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Investigation of Feasible Pavement Design Alternatives for WisDOT

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INVESTIGATION OF FEASIBLE PAVEMENT DESIGN ALTERNATIVES FOR WISDOT

FINAL REPORT

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16. Abstract

The current pavement design and selection process of WisDOT for all new pavements or reconstructions of existing pavement structures provides for the design of one asphaltic concrete (AC) and one portland cement concrete (PCC) pavement alternative. Life-cycle costs analyses are then used to determine the preferred alternative for construction. Previous restrictions in the WisDOT pavement selection process have essentially excluded the construction of thick AC (AC thickness > 150 mm) and thin PCC (PCC thickness < 225 mm) pavements and thus the validity of current life-cycle cost inputs for these pavement types is under question.

This report presents a performance analysis of existing thick AC and thin PCC pavements constructed in and around Wisconsin. The performance trends developed indicate current design assumptions utilized by WisDOT, related to the expected service life to first rehabilitation of AC and PCC pavements, may also be used for thick AC and thin PCC pavements.

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INVESTIGATION OF FEASIBLE PAVEMENT DESIGN ALTERNATIVES FOR WISDOT

FINAL REPORT WI/SPR-15-99
WisDOT Highway Research Study # 94-13
SPR # 0092-45-64

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for

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DIVISION OF TRANSPORTATION INFRASTRUCTURE DEVELOPMENT
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1.0 INTRODUCTION

This report has been prepared based on research findings for Wisconsin Department of Transportation (WisDOT) Research Project 94-13, “Investigation of Feasible Design Alternatives for WisDOT.” The primary focus of this project is to provide performance data necessary for conducting rational, objective, and justifiable life cycle costs analyses for thick asphalt concrete (AC) pavements (AC thickness > 150 mm) and thin portland cement concrete (PCC) pavements (PCC thickness < 225 mm). A secondary objective is to summarize the design and selection procedures utilized by surrounding States.

1.1 Problem Statement

The increasing trend of construction costs coupled with decreasing highway revenues over the past years has led many highway agencies to place greater emphasis on improved pavement management techniques with the objective of getting the best performance for the investments made in the highway infrastructure. Consequently, there has been greater interest in the generation and evaluation of various pavement design alternatives.

The current pavement design and selection process of WisDOT for all new pavements or reconstructions of existing pavement structures provides for the design of one asphaltic concrete (AC) and one portland cement concrete (PCC) pavement alternative. Life-cycle costs analyses are then used to determine the preferred alternative for construction. Previous restrictions in the WisDOT pavement selection process have essentially excluded the construction of thick AC and thin PCC pavements and thus the validity of current life-cycle cost inputs for these pavement types is under question. In order for WisDOT to conduct a rational, defensible, and objective life cycle cost analysis involving thick AC and thin PCC pavements, there is the need to review the performance of these pavement types.

1.2 Objectives

The specific objectives of this research are to:

- Broaden the WisDOT knowledge base regarding the performance of thick AC and thin PCC pavements
- Provide justification for life-cycle costs analysis inputs used for thick AC and thin PCC pavements, and
- Broaden the options available during the pavement selection process.

This report presents pertinent literature on current pavement design and selection practices used by other agencies and states -- with special attention to practices in states bordering Wisconsin, including Illinois, Indiana, Iowa, Ohio, Minnesota, and Michigan. The report also examines WisDOT pavement design/selection practices and presents performance models developed from existing pavement sections in Wisconsin and surrounding States. These models are used for analyzing the life- cycle cost inputs for thick AC and thin PCC pavements in Wisconsin.

2.0 SUMMARY OF PAVEMENT DESIGN PRACTICES USED IN WISCONSIN AND SURROUNDING STATES

A survey of surrounding States was conducted to determine practices used for new or reconstructed pavement thickness design and to determine inputs used for the selection of preferred pavement design alternatives. States providing information include Illinois (IDOT), Indiana (InDOT), Michigan (MiDOT), Minnesota (MnDOT), and Ohio (ODOT).

2.1 Thickness Design Procedures Used in Surrounding States

A variety of pavement thickness design methods for new or reconstructed pavements are currently in use within Wisconsin and surrounding States. These design methods include empirical, mechanistic, or a combination of these methods. Table 2.1 summarizes the pavement design procedures used in the surrounding States surveyed.

Table 2.1: Pavement Design Practices Used by Surrounding States

State	AC/PCC Pavement Design Procedure
Illinois DOT	Mechanistic
Indiana DOT	1986/1993 AASHTO
Michigan DOT	1986/1993 AASHTO
Minnesota DOT	1972 AASHTO (1)
Ohio DOT	1986/1993 AASHTO

(1) In-house procedure developed based on AASHO Road Test results.

The empirical method of pavement design utilizes observed pavement performance under trafficking to establish structural requirements for new or reconstructed pavements. The 1972 AASHO design procedure, based on test results of the AASHO road test conducted near Ottawa, Illinois in late 1950s and early 60s, is an example empirical design method which is currently utilized by MiDOT and WisDOT for both rigid and flexible pavement design. The essence of this design method is in fixed charts and equations, developed through regression analysis of variations in pavement performance, as measured by the Present Serviceability Index, with axle load applications.

Mechanistic principles were applied to the 1972 AASHO design procedures to expand it's applicability to other locations, materials, and/or loading regimes, resulting in the 1986/1993 AASHTO design procedures. The 1986/1993 AASHTO design procedure is currently used by InDOT, MiDOT and ODOT.

The mechanistic design method models elements of the pavement structure using multi-layer elastic theory or finite element analysis to determine the stresses, strains, and/or deflections resulting from applied load configurations. This method draws heavily on mechanics of materials and relationships between stress/strain and fatigue life consumption. The load-induced stress/strain values are typically used to determine the allowable number of applications prior to fatigue failure. Failure may be defined based on surface layer cracking, subgrade rutting, or other specific distress manifestations. Oftentimes it is necessary to employ "shift factors" to calibrate mechanistic fatigue models to better predict observed pavement performance, resulting in mechanistic-empirical design procedures. Mechanistic-empirical design procedures are currently in use by IDOT for both rigid and flexible pavement design.

2.2 Wisconsin Department of Transportation Procedures

The WisDOT pavement thickness design procedures, which are based on the 1972 AASHO design procedures, are provided in the WisDOT Facilities Development Manual (7). Many of the design input parameters have been standardized for use throughout the State. Key design parameters which are site specific include pavement design class, terminal serviceability, subgrade support, and 20-year design lane ESALs. While the standardized design process allows for efficient design thickness determinations, certain restrictions in design input selection limits the flexibility of usage for these procedures, particularly in the area of design reliability, and in some instances results in inconsistencies between rigid and flexible thickness design assumptions.

2.2.1 Flexible Pavement Thickness Design Procedures

The current WisDOT flexible pavement thickness design procedures allow designers to select terminal serviceability values of 2.0 or 2.5 depending on the design class of the pavement under consideration. Soil support values are determined in the soils report and used as specified. A regional factor of 3.0 is utilized throughout the State. Design lane ESALs are determined based on site specific truck distributions using standard flexible pavement ESAL factors established by WisDOT. On minor highways where truck classification data are not available, an average truck factor of 0.9 may be used. No inputs for design reliability are provided.

To assess the level of design reliability inherent within the WisDOT procedure, comparisons were made between the current WisDOT procedure and the 1986/1993 AASHTO design procedures. In order to make these comparisons, a conversion between the WisDOT soil support value (SSV) and the soil resilient modulus (Mr) was needed. Using the nomograph originally developed by Van Til, et al., the following simple correlation equation was developed:

$$\text{Log (Mr)} = 3 + 0.16\ (\text{SSV})$$

The required flexible pavement structural number, based on the current WisDOT design equation, was first determined for a wide variety of SSV and design ESAL values. The 1986/1993 AASHTO design equation was then utilized to determine the standard normal deviate (Zr) which would provide equal design ESALs, using Mr values determined from the above equation and setting the overall standard deviation (So) to the midrange value of 0.45. This process yielded a Zr value of -1.070 for all design SSV, ESAL or serviceability levels, which corresponds to a design reliability level of 85.8%. To allow for the flexibility of selecting varying levels of design reliability within the current WisDOT procedures, modifications to the flexible pavement ESAL factors would be required. These modifications would result in the selection of design ESAL values targeted to the desired level of design reliability, a modification which is not inconsistent with the primary function of the design reliability level in the 1986/1993 AASHTO design procedures. Table 2.2 provides a comprehensive range of flexible pavement ESAL factors which are consistent with current WisDOT ESAL factors while providing the added benefit of a user selected level of design reliability.

Table 2.2: Flexible Pavement ESAL Factors for Varying Levels of Design Reliability

Truck Type	Design Reliability Level, %						
	50	75	85	85.8	90	95	99
2D	.10	.20	.29	0.3[1]	.37	.54	1.10
3SU	.26	.53	.77	0.8[1]	1.00	1.45	2.94
2-S1,2-S2	.16	.33	.48	0.5[1]	.62	.91	1.84
3-S2 &Above	.30	.60	.87	0.9[1]	1.12	1.63	3.31
Dbl Bottoms	.66	1.33	1.93	2.0[1]	2.49	3.63	7.35

[1] Current WisDOT flexible pavement design ESAL factor

2.2.2 Rigid Pavement Thickness Design Procedures

The current WisDOT jointed rigid pavement thickness design procedures allow only for a terminal serviceability value of 2.5. The design modulus of subgrade reaction k-values is determined in the soils report and used as specified. Design inputs for the concrete modulus of rupture, working stress, and modulus of elasticity are fixed at 650 psi, 490 psi and 4,200,000 psi, respectively. Doweled transverse joints are specified for all rigid pavement designs, regardless of thickness. Design lane ESALs must be determined based on site specific truck distributions using standard rigid pavement ESAL factors established by WisDOT. No average truck factors are provided for use where truck classification data are not available nor are inputs for design reliability provided.

To assess the level of design reliability inherent within the WisDOT procedure, comparisons were made between the current WisDOT procedure and the 1986/1993 AASHTO design procedures. In order to make these comparisons, a J-factor of 3.0 was selected to represent the standard WisDOT design of doweled joints and widened outer lanes. Furthermore, a drainage factor (Cd) of 1.0 was utilized throughout. The required rigid pavement thickness, using the current WisDOT design equation, was first determined for a wide variety of k-value and design ESAL values. The 1986/1993 AASHTO design equation was then utilized to determine the standard normal deviate (Zr) which would provide equal design ESALs, using an overall standard deviation (So) set to the midrange value of 0.35. This process yielded a Zr value of -1.47 for all design k-value or ESAL values, which corresponds to a design reliability level of 92.9%.

