measure Cracks	Measurement Interval	No. of Fault Measurements per lane-km (lane-mile)
<3.65 (<12)	No	every 9th joint	>32 (>50)
3.65-4.57 (12-15)	No	every 7th joint	32-40 (50-63)
4.57-6.10 (15-20)	No	every 5th joint	33-44 (53-70)
6.10-9.14 (20-30)	Yes	every 4th joint	28-42 (44-66)*
>9.14 (>30)	Yes	every 4th joint	>19 (>30)*

* Include transverse cracks with joint fault measurements.

concrete surface. The increased macrotexture initially provides high skid numbers that decrease quickly over the first few years. However, adequate macrotexture is typically maintained for 8 years in freeze climatic regions and 12 years in non-freeze climatic regions. Spacing of diamond saw blades could be changed to offset the higher rate of wear of aggregates that are susceptible to polishing.

The relative benefits of increased macrotexture and friction are greater for pavements that exhibit very poor surface texture and skid characteristics. The increased macrotexture of the pavement surface provides for improved drainage of water at the tire–pavement interface, especially with the use of worn tires, thus reducing the potential for wet weather accidents. Pavements with polished concrete surface also have poor cornering friction characteristics. Diamond grinding improves cornering friction numbers, providing directional stability by tire tread–pavement groove interlock. Although aggregates susceptible to polishing may exhibit a high rate of wear of macrotexture after grinding, the tire tread–pavement groove interlock can still be expected to maintain higher cornering friction numbers and, consequently, greater directional stability than a polished nonground pavement surface.

Removal of Slab Warping

In very dry climates, slabs can get permanently warped at the joints. Long joint spacing and stiff base support result in curled slabs that are higher at the panel ends (joints) than midpanel, resulting in a bumpy ride. Diamond grinding can be used to restore smoothness and level off the surface of a warped pavement. As in the case of faulted pavements, roughness measurements using pavement profile data can be used to set trigger values. The difference in elevation between the center of the slab and joints can also be used.

Improvement of Transverse Slope

Good surface drainage is required to minimize the infiltration of surface water into the pavement. All tangent sections of highways usually have a transverse slope to facilitate surface drainage of water. Typical transverse slopes for newly constructed pavements range from 1.5 to 3 percent, which corresponds to 15 to 30 mm/m (3/16 to 3/8 in./ft) (Huang 1993). Minor changes in the pavement cross slope are also facilitated by diamond grinding. Proper cross slope facilitates transverse drainage and reduces the potential for hydroplaning, especially if studded tire wear has produced ruts in the pavement.

EFFECTIVENESS AND LIMITATIONS

For concrete pavements that have developed excessive roughness, diamond grinding can be an effective means of restoring smoothness and extending service life. The survival analysis results show that a diamond-ground surface can provide 8 to 10 years of service, after which the pavement can be reground to further extend its service life. Diamond grinding can remove faulting and surface irregularities and can restore a smooth riding surface with desirable surface texture on concrete pavements that have developed excessive roughness. Excessive roughness is the most common reason for diamond grinding, and smoothness that is generally better than that of a new pavement can be achieved through diamond grinding.

Diamond grinding is used frequently in conjunction with other CPR techniques, such as full-depth repairs, partial-depth repairs, retrofit edgedrains, and slab stabilization, as part of a comprehensive pavement restoration program.

Diamond grinding offers numerous advantages over other rehabilitation alternatives, including the following:

- Typically costs substantially less than an overlay (Pierce 1995; McGovern 1995).
- Enhances surface friction and safety of an old pavement surface.
- Can be performed during off-peak hours with short lane closures.
- Eliminates the need for taper, which is required with the overlay alternative at highway entrances, exits, and at side streets.
- Does not affect overhead clearances underneath bridges or hydraulic capacities of curbs and gutters on municipal streets.
- Grinding of one lane does not require grinding of the adjacent lane, which may have perfectly acceptable surface characteristics.
- Provides at least 8 to 10 years of service life.

For concrete pavements with predominantly roughness problems, diamond grinding can be a highly effective and economical rehabilitation alternative, especially if the pavement is in good structural condition. It may also be appropriate to consider diamond grinding as an economical short-term (less than 5 years) solution for pavements with structural problems to address roughness problems until a more comprehensive rehabilitation can be applied.

However, not all concrete pavements that have developed excessive roughness are good candidates for diamond grinding, and grinding alone may not be enough to address the existing problems. Structural distresses such as pumping, loss of support, corner breaks, working transverse cracks, and shattered slabs will require repair before grinding is conducted. If the cause of roughness is not treated prior to grinding, quick redevelopment of the roughness is likely. Widespread material problems such as D-cracking, reactive aggregate, or freeze-thaw damage indicate that diamond grinding may not be a suitable restoration technique and that a more comprehensive rehabilitation approach needs to be considered.

ASSESSING FEASIBILITY

Feasibility of diamond grinding should be assessed based on the collection and analysis of pavement condition and roughness data. The most important factor in determining the cost-effectiveness of the repair strategy is a thorough evaluation of the collected pavement condition data (Roman et al. 1985). Structural distresses such as pumping, loss of support, corner breaks, working transverse cracks, and shattered slabs will require repair before grinding is conducted. The presence of widespread distress related to concrete durability, such as D-cracking, reactive aggregate, or freeze-thaw damage, may indicate that diamond grinding is not a suitable restoration technique. The following factors should be considered in determining the feasibility of diamond grinding for a particular project (ERES 1993):

- If there is evidence that a severe drainage or erosion problem exists, as indicated by significant faulting (> 3.3 mm [0.13 in.]) or pumping, appropriate restoration should be performed to alleviate the problem prior to diamond grinding. Concurrent CPR such as full-depth repairs, load transfer restoration, edgedrain installation, slab stabilization, retrofit PCC shoulders, and joint/crack sealing could be used to address pumping/erosion problems.
- The presence of progressive transverse slab cracking (all severity levels for JPCP and only deteriorated cracks for JRCP) and corner breaks indicates a structural deficiency in the pavement. Slab cracking—and the faulting of these cracks—will continue after grinding and will reduce the life of the restoration project, resulting in a rapid development of roughness. Even in such cases, however, it may be appropriate to consider diamond grinding as an economical short-term solution to a roughness problem until the pavement section can be overlaid or reconstructed. For example, California DOT has used diamond grinding on highly cracked pavements to extend the service life of existing pavements before complete reconstruction of the pavement is warranted.
- The hardness of the aggregate has a direct relationship on the cost of the grinding project. Grinding of pavements with extremely hard aggregate (quartzite) takes more time and effort than grinding of PCC produced with a softer aggregate (limestone). If an extremely hard aggregate is present on a project, grinding costs may be very high.
- Concrete experiencing medium- to high-severity durability problems, such as D-cracking or alkali-aggregate reactivity, should not be rehabilitated through grinding. If the extent and severity of durability distresses are very low, then significant and cost-effective life extension can be attained by full-depth replacement of deteriorated areas, followed by grinding to restore rideability.
- Significant slab replacement and repairs may indicate continuing progressive deterioration that grinding would not remedy, resulting in a decrease in cost-effectiveness of the CPR project.
- Any significant depressions should be corrected by slab jacking before diamond grinding. It is not cost-effective to remove depressions by diamond grinding.

Table 22 lists the information that is needed and desirable to assess the feasibility and make decisions regarding the use of diamond grinding as a rehabilitation strategy for a given project.

CONCURRENT WORK

Diamond grinding does not address structural problems that may have caused the decrease in ride quality. If structural and durability problems are addressed solely by diamond grinding, the ride quality improvement will be short-lived because the cause of the distress has not been remedied. Therefore, diamond grinding is often combined with at least one other CPR procedure that addresses the structural problems. Table 23 shows a list of common CPR techniques and the distresses that they address.

Full-Depth Repairs

Full-depth repairs are placed at deteriorated joints and cracks in concrete pavements to restore ride quality, repair isolated structural defects, and prevent further deterioration of distressed areas. Full-depth repairs are cast-in-place repairs that extend the full depth of the existing slab and are typically a minimum of 1.8 m (6 ft) long and a full lane-width wide. The severity and extent of the overall deterioration of the pavement will determine whether the full-depth repairs will be cost-effective. It is generally more cost-effective to replace an entire slab if a significant portion is highly distressed.

Table 22. Suggested Data Collection Needs for Designing and Construction of Diamond Grinding Rehabilitation Alternative

Pavement Factor Information	Necessary Information	Additional	Typically Used
Pavement design	✓		Overall rehabilitation strategy, future performance prediction, grinding costs/plans
Original construction data		✓	Construction-related distresses (e.g., misaligned dowels, steel placement in reinforced pavements)
Age		✓	Cracking performance
Materials properties	✓		Grinding costs (aggregate hardness)
Traffic loadings and volumes	✓		Future performance prediction
Distress	✓		Concurrent rehabilitation, grinding costs
Skid		✓	Texture
Accidents		✓	Texture
Destructive testing/sampling		✓	Durability distresses (D-cracking, ASR)
Roughness		✓	Grinding costs, specifications
Surface profile	✓		Grinding costs, specifications
Drainage	✓		Future performance prediction
Previous maintenance		✓	Rehabilitation strategy
Traffic control options	✓		Grinding costs/plans

Partial-Depth Repairs

Partial-depth repair entails the removal of shallow areas of deteriorated concrete and replacement with a suitable repair material that is comparable in both strength and volume stability to the concrete of the existing slab. Partial-depth repairs extend the life of PCC pavements by restoring ride quality to pavements exhibiting spalled joints or other shallow distressed areas. Partial-depth repairs also result in well-defined, uniform joint sealant reservoirs that facilitate joint resealing.

Load Transfer Restoration

Load transfer is defined as the ability of a joint or a crack in a concrete pavement to transfer load from one slab to another. Good load transfer results in a reduction in:

- Deflections across transverse joints or cracks and corner deflections which, in turn, results in a reduction in pavement deterioration due to joint pumping, faulting, spalling, and subsequent cracking.
- Tensile stresses in concrete, thus reducing potential for cracking and corner breaks.
- Differential deflections, thus reducing reflection crack deterioration in overlays.

Load transfer restoration, also known as dowel bar retrofit, consists of retrofitting load transfer devices in JCP to improve load transfer. The ability of a joint or crack to transfer load from the approach side to the leave side is referred to as the *load transfer efficiency*. LTE is a major factor in the structural performance of a joint or crack and has a profound effect on the smoothness and longevity of the overall pavement structure. Load transfer restoration followed by diamond grinding results in a smooth pavement with good load transfer capabilities and, consequently, lower faulting over the future life of the pavement. Dowel bar retrofit is a very cost-effective, long-term solution for increasing service life and maintaining good ride quality on an existing pavement.

Edgedrain Installation

The optimum time to address subsurface drainage is during construction, when effective and long-lasting subdrainage systems can be built for minimal added cost over conventional designs. However, in some

Table 23. CPR Techniques and Distresses Addressed (ACPA 1990)

Procedure	Distresses Addressed Measure	Repair or Preventive
Diamond grinding	Faulting Curling roughness Polishing Studded tire wear Poor drainage slope	Repair Repair Preventive Repair Preventive
Full-depth repair Slab replacement	Faulting Joint spalling Pumping Transverse cracking Longitudinal cracking Shattered slab Blow-up D-cracking	Repair Repair Repair Repair Repair Repair Repair Repair
Partial-depth repair	Joint spalling Crack spalling Slab spalling	Repair Repair Repair
Load transfer restoration	Faulting Pumping Transverse cracking	Preventive Preventive Preventive
Cross stitching	Faulting Longitudinal cracking	Preventive Repair
Edgedrain installation	Faulting Pumping	Preventive Preventive
Slab stabilization	Faulting Pumping Transverse cracking	Preventive Repair Preventive
Retrofit PCC shoulders	Faulting Pumping	Preventive Preventive
Joint/crack resealing	Faulting Joint spalling Pumping Blow-up Transverse cracking Longitudinal cracking	Preventive Repair Preventive Preventive Repair Repair

cases, older pavements without effective drainage can be modified to improve drainage conditions by installing retrofitted edgedrains.

Moisture beneath pavements contributes significantly to pavement deterioration and the prevalence of moisture-related distress. The main effect of excess moisture in the pavement structure and subgrade on PCC pavement response is greatly increased deflections. Deflections at both slab edge and corner are substantially greater under saturated conditions than under dry conditions. Therefore, subsurface drainage may be expected to have the most significant influence on the development of deflection-related distresses on PCC pavements, such as faulting and crack deterioration. Free water within the pavement structure can also act as a source of water for frost activity and aggravate material-related problems such as D-cracking. Therefore, reduction of moisture in pavements can be expected to improve pavement performance. The important question, however, is whether edgedrains (particularly retrofit edgedrains) are effective in removing water from beneath the pavements.

Proper construction and routine maintenance are critical for obtaining the benefits associated with retrofitted edgedrains. Nonfunctional edgedrains can adversely affect pavement performance, rather than providing any benefit. An edgedrain that is practically never maintained becomes almost totally inoperative after about 10 years (Christory 1990). Even edgedrains that are maintained can get partially clogged when fines from beneath the pavement are pumped into the edgedrain. Video inspections show that only about 30 percent of edgedrains in in-service pavements are fully functional (Sawyer 1995; Daleiden 1998). The leading cause of this problem is poor maintenance, but improper design and construction also contribute to the problem (ERES 1998). This study found that the short-term benefits of edgedrains (5 years) did not continue with time.

Slab Stabilization

Voids under PCC pavements result in rapid development of various distresses and a loss in ride quality. Voids generally occur near cracks or joints, or along the pavement edge, and are partly a function of intruded moisture. The depth of the voids generally is not greater than 6.4 mm (0.25 in.), but voids are extremely detrimental to the serviceability of the pavement (ERES 1993). Distresses that are commonly associated with voids include pumping, faulting, and cracking. Slab stabilization is the preferred restoration method for correcting voids if full-depth repairs are not warranted. Within the past 10 years, slab stabilization techniques have become highly specialized, with improved materials and equipment.

Slab stabilization is the pressure insertion of material beneath the slab or stabilized base to restore slab support. The material injected beneath the slab is intended to fill voids and prevent further migration of water and support layer fines. Slab stabilization is also referred to as pressure grouting, undersealing, or subsealing. Slab jacking is a similar procedure, but it should not be confused with slab stabilization. Slab jacking involves injecting material beneath a slab to raise the profile of the pavement surface to remove a depression and improve ride quality.

Retrofit PCC Shoulders

PCC shoulders address many of the problems associated with bituminous shoulders adjacent to PCC pavements. The tied PCC lane–shoulder joint is easily sealed and maintained, and the PCC shoulder provides edge support to the mainline pavement. A tied shoulder reduces critical edge stresses and can increase the life of the mainline pavement substantially, as shown in a 1979 study that analyzed the stresses in a PCC shoulder / pavement and the influence on the fatigue life of the mainline pavement (Crovetti and Darter 1985). A shoulder / pavement load transfer of zero indicates a free edge condition or a condition where a bituminous shoulder would exist contiguous to the mainline pavement. When the load transfer is low, the edge stresses produced in the slab are high. If good load transfer exists between the slab and shoulder, the stresses in the slab will be decreased. However, it is very difficult to retrofit good load transfer between the slab and the shoulder, thus retrofit shoulders should not be relied upon for significant reduction in edge stresses.

Joint/Crack Resealing

Joint and crack sealing are common pavement maintenance and restoration activities. The primary objective of joint or crack sealing is to reduce the amount of moisture that can infiltrate into the pavement, thereby reducing moisture-related distresses. Free water entering a joint or crack can accumulate beneath the slab, causing distresses such as loss of support, faulting, and corner breaks.

A second objective of joint and crack sealing is to prevent the intrusion of incompressible materials so that pressure-related distresses (such as spalling) are prevented. Incompressible materials that infiltrate poorly sealed joints or cracks in PCC pavements interfere with normal opening and closing movements, thus creating compressive stresses in the slab and increasing the potential for spalling. If the compressive stress exceeds the compressive strength of the deteriorated pavement, blowups or buckling may occur. Even if blowups do not occur, continued intrusion of incompressibles may cause significant longitudinal expansion of the pavement. This expansion may result in movement of nearby bridge abutments, thereby necessitating expensive bridge rehabilitation.

Full-depth repair and partial-depth repair are the most common CPR procedures performed concurrent to diamond grinding. All CPR procedures except for joint and crack routing and sealing are performed prior to diamond grinding. Grinding is performed after all the structural repairs are completed, resulting in a smooth pavement. Joint resealing follows grinding because the cutting action of the diamond saw blades can wear joint sealant material.

CONSTRUCTION

Diamond grinding equipment uses diamond saw blades that are gang-mounted on a cutting head. Diamond grinding equipment can significantly affect the efficiency and rate of grinding (and, therefore, time required to complete a grinding project). The three most important machine-related factors in the diamond grinding process are (ACPA 1990):

Table 24. Recommended Dimensions for Grinding Texture (ACPA 1990)

	Range of Values, mm (in.)	Aggregate Type	
		Hard, mm (in.)	Soft, mm (in.)
Grooves	2.3-3.8 (0.09-0.15)	2.5-3.8 (0.10-0.15)	2.5-3.8 (0.10-0.15)
Land area	1.5-3.3 (0.06-0.13)	2.0 (0.08)	2.5 (0.10)
Height	1.6 (0.0625)	1.6 (0.0625)	1.6 (0.0625)
Blade spacing	164 to 197 per m (50 to 60 per ft)	174 to 184 per m (53 to 57 per ft)	164 to 177 per m (50 to 54 per ft)

- Weight of the grinding machine.
- Horsepower available to the grinding head.
- The grinding head itself.

The weight of the equipment and the horsepower available to the cutting head influence the depth of cut possible and the grinding effort required to compensate for the varying degrees of hardness of the aggregate in the PCC. The grinding head that cuts the concrete typically has a width ranging from 0.91 to 0.97 m (36 to 38 in.). Usually, from 160 to 195 blades per m (54 to 59 blades per ft) are required to produce a level surface with a corduroy-type texture (IGGA 1989). As the grinding machine moves over the pavement, the grinding head rotates and removes from 1.5 to 19.0 mm (0.06 to 0.75 in.) of the surface (Banasiak 1996). The cutting head can be adjusted to the desired depth, width, cross slope, and textural pattern, depending on the type of equipment used.

Before beginning a grinding project, a contractor must make the proper blade selection. The bond hardness, diamond size, and diamond concentration affect productivity, cost, and quality of the ground surface. Bond hardness determines the rate at which support provided by a metal matrix that is responsible for holding the diamonds is lost as the diamonds become worn. This is why bond hardness affects the cutting speed and life of a grinding head. It is important to match the proper bond hardness to the aggregate being ground in order to maintain maximum cutting efficiency. Diamond size also effects the life and cutting speed of the grinding head. When grinding soft aggregates, choose large diamond particles. For harder aggregates, use smaller diamonds. Diamond concentration is the most important factor because it can disguise or overshadow the effects of bond hardness or diamond size. More diamonds make a harder grinding head and allow for more efficient cutting.

Frictional resistance provided by diamond grinding can be altered by varying the land area, or the raised area between the sawed grooves, by increasing the saw blade spacing. The groove is defined as the recessed area created by the saw blade. The height is defined as the depth from the top of the land area to the bottom of the groove. Table 24 displays the recommended dimensions for hard and soft aggregate.

A key factor in successful grinding is depth control. If grinding is too deep, more concrete than necessary is removed. That is costly and unnecessarily reduces slab thickness. If grinding is too shallow, many of the low spots are missed, resulting in a pavement with a non-uniform texture (IGGA 1989).

Diamond grinding should begin and end at lines normal to the pavement centerline. Grinding is always longitudinal. The direction of grinding does not affect smoothness or quality of texture or joints, so no directional requirements should be imposed; specifications for grinding projects should allow the contractor to grind in either direction. This will give a contractor the flexibility to sequence grinding and CPR work in the most efficient manner to enable the pavement to be fully opened for peak traffic. Several machines can be used together to allow a lane to be completed in one pass, improving productivity and minimizing lane closures on large projects. On smaller projects, it may not be cost-effective to use multiple machines. The contractor would then need to make several passes with one machine to cover the entire width of the lane.

The grinding operation produces a slurry of ground concrete and water used to cool the blades. Modern grinding equipment contains on-board wet vacuums, which vacuum out the slurry from the ground surface. This slurry is chemically inert, poses no chemical threat to vegetation, and can be discarded onto shoulder areas or vacuumed into dishrag trucks, depending on the location of the project and the local governing legislation regarding grinding waste (IGGA 1990). The slurry should be prevented from flowing across lanes occupied by traffic or into gutters or other drainage facilities.

PLANS AND SPECIFICATIONS

To develop an effective plan for CPR with grinding, the agency must provide information necessary for the contractor to prepare a bid. Many factors affect the

Table 25. Pavement Information Required for Preparing an Accurate Bid and Effective Grinding Plan

Type	Pavement Factor	Necessary Information	Additional Information
Design	Year the pavement was built		✓
	Pavement type (plain, reinforced)	✓	
	Transverse joint spacing	✓	
Aggregate	Aggregate sources		✓
	Aggregate hardness	✓	
	Aggregate/sand abrasiveness		✓
	Aggregate size and amount of exposure	✓	
Distress	Faulting index (average faulting)		✓
	Existing pavement profile (roughness)	✓	
	Studded tire rut depth	✓*	
	Amount of warping	✓**	
Other information	Partial- and full-depth repair quantities/locations		✓
	Average depth of removal	✓	
	Slurry disposal regulations	✓	
	Traffic control options	✓	

*For removal of rutting due to studded tire wear.
**For restoration of smoothness to warped pavements.

mechanical effort required to diamond grind the pavement. Table 25 lists the information that is necessary and useful for a contractor to prepare an accurate bid and an effective CPR plan. This information is used to estimate grinding head life, grinding machine speed (productivity), and repair quantities.

Diamond grinding is typically used concurrently with other CPR techniques, so plans for diamond grinding should be made so that the entire CPR process is done in the most cost-effective and efficient manner with a minimum amount of annoyance (lane closures) to the road user. In addition, grinding effort and costs can vary significantly depending on aggregate size and hardness. Therefore, this information, along with information on partial- and full-depth repair quantities / locations, is useful for a contractor to plan the most appropriate time and location to start diamond grinding after placement of the patches.

Grinding should be performed across the entire lane surface to achieve the highest degree of ride quality, uniform skid resistance, and uniform appearance. Spot grinding is not recommended. Grinding specifications may allow for some isolated low areas that were built into the concrete pavement during construction. Isolated low spots that measure less than 0.2 m^2 (2 sq ft) need not be retextured. Generally, it is required that a minimum of 95 percent of the area within any 0.9 m x 30.5 m (3 ft x 100 ft) test area be textured by the grinding operation.

Typically, four passes of one or more grinding machines are made to cover the entire width of a lane. The maximum overlap between passes should be 50 mm (2 in.) (Snyder et al. 1989). If not all the pavement lanes are to be ground, the machine should feather the adjacent lane to level with the depth of cut within approximately 0.9 m (3 ft) of the construction joint.

Transverse slope is also important and should be specified to obtain the desired grade. In general, a transverse slope of 1.7 percent, or 6.4 mm (0.25 in.) over 3.66 m (12 ft), is recommended. Grinding limits and transitions or stop lines at bridges and ramps should be marked clearly on the plans.

The quality acceptance criteria for diamond grinding vary between agencies, but there are some generally accepted criteria. The smoothness specified for newly constructed PCC pavements should be used as the criterion for assessing the ride quality of diamond grinding projects (Snyder et al. 1989; ERES 1993). AASHTO recommends 0.11 to 0.16 m / km (7 to 10 in. / mi) on new construction. Roughness can be measured using a variety of devices, such as K.J. Law Profilometer, California Profilograph, or Mays Meter. Profile traces obtained prior to grinding can be compared to profiles obtained after grinding to determine and document improvements in ride quality.

Diamond grinding lends itself to the use of an incentive-type specification for pavement smoothness. Modern equipment and experienced contractors have enabled smoother grinding projects to be obtained in recent years. The level of smoothness required has a great effect on the cost of the grinding operation. The setting of unrealistic levels that require extensive removal or additional grinding will cause a large increase in cost with limited returns. An incentive approach to smoothness specifications will help assure that the smoothest possible result will be achieved for a given set of conditions. The thrust of the contractors concern will not be just to meet the minimum specification requirement but to fine-tune grinding operation and equipment to earn the incentive payments that are available. However, all sudden changes in elevation (e.g., joint faulting, crack faulting, full-depth and partial-depth repair joints) should be eliminated to comply with the smoothness specifications. Since smoother pavements are expected to have a longer life, there is a direct payoff in performance to be realized by the incentive payment.

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REFERENCES

AASHTO, *AASHTO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, D.C., 1993.

ACPA, *The Concrete Pavement Restoration Guide – Procedures for Preserving Concrete Pavements*, Technical Bulletin TB020P, American Concrete Pavement Association, Skokie, Ill., 1997.

ACPA, *Diamond Grinding and Concrete Pavement Restoration 2000*, Technical Bulletin TB-008.0, American Concrete Pavement Association, Arlington Heights, Ill., and International Grooving and Grinding Association, Westlake, Texas, 1990.

Ardani, A., "Portland Cement Concrete Pavement Texturing Methods," *Transportation Research Record 1544*, TRB, National Research Council, Washington, D.C., 1996.

Banasiak, D., "Georgia Boasts Smoothest Roads," *Roads & Bridges*, Vol. 34, Issue 4, Scantron-Gillette, Des Plaines, Ill., 1996, pp. 32-34.

Bradbury, A. and Kazmierowski, T. J., "Seven Year Performance Evaluation of a CPR Project in Canada," *Proceedings, Sixth International Purdue Conference on Concrete Pavement Design and Materials for High Performance*, Purdue University, West Lafayette, Ind., 1997.

Caestecker, C. and Heleven, L., "Grinding of a Transversely Grooved Cement Concrete Pavement of the European Motorway E40 at Bierbeek," *Proceedings, Fifth International Conference on Concrete Pavement Design and Rehabilitation*, Purdue University, West Lafayette, Ind., 1993.

Christory, J. P., "Assessment of PIARC Recommendation on the Combating of Pumping in Concrete Pavements," *Sixth International Symposium on Concrete Roads*, The European Cement Association, Madrid, Spain, 1990.

Crovetti, J. A. and Darter, M. I., "Void Detection for Jointed Concrete Pavements," *Transportation Research Record 1041*, TRB, National Research Council, Washington, D.C., 1985.

Daleiden, J. F., *Video Inspection of Highway Edgedrain Systems*, Report FHWA-SA-98-044, Federal Highway Administration, Washington, D.C., 1998.

Defrain, L., *Noise Analysis of Ground Surface on I-69 WB Near Lowell Road C.S. 19043 – Research Project 88 TI-1342*, Office Memorandum, Michigan Department of Transportation, Lansing, Mich., 1989.

Drakopoulos, A., Wenzel, T. H., Shober, S. F., and Schmiedlin, R. B., "Comparison of Accident Experience Between Tined and Continuously Ground Portland Cement Concrete Pavements," Presented at the 77th Annual TRB Meeting Committee on Surface Properties – Vehicle Interaction (A2B07), TRB, National Research Council, Washington, D.C., 1998.

ERES Consultants, *Techniques for Pavement Rehabilitation*, Participant's manual for NHI training course, 5th ed., National Highway Institute, Washington, D.C., 1993.

ERES Consultants, *Pavement Subsurface Drainage Design*, Draft participant's manual for NHI training course, National Highway Institute, Washington, D.C., 1998.

Farnsworth, E. E., "Continuing Studies of Pavement Grooving in California," *Improving Pavement and Bridge Deck Performance*, Highway Research Board Special Report 116, HRB, National Research Council, Washington, D.C., 1971.

Grady, J. E. and Chamberlin, W. P., "Groove-Depth Requirements for Tine-Textured Pavements," *Transportation Research Record 836*, Transportation Research Board, Washington, D.C., 1981.

Gulden, W., (Unpublished) Presentation, CPR3 National Workshop, Harrisburg, Pa., 1996.

Gulden, W. and Thornton, J. B., *Restoration Methods for Jointed Concrete Pavements*, Research Report 7504, Georgia Department of Transportation, Forest Park, Ga., 1985.

Henry, J. J., "Use of Blank and Ribbed Test Tires for Evaluating Wet-Pavement Friction," *Transportation Research Record 788*, Transportation Research Board, Washington, D.C., 1980.

Henry, J. J. and Saito, K., "Skid-Resistance Measurements with Blank and Ribbed Test Tires and Their Relationship to Pavement Texture," *Transportation Research Record 946*, Transportation Research Board, Washington, D.C., 1983.

Henry, J. J. and Wambold, J. C., "Use of Smooth-Treaded Test Tire in Evaluating Skid Resistance," *Transportation Research Record 1348*, Transportation Research Board, Washington, D.C., 1992.

Horne, W. B., "Results From Studies of Highway Grooving and Texturing at NASA Wallops Station," NASA SP-5073, 1969, pp. 425-464.

Horne, W. B., *Safety Grooving, Hydroplaning and Friction*, Technical report prepared for the International Grooving and Grinding Association, International Grinding and Grooving Association, New York, N.Y.

Huang, Y. H., *Pavement Analysis and Design*, Prentice-Hall, Inc., Englewood Cliffs, N.J., 1993.

Hultqvist, B-A. and Carlsson, B., *Maintenance of Concrete Roads – Diamond Grinding on the Malmo-Vellinge Section of the E6*, Report VTI Meddelande N. 736, 1994.

IGGA, *A Level Road Rides Better, Lasts Longer*, International Grooving and Grinding Association, New York, N.Y., 1989.

Janssen, D.J. and Snyder, M.B., *Resistance of Concrete to Freezing and Thawing*, Report SHRP-C-391, TRB, National Research Council, Washington, D.C., 1994.

Korynata, A., "Dowel Retrofit Cuts Rehab Costs in Washington," *Better Roads*, Vol. 64, Number 8, Park Ridge, Ill., 1994, pp. 16-18.

Mack, J. W., "Dowel Retrofit Restores Pavement Load Transfer," *Concrete Repair Digest*, The Aberdeen Group, Addison, Ill., 1995.

Marquette University, *Noise and Texture on PCC Pavements – Results of a Multi-State Study*, Copy distributed as ACPA Technorandum, American Concrete Pavement Association, Skokie, Ill., 1998.

McGovern, M. S., "CPR Breathes New Life into North Carolina Pavement," *Concrete Repair Digest*, The Aberdeen Group, Addison, Ill., 1995.

Mindess, S. and Young, J.F., *Concrete*, Prentice-Hall, Inc., Englewood Cliffs, N.J., 1981.

Mosher, L. G., "Restoration of Final Surface to Concrete Pavement by Diamond Grinding," *Proceedings, Third International Conference on Concrete Pavement Design and Rehabilitation*, Purdue University, West Lafayette, Ind., 1985.

Neal, B. F. and Woodstrom, J. H., *Rehabilitation of Faulted Pavements by Grinding*, Report No. CA-DOT-TL-5167-4-76-18, California Department of Transportation, Sacramento, Calif., 1976.