To allow for the flexibility of selecting varying levels of design reliability within the current WisDOT procedures, modifications to the rigid pavement ESAL factors would again be required. These modifications would result in the selection of design ESAL values targeted to the desired level of design reliability, a modification which is not inconsistent with the primary function of the design reliability level in the 1986/1993 AASHTO design procedures. Table 2.3 provides a comprehensive range of rigid pavement ESAL factors which are consistent with current WisDOT ESAL factors while providing the added benefit of a user selected level of design reliability.

Table 2.3: Rigid Pavement ESAL Factors for Varying Levels of Design Reliability

Truck Type	Design Reliability Level, %						
	50	75	85	90	92.9	95	99
2D	.09	.16	.21	.26	.3[1]	.34	.59
3SU	.36	.63	.84	1.02	1.2[1]	1.37	2.37
2-S1,2-S2	.18	.31	.42	.51	.6[1]	.68	1.19
3-S2 & Above	.49	.83	1.12	1.36	1.6[1]	1.83	3.16
Dbl Bottoms	.64	1.10	1.47	1.79	2.1[1]	2.40	4.15

[1] Current WisDOT rigid pavement design ESAL factor

2.3 Life-Cycle Cost Analysis

Inputs used for life-cycle costs analysis were obtained from the MiDOT, IDOT and ODOT. Primary inputs include initial construction costs, future maintenance costs, analysis period, and discount rate. Decision matrices, including both economic and non-economic factors, are also used in the final pavement selection process.

2.3.1 Maintenance Activities

Scenarios for future maintenance activities were reported based on pavement type and traffic level. These scenarios were developed based on observed early-life performance and extrapolated where necessary. Pavement performance continues to be monitored to determine if revisions are needed to keep the scenarios current and to reflect current technology and maintenance expenditures.

Current maintenance scenarios for both rigid and flexible pavements are summarized in Tables 2.4 and 2.5.

2.3.2 Analysis Period

Reported analysis periods ranged from 30 to 40 years. In all states, equal analysis periods were used for both flexible and rigid pavement alternatives.

2.3.3 Discount Rate

A fixed discount rate of 3% is used by IDOT Illinois during life-cycles analysis. MiDOT and ODOT use variable discount rates, ranging from 0% to 6%, to test the sensitivity of this input on the life-cycle costs.

2.3.4 Decision Matrices

Decision matrices are used in the pavement selection process to allow for the inclusion of both economic and non-economic factors. These matrices are commonly used as a final tool to select a preferred design alternative from amongst all alternatives which have life-cycle costs differing by no more than preselected values. Economic factors used include life-cycle costs, user costs, energy savings, and engineering costs. Non-economic factors used include reliability, materials availability, noise, continuity of pavement, local preference, stage construction potential, balancing of business, and others.

Table 2.4: Maintenance Strategies for PCC Pavements

Maintenance Activity	Illinois DOT All Traffic Levels	Michigan DOT		Ohio DOT
		<3000 Trucks/day[1]	> 3000 Trucks/day[1]	
Longitudinal CL Joint Seal	100%@ 10, 20, 30 yr			
Shoulder Joint Seal	100%@ 0, 10, 20, 30 yr			
Transverse Joint Seal	100%@ 10, 20, 30 yr [2] 10%@10, 30[3], 70%@20 yr	2%@5, 20%@13, 75%@20, 2%@25 yr	2%@5, 4%@10, 20%@15, 75%@20, 5%@25 yr	100%@20 yr
Crack Seal		5%@13, 10%@20,25, 15%@ 30 yr	4%@10, 5%@15, 30%@20, 25%@25 yr	
Slab Replacement		0.04%@5 yr	0.04%@5, 0.08%@10 yr	
Full-Depth Joint Repair	0.5%@7, 1%@10, 1.5%@15,25, 4%@20, 2.5%@30, 3.5%@35 yr	4%@20, 3%@25, 2%@30 yr	3%@15, 5%@20, 5%@25, 2%@30 yr	2%@20 yr
Surface Grinding	100%@20yr		500SY/lm@15,20 yr	100%@20 yr
Undersealing	70%@20 yr			
AC Overlay				3"@30 yr

(1) Average two-way design daily truck volume
(2) Rout and seal transverse hinge joints with ASTM D 3405
(3) Remove and replace preformed elastomeric joint seal

Table 2.5: Maintenance Strategies for AC Pavements

Activity	Illinois DOT (Full-Depth AC)				Michigan DOT		Ohio DOT
	TF > 34 Rural (1) TF > 22.7 Urban	24.5<TF<34 R 16.3<TF<22.7 U	15<TF<24.5 R 10<TF<16.3 U	TF < 15 R TF < 10 U	< 3000 Trucks/day (2)	> 3000 Trucks/day (2)	
Centerline Crack Seal	100%@3,6,14, 21,29,37 yr	100%@3,8,15, 21,31 yr	100%@3,11, 21,33 yr	100%@3,12, 21,31 yr			
Transverse Crack Rout&Seal	(100 ft spacing) 15%@3,50%@6 100%@14,21 29,37 yr	(100 ft spacing) 15%@3 50%@8,12,15 100%@21,31 yr	(100 ft spacing) 15%@3 50%@6 100%@11,21,33	(100 ft spacing) 15%@3 50%@6 100%@12,21,31	(lf/mile) 250 @8 500 @15 750 @25,30yr (3)	(lf/mile) 150 @5 1000 @10,23 500 @25 750 lf/lm @30yr	
Random Crack Rout&Seal	(100 lf/sta) 50%@6,14 21,29,37 yr	(100 lf/sta) 50%@8,15 21,31 yr	(100 lf/sta) 50%@11, 21,33 yr	(100 lf/sta) 50%@12,21 31 yr			
Patching	0.5%@5 2%@13,28,36 4%@20 yr	0.5%@5 3%@7,30 4%@20 yr	0.5%@5 3%@10,32 4%@20 yr	0.5%@5 3%@10,32 4%@20 yr	150 lf/lm @15,30 yr(3)		
Surface Mill	3/4" @5,13 20,28,36 yr	3/4" @7, 20,30 yr	3/4" @10, 20,32 yr	3/4" @20 yr		3" @18,28 yr	
AC Overlay	1-1/2"@5,13, 28,36 yr 3-1/4"@20 yr	1-1/2"@7,30 yr 3-1/4"@20 yr	1-1/2"@10,32 3-1/4"@20 yr	3-1/4"@20yr	OL @ 20 yr	3" @ 18,28 yr	1-1/4"@10 3"+3"ATB@20 1-1/4"@30 yr

(1) TF = Traffic Factor expressed in millions of ESALS
(2) Average two-way design daily truck volume
(3) Either crack seal or skip patch used

3.0 SURVEY ON THICK ASPHALT PAVEMENT USAGE IN WISCONSIN

Thick asphalt pavements have long been recognized to play a key role in minimizing the damaging effects of heavy traffic loads on pavement structures. Their use and performance characteristics, however, have not been documented in Wisconsin. To acquire knowledge on the extent and usage characteristics of thick AC pavements in Wisconsin, a survey of City governments and WisDOT district offices was conducted.

Candidate City governments were selected based on pavement information supplied by the Wisconsin Asphalt Pavement Association through Mr. Gerald Waelti. The Cities of La Crosse, Brookfield, Kenosha, and Waukesha indicated the use of thick AC pavements; however, the majority of these pavement were not designed following standard WisDOT pavement design procedures. Instead, pavement thicknesses were determined on the basis of city government policies established through years of experience and engineering judgement. Section 3.3 of this report presents a procedure used to validate the inclusion of these pavements into the pavement performance database.

In the *City of Kenosha*, all pavements with cross-sections larger than 12.8 m either receive a 250 mm thickness of AC or a 200 mm thickness of PCC while those with cross-sections less than 12.8 m receive a 225 mm AC or 175 mm PCC.

In the *City of Brookfield*, thickness consists of 190 mm full-depth pavements constructed over the subgrade. This standard thickness is based on a city policy established in 1968. This practice, however, has led to a number of premature failures, attributed to heavy loads from home construction and poor construction practice on weak subgrades. City policy was revised in 1993 to require that all new pavements be designed following standard WisDOT pavement design guidelines.

In the *City of La Crosse* a standard AC thickness of 175 mm is used while the *City of Waukesha* uses a standard maximum thickness of 150 mm of AC over a 250 mm gravel base on major traffic carrier routes, including downtown streets and industrial/business parks.

Two of the eight WisDOT district offices surveyed indicated a minimum use of thick AC pavements in their jurisdictions. *District 4* reported only two projects of approximately 7.6 km classified as thick AC pavements. A 7.1 km section exists in Waushara county on STH 49 between Green Lake County Line and STH 21. This section was originally constructed in 1976 as a 225 mm AC pavement placed directly on the subgrade. It was overlaid in 1988 with 150 mm limestone and 75 mm of AC. No maintenance records, however, exist for this pavement. The remaining 0.5 km section is located on USH 51 in Portage county in the vicinity of CTH XX. It was originally constructed in 1974 as a 200 mm AC resting on a 150 mm dense graded crushed aggregate base course on top of a clay subgrade. The pavement was milled and overlaid in 1990. No maintenance records exist for this pavement. In *District 3*, approximately 4.4 km of highway in Shawano county along STH 29 was identified as thick AC pavement. It was originally constructed in 1964 as a 200 mm AC over a 125 mm crushed aggregate base course. It was overlaid in 1990 with a 50 mm AC.

The survey, in addition, showed a wide range of other characteristics associated with thick asphalt pavements. These characteristics include: pavement foundation type, traffic level, age, thickness, location, and length. A summary of these characteristics are provided in Appendix A.

3.1 Field Condition Survey of Thick Asphalt Pavements

Based on the results of the survey, 85 pavement segments were initially considered for inclusion in this study. A field condition survey was initially conducted for each segment to acquire data on the severity and extent of the different kinds of distresses existing on the pavements, as well as their drainage characteristics. The observed distresses were in turn converted to pavement distress indices (PDI) using procedures outlined in WisDOT Pavement Surface Distress Survey Manual for pavements (1). The field survey indicated that the predominant modes of distress on thick asphalt pavements include transverse and block cracking; these distresses dictated to a large extent the magnitude of the PDI values for the segments.

3.2 City Government AC Pavement Data Validation

A fundamental objective of the city government survey was to capture important pavement design parameters, including traffic loading levels and soil characteristics , to enable performance trends to be established for thick AC pavements. Discussion with City engineers indicated that actual pavement design records were unavailable since all existing pavement thicknesses were determined on the basis of experience and policy rather than established WisDOT design methods. However, they indicated that they were very familiar with the segments under study and could give information on type of pavement foundation and traffic loading range (e.g. low, medium, or high). Traffic loading ranges used were those established for the research factorial.