Perenchio, W., "Effect of Surface Grinding and Joint Sawing on the Durability of Paving Concrete," *Journal of the PCA Research and Development Laboratories*, Vol. 6, No.1, Portland Cement Association, Skokie, Ill., 1964.

Pierce, L. M., "Portland Cement Concrete Pavement Rehabilitation in Washington State: Case Study," *Transportation Research Record 1449*, Transportation Research Board, Washington, D.C., 1995.

Roman, R. J., Darter, M. I., and Snyder, M. B., *Procedures to Determine the Optimum Time* to *Restore Jointed Concrete Pavements*, American Concrete Pavement Association / General Electric Company, Arlington Heights, Ill., 1985.

Sawyer, S., *Establishment of Underdrain Maintenance Procedure*, Report FHWA / OK95(04), Federal Highway Administration, Oklahoma City, Okla., 1995.

Snyder, M. B., Reiter, M. J., Hall, K. T., and Darter, M. I., *Rehabilitation of Concrete* Pavements, *Volume I : Repair Rehabilitation Techniques*, Report FHWA-RD-88-071, Federal Highway Administration, Washington, D.C., 1989.

Thornton, J. B. and Gulden, W., "Pavement Restoration Measures to Precede Joint Resealing," *Transportation Research Record 752*, Transportation Research Board, Washington, D.C., 1980.

Tyner, H. L., "Concrete Pavement Rehabilitation: Georgia Methodology," *Proceedings*, National *Seminar on PCC Pavement Recycling and Rehabilitation*, Report FHWA-TS-82-208, Federal Highway Administration, Washington, D.C., 1981.

Walters, W. C., *Investigation of Accident Reduction by Grooved Concrete Pavement*, Report FHWA / LA-79 / 133, Louisiana Department of Transportation, Baton Rouge, La., and Federal Highway Administration, Washington, D.C., 1979.

Wells, G.K. and Marsh, R., *Evaluate Performance of Grinding PCCP*, Minor Research Report 65332-638005-32135, California Department of Transportation, Sacramento, Calif., 1993.

Wu, C. and Nagi, M. A., *Optimizing Surface Texture of Concrete Pavement*, PCA Research and Development Bulletin RD111T, Portland Cement Association, Skokie, Ill., 1995.

Yu, H.T., Smith, K.D., Darter, M.I., Jiang, J., and Khazanovich, L., *Performance of Concrete Pavements Volume III—Improving Concrete Pavement Performance*, Report FHWA-RD-95-111, Federal Highway Administration, Washington, D.C., 1997.

Yu, H. T., Khazanovich, L., Rao, S., Darter, M. I., and Von Quintus, H., *Guidelines for Subsurface Drainage Based on Performance*, Final Report Prepared for National Cooperative Highway Research Program Project 1-34. NCHRP, TRB, National Research Council, Washington, D.C., 1998.

APPENDIX A
Faulting Model

Faulting

Transverse joint faulting is a difference in slab elevations across the joint and is the result of a combination of heavy axle loads, insufficient load transfer between adjacent slabs, free moisture beneath the pavement, and erosion of the supporting base or subgrade material from beneath the slab. Erosion occurs when excess moisture is ejected from beneath the leave slab corner under the influence of a vehicle load. The moisture that is ejected carries base and / or subgrade fines with it, resulting in the development of a void beneath the pavement at the leave slab corner. In addition, there is a corresponding deposition of this material under the approach slab corner. Due to the build-up of material beneath the approach corner and the loss of support under the leave corner, faulting and corner cracking can develop (Simpson et al. 1994).

Transverse joint faulting is an important deterioration mechanism for JCP because of its highly negative impact on ride quality. Significant joint faulting has a major impact on the life cycle costs of the pavement in terms of rehabilitation and vehicle operating costs.

The objective of this part of the study was to quantify the long-term effects of diamond grinding of JCP. The ability to quantify the effectiveness of subsurface drainage would enable objective determination of the cost-effectiveness of this rehabilitation alternative. This was accomplished by developing a faulting prediction model based on performance data collected from in-service pavement sections.

This section begins with a review of the existing faulting models for new pavements, followed by the presentation of the proposed faulting model. Model calibration is presented, along with an example calculation and a brief sensitivity analysis. The mechanistic-based faulting model was developed by performing an iterative nonlinear regression analysis to minimize the error between the field faulting data and the predicted faulting data.

Previous Studies

Several faulting models are available for new pavements. These models can be categorized as either empirical or mechanistic-empirical. Empirical models predict JCP faulting based on observations taken from a database. The range of variables and number of cases in the database limit the usefulness of empirical models. Furthermore, model applications outside of the database inference space may be inaccurate. Thus, the best empirical distress models are often those that are derived from large, nationwide databases.

Several empirical models were derived from national databases under the NCHRP 1-19 study (COPES), LTPP Data Analysis Study, and a recently completed FHWA-sponsored project on jointed concrete pavements (RIPPER)(Darter et al. 1985; Simpson et al. 1994; Yu et al. 1997). These models identified several pavement design features and site conditions that affect transverse joint faulting. For example, the RIPPER models developed for nondoweled JPCP and doweled JCP, respectively, are:

$$FaultND = CESAL^{0.25} * [0.2347 - 0.1516*C_d - 0.000250 * Hpcc^2 / JTSPACE^{0.25} - 0.0115 * BASE + 0.7784 * 10^{-7} * FI^{1.5}*PRECIP^{0.25} - 0.002478 * DAYS90^{0.5} - 0.0415 * WIDENLANE] \quad (A-1)$$

N=131
R^2=0.45
RSE=0.034 in(0.86 mm)

$$FaultD = CESAL^{0.25} * [0.0628 - 0.0628 * C_d + 0.3673*10^{-8} * BSTRESS^2 + 0.4116 * 10^{-5} * JTSPACE^2 + 0.7466 * 10^{-9} * FI^2 * PRECIP^{0.5} - 0.009503 * BASE - 0.01917 *WIDENLANE + 0.0009217 * Age] \quad (A-2)$$

N=146
R^2=0.60
RSE=0.022 in (0.56 mm)

where:

Age	=	Pavement age, years.
BASE	=	Base type (0 = nonstabilized base; 1 = stabilized base).
BSTRESS	=	Maximum dowel / concrete bearing stress, lb / in^2.
C_d	=	Modified AASHTO drainage coefficient, calculated from database information.
CESAL	=	Cumulative 18-kip (80-kN) equivalent single axle loads, millions.
DAYS90	=	Average number of days with maximum temperature above 90 °F (32 °C).
FI	=	Mean annual freezing index, degree-days.
JTSPACE	=	Mean transverse joint spacing, ft.
Hpcc	=	PCC slab thickness, in.
WIDENLANE	=	Widened lane (0 = not widened, 1 = widened).
PRECIP	=	Mean annual precipitation, in.

These models generally agree with the LTPP Early Analysis and NCHRP 1-19 models (Darter et al. 1985; Simpson et al. 1994). All of these models predict faulting as a function of site conditions and pavement design features. The parameter with the greatest effect on joint faulting is the presence and configuration of dowels. The models show that heavy traffic loading is strongly correlated with faulting (i.e., increases in the number of heavy axle loads correspond to increased joint faulting). The models also suggest that pavement design features such as drainage, joint spacing, base type, and the presence of a widened lane have significant effects on faulting. Climatic conditions, such as precipitation and freeze-thaw (characterized by the Freezing Index and measures of annual precipitation) also influence faulting significantly.

The faulting of nondoweled and doweled concrete pavements has also been modeled using M-E procedures for several years. Faulting models for doweled concrete pavements typically consider computed dowel / concrete bearing stress as the critical response output from a structural pavement model and relates that to joint faulting through correlation with field data (Yu et al. 1997). Faulting models for nondoweled pavements typically utilize corner deflection or load transfer across the joint as the critical pavement response.

Two M-E faulting models were developed by the ACPA (Wu et al. 1993). These models are extensions of the PCA faulting models developed by Packard (1977) and use erodibility as the main factor influencing faulting. Using Miner's linear damage hypothesis, the percent of erosion damage occurring at the slab corner was defined as:

$$EROSION = 100*C_2*\sum(n_i / N_i) \quad (A-3)$$

where:

EROSION	=	Percent erosion damage.
n_i	=	Expected number of axle load repetitions for axle group "i."
N_i	=	Allowable number of repetitions for axle group "i."
C_2	=	Constant that varies with the degree of edge support provided (0.06 for pavements without a shoulder and 0.94 for pavements with a tied concrete shoulder).

The allowable number of load applications for a given axle group, N_i, was related to the power or rate of work, P, of each axle pass at the corner of the slab as:

$$\log N = 14.524 - 6.777 * (C_1 * P - 9.0)^{0.103} \quad (A-4)$$

where:

N	=	Allowable load repetitions to end of design period.
P	=	Power.
C_1	=	1 - (KSTATIC / 2000*4 / Hpcc t)^2.

The following equation was used to calculate the power of each axle pass at the corner of the slab:

$$P = 268.7 * p^2 / Hpcc / KSTATIC^{0.73} \quad \text{(A-5)}$$

where:

P = Power (rate of work).
p = Pressure at slab-foundation interface, psi.
Hpcc = Slab thickness, in.
KSTATIC = Modulus of subgrade reaction, psi/in.

The following models were then developed through regression techniques for predicting faulting of undoweled and doweled pavements, respectively:

$$FAULTND = EROSION^{0.25} [9.75873x10^{-4}*(PRECIP)^{0.91907} + 0.0060291*JTSPACE^{0.54428} - 0.016799*DRAIN] \quad \text{(A-6)}$$

N=582
R^2=0.743
RSE=0.055 in(0.86 mm)

$$FAULTD = EROSION^{0.25} [0.0038332*(PRECIP / 10)^{1.84121} + 0.0057763*JTSPACE^{0.38274}] \quad \text{(A-7)}$$

N=281
R^2=0.703
RSE=0.047 in (0.86 mm)

where:

FAULTND = Faulting at undoweled transverse pavement joints, in.
FAULTD = Faulting at doweled transverse pavement joints, in.
EROSION = Calculated accumulated erosion.
PRECIP = Annual precipitation, in.
JTSPACE = Joint spacing, ft.
DRAIN = Dummy variable for the presence of edge drains (0 when edge drains are not present, 1 when edge drains are present).

These models generally agree with the RIPPER and LTPP Early Analysis faulting models. In addition, they identify the PCC thickness as a significant parameter that is negatively correlated with faulting.

In the recent Nationwide Pavement Cost Model (NAPCOM) refinement study (Owusu-Antwi et al. 1997), the following M-E faulting model was developed:

$$FAULT = DAMAGE^{0.23}(0.35-0.0277 *BASE – 0.25C_d + 2.17E-5*FI) \quad \text{(A-8)}$$

N=101
R^2=0.52
RSE=0.03 in (0.86 mm)

where:

FAULT = Mean transverse joint faulting, in.
Damage = $\sum n_i / N_i$.
n_i = Number of applications for each axle group, i.
N_i = Allowable number of load repetitions for each axle group, i.
C_d = Drainage coefficient.
BASE = Base type (0 = nonstabilized base; 1 = stabilized base).
FI = Freezing index .

The allowable number of load repetitions is defined as follows:

$$\text{Log } N = 4.27 – 1.6*\text{Log}(DE – 0.002) \quad \text{(A-9)}$$

where:

N = Allowable number of load repetitions.
DE = Differential of subgrade elastic energy density.

The model predicts faulting for doweled and nondoweled pavements and illustrates that the presence of dowels significantly reduces faulting through reduction of DE. In addition, it indicates that a stabilized base, stiff subgrade, and improved drainage are negatively correlated with faulting.

In the recent LTPP study (Titus-Glover et al. 1997), the NAPCOM faulting model was recalibrated using data from the LTPP database. The following model was obtained:

$$FAULT=DAMAGE^{0.3}*[0.05+0.0004*WETDAYS –0.0024*DOWDIA– 0.025*C_d*(0.5+BASE)] \quad \text{(A-10)}$$

N=120
R^2=0.56
RSE=0.03 in (0.86 mm)

where:

Damage = CESAL / N.
CESAL = Cumulative ESALs.
N = Allowable number of 80-kN (18-kip) load repetitions.
C_d = Drainage coefficient.
BASE = Base type (0 = nonstabilized base; 1 = stabilized base).
WETDAYS = Average number of wet days per year.
DOWDIA = Dowel diameter.

The main difference in these last two models is that the LTPP model characterizes traffic in terms of ESALs, whereas the NAPCOM model uses actual axle loads. In addition, the NAPCOM model considers the freezing index to be an important climatic parameter, whereas the LTPP model identifies the number of wet days per year as positively correlated with faulting.

The most recent modification of the NAPCOM model was developed under the NCHRP 1-34 study (Yu et al. 1998). In this study, it was postulated that an increase in differential energy leads to a reduction in the number of allowable load applications for any given level of faulting. However, the presence of a drainage system may increase the allowable number of load applications by keeping the base / subgrade dry, thereby making the system more erosion-resistant and significantly reducing the rate of base / subgrade deterioration. This effect was significant only for nondoweled pavements; the dowel load transfer mechanism is apparently less affected by drainage effects.

The following model was proposed for estimating the allowable number of ESAL applications:

$$\text{Log } N = 0.785983 - 0.9291{*}\text{Log}(DE) {*}(1 + 0.4{*}PERM{*}(1\text{-}DOWEL)) \tag{A-11}$$

where:

N = Allowable number of ESAL applications.
DE = Differential energy density.
DOWEL = Dummy variable (= 1 if dowels present; = 0 otherwise).
PERM = Base permeability (0 = not permeable, 1 = permeable).

The final model developed under NCHRP 1-34 for joint faulting was:

$$FAULT = DAMAGE^{0.2475}{*}[0.2405 - 0.00118{*}DAYS90 + 0.001216{*}WETDAYS\text{-}0.04336{*}BASE - (0.004336 + 0.007059{*}(1 - DOWEL)){*}LCB) \tag{A-12}$$

N=391
R^2=0.50
RSE=0.035 in

where:

FAULT = Mean transverse joint faulting, in.
BASE = Base type (=0 if nonstabilized or lean concrete base; = 1 otherwise).
LCB = Lean concrete base (= 1 if lean concrete base, = 0 otherwise).
WETDAYS = Average number of wet days per year.
DAYS90 = Average number of days with the maximum temperature above 90 °F (32 °C)
DOWEL = Presence of dowels (=0 if no dowels present, =1 otherwise).
Damage = n / N.
n = Number of cumulative equivalent single axle load applications, millions.
N = Allowable number of 18-kip (80-kN) load applications.

Model Development

<u>General Description of Data Used in Model Development</u>

The pavement sections analyzed in this study are located throughout the United States, with the majority located in the upper Midwest. A variety of design features (slab thickness, joint spacing, load transfer, and so on) are present on these pavement sections. The range of design features encountered in these projects is summarized in table A-1. This table shows that a significant range of variables exists, although often these ranges occur over different projects located in different climates. The inference space (factorial matrix) of the 89 pavement sections is also shown in table A-1.

<u>Explanatory Variables Initially Selected</u>

The dependent variable in the nondoweled joint faulting model is the mean transverse joint faulting measured about 1 ft (0.3 m) from the slab edge on each section. Based on the previous studies of joint faulting modeling, a set of variables was selected. The explanatory variables that were initially considered are as follows:

CESAL: Cumulative 18-kip (80-kN) equivalent single axle loads, millions.
Jointspace: Mean transverse joint spacing, ft.
Slabthick: PCC slab thickness, in.
Base: Base type (0 = nonstabilized base; 1 = stabilized [asphalt, cement] base
PERM: Base permeability (0 = not permeable, 1 = permeable).
DRAIN: Drainage provisions (0 = no edge drains; 1 = edge drains).
Soiltype: Subgrade soil type (0 = fine grained; 1 = coarse grained).
Kstatic: Static backcalculated k-value, $lb/in^2/in$.
FI: Mean annual freezing index, degree-days.
Precip: Mean annual precipitation, in.