Pavement thickness data was used to calculate the design structural number of the pavement using standard WisDOT layer coefficients. Soils information and County Soils Maps were used to establish Soil Support Values for each segment. These two values were combined with the design terminal serviceability (p_t=2.5) to calculate the allowable 20-year Design ESALs using the WisDOT design equation.

The allowable 20-year Design ESAL values were then converted to Allowable Daily Design ESAL values and verified using the WisDOT design nomograph. The Allowable Daily Design ESAL values were used to assign each pavement section into the appropriate factorial cell traffic group. These assignments were compared with the cell assignments based on traffic levels specified by city engineers. Table 3.3 shows the distribution of segments by allowable and design traffic levels. Overall, of the 66 non-overlaid pavement sections, 34 were found to be in agreement between Design and Allowable Daily ESAL levels, 23 were considered under designed (Design Daily ESALs significantly exceeds Allowable Daily ESALs), and 9 were considered over designed (Design Daily ESALs significantly less than Allowable Daily ESALs). Only those 34 pavement segments which were found to be in agreement were considered appropriate for inclusion in this study.

Table 3.3: Distribution of Segments by Design and Actual Traffic Levels

Allowable Daily ESALs	Design Daily ESALs		
	50-250	250-450	> 450
50 - 250	9*	13**	4**
250 - 450	2**	22*	6**
> 450	4**	3**	3*

* used for analysis; ** excluded from analysis (under or over designed)

3.3 Performance Analysis of Thick Asphalt Pavements

The concept of pavement performance as a measure of highway deterioration has been widely analyzed and discussed by many researchers using various performance indicators. The use of combined indices such as the present serviceability index (PSI), pavement condition index (PCI), pavement quality index (PQI), and pavement distress index (PDI) is popular among pavement engineers. Recent modeling techniques, however, are shifting from the combined index approach to a more versatile approach in which major distress modes are individually modeled to better analyze and explain the relationship between distress and pavement serviceability.

The indicator of pavement performance used in this research was the PDI. It ranges from 0 to 100; 0 being the best and 100 being the worst. A zero rating for example, will correspond to a newly constructed pavement with no surface distress while higher PDI values indicate increased pavement surface distresses.

A fundamental assumption was made in this study that rehabilitation practice in the form of an overlay affects the overall performance of pavements. Therefore, the data was stratified to enable trends and pertinent analyses to be performed separately for non-overlaid and overlaid thick asphalt pavements. Sixty-six non-overlaid pavements had initially being surveyed for inclusion in this study but after the data validation process previously outlined, only thirty-four were found appropriate. For the purpose of this analysis, *n*on-*o*verlaid *t*hick *a*sphalt *p*avements (NOTAP) are defined as thick AC pavements which have not received any form of overlay in their life history while *o*verlaid *t*hick *a*sphalt *p*avements (OTAP) have received at least one AC overlay in their life history. The main factors assumed to affect the performance of each of the two pavement categories include :

a) *Thickness grouping*: Group A denoting thickness greater than 200 mm (8 inches) and Group B denoting thickness in the range of 150 mm -200 mm (6-8 inches)

b) *Traffic level*: Light traffic level representing segments subject to an approximate daily ESAL of 50-250; Medium traffic corresponds to daily ESAL range of 250-450; and Heavy traffic level corresponds to daily ESAL greater than 450

c) *Roadbed condition* was indicated by the soil support value (SSV).
Three levels indicating poor, fair, and good conditions were defined to have respective SSV ranges of 3-4, 4-5, and 5-6.

The above factors were used in designing the general factorial chart shown in Figure 3.1.

		SOIL SUPPORT VALUE		
	Pavement Thickness (mm)	3 - 4	4 - 5	5 - 6
Light Traffic 50-250 ESALs/day	150 - 200			
	> 200			
Medium Traffic 250-450 ESALs/day	150 - 200			
	> 200			
Heavy Traffic >450 ESALs/day	150 - 200			
	> 200			

Figure 3.1: Factorial Chart Showing Thick AC Pavement Performance Factors

3.4 Analysis of Non-Overlaid Thick Asphalt Pavements (NOTAP)

The analysis of NOTAP consisted of two phases: a preliminary phase and a model building phase. The former used analysis of variance, scatter plots, and regression techniques to select key variables for the model building phase. Additional regression analyses were performed in the model-building phase on the variables that were found to influence pavement performance.

The first step involved in the preliminary analysis was to determine the number of segments pertaining to specific combinations of traffic level, SSV, and thickness group. The purpose was to observe PDI trends, where possible, for all combinations of the factors affecting PDI. Figure 3.2 shows a factorial chart depicting cells represented in the database for thick asphalt pavements. The next step in the preliminary phase was to examine the extent to which pavement foundation condition indicator (SSV), pavement thickness group, and traffic levels affect the performance indicator (PDI) using analysis of variance (ANOVA) technique. Since only one segment was of thickness greater than 200 mm, the effect of thickness was not considered. The effect of pavement age on performance was controlled in the preliminary analysis to enable the assumed main influential factors (traffic, SSV) defined in Figure 3.2 to be examined. This was done by normalizing the PDI data, i.e. PDI was transformed into PDI per year (PDI/pavement age).

		SOIL SUPPORT VALUE		
	Pavement Thickness (mm)	3 - 4	4 - 5	5 - 6
Light Traffic 50-250 ESALs/day	150 - 200	X	X	
	> 200			
Medium Traffic 250-450 ESALs/day	150 - 200		X	X
	> 200	X		
Heavy Traffic >450 ESALs/day	150 - 200			X
	> 200			

Figure 3.2 : Cells Represented in **Non-Overlaid** Thick Asphalt Pavement Database

3.5 Analysis of Variance, Scatter Plots, and Trend

In the ANOVA, the objective is to determine the extent to which the different levels of SSV and traffic affect the mean value of the PDI/year. For example, considering SSV, the basic null hypothesis of interest is given by:

H_0: $\mu_{PDI/yeargood} = \mu_{PDI/yearfair} = \mu_{PDI/yearpoor}$

and the alternative hypothesis is:

H_A: H_0 is not true i.e. at least one of the mean PDI/year is different,

$\mu_{PDI/yearpoor}$ = mean PDI/year for pavements on poor soils

$\mu_{PDI/yearfair}$ = mean PDI/year for pavements on fair soils

$\mu_{PDI/yeargood}$ = mean PDI/year for pavements on Good soils.

The null hypothesis of equal means is tested using an F-test statistic. The overall results of the analysis of variance is shown in Figure 3.3 and Figure 3.4. Figure 3.3 shows that AC pavements on good soils have lower mean PDI/year values compared to those located on fair and poor soils. Figure 3.4 also indicates that pavements subjected to high traffic loads exhibit higher average PDI/year compared to those subjected to medium and low traffic loads. These results are further summarized in Table 3.4. In Table 3.4 the P-value measures whether the F-statistic is significantly large or the probability of obtaining a value of the F-statistic at least as unfavorable to the null hypothesis as the observed values (3). Small P-values (less than 0.05 for 95% confidence level) indicate that the mean PDI/year for the various levels of a specific variable differ significantly. In other words a significant difference exists among sample PDI/year means representing specific levels of a variable.

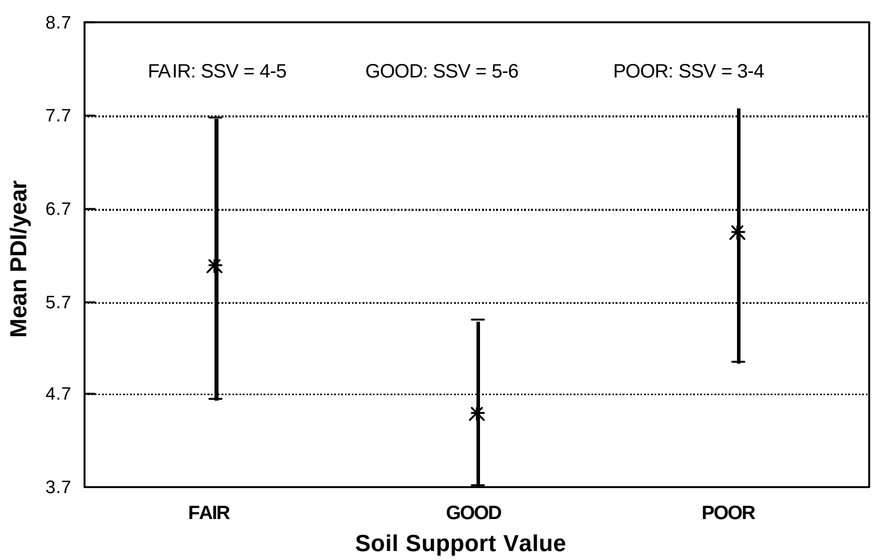

Figure 3.3: Effect of Roadbed Condition on Mean PDI/year and 95% Confidence Intervals

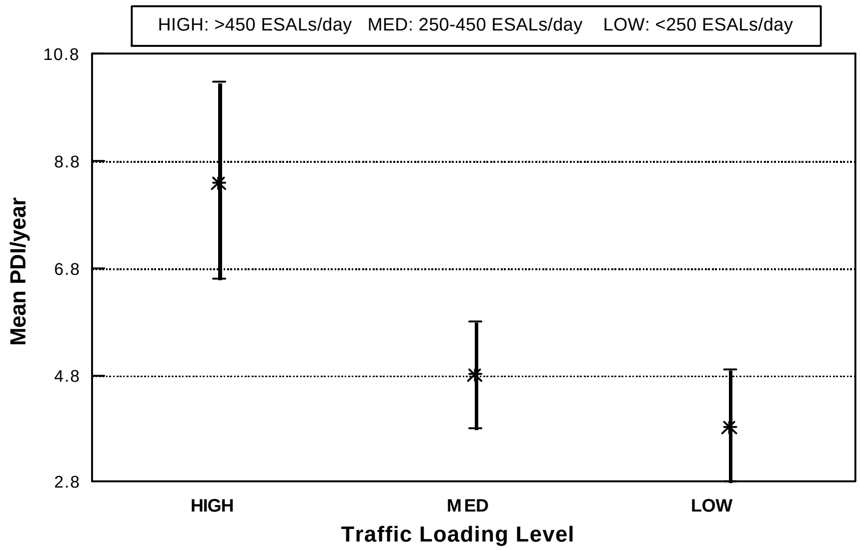

Figure 3.4: Effect of Traffic Loading Level on Mean PDI/year

Table 3.4. ANOVA Summary for NOTAP Performance Factors

Variable	Computed F-statistic	P-value
ESAL	6.36	0.0058
SSV	1.00	0.3824

In addition to the ANOVA, preliminary scatter plots and regression analyses were conducted to examine the relationship between PDI and other factors including pavement age and base course thickness. Scatter plot and trend line showing the relationship between PDI and pavement age is shown for all NOTAP pavements in Figure 3.5. The regression model and corresponding characteristics pertaining to a confidence level of 95% are summarized in Table 3.6. Figure 3.5 shows an increasing trend in PDI with increasing pavement age but in a nonlinear fashion. The rate of deterioration (change in PDI with age) is high initially and slows down as the pavement gets older and approaches a terminal value. The deterioration rate is approximately 5.3 PDI/year between ages 1 and 5, compared to 2.2 PDI/year between ages 5 and 15, and approximately 1.0 PDI/year between age 15 and 25. The high initial deterioration rate suggests that distresses appear quicker on these pavement types; this could probably be due to pavement material characteristics reacting initially to severe or new environmental exposure.