In this study, the NCHRP 1-34 model for new was pavements was modified and recalibrated using the database of faulting data for nondoweled diamond grounded pavements. The new model also employs the differential subgrade energy parameters, DE, as the main mechanistic response parameters. The DE parameter can be calculated using the following step-by step procedure (Yu et al. 1998):

Step 1. Calculate the radius of relative stiffness, l.

$$l = \left(\frac{Epcc^{*}\ Hpcc^{3}}{12(i-\mu^{2})k} \right)^{0.25} \qquad \text{(A-13)}$$

where:

Hpcc = PCC slab thickness, in.
Epcc = PCC modulus of elasticity, psi.
µ = PCC Poisson=s ratio (usually assumed to be equal to 0.15).
k = Modulus of subgrade reaction, psi/in.

Step 2. Calculate the nondimensional effective aggregate interlock stiffness.

$$AGG^{*} = 2.3\ \mathrm{Exp}(1\text{-}1.987\ JTSPACE/l) \qquad \text{(A-14)}$$

where:

AGG* = Nondimensional aggregate load transfer factor.
k = Modulus of subgrade reaction.
l = Slab=s radius of relative stiffness.
JTSPACE = Slab length.

Table A-1. Range of design features included in study.

Design Feature	Range of Design Feature	Distribution of Design Feature	
		Categories	**Number in Category**
Slab Thickness	7 to 15 in	<9 9 to 9.9 in 10 to 11.9 in ≥ 12 in	35 40 12 2
Joint Spacing	5 to 70 ft	£15 ft 15.1 to 20 ft 20.1 to 25 ft ≥25 ft	24 33 10 22
Joint Orientation		Non-skewed Joints Skewed Joints	79 10
Base Type		Non-stabilized Stabilized	45 44
Drainage		Edgedrains Permeable Base	17 19

*Note:*1 in = 25.4 mm; 1 ft = 0.305 m

Step 3. Calculate maximum corner deflections of the loaded and unloaded slabs assuming that the 18-kip single axle loaded is located 12 in from the corner (36 in from the corner if widened slab is used). A finite element program such as ILLI-SLAB can be used for this purpose, or the following simplified equations might be used:

Step 3.1. Calculate free edge corner deflection (no load transfer to the adjacent slab), $w_{fe,0}$, assuming that the load is placed right at the slab corner using the following equation:

$$w_{fe,0} = (0.0000864*l^2 + 0.002824*l + 0.2953)*18000/kl^2 \quad \text{(A-15)}$$

Step 3.2. Calculate free edge corner deflection (no load transfer to the adjacent slab), $w_{fe,36}$, assuming that the load is placed at the transverse joint 36 in away from the slab corner:

$$w_{fe} = (0.0000648*l^2 + 0.003934*l - 0.02548)*18000/kl^2 \quad \text{(A-16)}$$

Step 3.3. Using an equation developed by Crovetti (1995), calculate LTE for a load placed at the slab corner:

$$LTE_{x=0}(\%) = 1 / [0.01 + 0.012*(AGG/Kl)^{-0.849}] \quad \text{(A-17)}$$

Step 3.4. Using the following modified Crovetti's equation, calculate LTE at the slab corner for a load placed 36 in away from the slab corner:

$$LTE_{x=36}(\%) = 1 / [0.01 + 0.003483*(AGG/Kl)^{-1.13677}] \quad \text{(A-18)}$$

Step 3.5. Using the corresponding LTE and corner deflections, calculate the corner deflection of the unloaded slab for a load placed at 0 and 36 in from the slab corner, $w_{UL,0}$ or $w_{UL,36}$, respectively, using the following relationship:

$$w_{UL,0\text{ or }36} = w_{fe,0\text{ or }36} *(LTE_{x=0\text{ or }36})/(1 + LTE_{x=0\text{ or }36}) \quad \text{(A-19)}$$

Step 3.6. Calculate the corner deflection of the unloaded slab, w_{UL}, for an axle load located at the transverse joint in the wheelpath (12 in from the slab corner for a non-widened slab) by interpolating between the unloaded slab deflections at x = 0 and 36 in.

$$w_{UL} = w_{UL,0} +(w_{UL,36}-w_{UL,0})*x/36 \quad \text{(A-20)}$$

where x is the distance from the wheelpath to the slab corner.

Step 3.6. Calculate the LTE at the slab corner for an axle load located at the transverse joint in the wheelpath (12 in from the slab corner for a non-widened slab) by interpolating between the LTEs at x = 0 and 36 in.

$$LTE = LTE_0 +(LTE_{36}-LTE_0)*x/36 \quad \text{(A-21)}$$

Step 3.7. Calculate the loaded corner deflection, w_L, for the load located in the wheelpath using the following relationship:

$$w_L = w_{UL} / LTE \quad \text{(A-22)}$$

Step 3.8 Calculate the differential energy density, DE:

$$DE=1/2\ k\ (w_L^2 - w_{UL}^2) \quad \text{(A-23)}$$

Step 4. Calculate the allowable number of ESAL applications using equation A-24.

The model for estimating the allowable number of ESAL applications remained intact; however, no credit was given to permeable bases. Indeed, since the pavement was ground before it exhibited significant faulting, it reasonable to assume that the base could be clogged or poorly compacted and, therefore, malfunctioning. Therefore, the following model was proposed for estimating the allowable number of ESAL applications:

$$\text{Log } N = 0.785983 - 0.9291*\text{Log}(DE) \quad \text{(A-24)}$$

where:

N = Allowable number of ESAL applications.
DE = Differential energy density.

The faulting final model for diamond-ground pavements has the following form:

$$FAULT = DAMAGE^{0.35}*[0.31 + 0.00474*PRECIP] \quad \text{(A-25)}$$

N=89
R^2=0.53
RSE=0.042 in

where:

PRECIP = Annual precipitation, in.
DAMAGE = CESAL / N.
CESAL = Cumulative 18,000-lb (80-kN) ESALs in traffic lane, millions.

Figure A-1 provides a plot of the predicted versus actual measured faulting. The residual versus predicted faulting plot (goodness-of-fit) is given in figure A-2. These figures show that prediction errors are fairly symmetrically distributed around zero, with a few apparent outliers.

- Fifty-three percent of the total variation of joint faulting can be explained by the included variables.
- Some of the unexplained variation is caused by errors in the independent variables used to develop the model, such as the traffic estimates.
- The Aaverage@ error in predicting faulting is 0.042 in (1.0 mm).
- There are no discernible patterns in the residuals.
- Data from 89 JCP sections across the United States were used to develop the model.
- Each independent variable and the overall model were significant at a level of significance of 5 percent or lower.
- The sensitivity analysis shows that all of the explanatory variables have a plausible effect on faulting that agrees with theoretical expectations and previous empirical field results.

Figures A-3 through A-6 show sensitivity analyses of key variables in the faulting model for diamond-ground pavements. The plots show that faulting always increases with traffic; an increase in PCC thickness decreases predicted faulting, whereas increases in joint spacing and precipitation increase faulting.

It should be noted that edgedrains were not found to significantly affect faulting. This contradicts the conclusion of the study conducted by the University of Illinois (Snyder et al. 1989). To resolve this issue, the model was recalibrated using the University of Illinois data only. It was confirmed that for those sections, the presence of edgedrains was indeed a significant factor reducing faulting. However, the edgedrains on these sections were newly installed; with time, they could be clogged and may actually increase faulting by trapping water inside the pavement system and increasing erodibility of the base and subgrade. This observation would be more in line with the findings from the NCHRP 1-34 study.

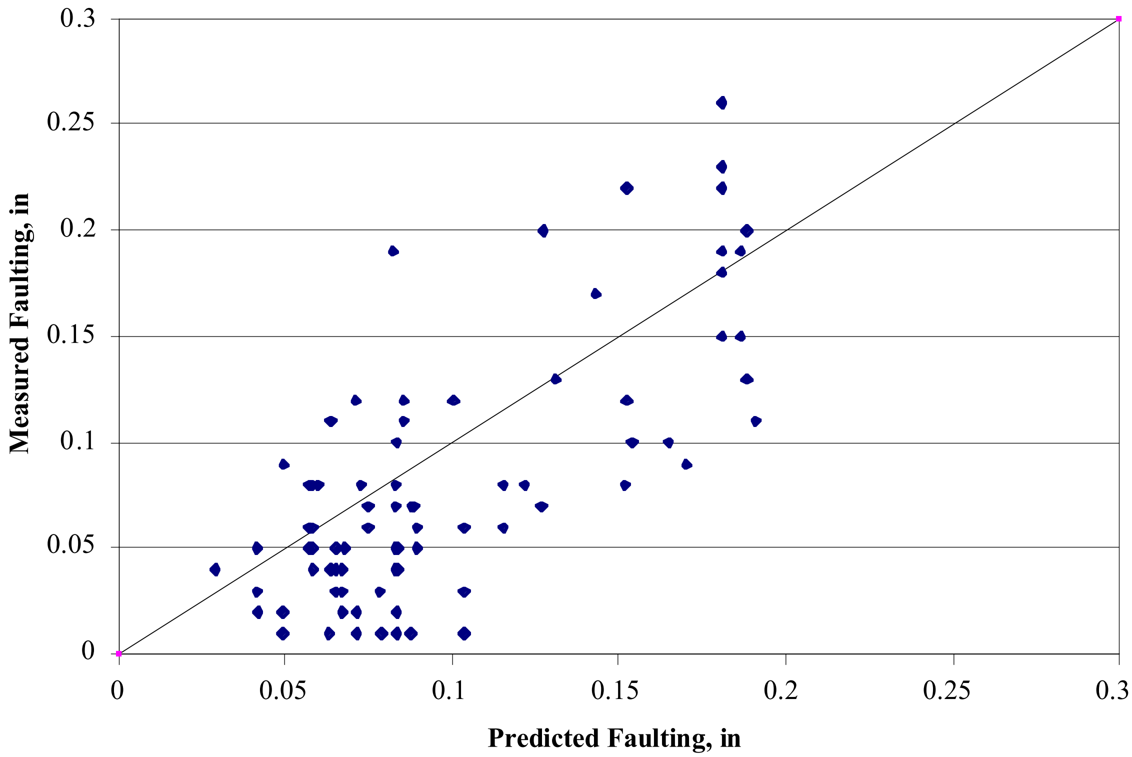

Figure A-1. Predicted faulting vs. measured faulting for nondoweled diamond-ground pavements.

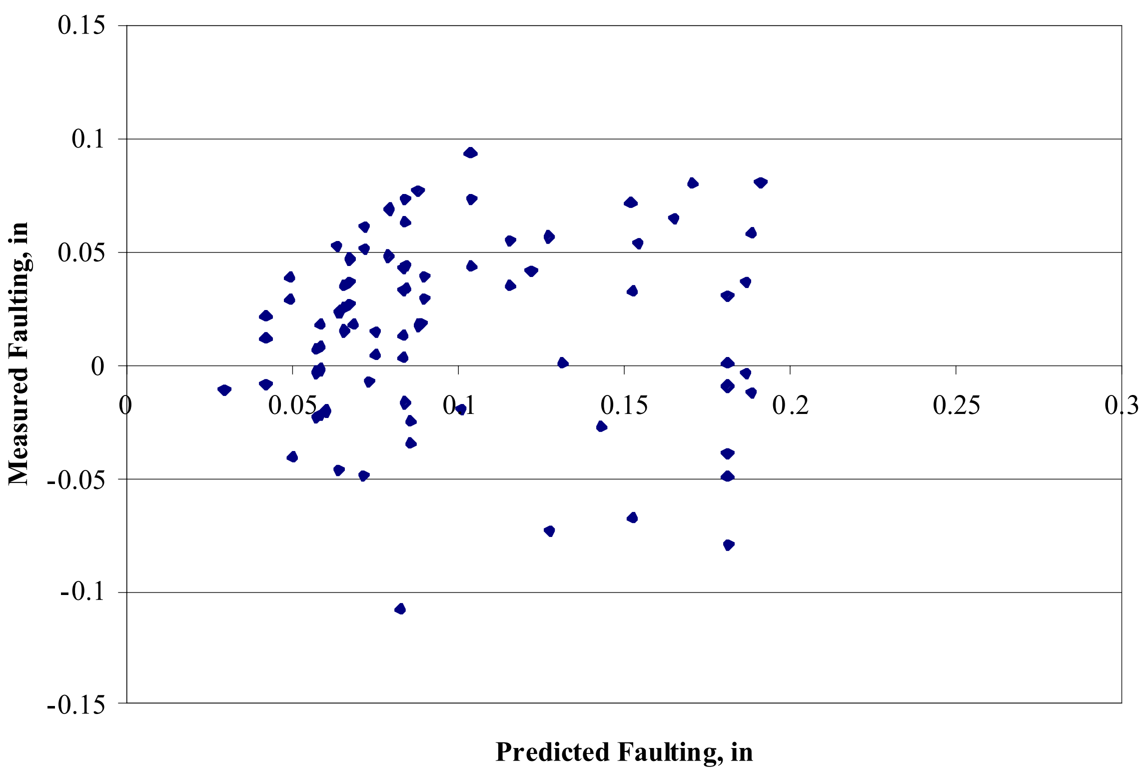

Figure A-2. Residual plot vs. predicted faulting for diamond-ground pavements.

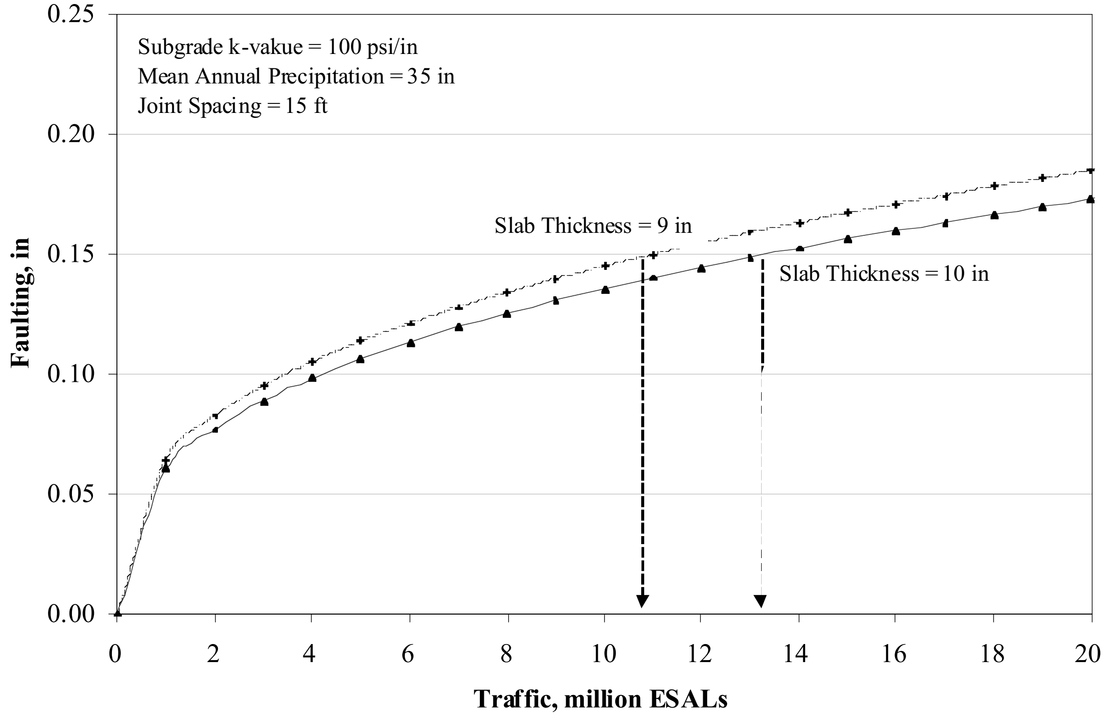

Figure A-3. Effect of traffic and PCC thickness on joint faulting.