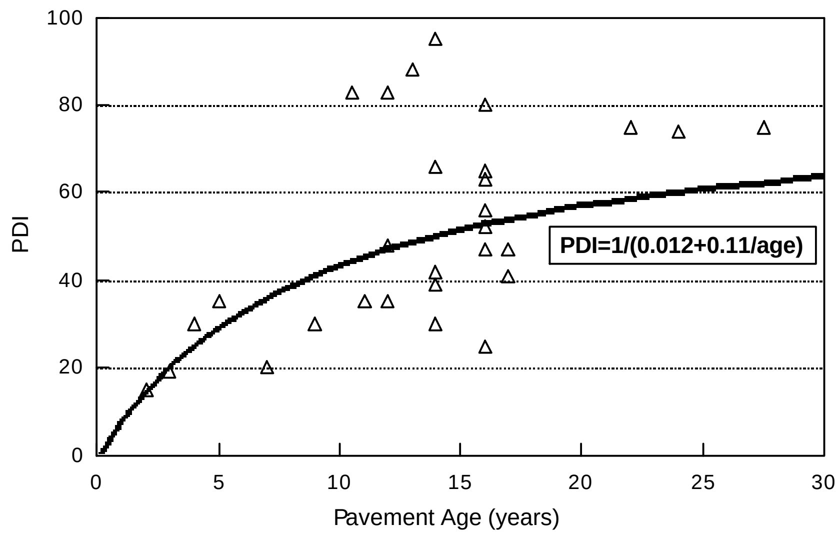

Figure 3.5: PDI Trend Plot for Non-overlaid Thick Asphalt Pavements in Wisconsin

Field observations indicated transverse and block cracking as the predominant mode of distress on all of the pavements surveyed. Such distresses result from an environmental fatigue process that is determined largely by material characteristics and the temperature regime. Once these distresses develop, the deterioration will continue if not rectified.

3.6 Overlaid Thick Asphalt Pavement Performance Modeling

PDI variations for overlaid thick pavements had been examined in relation to pavement age prior to overlay, existing pavement surface age, and overlay thickness. The general characteristics of all OTAP segments are shown in Table 3.5. Table 3.5 indicates that PDI changes an average of about 8 per year for existing overlaid thick asphalt pavements and overlays are used when the original pavement age is about 17 years old. However, an age range of 10 to 24 years prior to overlay consideration (mean ± one standard deviation) is not uncommon.

Table 3.5. Summary Statistics for OTAP Database

Variable	Range	Mean	Standard Deviation, σ
PDI/year	4.1-14	8.2	2.8
Pvt. Surface Age	1-12	6.3	3.2
Age Prior to Overlay	8.5-26	17.5	6.9
Overlay thickness	2-4	2.7	0.75

A scatter plot and trend line showing the variation of PDI with age for OTAP segments is presented in Figure 3.6. The overall results indicate that for OTAP, pavement surface age alone may be sufficient for explaining the variation in PDI. This is certainly the case when field observations of these pavements reveal that PDI values are mainly triggered by the existence of transverse and block cracking which result from an environmental fatigue process as previously mentioned. The overall regression model developed for OTAP is as given in Table 3.6 with the corresponding statistical characteristics.

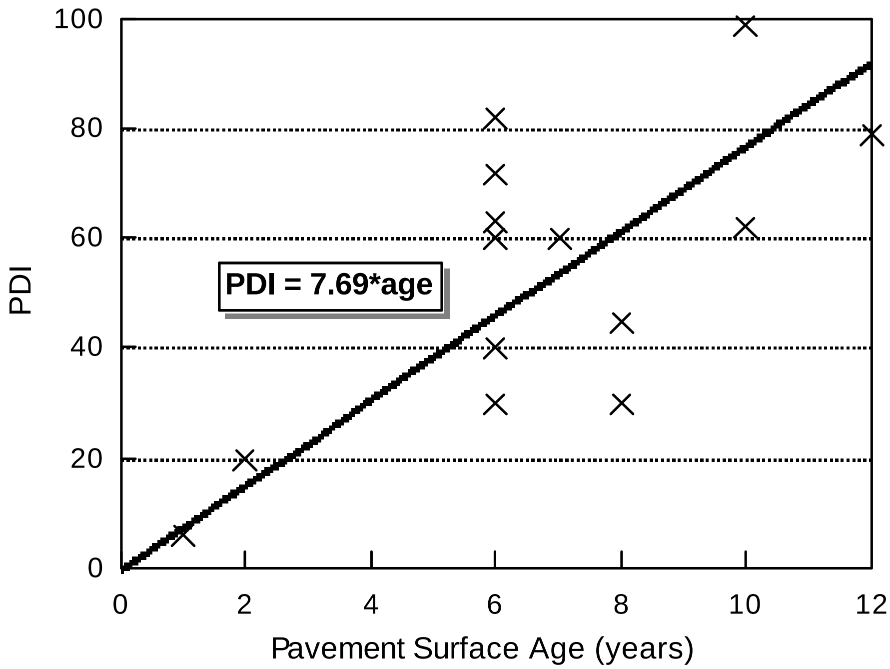

Figure 3.6: PDI Scatter Plot and Trend for Overlaid Thick Asphalt Pavements

Table 3.6: Statistical Characteristics of Thick Asphalt Pavement Performance Models

Paveme nt Class	Traffic Level	Performance Model*	No. of Observation s	Model R^2	Model F-value	Model P-value
NOTAP	All Traffic	*PDI(t) = 1/(0.012 + 0.11/**t**)*	30	0.67	57.1	0.0000
OTAP	All Traffic	*PDI(t) = 1/(-0.002+0.14/t)*	15	0.96	319.7	0.0000

*PDI (t) = PDI corresponding to pavement age t

3.7 Engineering Implications of Performance Models

Performance prediction over the life cycle of a pavement is critical for both pavement management and design. The performance models for NOTAP and OTAP would provide means for pavement service life estimates based on critical PDI levels for pavement maintenance and rehabilitation interventions. These service life estimates would in turn provide a basis for life cycle cost analyses involving thick asphalt pavements and other pavement options. Service life estimates based on the proposed models are as shown in Table 3.7.

Table 3.7. Service Life Estimates by Traffic Level at Critical PDI Levels

Pavement Class	PDI	Approx. PSI	Service Life (years)
NOTAP	48	3.0	12
	62	2.5	26
OTAP	48	3.0	6
	62	2.5	8

Current pavement service lives recommended by WisDOT for life cycle cost analysis involving newly constructed pavements prior to first overlay range from 12-16 years for undrained pavements and 15-20 years for drained pavements. A drained pavement by definition, constitutes a pavement structure designed to minimize moisture induced damage by draining free water from the pavement through utilization of an open graded base and edge drain system (4). The existing values in their present form do not directly account for the effect of pavement thickness and serviceability level. However, the results of this study suggest that service life prior to first overlay is dependent on the desired serviceability level as shown in Table 3.7. The service life is found to be highly sensitive to serviceability especially for NOTAP. At a PSI of 2.5 (PDI$\approx$62), a service life of 26 years is expected for undrained NOTAP segments compared to 12 years for a PSI of 3.0 (PDI≈48). The 26 year service life for undrained NOTAP significantly exceeds the 20 year service life currently in use by WisDOT for even drained pavements. This study did not look at thin pavements (<150 mm), however, the results of this study suggest the need for WisDOT to examine the appropriateness of the existing service life criteria in relation to both thin and thick AC pavements.

In the case of overlaid thick asphalt pavements, service lives of 6 and 8 years are suggested respectively for PDI= 48 (PSI=3.0) and PDI =62 (PSI =2.5) based on the results of this study compared to the existing WisDOT values of 10 to 12 years.

3.8 Minnesota Department of Transportation Models for Thick Asphalt Pavements

The Minnesota Department of Transportation developed performance models for pavements in Minnesota based on their functional classification. Table 3.8 shows models developed for non-overlaid thick asphalt pavements on aggregate bases and their prediction capabilities at critical serviceability levels. The low R^2 values for minor arterials and collectors indicate that age is not a predictor of performance for thick AC pavements representing these functional classifications but it is a critical predictor of performance for Interstate pavements and principal arterials. The models, in addition, indicate that service lives at the two critical serviceability levels for any one functional class is significantly different (e.g. 17 years for PSI =3.0 versus 24 years for PSI =2.5 for Interstate pavements). Similar observation is made for the pavements studied in Wisconsin for NOTAP (see Table 3.7). The models also show significant differences in service life between Interstate and principal arterials at PSI = 2.5. The above observations suggest the need for WisDOT to review and revise existing service life criteria to reflect the effect of serviceability and functional classification.

Table 3.8: MnDOT Models for Thick Asphalt Pavements*

Functional Class	Performance Model	R^2	Age@PSI=3.0	Age@PSI=2.5
Interstate	PSI = 4.18 - 0.0712t	0.719	17	24
Principal Arterial	PSI = 3.58 - 0.0310t	0.570	19	35
Minor Arterial	PSI = 3.06 - 0.0122t	0.075	5	46
Collector	PSI = 3.40 - 0.0229t	0.310	17	39

**Reproduced from report submitted by Art Hill to MnDOT Full-Depth Asphalt Pavement Committee (see reference 5)*

In the Wisconsin study of NOTAP, there was not enough data pool to analyze pavements by functional class as done in the Minnesota study. Hence, a direct comparison of the service life values at the two critical serviceability values between NOTAP and the MnDOT pavements may not be appropriate. However, the MnDOT study suggests that for thick asphalt pavements, service life in the range of 17 to about 30 years may be common. Thus a 26 year service life as estimated by the NOTAP model at PDI = 62 (PSI≈2.5) may not be unreasonable.

3.9 Overview of Thick Asphalt Pavement Usage

Thick asphalt pavements have been constructed by many agencies in one of two forms to withstand the damaging effects of heavy traffic loads. These forms consist of either construction of full-depth asphaltic concrete (AC) directly on the roadbed, or construction of the AC on aggregate bases.