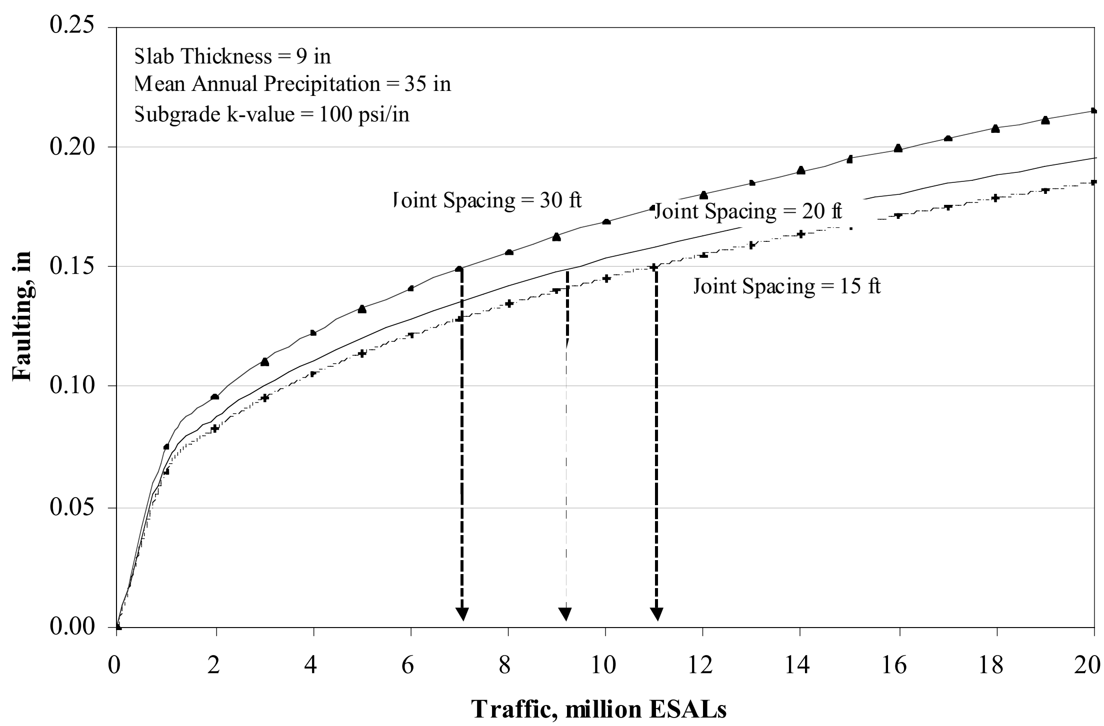

Figure A-4. Effect of traffic and joint spacing on joint faulting.

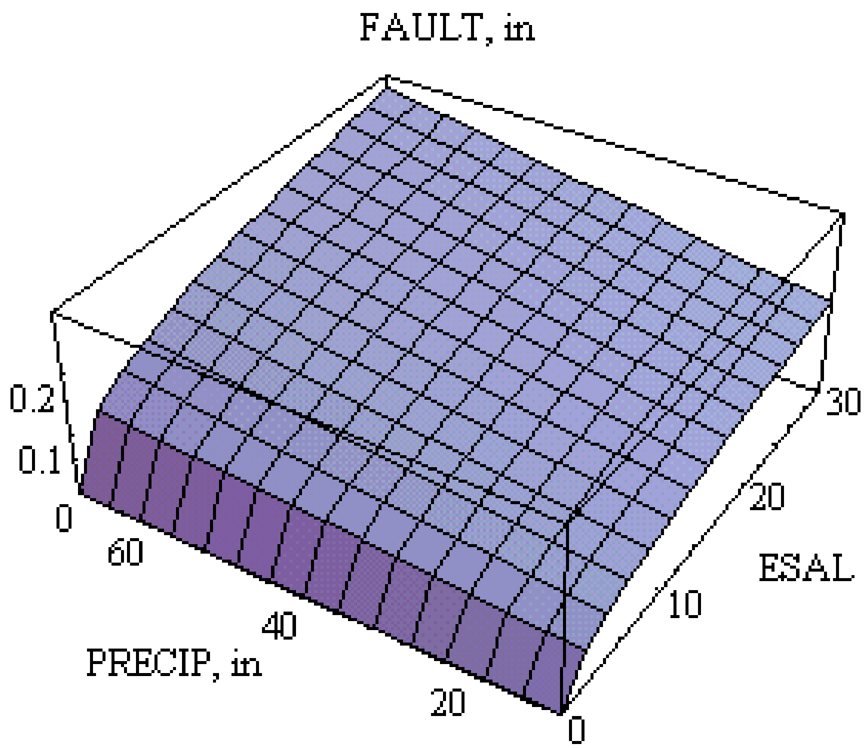

Figure A-5. Effect of traffic and precipitation on joint faulting (fine subgrade).

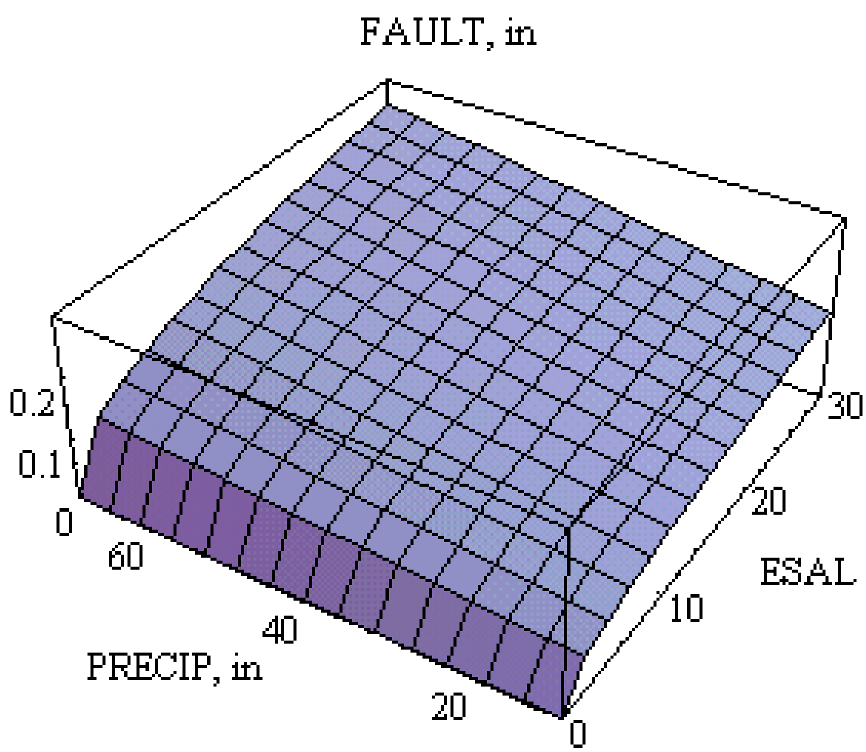

Figure A-6. Effect of traffic and precipitation on joint faulting (coarse subgrade).

REFERENCES

Crovetti, J. A. 1995. *Analysis of Support Conditions Under Jointed Concrete Slabs Along USH 18/151*. Report WI / SPR-01-95. Madison, WI: Wisconsin Department of Transportation.

Darter, M. I., J. M. Becker, M. B. Snyder, and R. E. Smith. 1985. *Concrete Pavement Evaluation System (COPES).* NCHRP Report 277. Washington, DC: Transportation Research Board.

Owusu-Antwi, E. et al. 1997. *Development and Calibration of Mechanistic-Empirical Distress Models for Cost Allocation*. Draft Final Report. Washington, DC: Federal Highway Administration.

Packard, R.G. 1977. "Design Considerations for Control of Joint Faulting of Undoweled Pavements." *Proceeding, First International Conference on Concrete Pavement Design*. West Lafayette, IN: Purdue University.

Simpson, A. L., Owusu-Antwi, E. O., Pendelton, O. J., and Y. Lee. 1994. *Sensitivity Analysis for Selected Pavement Distresses*. SHRP Report No. SHRP-P-393. Washington, DC: Strategic Highway Research Program, National Research Council.

Titus-Glover, L., E. Owusu-Antwi, and T. E. Hoerner. 1997. *Long Term Pavement Performance Data Analysis. Volume II: Design and Construction of PCC Pavements—Improved PCC Performance Models*. Washington, DC: Federal Highway Administration.

Snyder, M. B., M. J. Reiter, K. T. Hall, and M. I. Darter. 1989. *Rehabilitation of Concrete Pavements, Volume I : Repair Rehabilitation Techniques*. Report FHWA-RD-88-071. Washington, DC: Federal Highway Administration.

Wu, C. L., J. W. Mack, P. A. Okamoto, and R. G. Packard. 1993. "Prediction of Faulting of Joints in Concrete Pavements." *Proceedings, Third International Conference on Concrete Pavement Design and Rehabilitation*. West Lafayette, IN: Purdue University.

Yu, H.T., K.D. Smith, M.I. Darter, J. Jiang, and L. Khazanovich. 1997. *Performance of Concrete Pavements Volume III—Improving Concrete Pavement Performance*. Report FHWA-RD-95-111. Washington, DC: Federal Highway Administration.

Yu, H. T., L. Khazanovich, S. Rao, M. I. Darter, and H Von Quintus. 1998. *Guidelines for Subsurface Drainage Based on Performance*. Final Report Prepared for National Cooperative Highway Research Program Project 1-34. Washington, DC: NCHRP, TRB, National Research Council.

APPENDIX B
Summary Tables

The tables included in this appendix are listed below:

Table B-1. Project identification and location information for survey sections.

Sect. ID (Long)*	County	Begin MP	End MP	Hwy. Type	Hwy. No.	Dir.	No. of Lanes	Sample Unit No.	Sample Unit Length, ft	Year Ground	FHWA Study Section?
ALABAMA											
AL-IH-20E-157.5	St. Clair	157.5	157.6	IH	20	E	2	1	560	85	No
AL-IH-20E-183.0	Calhoun	183.0	183.1	IH	20	E	2	1	480	86	No
AL-IH-59N-185.1	Etowah	185.1	185.2	IH	59	N	2	1	500	91	No
AL-IH-59N-235.5	Dekalb	235.5	235.6	IH	59	N	2	1	500	83	No
ARIZONA											
AZ-IH-17S-199.3	Maricopa	199.3	199.2	IH	17	S	3	1	310	91	Yes
CALIFORNIA											
CA-IH-80W-9.0	Nevada/Placer			IH	80	W	2	1	540	97	No
CA-IH-80W-37.9	Solano	37.9	37.8	IH	80	W	3	1	560	91	No
CA-IH-80W-37.8	Solano	37.8	37.7	IH	80	W	3	1	430	91	No
CA-US-101N-84.9	Monterey	84.9	85.0	US	101	N	2	1	510	80	Yes
CA-US-101S-56.7	Monterey	56.7	56.6	US	101	S	2	1	510	80	No
CA-US-101N-39.2	San Luis Obispo	39.2	39.3	US	101	N	2	1	530	97	Yes
CA-US-101S-39.5	San Luis Obispo	39.5	39.4	US	101	S	2	1	510	97	Yes
CA-IH-10E-17.4	San Bernardino	17.4	17.5	IH	10	E	4	1	500	84	No
CA-IH-8E-43.4	Imperial	43.4	43.6	IH	8	E	2	1	1010	97	No
FLORIDA											
FL-IH-95N-358.3	Duval	358.3	358.4	IH	95	N	2	1	500	96	No
FL-IH-10W-230.8	Jefferson	230.8	230.7	IH	10	W	2	1	500	92	No
FL-IH-10E-214.7	Leon	214.7	214.8	IH	10	E	2	1	540	92	No
FL-IH-10W-106.5	Washington	106.5	106.4	IH	10	W	2	1	500	91	No
FL-IH-10E-99.0	Holmes	99.0	99.2	IH	10	E	2	1	1000	91	No
FL-IH-10W-1.2	Escambia	1.2	1.0	IH	10	W	2	1	1000	94	Yes
GEORGIA											
GA-IH-75S-66.4	Tift	66.4	66.2	IH	75	S	2	1	780	94	Yes
GA-IH-16W-59.9	Laurens	59.9	59.8	IH	16	W	2	1	600	97	Yes
GA-IH-75N-193.6	Monroe	193.6	193.7	IH	75	N	3	1	600	92	Yes
GA-IH-75N-227.7	Clayton	227.7	227.8	IH	75	N	3	1	300	91	Yes
GA-IH-985S-6.7	Gwinnet	6.7	6.6	IH	985	S	2	1	720	81	Yes
IOWA											
IA-IH-80W-87.7	Adair	87.7	87.6	IH	80	W	2	1	500	84	No
ILLINOIS											
IL-IH-72W-168.9	Champaign	168.8	169.0	IH	72	W	2	1	1000	97	No
IL-IH-74E-8.0	Henry	8.0	8.2	IH	74	E	2	1	1000	84	Yes
IL-IH-74W-9.0	Henry	9.0	9.2	IH	74	W	2	1	1000	84	Yes
IL-IH-280E-16.0	Rock Island	16.0	16.2	IH	280	E	2	1	1000	84	Yes
IL-IH-280E-17.0	Rock Island	17.0	17.2	IH	280	E	2	2	1000	84	Yes
MINNESOTA											
MN-US-10E-208.0	Sherburne	208.0	208.1	US	10	E	2	1	500	82	Yes
MN-US-10W-205.5	Sherburne	205.5	205.4	US	10	W	2	1	500	82	Yes
MN-US-10W-88.4	Wadena	88.4	88.3	US	10	W	1	1	480	85	No
MN-US-10W-87.8	Wadena	87.8	87.7	US	10	W	1	2	480	85	No

Table B-1. Project identification and location information for survey sections.