In Minnesota both forms have been used for more than two decades, with the majority consisting of construction of full-depth AC on a wide range of roadbeds including clay-loam, silty-loam, and granular soils. Thickness for full-depth pavements commonly range from 7" to 16.75" whilst those on aggregate bases range from 6" to 13". Figures 3.7 and 3.8 show performance comparisons of Full-depth AC and aggregate-base AC pavements constructed between 1976-1980 and 1981-1985.

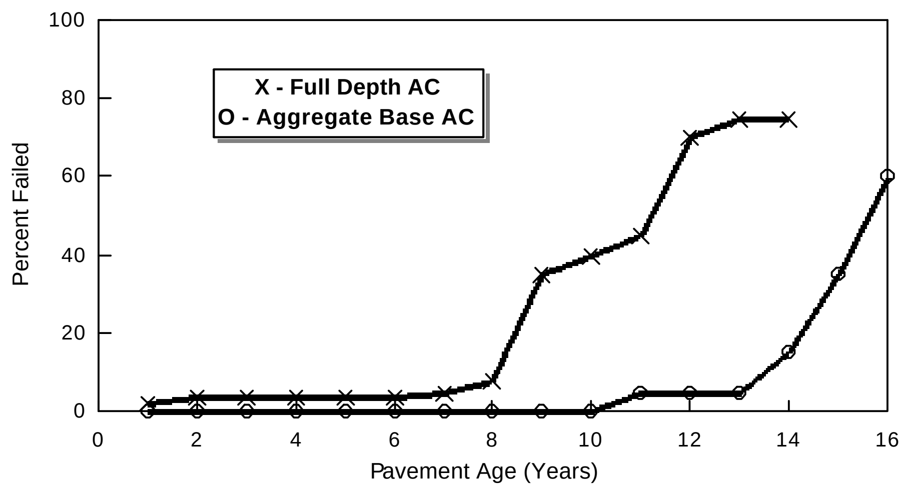

Figure 3.7: Survival Curves for 1976-1980 Minnesota Asphalt Pavements *(Reproduced from reference 5)*

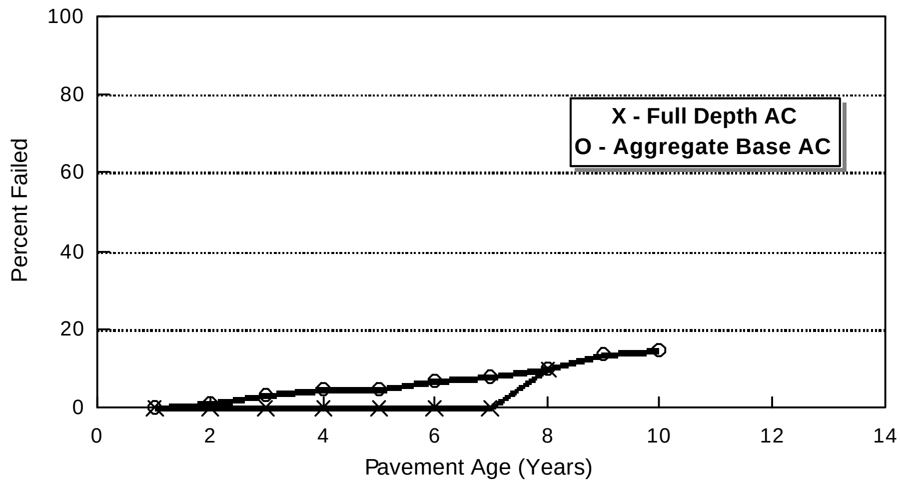

Figure 3.8: Survival Curves for 1981-1985 Minnesota Asphalt Pavements *(Reproduced from reference 5)*

Figure 3.7 indicates that approximately 70% of the full-depth pavements were rehabilitated by age 12 years and nearly 80% by age 14 years compared to about 15% at age 14 years for aggregate-base pavements. Figure 3.8 also shows higher percentage failure for full-depth pavements compared to aggregate-base AC pavements.

On the average a service life range of 10-15 years has been obtained from full-depth AC pavements, far short of their expected 20-year design life. According to Hill (5), in general asphalt pavements over aggregate bases exhibit better long-term ride qualities than full-depth pavements. The perceived poor performance of full-depth AC pavements and the better performance of aggregate base AC pavements have prompted the Minnesota Department of Transportation (MNDOT) to place emphasis on providing a high quality well-drained select granular material directly under the pavement structure. The new flexible pavement design method uses an average of 10”-20” thickness of select granular material (with 12% passing # 200) under the base course of the pavement structure. Hill also points out that due to the poor performance of full-depth pavements, no other state or province in the area immediate surrounding of Minnesota is building full-depth AC as a regular option; South Dakota, Alberta, Saskatchewan, and Kansas have all quit building full-depth asphalt pavements.

In Orange County, California, a pavement type selection study had been conducted by a pavement advisory committee to determine the most economical and durable material for 68 miles of new highways in three toll corridors (23 miles for the Eastern Transportation corridor, 30 miles for the Foothill corridor, and 15 miles for the San Joaquin Hills Transportation corridor). The study employed four different pavement thickness design procedures including the Portland Cement Association, Caltrans, the Asphalt Institute and the AASHTO procedures. This resulted in an initial design analysis of 350 different combinations of design methods, pavement types, structural sections, subgrade conditions, bases and subbases. The initial analysis indicated that for the same traffic and subgrade conditions the difference in asphalt pavement thickness was more than 25%, depending on the design method used; in addition, it was revealed through the analysis that the Caltrans method was the most conservative and the AASHTO method the least conservative.

On the basis of this preliminary analysis, 22 pavement options were selected for further analysis. These included 10 thick asphalt pavements ranging in surface thicknesses from 10.5” to 20” and 8” to 14.5” respectively for five Caltrans and five AASHTO designs. The 12 concrete pavements consisted of eight AASHTO and Four Caltrans designs . A life cycle cost analysis involving these options concluded that in all cases asphalt pavements showed significantly lower life cycle costs. In addition to pavement structural thickness and economic analysis, many other factors were subjected to engineering judgement in the final pavement type selection. These factors included: the performance of similar pavements under similar conditions in the area, availability of local materials, recyclability of materials, potential for differential settlement, variation in subgrade conditions, etc. Consequently, the pavement advisory group recommended for all mainline road sections an asphalt pavement consisting of 12 inches of asphalt concrete on 3 inches of asphalt-treated permeable base placed on 15 inches of aggregate base. Although this study recommended the use of asphalt on Orange county toll roads, it may be pointed out that cost components for life cycle costs analysis and other judgmental factors involving pavement options may differ significantly from agency to agency and thus the outcome of a similar study conducted in Wisconsin, for example, could be different from or similar to the above.

In Illinois the policy regarding full-depth pavements is to overlay them every 20 years (6). The frequency and quantity of other maintenance strategies for use in a life cycle cost analysis are, on the other hand, determined on the basis of a traffic factor estimated by functional class of road for a given design period and traffic composition. The maintenance strategies currently employed by Illinois are also distress specific i.e. a strategy is triggered by the encountered distress type rather than a combined index such as a pavement distress index which requires determining the average amount of distress from many different combinations of distresses.

4.0 SURVEY ON THIN PCC PAVEMENTS

The use of thin PCC pavements (PCC thickness less than 225 mm) in Wisconsin has for the most part been limited to urban areas. For example, the Cities of Appleton and Milwaukee have shown preference for thin PCC pavements due to their perceived superior performance in areas with poor quality subgrades. Discussions with city officials indicate that pavements are designed following WisDOT guidelines and in some cases thin PCC pavements are selected instead of AC pavements based solely on their judgement that the PCC pavement will provide superior performance. Slab thicknesses of 200 mm are predominantly used, even if the pavement design worksheet indicates a thinner structure may be appropriate. The City of Milwaukee typically constructs undoweled PCC pavements over a 100 mm aggregate base. The City of Appleton also typically constructs undoweled PCC pavements but rarely uses aggregate bases.

Programmed design lives for these PCC pavements prior to overlay are typically 25 years and in some cases design lives of 30 years are used. The predominant programmed maintenance activity for PCC pavements in the above cities is crack and joint sealing, typically performed on 3 to 5 year intervals.

After discussions with officials from both cities, and a review of construction records, a total of 25 thin PCC pavement sections were considered appropriate for inclusion into this study. Six pavement sections, constructed between 1980 and 1989, were selected from the City of Appleton and the remaining 19 sections, constructed between 1979 and 1993, were selected from the City of Milwaukee. Appendix B provides a listing of these sections.

A survey of WisDOT district offices indicated that only District 4 has constructed significant lengths of thin PCC pavements. A review of pavement design records indicated 11 pavement sections appropriate for inclusion into this study. Included are 5 pavement sections located along STH 13 with a total length of 42 km. Four of these pavement sections include an 200 mm, undoweled PCC slab over a 150 mm aggregate base, constructed between 1957 and 1983. The remaining section along STH 13 includes an 200 mm, undoweled PCC slab over a 100 mm open graded base constructed in 1989. Two pavement sections, totaling 9.6 km, are located along STH 21. These sections were constructed in 1988 and include an 200 mm, undoweled PCC slab over a 150 mm aggregate base. One 12.9 km section of STH 52 constructed in 1990 is also included. This section is composed of an 200 mm, doweled PCC slab over a 100 mm open graded aggregate base. The three remaining sections are located along USH 10 and include a 6.4 km section constructed in 1951 and two sections totaling 17.7 km constructed in 1969. The oldest section includes an 200 mm, undoweled PCC slab constructed over a 100 mm aggregate base. The two newer sections include an 200 mm, doweled PCC slab over a 150 mm aggregate base. Appendix B provides a listing of these pavement sections.

A pavement database from the State of Iowa was supplied by Mr. Chris Brakke, Assistant Pavement Engineer. A review of this database indicated that 101 pavement sections, constructed between 1968 and 1984, were appropriate for inclusion into this study. These pavements include both State and US highway segments ranging in length from 1.1 to 22.2 km. All pavements were constructed with 6.1 m transverse joint spacings with pavement thicknesses ranging from 150 to 216 mm. Pavements with PCC thicknesses of 200 mm or greater include 32 mm diameter dowels at 300 mm spacings. Appendix B provides a listing of these pavement sections.

4.1 Field Condition Survey of Pavements Outside Wisconsin

Performance indicators for PCC pavement sections located in Iowa were provided in terms of quantity of surveyed and catalogued distress. The supplied data were reviewed and methodology developed to convert the data to comparable PDI values. These methodologies are described in Appendix C.

The PDI conversion process for Iowa pavements was validated through a field survey in Northeastern Iowa. A total of 12 pavement sections in 7 counties which were included in the Iowa database were surveyed, resulting in a total of 18 survey segments. Pavement distress information was recorded in each section following the established PDI survey procedures. In addition, transverse cracking, surface patching, and transverse joint faulting data were obtained over 0.81 km segments of each section. The validation survey indicated the following major points:

- Individual surface patches were, on average, larger in size than originally considered
- 1-2 low severity Distressed Joints/Cracks per station were noted in 1/3 of the PDI survey segments
- Low severity Surface Distress was noted in 1/6 of the PDI survey segments
- The majority of transverse cracks were of low severity and occurred at the rate of one per slab affected

4.2 Performance Analysis of Thin PCC Pavements

The performance analysis of thin PCC pavements was based on only those pavement sections where actual PDI values were obtained, including all Wisconsin sections and those surveyed in Northeastern Iowa. For all of these pavements, the design ratio between calculated Allowable Daily ESALs and the Design Daily ESALs was determined. Only those pavements with a design ratio between 0.8 to 1.2 were considered as properly designed and thus included during the development of regression performance models. Performance data was further segregated based on transverse joint design, resulting in separate performance models for undoweled and doweled pavements.

Figures 4.1 and 4.2 illustrate the PDI vs Age trends for the included undoweled and doweled pavements sections, respectively.

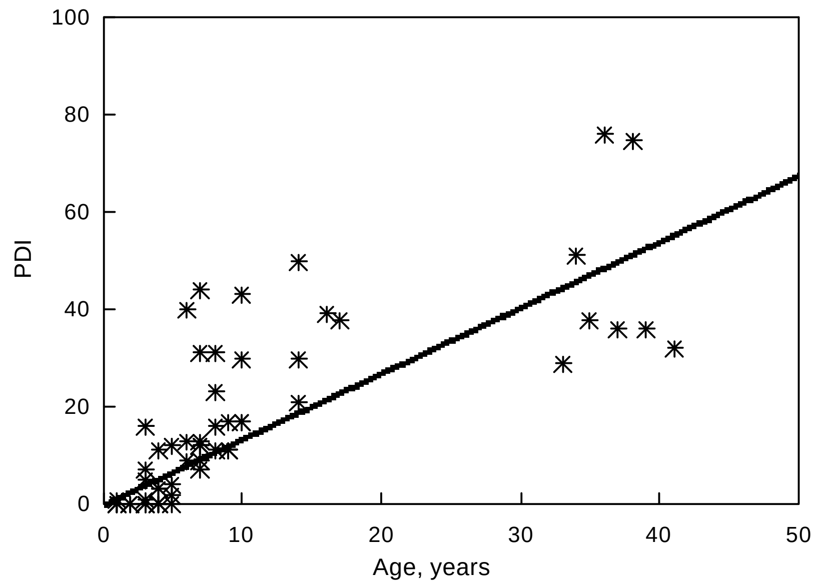

Figure 4.1: PDI vs Age for Undoweled Thin PCC Pavements

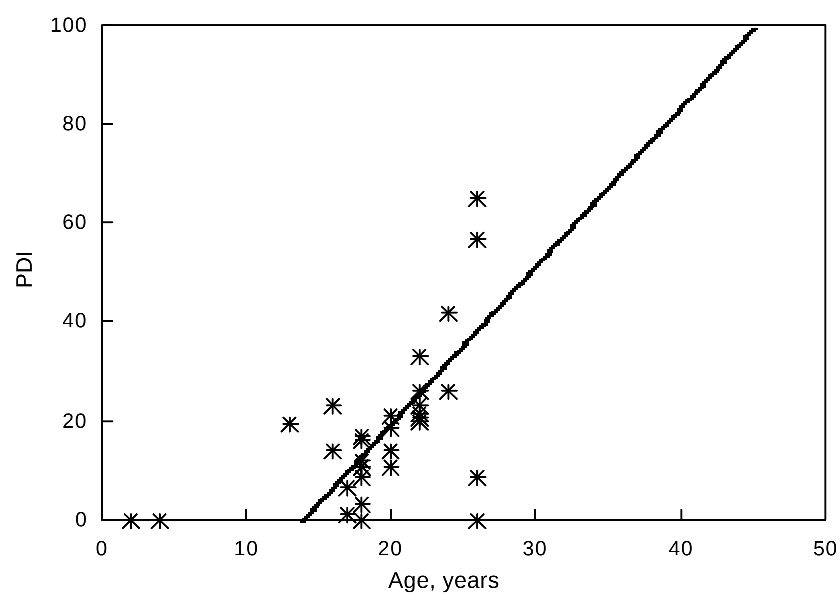

Figure 4.2: PDI vs Age for Doweled Thin PCC Pavements

Linear performance models were developed based on the data illustrated and are of the general form:

$$PDI(t) = At + B$$

where: PDI(t) = PDI at time t

t = Age of pavement, years

A = slope of regression line

B = PDI intercept

Performance models were developed using unconstrained and constrained (B=0) modeling. Regression values for each model are provided in Table 4.1.

Table 4.1: Regression Values for Thin PCC Pavements in Wisconsin and Northeast Iowa

Pavement Subgroup	Traffic Level	Slope A	Intercept B	R^2
Undoweled	All	1.19	4.56	0.644
Undoweled	All	1.35	0	0.622
Doweled	All	3.19	-44.2	0.384
Doweled	All	1.09	0	0.214

From the available data it is concluded that the constrained performance model is more appropriate for the undoweled pavement group and the unconstrained model for the doweled pavement group. The implication of the unconstrained model is an average projected delay of 13.9 years (-B/A) until the start of PDI accumulation.

Performance data from those surveyed sections not included in the model development were investigated by first adjusting the pavement age by the calculated design ratio.

The adjusted pavement age was determined as the actual pavement age divided by the calculated design ratio. Thus, for an over designed pavement with a design ratio of say 3.0 (Allowable ESAL/Design ESAL = 3.0) and an actual age of say 9 years, the adjusted age would be calculated as 9 / 3.0 = 3 years. Similarly, for an under designed pavement with a design ratio of say 0.6 (Allowable ESAL/Design ESAL = 0.6) and an actual age of say 9 years, the adjusted age would be calculated as 9 / 0.6 = 15 years.

Performance data collected from the under and over designed pavements are plotted in Figures 4.3 and 4.4. PDI trend lines established from the regression analysis have been included for comparative purposes. These data were not included in the original model building because it was felt that their performance, without age adjustment, would not be considered representative of properly designed pavements. Furthermore, the age adjustment described is considered to be only a rough estimate of the effect of under or over design on performance.

Figures 4.5 and 4.6 illustrate PDI vs Age trends for the Iowa pavements not surveyed by the research team, with age values adjusted based on the calculated design ratio (Allowable ESAL/Design ESAL).