Sect. ID (Long)*	County	Begin MP	End MP	Hwy. Type	Hwy. No.	Dir.	No. of Lanes	Sample Unit No.	Sample Unit Length, ft	Year Ground	FHWA Study Section?
MISSISSIPPI											
MS-IH-55N-11.7	Pike	11.7	11.8	IH	55	N	2	1	500	87	No
MS-IH-55N-9.8	Pike	9.8	9.9	IH	55	N	2	2	500	87	No
MS-IH-59S-158.4	Lauderdale	158.4	158.2	IH	59	S	2	1	1020	86	No
NORTH CAROLINA											
NC-IH-26E-19.0	Henderson	19.0	19.2	IH	26	E	2	1	1020	93	No
NEBRASKA											
NE-IH-80W-420.1	Cass	420.1	420.0	IH	80	W	2	1	510	89	No
NEVADA											
NV-IH-80E-179.7	Humboldt	179.7	179.9	IH	80	E	2	1	930	87	No
NV-IH-80W-111.9	Pershing	111.9	111.7	IH	80	W	2	1	930	94	No
OHIO											
OH-CONTECH	Lorain	2.8	2.6	SR	2	W	2	1		92	No
SOUTH CAROLINA											
SC-IH-85N-27.0	Anderson	27.0	27.1	IH	85	N	2	1	570	91	Yes
SC-IH-85N-31.0	Anderson	31.0	31.1	IH	85	N	2	2	570	91	Yes
SC-IH-20E-2.0	Aiken	2.0	2.1	IH	20	E	2	1	570	84	Yes
SC-IH-20E-4.0	Aiken	4.0	4.1	IH	20	E	2	2	600	84	Yes
SC-IH-20E-55.6	Lexington	55.6	55.7	IH	20	E	2	1	570	97	No
SC-IH-95N-162.3	Florence	162.3	162.5	IH	95	N	2	1	950	93	No
SOUTH DAKOTA											
SD-US-14W-402.6	Kingsbury	402.6	402.4	US	14	W	2	1	1040	91	No
SD-IH-29S-174.0	Codington	174.0	173.8	IH	29	S	2	1	930	90	No
SD-IH-29S-168.5	Hamin	168.5	168.3	IH	29	S	2	2	930	90	No
SD-IH-90E-290.0	Brule	290.0	290.2	IH	90	E	2	1	1020	97	Yes
SD-IH-90E-61.0	Pennington	61.0	61.2	IH	90	E	2	1	1170	81	Yes
SD-IH-90W-42.5	Meade	42.5	42.3	IH	90	W	2	1	980	81	Yes
TENNESSEE											
TN-IH-65S-91.9	Davidson	91.9	91.8	IH	65	S	3	1	600	89	No
WISCONSIN											
WI-US-61N-0	Boscobel			US	61	N	1	1	580	81	Yes
WI-IH-43N-2.7	Rock	2.7	2.9	IH	43	N	2	1	900	94	No
WYOMING											
WY-IH-90W-191.2	Crook	191.2	191.0	IH	90	W	2	1	930	87	No
WY-IH-90W-131.2	Campbell	131.2	131.0	IH	90	W	2	1	930	85	No

* Note: Section ID (Long) format - "State-Highway Type-Highway Number and Direction-Starting Milepost."

Table B-2. Climatic information for survey sections.

Sect. ID	Climatic Region	Latitude	FI	Average Annual Temp.	Average Annual Max. Temp.	Average Annual Min. Temp.	Average Annual Precip.
ALABAMA							
AL-IH-20E-157.5	WN	34	0	62.0	73.4	50.5	52.8
AL-IH-20E-183.0	WN	34	0	62.0	73.4	50.5	52.8
AL-IH-59N-185.1	WN	34	0	60.7	72.2	49.3	54.8
AL-IH-59N-235.5	WN	35	0	56.7	69.5	43.9	58.3
ARIZONA							
AZ-IH-17S-199.3	DN	33	0	71.2	85.1	57.3	7.1
CALIFORNIA							
CA-IH-80W-9.0	DF	39	0	42.6	58.7	26.5	39.5
CA-IH-80W-37.9	DN	39	0	59.8	73.5	46.1	18.1
CA-IH-80W-37.8	DN	39	0	59.8	73.5	46.1	18.1
CA-US-101N-84.9	DN	36	0	57.7	68.3	47.1	14.1
CA-US-101S-56.7	DN	36	0	58.5	74.0	42.8	11.4
CA-US-101N-39.2	DN	35	0	59.1	77.2	40.9	30.9
CA-US-101S-39.5	DN	35	0	59.1	77.2	40.9	30.9
CA-IH-10E-17.4	DN	34	0	66.2	79.6	52.7	15.6
CA-IH-8E-43.4	DN	33	0	72.0	88.4	55.6	2.7
FLORIDA							
FL-IH-95N-358.3	WN	31	0	68.0	78.7	57.2	52.8
FL-IH-10W-230.8	WN	30	0	68.3	79.8	56.7	52.2
FL-IH-10E-214.7	WN	30	0	68.3	79.8	56.7	52.2
FL-IH-10W-106.5	WN	31	0	66.2	78.0	54.4	56.9
FL-IH-10E-99.0	WN	31	0	66.2	78.0	54.4	56.9
FL-IH-10W-1.2	WN	30	0	68.0	76.8	59.1	61.2
GEORGIA							
GA-IH-75S-66.4	WN	31	0	65.6	76.8	54.4	46.6
GA-IH-16W-59.9	WN	33	0	64.5	77.7	51.2	45.9
GA-IH-75N-193.6	WN	33	0	64.7	76.5	52.9	44.9
GA-IH-75N-227.7	WN	34	0	61.2	71.3	51.1	48.6
GA-IH-985S-6.7	WN	34	0	59.4	70.3	48.6	55.8
IOWA							
IA-IH-80W-87.7	WF	41	700	50.3	61.2	39.3	34.4
ILLINOIS							
IL-IH-72W-168.9	WF	40	100	51.5	61.1	41.9	39.7
IL-IH-74E-8.0	WF	41	550	49.6	59.7	39.5	39.1
IL-IH-74W-9.0	WF	41	550	49.6	59.7	39.5	39.1
IL-IH-280E-16.0	WF	41	550	49.6	59.7	39.5	39.1
IL-IH-280E-17.0	WF	41	550	49.6	59.7	39.5	39.1
MINNESOTA							
MN-US-10E-208.0	WF	45	1750	43.6	54.3	32.9	28.2
MN-US-10W-205.5	WF	45	1750	43.6	54.3	32.9	28.2
MN-US-10W-88.4	WF	46	2100	39.3	50.4	28.2	26.2
MN-US-10W-87.8	WF	46	2100	39.3	50.4	28.2	26.2

Table B-2. Climatic information for survey sections.

Sect. ID	Climatic Region	Latitude	FI	Average Annual Temp.	Average Annual Max. Temp.	Average Annual Min. Temp.	Average Annual Precip.
MISSISSIPPI							
MS-IH-55N-11.7	WN	31	0	66.1	78.3	53.8	63.4
MS-IH-55N-9.8	WN	31	0	66.1	78.3	53.8	63.4
MS-IH-59S-158.4	WN	32	0	64.0	76.2	51.7	56.7
NORTH CAROLINA							
NC-IH-26E-19.0	WN	35	0	55.6	67.5	43.8	55.8
NEBRASKA							
NE-IH-80W-420.1	DF	41	550	50.6	62.7	38.4	29.3
NEVADA							
NV-IH-80E-179.7	DF	41	200	49.1	66.0	32.2	8.2
NV-IH-80W-111.9	DF	40	150	52.2	68.0	36.4	5.8
OHIO							
OH-CONTECH	WF	41	350	51.1	61.4	40.7	36.0
SOUTH CAROLINA							
SC-IH-85N-27.0	WN	35	0	61.3	72.9	49.7	51.4
SC-IH-85N-31.0	WN	35	0	61.3	72.9	49.7	51.4
SC-IH-20E-2.0	WN	33	0	63.2	75.5	50.9	43.1
SC-IH-20E-4.0	WN	33	0	63.2	75.5	50.9	43.1
SC-IH-20E-55.6	WN	34	0	63.1	75.2	51.0	49.8
SC-IH-95N-162.3	WN	34	0	62.3	73.5	51.1	45.7
SOUTH DAKOTA							
SD-US-14W-402.6	DF	44	1350	42.5	54.4	30.6	21.7
SD-IH-29S-174.0	DF	45	1500	43.3	56.0	30.5	22.4
SD-IH-29S-168.5	DF	45	1500	43.3	56.0	30.5	22.4
SD-IH-90E-290.0	DF	44	1100	47.0	59.8	34.1	21.0
SD-IH-90E-61.0	DF	44	800	46.7	59.3	34.0	16.3
SD-IH-90W-42.5	DF	44	800	46.7	59.3	34.0	16.3
TENNESSEE							
TN-IH-65S-91.9	WN	36	0	59.0	69.7	48.3	47.3
WISCONSIN							
WI-US-61N-0	WF	43	1000	48.1	59.1	37.1	30.9
WI-IH-43N-2.7	WF	42	800	47.0	57.2	36.7	33.1
WYOMING							
WY-IH-90W-191.2	DF	44	1000	43.9	56.1	31.7	17.8
WY-IH-90W-131.2	DF	44	950	44.2	56.7	31.7	16.6

Table B-3. Traffic information for survey sections.

Sect. ID	ADT (Year Surveyed)	Percent Trucks (Year Surveyed)	Cumulative Outer Lane million vehicles (CPR to Survey)	Cumulative Outer Lane million trucks (CPR to Survey)	Cumulative Outer Lane million ESALs (CPR to Survey)	Cumulative Outer Lane million ESALs (Const. to CPR)	Cumulative Outer Lane million ESALs (Const. to Survey)
ALABAMA							
AL-IH-20E-157.5	37769	28.0	54.8	15.4	18.4	13.5	31.9
AL-IH-20E-183.0	29400	36.0	40.9	14.7	17.7	10.5	28.1
AL-IH-59N-185.1	12345	33.0	11.6	3.8	4.6	9.9	14.5
AL-IH-59N-235.5	10962	36.0	20.2	7.3	8.7	5.2	13.9
ARIZONA							
AZ-IH-17S-199.3	130000	10.0	87.9	8.8	15.9	44.8	60.7
CALIFORNIA							
CA-IH-80W-9.0							
CA-IH-80W-37.9							
CA-IH-80W-37.8							
CA-US-101N-84.9	30000	12.0	69.4	8.3	13.7	11.4	25.1
CA-US-101S-56.7	21000	8.0	38.6	3.1	5.1	3.1	8.2
CA-US-101N-39.2	35500	9.0	2.6	0.2	0.3	11.8	12.1
CA-US-101S-39.5	35500	9.0	2.6	0.2	0.3	11.8	12.1
CA-IH-10E-17.4	158300	14.1	165.3	23.2	24.3	19.3	43.6
CA-IH-8E-43.4	11000	23.0	0.9	0.2	0.3	8.7	9.0
FLORIDA							
FL-IH-95N-358.3	58000	7.6	9.8	0.7	0.8	21.6	22.3
FL-IH-10W-230.8	16800	32.1	14.0	4.4	6.9	13.7	20.5
FL-IH-10E-214.7	19000	28.6	17.8	4.9	7.7	16.3	24.0
FL-IH-10W-106.5	13000	27.2	12.4	3.2	5.1	8.8	13.8
FL-IH-10E-99.0	12600	30.9	13.8	4.0	6.3	8.9	15.2
FL-IH-10W-1.2	24900	15.3	11.2	1.7	2.2	18.1	20.3
GEORGIA							
GA-IH-75S-66.4	35800	24.2	16.2	3.6	8.4	21.1	29.5
GA-IH-16W-59.9	12600	25.1	1.0	0.3	0.6	13.7	14.3
GA-IH-75N-193.6	48500	23.4	24.8	5.6	10.0	21.5	31.5
GA-IH-75N-227.7	69000	11.6	41.5	5.1	11.9	29.1	41.0
GA-IH-985S-6.7	37200	8.2	43.5	3.9	8.9	2.2	11.1
IOWA							
IA-IH-80W-87.7	17700	30.0	59.6	17.9	21.2	3.5	24.7
ILLINOIS							
IL-IH-72W-168.9	12384	21.0	0.0	0.0	0.0	9.2	9.2
IL-IH-74E-8.0	16332	23.3	27.9	6.8	12.5	7.2	19.7
IL-IH-74W-9.0	16332	23.3	27.9	6.8	12.5	7.2	19.7
IL-IH-280E-16.0	25526	17.8	48.3	8.5	13.5	7.3	20.7
IL-IH-280E-17.0	25526	17.8	48.3	8.5	13.5	7.3	20.7
MINNESOTA							
MN-US-10E-208.0	11550	10.0	25.1	3.7	2.5	2.7	5.2
MN-US-10W-205.5	11550	10.0	25.1	3.7	2.5	2.7	5.2
MN-US-10W-88.4	5300	13.0	8.9	1.0	0.5	0.5	0.9
MN-US-10W-87.8	5300	13.0	8.9	1.0	0.5	0.5	0.9

Table B-3. Traffic information for survey sections.

Sect. ID	ADT (Year Surveyed)	Percent Trucks (Year Surveyed)	Cumulative Outer Lane million vehicles (CPR to Survey)	Cumulative Outer Lane million trucks (CPR to Survey)	Cumulative Outer Lane million ESALs (CPR to Survey)	Cumulative Outer Lane million ESALs (Const. to CPR)	Cumulative Outer Lane million ESALs (Const. to Survey)
MISSISSIPPI							
MS-IH-55N-11.7	16800	31.0	24.8	7.7	9.2	12.5	21.7
MS-IH-55N-9.8	16800	31.0	24.8	7.7	9.2	12.5	21.7
MS-IH-59S-158.4	32420	24.0	44.6	10.7	12.8	9.3	22.1
NORTH CAROLINA							
NC-IH-26E-19.0	42270	19.5	23.8	4.4	6.4	13.8	20.2
NEBRASKA							
NE-IH-80W-420.1	32500	19.0	31.5	6.1	10.7	16.7	27.4
NEVADA							
NV-IH-80E-179.7	7280	36.8	9.5	3.4	5.6	2.4	8.0
NV-IH-80W-111.9	7380	46.0	3.8	1.7	2.5	6.4	8.9
OHIO							
OH-CONTECH	20260	22.0	14.4	3.2	6.2		
SOUTH CAROLINA							
SC-IH-85N-27.0	25900	48.0	21.7	10.1	12.4	30.8	43.2
SC-IH-85N-31.0	25900	48.0	21.7	10.1	12.4	30.8	43.2
SC-IH-20E-2.0	20700	15.0	39.6	5.8	7.1	5.3	12.4
SC-IH-20E-4.0	20700	15.0	39.6	5.8	7.1	5.3	12.4
SC-IH-20E-55.6	39000	15.0	4.3	0.6	0.6	14.3	14.8
SC-IH-95N-162.3	42000	25.0	24.1	6.0	7.8	22.2	30.0
SOUTH DAKOTA							
SD-US-14W-402.6	2452	13.9	2.5	0.3	0.4	1.1	1.5
SD-IH-29S-174.0	5896	20.1	6.7	1.4	2.9	5.0	7.9
SD-IH-29S-168.5	6046	22.3	6.8	1.5	3.0	5.2	8.2
SD-IH-90E-290.0	6384	24.0	0.5	0.1	0.3	8.3	8.5
SD-IH-90E-61.0	23470	8.0	41.6	4.0	8.3	2.8	11.1
SD-IH-90W-42.5	15124	12.0	24.9	4.9	8.4	2.7	11.0
TENNESSEE							
TN-IH-65S-91.9	97190	17.0	76.6	10.9	15.5	16.4	31.9
WISCONSIN							
WI-US-61N-0	7850	12.6	20.0	2.3	2.9	2.5	5.4
WI-IH-43N-2.7	11860	24.5	6.0	1.5	1.8	4.0	5.7
WYOMING							
WY-IH-90W-191.2	3500	20.0	5.3	1.1	2.2	1.4	3.7
WY-IH-90W-131.2	4600	20.0	8.0	1.6	3.3	1.3	4.6

Table B-4. Design information for survey sections.