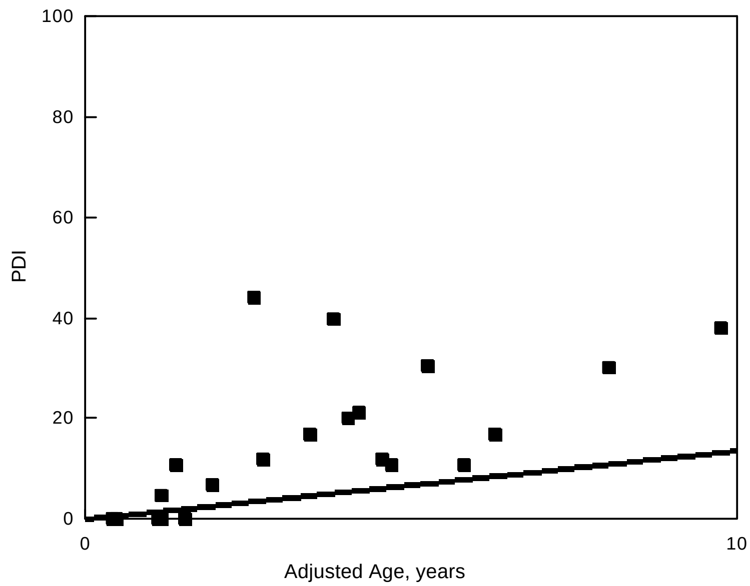

Figure 4.3: PDI vs Adjusted Age for Undoweled Thin PCC Pavements

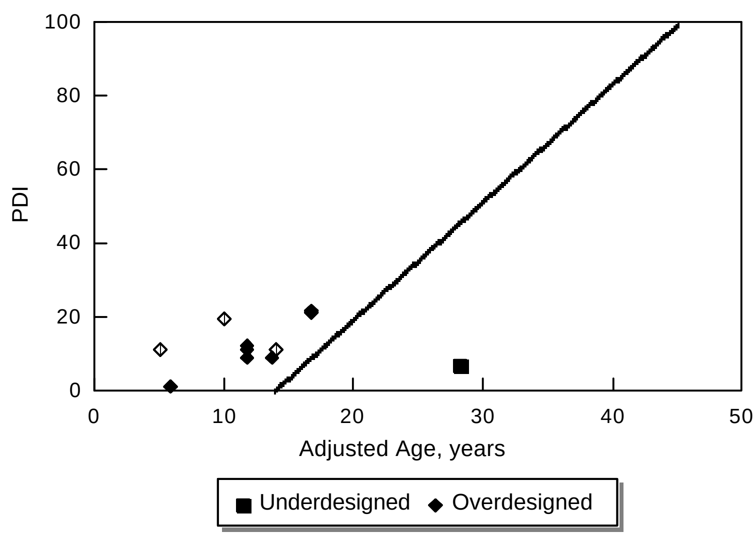

Figure 4.4: PDI vs Adjusted Age for Doweled Thin PCC Pavements

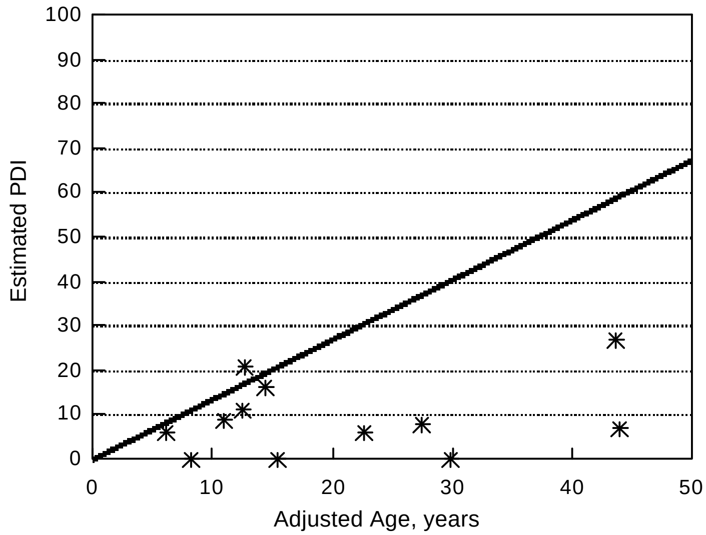

Figure 4.5: PDI vs Adjusted Age for Undoweled Thin Iowa PCC Pavements

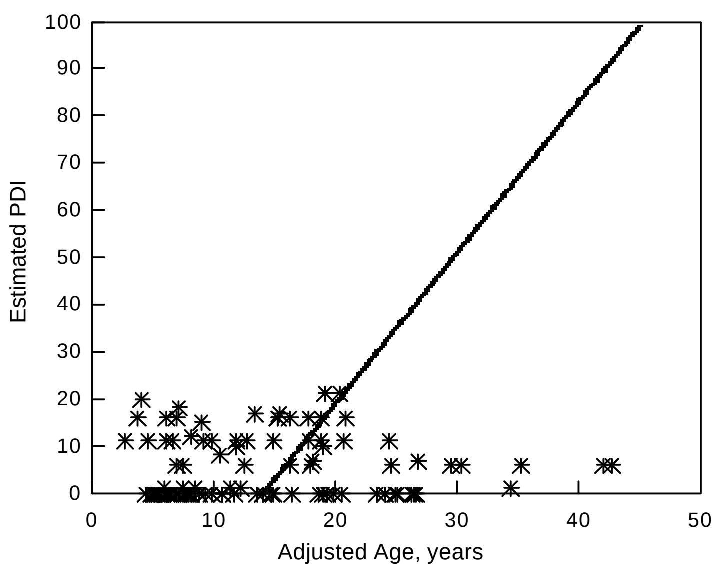

Figure 4.6: PDI vs Adjusted Age for Doweled Thin Iowa PCC Pavements

4.3 Conclusions and Recommendations Based on The Thin PCC Database

A review of the life cycle cost analysis (LCCA) input parameters as identified in Procedure 14-1-30 of the Facilities Design Manual, augmented by discussions with WisDOT personnel, indicates the current practice for conducting life cycle cost analyses for jointed plain concrete (JPC) pavements includes the parameters and assumptions shown in Table 4.2.

Table 4.2: Current LCCA Input Parameters for JPC Pavements

JPC Pavement Type	Assumed Service Life, Yrs	Critical PDI at First Rehab	Assumed Average PDI/yr Accumulation
Undoweled - Undrained	16	38	2.4
Doweled - Undrained	24	24	1.0
Undoweled - Drained	20	38	1.9
Doweled - Drained	30	24	0.8

The thin PCC database obtained from Wisconsin and Northeastern Iowa and included in the model building contains 49 JPC data elements, including 28 undoweled-undrained sections and 21 doweled-undrained sections. For the purposes of relative performance comparisons, pavement sections are considered to be performing better than assumed if the elapsed time to critical PDI exceeded the service life estimates listed in Table 4.2 or, for those pavements which have not yet reached critical PDI, the PDI/yr accumulation is less than Table 4.2 values. Conversely, those sections that reached critical PDI prior to their estimated service life or those sections which are accumulating PDI points at an accelerated rate are considered to be performing poorly.

The 28 undoweled, undrained thin PCC pavement sections include 24 sections (86%) which have not yet reached critical PDI = 38. Of these sections, 20 (83%) would be considered as performing better than anticipated, having an average PDI accumulation of less than 2.38 PDI/yr. Furthermore, 11 (46%) have an average PDI accumulation of less than 1 PDI/yr, and 7 (29%) have an average PDI accumulation between 1 PDI/yr to 2 PDI/yr. The remaining 4 sections (17%) had average PDI accumulations ranging from 2.4 PDI/yr to 5.33 PDI/yr. Of the 4 sections with PDI values greater 38, 3 (75%) have average PDI accumulations less than 2.38 PDI/yr, ranging from 1.5 PDI/yr to 2.11 PDI/yr. The remaining section had an average accumulation of 3.57 PDI/yr.

The 21 doweled, undrained pavement sections include 15 sections (71%) which have not yet reached the critical PDI = 24. Of these sections, 12 (80%) are performing better than anticipated, having average accumulations of less than 1 PDI/yr. The remaining 3 sections had average accumulations ranging from 1.05 PDI/yr to 1.44 PDI/yr. Of the 6 sections with PDI values greater than 24, 4 (67%) had average accumulations between 1.08 PDI/yr and 1.756 PDI/yr, and 2 had average accumulations ranging from 2.19 PDI/yr to 2.50 PDI/yr.

The drained thin PCC database includes only two sections, one doweled and one undoweled JPC. The undoweled pavement is 5 years old and has a PDI = 2 (PDI/yr = 0.4) while the doweled pavement is 4 years old and remains at PDI = 0. While in both cases PDI accumulations for these drained pavements are lower than assumed (assumed PDI/yr = 1.9 for undoweled JPC and 0.8 for doweled JPC), these pavements are still early in their life cycle and no definitive conclusions can be drawn as to estimated performance or service life.

Based on the analyses presented above, it is concluded that the currently assumed LCCA inputs, as listed in Table 4.2, are valid for undoweled-undrained and doweledndrained thin PCC pavements with little or no modifications. Based on observed average PDI accumulation rates, 82% of the undoweled-undrained and 57% of the doweled-undrained pavements are performing better than anticipated. Because of their relatively young age, additional performance data for drained pavement sections needs to be collected to determine if the assumed 25% increase in service life due to drainage is valid for these thin PCC pavement types.

5.0 CONCLUSIONS AND RECOMMENDATIONS

This report has been prepared based on research findings related to the design and performance of thick AC (AC thickness > 150 mm) and thin PCC (PCC thickness < 225 mm) pavements. A database of thick AC and thin PCC pavements constructed in Wisconsin has been developed to assess the performance of these pavements as identified by the accumulation of PDI points with age. This database is limited in size and has been augmented with thin PCC pavements constructed in Iowa. Even after augmentation, the database is still limited and thus conclusions drawn may be biased by size effects.

Current WisDOT service lives for AC and PCC pavements are characterized as interim and are defined as ranges of values. Using the pavement database developed for this project, pavement performance models have been developed for undrained non-overlaid thick asphalt pavements (NOTAP), overlaid thick asphalt pavements (OTAP) as well as doweled and undoweled thin PCC pavements. These models provide specific service life values at any desired terminal serviceability and provide an appropriate basis for conducting and comparing life cycle costs associated with thick AC and thin PCC pavements.

The developed performance models indicate that current WisDOT design assumptions for the service lives of undrained AC and PCC pavements to first major rehabilitation, developed based on the performance of thin AC (AC < 150 mm) and thick PCC pavements (PCC > 225 mm) constructed in Wisconsin, may be extended to thick AC and thin PCC pavements without significant error. No conclusions as to the performance of drained pavements (thick AC or thin PCC) can be made at this time due to limited performance data.

The results of this study further indicate that service lives for AC pavements are dependent on the desired pavement terminal serviceability level and functional classification. Hence, in current WisDOT pavement modeling efforts there is the need to expand existing service life criteria to include pavement service life values which reflect the effect of functional classification and serviceability. The review of current WisDOT pavement thickness design procedures has also indicated a need to incorporate design reliability for both AC and PCC pavement design.

Based on the results of this study, the following recommendations are made:

1. It is recommended that both thick AC and thin PCC pavements be considered as viable pavement designs as developed following current WisDOT design procedures presented in the Facilities Development Manual. Guidelines should be developed for establishing minimum soil support values needed to ensure proper placement and compaction of thick AC pavements. Furthermore, guidelines should be developed for establishing maximum soil/base support values for unbonded thin PCC pavements to limit the potential for cracking due to temperature curling.

2. It is recommended that design reliability be incorporated into the WisDOT Facilities Development Manual for both rigid and flexible pavement design. This could be accomplished by providing tabular values of ESAL factors for varying levels of design reliability, as shown in Tables 2.2 and 2.3 of this report, or through the adoption of the current AASHTO design equation which includes design reliability. This change would allow designers to better match the pavement design scenario and would provide a consistent design frame for flexible and rigid pavement alternatives.

3. For flexible pavement thickness design, a correlation between soil support value and resilient modulus should be developed for Wisconsin soils to allow for a

transition between current WisDOT design procedures and the existing 1993 AASHTO design procedures. This change would also facilitate an easier transition, if required, to the 2002 AASHTO design procedures currently being developed or to a WisDOT-specific mechanistic design procedure.

4. For rigid pavement thickness design, an additional nomograph providing for a terminal serviceability of 2.0 should be added to provided consistency between pavement type selection.

5. For rigid pavement thickness design, the thickness design nomograph should be modified by replacing the working stress (f_t) scale with a scale based on the 28-day compressive strength of concrete. The current assumed WisDOT design values of Mr=650 psi and E=4,200,000 psi represent a 28-day compressive strength (f'_c) of approximately 5,300 psi. In many instances, concrete with significantly lower 28-day compressive strengths are utilized during construction without modification to the design slab thickness, potentially leading to a reduced pavement fatigue life.

6.0 REFERENCES

1. Wisconsin Department of Transportation; “Pavement Surface Distress Survey Manual”, Division of Highways, Pavement Management Section, Madison WI 1993

2. Wisconsin Department of Transportation; “Instructions on Producing Geotechnical Investigations, Analyzing Data, and Reporting Result”. Geotechnical Bulletin No.1

3. Kleinbaum,David G., Kupper, L.L., and Muller, K.E.; “Applied Regression Analysis and Other Multivariable Methods”, PWS-KENT Publishing Company, Boston, MA, 1988.

4. Wisconsin Department of Transportation; Facilities Development Manual-Chapter 14 Proc.14-1-30, WisDOT, Madison, February 1995

5. Hill, Art “Full-depth Asphalt Pavement Study”- *Status Report* Submitted to the Full-depth Asphalt Pavement Steering Committee, MnDOT, 1994

6. IDOT; “ Mechanistic Pavement Design -Supplement to Section 7 of Design Manual,” Illinois Dept. of Transportation, 1989

7. Facilities Development Manual, Wisconsin Department of Transportation, Chapter 14, 1995.

APPENDIX A

THICK AC DESIGN FEATURES

Non-Overlaid Thick Asphalt Pavement Characteristics

City	Street	Location	Length (km)	Pvt AGE @ 1996	Thickness (mm)	ESAL	Subgrade Type	Base Type
Kenosha	55th St	63rd Ave - 64th Ave	0.06	17	225	Light	Clay	150 mm Stone
Kenosha	54th ave.	65th Ave - 67th	0.26	17	225	Light	Clay	150 mm Stone
Kenosha	67th St	60th Ave - 62nd Ave	0.21	17	225	Medium	Clay	150 mm Stone
Kenosha	68th St	58th Ave 60th Ave	0.16	12	200	Light	Clay	150 mm Stone
Kenosha	44th ave	73rd St - C.N.W.R.	0.16	12	200	Light	Clay	150 mm Stone
Kenosha	45th St	Hwy 31 - 56th Ave	0..39	12	250	Light	Clay	150 mm Stone
Kenosha	89th St	35th Ave -39thAve	0..26	12	200	Medium	Clay	150 mm Stone
Kenosha	Pershing Blvd	53rd St - 56th St	0.26	10	250	Light	Clay	150 mm Stone
Kenosha	81 St	43 Ave - 41CT	0..35	9	200	Light	Clay	150 mm Stone
Kenosha	47th Ave	52nd St - Washington	1.6	7	250	Light	Clay	150 mm Stone
Kenosha	46th St	56th Ave- Cul-DeSac	0.26	6	250	Light	Clay	150 mm Stone
Brookfield	Turnberry	Pilgrim - Bit.Sweet	0.48	8	190	Light	Clay	None
Brookfield	Bishops Way	Bl.mound End	0.8	9	200	Medium	Clay	None
Brookfield	Tnglwood	Calhoun - Havnwood	0.8	19	190	Light	Clay	None

Non-Overlaid Thick Asphalt Pavement Characteristics (Cont.)