Sect. ID	Year Const.	Pvmt Type	Slab Thick, in	Base Type	Base Thick, in	Subbase Type	Subbase Thick, in	Joint Spacing, ft	Skew	Dowel	Dowel Dia., in			Drain Type
ALABAMA														
AL-IH-20E-157.5	60	JPCP	10.0					20	n	y	1.50	Fine	AC	Long. Pipe
AL-IH-20E-183.0	72	JPCP	10.0	Stone	6.0	Soil Aggregate	12.0	20	n	n		Fine	AC	Long. Pipe
AL-IH-59N-185.1	63	JPCP	10.0					20	n	n		Fine	AC	Long. Pipe
AL-IH-59N-235.5	66	JPCP	10.0	Soil Aggregate	6.0	Soil Aggregate	6.0	20	n	y	1.13	Fine	AC	None
ARIZONA														
AZ-IH-17S-199.3	61	JPCP	9.0	Soil Cement	3.0	Soil Cement	6.0	14-17-14-16	n	n		Fine	AC	Long. Pipe
CALIFORNIA														
CA-IH-80W-9.0		JPCP	8.0					13-15-12-14	y	n		Coarse	AC	None
CA-IH-80W-37.9		JPCP	8.0					14-15-17-16	n	n		Coarse	AC	None
CA-IH-80W-37.8		JPCP	8.0					14-15-17-16	n	n		Coarse	AC	None
CA-US-101N-84.9	54	JPCP	8.0	Soil Cement	4.0	Soil Aggregate	6.0	15	n	n		Coarse	AC	None
CA-US-101S-56.7	54	JPCP	8.0	Open Asphalt Base	4.0			15	n	n		Coarse	AC	None
CA-US-101N-39.2	54	JPCP	8.0	Open Asphalt Base	4.0			15	n	n		Fine	AC	None
CA-US-101S-39.5	54	JPCP	8.0	Open Asphalt Base	4.0			15	n	n		Fine	AC	None
CA-IH-10E-17.4	46	JPCP	8.0	Asphalt-Treated Base	4.0	Soil Aggregate	12.0	15	n	n		Coarse	AC	Long. Pipe
CA-IH-8E-43.4	69	JPCP	8.4					13-12-17-21	y	n		Coarse	AC	None
FLORIDA														
FL-IH-95N-358.3	61	JPCP	9.0	Open Asphalt Base	12.0			20	n	y	1.25	Fine	AC	None
FL-IH-10W-230.8	73	JPCP	9.0	Stabilized Soil	12.0			20	n	n		Fine	AC	Long. Pipe
FL-IH-10E-214.7	73	JPCP	9.0	Cement-Treated Base	6.0			20	n	n		Fine	AC	Long. Pipe
FL-IH-10W-106.5	73	JPCP	9.0	Cement-Treated Base	6.0			20	n	n		Fine	AC	Long. Pipe
FL-IH-10E-99.0	74	JPCP	9.0	Cement-Treated Base	6.0			20	n	n		Fine	AC	Long. Pipe
FL-IH-10W-1.2	68	JPCP	9.0	Open Asphalt Base	12.0			20	n	y	1.25	Fine	AC	Long. Pipe
GEORGIA														
GA-IH-75S-66.4	61	JPCP	9.0	Soil Cement	3.0	Stone	5.0	30	n	n		Fine	AC	Long. Pipe
GA-IH-16W-59.9	68	JPCP	10.0	Asphalt-Treated Base	3.0	Gravel	5.0	30	n	n		Fine	AC	Long. Pipe
GA-IH-75N-193.6	68	JPCP	9.0	Soil Cement	3.0	Gravel	5.0	30	n	n		Coarse	AC	None
GA-IH-75N-227.7	69	JPCP	9.0	Soil Cement	3.0	Stone	5.0	30	n	n		Fine	AC	None
GA-IH-985S-6.7	69	JPCP	9.0	Soil Cement	3.0	Gravel	5.0	30	n	n		Fine	AC	None
IOWA														
IA-IH-80W-87.7	81	JPCP	10.0	Asphalt Concrete	3.0	Stone	12.0	20	y	y	1.38	Coarse	AC	Long. Pipe

Table B-4. Design information for survey sections.

Sect. ID	Year Const.	Pvmt Type	Slab Thick, in	Base Type	Base Thick, in	Subbase Type	Subbase Thick, in	Joint Spacing, ft	Skew	Dowel	Dowel Dia., in			Drain Type
ILLINOIS														
IL-IH-72W-168.9	62	JRCP	10.0					100	n	n		Coarse	AC	None
IL-IH-74E-8.0	61	JRCP	10.0	Stone	6.0			100	n	y	1.25	Coarse	AC	Long. Pipe
IL-IH-74W-9.0	61	JRCP	10.0	Stone	6.0			100	n	y	1.25	Coarse	AC	Long. Pipe
IL-IH-280E-16.0	61	JRCP	10.0	Stone	6.0			100	n	y	1.25	Coarse	AC	Long. Pipe
IL-IH-280E-17.0	61	JRCP	10.0	Stone	6.0			100	n	y	1.25	Coarse	AC	Long. Pipe
MINNESOTA														
MN-US-10E-208.0	46	JPCP	8.0	Stone	3.0			15	n	n		Fine	AC	None
MN-US-10W-205.5	46	JPCP	8.0	Stone	3.0			15	n	n		Fine	PCC	None
MN-US-10W-88.4	48	JPCP	9.0					15	n	n		Coarse	AC	None
MN-US-10W-87.8	48	JPCP	9.0					15	n	n		Coarse	AC	None
MISSISSIPPI														
MS-IH-55N-11.7	61	JPCP	9.0	Cement-Treated Base	6.0	Gravel	6.0	21	n	n		Fine	AC	None
MS-IH-55N-9.8	61	JPCP	9.0	Cement-Treated Base	6.0	Gravel	6.0	21	n	n		Fine	AC	None
MS-IH-59S-158.4	63	JRCP	9.0	Cement-Treated Base	6.0	Gravel	6.0	64	n	y	1.00	Coarse	PCC	None
NORTH CAROLINA														
NC-IH-26E-19.0	67	JPCP	9.0	Stone	4.0			30	n	n		Fine	AC	None
NEBRASKA														
NE-IH-80W-420.1	60	JPCP	10.0	Stabilized Base	4.0			17	n	n		Fine	AC	None
NEVADA														
NV-IH-80E-179.7	81	JPCP	8.0	Cement-Treated Base	6.0	Gravel	3.0	12-13-19-18	y	n		Fine	AC	None
NV-IH-80W-111.9	81	JPCP	8.0	Cement-Treated Base	6.0			12-13-19-18	y	n		Coarse	AC	None
OHIO														
OH-CONTECH								30	n					
SOUTH CAROLINA														
SC-IH-85N-27.0	63	JPCP	9.0	Open Asphalt Base	4.0	Soil Cement	8.0	25	n	n		Fine	AC	None
SC-IH-85N-31.0	63	JPCP	9.0	Open Asphalt Base	4.0	Soil Cement	8.0	25	n	n		Fine	AC	None
SC-IH-20E-2.0	67	JPCP	9.0	Open Asphalt Base	4.0	Soil Cement	8.0	25	n	n		Coarse	PCC	None
SC-IH-20E-4.0	67	JPCP	9.0	Open Asphalt Base	4.0	Soil Cement	8.0	25	n	n		Coarse	PCC	None
SC-IH-20E-55.6	67	JPCP	9.0	Stone	6.0			25	n	n		Fine	AC	Long. Pipe
SC-IH-95N-162.3	68	JPCP	9.0	Cement-Treated Base	6.0			19-18-24-25	y	n		Fine	AC	None

Table B-4. Design information for survey sections.

Sect. ID	Year Const.	Pvmt Type	Slab Thick, in	Base Type	Base Thick, in	Subbase Type	Subbase Thick, in	Joint Spacing, ft	Skew	Dowel	Dowel Dia., in			Drain Type
SOUTH DAKOTA														
SD-US-14W-402.6	73	JPCP	7.0	Asphalt Concrete	2.0			16-17-21-20	y	n		Fine	AC	None
SD-IH-29S-174.0	76	JPCP	9.5	Lime-Treated Base	3.5			13-12-18-19	y	n		Fine	PCC	None
SD-IH-29S-168.5	75	JPCP	9.5	Asphalt Concrete	2.0			13-12-18-19	y	n		Fine	PCC	None
SD-IH-90E-290.0	65	JRCP	9.0	Gravel	3.0	Recycled AC	6.0	46.5	n	y	1.25	Fine	AC	None
SD-IH-90E-61.0	60	JRCP	9.0	Sand	3.0	Gravel	8.0	61.5	n	y	1.25	Coarse	AC	None
SD-IH-90W-42.5	60	JRCP	9.0	Sand	3.0	Gravel	8.0	61.5	n	y	1.25	Coarse	AC	None
TENNESSEE														
TN-IH-65S-91.9	70	JPCP	10.0	Stone	6.0			25	n	y	1.25	Coarse	AC	None
WISCONSIN														
WI-US-61N-0	53	JPCP	8.0	Stone	6.0	Gravel	9.0	20	y	n		Fine	AC	None
WI-IH-43N-2.7	76	JPCP	10.0					12-13-18-17	y	n		Fine	AC	None
WYOMING														
WY-IH-90W-191.2	76	JPCP	8.5					12-13-19-18	y	n		Fine	AC	None
WY-IH-90W-131.2	77	JPCP	8.5					12-13-19-18	y	n		Fine	AC	Long. Pipe

Table B-5. Distress information for survey sections.

Sect. ID	Pvmt Type	PSR	SU Length, ft	Avg. Joint Space, ft	Full-Depth Repairs per mile		Transverse cracks, % slabs				JRCP det. cracks per mile	Long. crack, % slabs	Spalling, % joints							
					Pre Ground	Post Ground	Low	Med.	High	Total			Low	Med.	High	Total				
ALABAMA																				
AL-IH-20E-157.5	JPCP	**3.90**	560	20.0	0	0	4%	7%	0%	11%		0%	0%	0%	0%	0%	4%	**0.06**	0.06	Two 4'-long LC
AL-IH-20E-183.0	JPCP	**3.65**	480	20.0	0	0	4%	58%	21%	83%		0%	0%	0%	0%	0%	0%	**0.13**	0.05	Pumping
AL-IH-59N-185.1	JPCP	**3.70**	500	20.0	0	0	0%	20%	0%	20%		0%	0%	0%	4%	4%	0%	**0.20**	0.08	
AL-IH-59N-235.5	JPCP	**4.05**	500	20.0	0	0	0%	4%	0%	4%		0%	4%	0%	0%	4%	0%	**0.05**	0.03	
ARIZONA																				
AZ-IH-17S-199.3	JPCP	**3.40**	310	15.0	85	85	0%	0%	0%	0%		29%	0%	0%	0%	0%	0%	**0.05**	0.04	
CALIFORNIA																				
CA-IH-80W-9.0	JPCP	**4.00**	540	13.5	0	0	5%	8%	5%	18%		15%	0%	2%	0%	2%	20%	**0.01**	0.02	Severe faulting on non DG areas
CA-IH-80W-37.9	JPCP	**3.70**	560	15.5	0	0	8%	8%	22%	39%		6%	16%	19%	5%	40%	0%	**0.12**	0.04	
CA-IH-80W-37.8	JPCP	**3.70**	430	15.5	0	0	7%	14%	36%	58%		50%	28%	3%	3%	35%	0%	**0.05**	0.04	Beginning ASR
CA-US-101N-84.9	JPCP	**3.60**	510	15.0	0	0	0%	3%	0%	3%		9%	11%	0%	6%	17%	6%	**0.05**	0.06	
CA-US-101S-56.7	JPCP	**3.40**	510	15.0	0	0	26%	41%	0%	68%		12%	0%	9%	0%	9%	0%	**0.07**	0.03	
CA-US-101N-39.2	JPCP	**3.80**	530	15.0	50	0	14%	48%	3%	65%		6%	3%	3%	0%	6%	25%	**0.11**	0.09	
CA-US-101S-39.5	JPCP	**3.80**	510	15.0	0	0	0%	0%	0%	0%		15%	3%	0%	0%	3%	3%	**0.02**	0.03	
CA-IH-10E-17.4	JPCP		500	15.0	11	0	96%	0%	0%	96%		3%	41%	0%	0%	41%	0%	**0.10**	0.06	
CA-IH-8E-43.4	JPCP	**3.90**	1010	16.0	0	0	2%	0%	0%	2%		0%	0%	0%	0%	0%	2%	**0.04**	0.03	
FLORIDA																				
FL-IH-95N-358.3	JPCP	**3.80**	500	20.0	11	0	0%	4%	4%	8%		4%	23%	8%	0%	31%	8%	**0.01**	0.03	AC Patch
FL-IH-10W-230.8	JPCP	**3.10**	500	20.0	0	0	12%	24%	12%	48%		0%	0%	0%	0%	0%	0%	**0.17**	0.09	
FL-IH-10E-214.7	JPCP	**3.70**	540	20.0	59	0	11%	0%	0%	11%		4%	18%	0%	0%	18%	4%	**0.08**	0.17	PD PCC
FL-IH-10W-106.5	JPCP	**3.70**	500	20.0	0	0	12%	8%	0%	20%		0%	0%	0%	0%	0%	8%	**0.12**	0.07	1 Broken slab
FL-IH-10E-99.0	JPCP	**3.60**	1000	20.0	11	0	0%	0%	0%	0%		0%	4%	12%	4%	20%	0%	**0.22**	0.13	
FL-IH-10W-1.2	JPCP	**3.90**	1000	20.0	0	16	0%	0%	0%	0%		4%	0%	0%	0%	0%	2%	**0.06**	0.05	57 2-3' LC
GEORGIA																				
GA-IH-75S-66.4	JPCP	**3.90**	780	30.0	0	0	4%	4%	4%	12%		0%	4%	0%	0%	4%	0%	**0.10**		
GA-IH-16W-59.9	JPCP	**4.05**	600	30.0	0	0	0%	0%	0%	0%		0%	0%	0%	0%	0%	0%	**0.01**		
GA-IH-75N-193.6	JPCP		600	30.0	0	0	0%	0%	0%	0%		20%	0%	0%	0%	0%	0%	**0.04**		4-5 two ft low long cracks
GA-IH-75N-227.7	JPCP	**3.90**	300	30.0	0	0	0%	0%	0%	0%		0%	0%	0%	0%	0%	0%	**0.07**	0.04	
GA-IH-985S-6.7	JPCP	**3.90**	720	30.0	0	0	0%	0%	4%	4%		8%	0%	0%	0%	0%	4%	**0.11**	0.03	
IOWA																				
IA-IH-80W-87.7	JPCP	**4.10**	500	20.0	0	0	0%	0%	0%	0%		0%	4%	0%	0%	4%	0%	**0.01**	0.01	
ILLINOIS																				
IL-IH-72W-168.9	JRCP		1000	100.0	0	0					0.0	0%					0%	**0.00**	0.00	
IL-IH-74E-8.0	JRCP	**3.70**	1000	100.0	5	16					15.8	0%	9%	27%	27%	64%	0%	**0.14**	0.14	
IL-IH-74W-9.0	JRCP	**3.60**	1000	100.0	0	21					0.0	0%	9%	9%	55%	73%	0%	**0.12**	0.13	
IL-IH-280E-16.0	JRCP	**3.60**	1000	100.0	5	16					0.0	0%	0%	36%	27%	64%	0%	**0.11**	0.11	D-crack
IL-IH-280E-17.0	JRCP	**3.50**	1000	100.0	11	11					42.2	0%	0%	0%	64%	64%	0%	**0.10**	0.15	
MINNESOTA																				
MN-US-10E-208.0	JPCP	**3.30**	500	15.0	0	0	0%	54%	3%	57%		57%	0%	0%	0%	0%	0%	**0.07**	0.03	
MN-US-10W-205.5	JPCP	**3.35**	500	15.0	0	0	0%	27%	3%	30%		0%	0%	0%	0%	0%	0%	**0.07**	0.03	
MN-US-10W-88.4	JPCP	**3.50**	480	15.0	33	0	0%	0%	0%	0%		0%	9%	27%	18%	55%	0%	**0.05**	0.04	D-crack at inside corner
MN-US-10W-87.8	JPCP	**3.50**	480	15.0	11	0	0%	0%	0%	0%		0%	0%	0%	0%	0%	0%	**0.03**	0.03	D-crack at inside corner

Table B-5. Distress information for survey sections.

Sect. ID	Pvmt Type	PSR	SU Length, ft	Avg. Joint Space, ft	Full-Depth Repairs per mile		Transverse cracks, % slabs				JRCP det. cracks per mile	Long. crack, % slabs	Spalling, % joints							
					Pre Ground	Post Ground	Low	Med.	High	Total			Low	Med.	High	Total				
MISSISSIPPI																				
MS-IH-55N-11.7	JPCP	**3.80**	500	21.0	0	0	0%	0%	0%	0%		0%	0%	0%	0%	0%	0%	**0.13**	0.06	
MS-IH-55N-9.8	JPCP	**3.60**	500	21.0	0	0	13%	4%	0%	17%		4%	4%	0%	0%	4%	4%	**0.20**	0.18	Pumping
MS-IH-59S-158.4	JRCP	**3.50**	1020	64.0	5	0					20.7	0%	0%	6%	0%	6%	6%	**0.34**	0.20	Map crack
NORTH CAROLINA																				
NC-IH-26E-19.0	JPCP	**3.80**	1020	30.0	5	0	0%	3%	0%	3%		0%	0%	0%	0%	0%	0%	**0.09**	0.07	Pumping
NEBRASKA																				
NE-IH-80W-420.1	JPCP	**3.60**	510	17.0	0	0	0%	3%	0%	3%		0%	0%	0%	3%	3%	0%	**0.13**	0.06	
NEVADA																				
NV-IH-80E-179.7	JPCP	**3.50**	930	15.5	0	0	7%	7%	22%	35%		0%	7%	2%	5%	13%	0%	**0.10**	0.06	ASR
NV-IH-80W-111.9	JPCP	**3.75**	930	15.5	0	0	10%	13%	5%	28%		2%	3%	0%	0%	3%	10%	**0.04**	0.03	Beginning ASR
OHIO																				
OH-CONTECH				30.0																D-cracking
SOUTH CAROLINA																				
SC-IH-85N-27.0	JPCP	**3.80**	570	25.0	28	0	0%	4%	26%	31%		0%	0%	0%	0%	0%	9%	**0.15**	0.09	
SC-IH-85N-31.0	JPCP	**3.50**	570	25.0	9	83	0%	0%	13%	13%		0%	0%	0%	0%	0%	0%	**0.19**	0.10	Pumping, shattered slab
SC-IH-20E-2.0	JPCP	**3.90**	570	25.0	9	0	0%	4%	0%	4%		0%	4%	4%	8%	17%	0%	**0.07**	0.07	
SC-IH-20E-4.0	JPCP	**3.80**	600	25.0	0	0	0%	0%	0%	0%		0%	0%	0%	0%	0%	0%	**0.07**	0.06	1 PCC PD
SC-IH-20E-55.6	JPCP	**4.05**	570	25.0	9	0	0%	0%	0%	0%		0%	0%	0%	21%	21%	0%	**0.05**	0.05	
SC-IH-95N-162.3	JPCP	**3.75**	950	21.5	67	0	5%	2%	0%	7%		0%	0%	0%	0%	0%	2%	**0.10**	0.13	
SOUTH DAKOTA																				
SD-US-14W-402.6	JPCP	**3.40**	1040	18.5	0	0	0%	0%	2%	2%		0%	2%	0%	2%	3%	0%	**0.16**	0.07	D-crack
SD-IH-29S-174.0	JPCP	**3.85**	930	15.5	6	0	0%	2%	0%	2%		0%	3%	0%	0%	3%	2%	**0.03**	0.03	1 D-crack
SD-IH-29S-168.5	JPCP	**3.85**	930	15.5	0	0	0%	0%	0%	0%		0%	3%	0%	0%	3%	0%	**0.01**	0.02	
SD-IH-90E-290.0	JRCP	**3.95**	1020	46.5	31	0					5.2	0%	0%	0%	0%	0%	0%	**0.01**	0.05	
SD-IH-90E-61.0	JRCP	**3.30**	1170	61.5	0	0					4.5	0%	10%	0%	65%	75%	0%	**0.03**	0.04	High spalling
SD-IH-90W-42.5	JRCP	**3.55**	980	61.5	5	0					48.5	0%	0%	6%	94%	100%	0%	**0.04**	0.04	High spalling
TENNESSEE																				
TN-IH-65S-91.9	JPCP	**4.00**	600	25.0	0	0	13%	25%	17%	54%		0%	0%	0%	0%	0%	0%	**0.02**	0.04	
WISCONSIN																				
WI-US-61N-0	JPCP	**3.60**	580	20.0	9	18	0%	7%	3%	10%		0%	3%	50%	27%	80%	0%	**0.12**	0.03	D-cracking
WI-IH-43N-2.7	JPCP	**3.55**	900	15.0	0	0	0%	0%	0%	0%		0%	97%	2%	0%	98%	0%	**0.12**	0.03	
WYOMING																				
WY-IH-90W-191.2	JPCP	**3.50**	930	15.5	0	0	0%	0%	0%	0%		0%	7%	0%	0%	7%	0%	**0.08**	0.03	
WY-IH-90W-131.2	JPCP	**3.30**	930	15.5	0	0	0%	0%	0%	0%		0%	3%	2%	2%	7%	0%	**0.19**	0.09	

Table B-6. Other survey information.

Sect. ID	Section Found	Pumping	Shoulder Cond.	Blade Space per ft	Mean Texture Depth, in
ALABAMA					
AL-IH-20E-157.5	At Grade to 6-16 ft Cut	None	Good	54	
AL-IH-20E-183.0	At Grade	Low	Poor	54	
AL-IH-59N-185.1	At Grade	None	Good	53	0.021
AL-IH-59N-235.5	At Grade	None	Good	52	0.024
ARIZONA					
AZ-IH-17S-199.3	16-40 ft Cut	None	Good		
CALIFORNIA					
CA-IH-80W-9.0	6-16 ft Cut	None	Good	58	
CA-IH-80W-37.9	At Grade	None	Good		
CA-IH-80W-37.8	At Grade	None	Good		
CA-US-101N-84.9	At Grade	None	Good	60	
CA-US-101S-56.7	At Grade	None	Fair		
CA-US-101N-39.2	At Grade	None	Fair	56	0.046
CA-US-101S-39.5	At Grade	None	Fair	56	0.037
CA-IH-10E-17.4	At Grade	None	Good	57	0.017
CA-IH-8E-43.4	At Grade	None	Good	57	0.036
FLORIDA					
FL-IH-95N-358.3	At Grade	None	Good	55	
FL-IH-10W-230.8	At Grade	None	Good	57	0.022
FL-IH-10E-214.7	6-16 ft Fill	None	Good	55	0.023
FL-IH-10W-106.5	At Grade	None	Good	56	0.019
FL-IH-10E-99.0	At Grade	None	Good	61	0.020
FL-IH-10W-1.2	At Grade	None	Good	58	0.033
GEORGIA					
GA-IH-75S-66.4	At Grade	None	Fair	57	0.035
GA-IH-16W-59.9	At Grade	None	Good	53	0.042
GA-IH-75N-193.6	At Grade to 6-16 ft Fill	None	Good	54	0.025
GA-IH-75N-227.7	At Grade	None	Fair	54	0.023
GA-IH-985S-6.7	At Grade	None	Good	54	
IOWA					
IA-IH-80W-87.7	At Grade	None	Good	53	
ILLINOIS					
IL-IH-72W-168.9	At Grade			54	0.066
IL-IH-74E-8.0	At Grade	None	Fair	53	0.014
IL-IH-74W-9.0	At Grade	None	Good	53	0.016
IL-IH-280E-16.0	6-16 ft Fill	None	Fair	53	0.013
IL-IH-280E-17.0	At Grade	None	Good	53	0.014
MINNESOTA					
MN-US-10E-208.0	At Grade	None	Fair	55	
MN-US-10W-205.5	At Grade	None	Fair	55	
MN-US-10W-88.4	At Grade	None	Good	55	0.017
MN-US-10W-87.8	At Grade	None	Good	55	0.019

Table B-6. Other survey information.

Sect. ID	Section Found	Pumping	Shoulder Cond.	Blade Space per ft	Mean Texture Depth, in
MISSISSIPPI					
MS-IH-55N-11.7	At Grade	Low	Fair	56	0.023
MS-IH-55N-9.8	At Grade	Medium	Fair	56	
MS-IH-59S-158.4	At Grade	None	Good	54	
NORTH CAROLINA					
NC-IH-26E-19.0	At Grade	Medium	Good	55	
NEBRASKA					
NE-IH-80W-420.1	At Grade	None	Fair	52	0.015
NEVADA					
NV-IH-80E-179.7	At Grade	None	Good	54	0.023
NV-IH-80W-111.9	At Grade	None	Good	52	0.028
OHIO					
OH-CONTECH	6-16 ft Fill	None	Good	52	0.017
SOUTH CAROLINA					
SC-IH-85N-27.0	At Grade	None	Good	55	0.024
SC-IH-85N-31.0	At Grade	Low	Fair	55	
SC-IH-20E-2.0	At Grade	None	Good	55	
SC-IH-20E-4.0	At Grade	None	Good	55	
SC-IH-20E-55.6	At Grade	None	Good	53	
SC-IH-95N-162.3	At Grade	None	Good	53	0.027
SOUTH DAKOTA					
SD-US-14W-402.6	At Grade	None	Fair	52	0.028
SD-IH-29S-174.0	At Grade	None	Good	54	0.018
SD-IH-29S-168.5	At Grade	None	Good	54	
SD-IH-90E-290.0	At Grade	None	Poor	57	0.034
SD-IH-90E-61.0	At Grade	None	Good	49	
SD-IH-90W-42.5	At Grade	None	Fair	49	0.012
TENNESSEE					
TN-IH-65S-91.9	6-16 ft Fill to 6-16 ft Cut	None	Good	50	0.023
WISCONSIN					
WI-US-61N-0	At Grade	None	Fair		
WI-IH-43N-2.7	6-16 ft Fill to 16-40 ft Cut	None	Fair	55	0.030
WYOMING					
WY-IH-90W-191.2	At Grade	None	Fair	53	0.015
WY-IH-90W-131.2	At Grade to 6-16 ft Fill	None	Fair	53	0.016

PALABRAS CLAVE: El moler por diamante, la rehabilitacíon de pavimento de concreto, el funcionamiento del pavimento, la falla, la textura de la superficie, la aspereza, la lisura.

SINOPSIS: El moler por diamante restaura una superficie lisa de conducir con las características deseadas de tracción en pavimento de concrete. Esta técnica fue empleada por primera vez en 1965 en una seccón 19 años de viejo de I-10 in el sur de California para eliminar la falla excesiva (Neal and Woodstrom 1976). Desde entonces, el moler por diamante ha sido un elemento importante de la restauración de pavimento de concreto. Sin embarg, a pesar de una historia larga de uso exitoso, mey poca documentación valida existe sobre el funcionamiento de pavimento molido por diamante. El propósito de este estudio era proporcionar tal documentación.

El estudio constó de un repaso comprensivo de la información existente del moler por diamante, la recolección de datos, el análisis de datos y la documentación de los descubrimientos del estudio. Fueron conducidas extensivas inspecciones científicas de campo para obtener datos del funcionamiento necesarios para el análisis. En total, fueron inspeccionadas 60 secciones de pavimento en 18 estados. Además, los datos de funcionamiento de 133 secciones fueron obtenidos de un estudio anterior del funcionamiento de pavimento molido por diamante (Snyder et al. 1989). También, los datos del programa de funcionamiento a plazo largo de pavimento, secciones (restauración de pavimento de concreto) fueron utilizados para hacer comparaciones directas lado a lado del funcionamiento de secciones de pavimento molido por diamante y las de otros métodos de restauración. Varios análisis fueron conducidos empleando los datos recogidos para documentar el funtionamiento de pavimento molido por diamante, incluso una evaluacíon de funcionamiento de falla, la longevidad de textura de pavimento molido por diamante y los efectos de moler por diamante en la vida de servicio del pavimento.

REFERENCIA: Rao, Shreenath; H. Thomas Yu and Michael I Darter. *La Longevidad y El Funcionamiento de los Pavimentos Molidos por Diamante*, Boletin de Investigación y Desarrollo RD118, Asociación de Cemento Portland, Skokie, Illinois, U.S.A., 1999, 112 Pages páginas.

SCHÜSSELWORT: Strassendeckeschleifen mit Diamant besetzten Räedern, Betonfahrbahnwiederherstellung Fahrbahnleistung, Fugenabstufen, Strassendeckegrobheit, Uneben, Griffigkeit, Ebenheit

ABSTRAKT: Strassendeckenscleifen mit Diamanten besetzten Schleifräedern stellt die Ebene und die erwüenschte Griffigkeit der Betonfahrbahnahrbahnfläechen wieder her. Diese Technik wurde in 1965 an Hand eines 19 Jahr alten Abschnitt der I-10 im süedlichem Teil von California gelegenen Autobahn züerst gebraucht um üebermäessige Fugenabstuffungen auszuebenen (Neal and Woodstrom, 1976). Seit dieser Zeit wurde das Strassendeckenschleifen mit Diamant besetztem Schleifraedern ein wichtiger Anteil der Betonfahrbahnwiederherstellung Arbeit. Trotz der langen und erfolgreichen Praxis existieren sehr wenige unbestreitbare documentierte Berichte üeber die Fahrbahnleistungen nach dieser Behandelung. Das Objective dieser Forschung war die Dokumentierung dieser Technik.

Die Untersuchung besteht aus einem umfassenden Rüeckblick existierender Informationen füer Strassendeckenschleifen, Datensammelung, Datenverarbeitung, Dokumentierung der Forschungergebnisse. Umfassende Untersuchungen der Betonfahrbahnstrecken wurden gemacht um die die Fahrbahnleistungswerte zu erringen. Im Ganzem wurden 60 Fahrbahnstrecken in 18 Staaten untersucht. Noch dazu wurden die Resultate einer Forschung der Fahrbahnleistung von 133 Fahrbahnstrecken die schon früeher nach Diamantschleifen beobachtet wurden (Snyder at al. 1989). Die Werte von dem langsfristigem Fahrbahnleistungprogram SPS – 6 (Betonfahrbahnwiederherstellung) wurden auch angewendet um direkte Vergleiche von Fahrbahnleistungen nach Diamantschleifen mit jenem von anderen Wiederherstellungarbeiten zu dokumentieren. Verschiedene Analyse wurden gemacht welche die Werte der Fahrbahnleistungen nach behandlung vom Diamantenschleifen dokumentierte. Dieser Arbeit waren auch die Fahrbahnleistung hinsichtlich des Fugenabstufen, der Lebensdaüer der Griffigkeit der Fahrbahndecke, und die Effekten des Diamantschleifen auf die Langlebigkeit der der Betongfahrbahn beigesetzt.

REFERENZ: Roa, Shreenath, Yu, H.Thomas, Darter, MichaelI., *Die Langlebigkeit und Betongfahrbahnleistung nach Strassendeckeschleifen mit Diamant besetzten Räedern*, Forschungs-und Entwicklungsbulletin RD118, Portlandzementverbands, Skokie, Illinois, U.S.A., 1999, 112 seiten

WARNING: Contact with wet (unhardened) concrete, mortar, cement, or cement mixtures can cause SKIN IRRITATION, SEVERE CHEMICAL BURNS (THIRD-DEGREE), or SERIOUS EYE DAMAGE. Frequent exposure may be associated with irritant and/or allergic contact dermatitis. Wear waterproof gloves, a long-sleeved shirt, full-length trousers, and proper eye protection when working with these materials. If you have to stand in wet concrete, use waterproof boots that are high enough to keep concrete from flowing into them. Wash wet concrete, mortar, cement, or cement mixtures from your skin immediately. Flush eyes with clean water immediately after contact. Indirect contact through clothing can be as serious as direct contact, so promptly rinse out wet concrete, mortar, cement, or cement mixtures from clothing. Seek immediate medical attention if you have persistent or severe discomfort.

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