City	Street	Location	Length (km)	Pvt Age @ 1996	Thickness (mm)	ESAL	Subgrade Type	Base Type
La Crosse	4th St	Badger - LaCrosse	0.13	14	175	Medium	Sand	125 mm Crushed Agg
La Crosse	La Crosse	2nd St - 4th St	0..10	14	175	Medium	Sand	125 mm Crushed Agg
La Crosse	3rd St	Cass St - Badger	0..93	14	175	Medium	Sand	125 mm Crushed Agg
La Crosse	West Ave	Adam St - Winnebago	0..47	13	175	Heavy	Sand	200 mm Crushed Agg
La Crosse	Copeland Ave	St Cloud - Clinton	0..68	12	175	Medium	Sand	150 mm Crushed Agg
La Crosse	Badger	3rd St - 4th St	0..06	12	175	Medium	Sand	125 mm Crushed Agg
La Crosse	Front St	Vine St North	0.19	12	175	Medium	Sand	150 mm Crushed Ag
La Crosse	Fanta Reed Pl	Fanta Reed Rd East	0..29	12	175	Medium	Sand	150 mm Crushed Agg
La Crosse	West Ave	Winnebago St - State St	0..76	12	175	Heavy	Sand	200 mm Crushed Agg
La Crosse	West Ave	Greenbay St - Adams St	0..69	11	175	Heavy	Sand	200 mm Crushed Agg
La Crosse	State St	4th St - 7th St	0..31	9	175	Medium	Sand	150 mm Crushed Agg
La Crosse	Main St	5th St - 7th St	0..24	7	175	Medium	Sand	125 mm Crushed Ag
La Crosse	Copeland Ave	S. of Hagar - S. of Cloud	0..13	7	175	Medium	Sand	125 mm Crushed Agg
La Crosse	Front St	State - Vine	0..10	7	175	Medium	Sand	150 mm Crushed Agg
La Crosse	King St	3rd St - 7th St	0..50	6	175	Heavy	Sand	None
La Crosse	Front St	Cameron - Jay	0..21	3	175	Medium	Sand	125 mm Crushed Agg
La Crosse	Ward St	Lasey - 32nd	0.81	3	175	Medium	Sand	150 mm Crushed Agg
La Crosse	La Crosse	3rd - 7th St	0.19	2	175	Medium	Sand	125 mm Crushed Agg

Non-Overlaid Thick Asphalt Pavement Characteristics (Cont.)

City	Street	Location	Length (km)	Pvt Age @ 1996	Thickness (mm)	ESAL	Sugbrade Type	Base Type
La Crosse	Ward Ave	South Ave-32nd St	1.61	28	175	Medium	Sand	150 mm Crushed Agg
La Crosse	State St	2nd - 3rd St	0..10	22	175	Medium	Sand	125 mm Crushed Agg
La Crosse	West Ave	State St - Badger	0..35	22	175	Medium	Sand	125 mm Crushed Agg
La Crosse	State St	Riverside Park -2nd St	0.35	22	175	Medium	Sand	125 mm Crushed Agg
La Crosse	Mt Vernon St	2nd St West	0..08	22	175	Medium	Sand	125 mm Crushed Agg
La Crosse	Vine St	Front St- 3rd St	0..10	20	175	Medium	Sand	150 mm Crushed Agg
La Crosse	4th St	Cass St - Vine	0..64	17	175	Medium	Sand	125 mm Crushed Agg
La Crosse	2nd St	Market St- Vine St	1.22	16	175	Medium	Sand	125 mm Crushed Agg
La Crosse	King St	Front St - 3rd	0.14	16	175	Medium	Sand	125 mm Crushed Agg
La Crosse	Jay St	Front St- 3rd St	0.16	16	175	Medium	Sand	125 mm Crushed Agg
La Crosse	Pearl St	2nd St - 3rd St	0.10	16	175	Medium	Sand	125 mm Crushed Agg
La Crosse	Main St	2nd St- 3rd St	0.10	16	175	Medium	Sand	125 mm Crushed Agg
La Crosse	Market St	Front St- 2nd	0.03	16	175	Medium	Sand	125 mm Crushed Agg
La Crosse	Cross St	Division St- Cameron Ave	0.13	16	175	Medium	Sand	125 mm Crushed Agg
La Crosse	Division St	Front St- 2nd St	0.06	16	175	Medium	Sand	125 mm Crushed Agg
La Crosse	Front St	Market St - Division St	0.21	16	175	Medium	Sand	125 mm Crushed Agg
La Crosse	West Ave	Badger St- LACrosse St	0.13	16	175	Medium	Sand	150 mm Crushed Agg
La Crosse	2nd St	Vine St - LACrosse	1.22	14	175	Medium	Sand	125 mm Crushed Agg

Non-Overlaid Thick Asphalt Pavement Characteristics (Cont.)

City	Street	Location	Length (km)	Pvt Age @ 1996	Thickness (mm)	ESAL	Subgrade Type	Base Type
La Crosse	Vine St	2nd St-3rd St	0.10	14	175	Medium	Sand	125 mm Crushed Agg
La Crosse	Grove St	2nd St-3rd St	0.13	14	175	Medium	Sand	125 mm Crushed Agg
La Crosse	3rd St	Badger St - LACrosse	0.19	14	175	Medium	Sand	125 mm Crushed Agg
Waukesha	Parklawn	CTH JJ-Moreland		8	150	Medium		250 mm Gravel
Waukesha	Abbott Dr	Kossow-Parklawn		16	150	Heavy		250 mm Gravel
Waukesha	E.Moreland Ct	Kossow-Cul-de-sac		9	150	Medium		250 mm Gravel
Waukesha	Kossow	Abbott-CTH JJ		17	150	Medium		250 mm Gravel
Waukesha	Springfield	Moreland-CTH JJ		19	150	Medium		250 mm Gravel
Waukesha	Wolf Rd	Stardust-Butler		11	150	Light		250 mm Gravel
Waukesha	Wolf Rd	Allen-Wolf Ct		18	150	Medium		250 mm Gravel
Waukesha	Jefferson	Gale-Oscar		24	150	Light		250 mm Gravel
Waukesha	Moreland	Michigan - Madison		21	150	Medium		250 mm Gravel
Waukesha	Madison	Grandview-Joellen		11	150	Medium		250 mm Gravel
Waukesha	Michigan	University-Greenmeadow		5	150	Light		250 mm Gravel
Waukesha	University Dr	Whitby-Michigan		4	150	Light		250 mm Gravel
Waukesha	University Dr	Sunkist-Northview		16	150	Medium		250 mm Gravel

Overlaid Thick Asphalt Pavement Characteristics in Wisconsin

City	Street	Location	Length (km)	Pvt ***Surface AGE*** @ 1996	Thickness (mm) [1]	ESAL	Subgrade Type	Base Type
Brookfield	Barker	Gebhardt-City Limits	0.81	6	275 (75)	High	SiltyClay Loam	N/A
Brookfield	Barker	Gebhardt - North	0..77	12	275 ( )	Medium	Silt Loam	N/A
Brookfield	Brookfield Road	Tamarack-Bluemound	1.77	3	450 (75)	Medium	Loam	Gravel
Brookfield	Brookfield Road	North-Riverview	0.75	7	350 ( )	High	Clay	N/A
Brookfield	Calhoun Road	Wisconsin-Gebhardt	0.81	2	300 (64)	Medium	Muck	N/A
Brookfield	Calhoun Road	RR Tracks -Burleigh	0.81	10	275 ( )	Medium	Muck	N/A
Brookfield	Calhoun Road	Burleigh - Capitol	1.61	2	300 (50)	Medium	Silty Clay	N/A
Brookfield	Parkside	Burleigh - Keefe	0.64	6	188 (50)	Light	Clay-Loam	None
Brookfield	LeChateau	Brookfield - North	0.81	1	188 ( )	Light	Clay	None
Brookfield	Forest Grove	Sunnyslap -Richard	0.48	10	188 ( )	Light	Clay	
Waushara Cty.	STH 49	Green Lk. Cty. Line - STH21	7.13	8	225 (100)	Light	Clay	None
Portage Cty	USH 51	CTH XX area	0.43	6	200 (75)	Heavy		150 mm Crushed Agg
Shawano Cty	STH 29	Laney - Birch Rd	4.46	6	200 (50)	Heavy		150 mm Crushed Agg

(1) Thickness of overlay shown in parenthesis.

APPENDIX B

THIN PCC DESIGN FEATURES

Urban Thin PCC Database

Location	Section	Const Year	Hpcc in	Dowels	Base Type	k pci	Daily ESAL
Appleton	Appleton	1980	8.0	No		100	83
Appleton	Glendale	1982	8.0	No		100	100
Appleton	Fremont	1989	8.0	No		100	100
Appleton	Ballard	1987	8.0	No		100	243
Appleton	Northland	1982	8.0	No		100	281
Appleton	Wisconsin	1986	8.0	No		100	356
Milwaukee	Howard	1992	8.0	No	4" CAB	100	142
Milwaukee	Howard	1992	8.0	No	4" CAB	100	159
Milwaukee	76th St.	1986	8.0	No	4" CAB	100	181
Milwaukee	Whitnall	1993	8.0	No	4" CAB	125	78
Milwaukee	60th St.	1989	8.0	No	4" CAB	150	162
Milwaukee	6th St.	1993	8.0	No	4" CAB	150	208
Milwaukee	Bradley	1979	8.0	No	4" CAB	150	217
Milwaukee	Fond Du lac	1987	8.0	No	4" CAB	150	258
Milwaukee	Bradley	1992	8.0	No	4" CAB	175	51
Milwaukee	Bradley	1992	8.0	No	4" CAB	175	138
Milwaukee	Morgan	1986	8.0	No	4" CAB	175	144
Milwaukee	84th St.	1989	8.0	No	4" CAB	175	162
Milwaukee	Clybourn	1993	8.0	No	4" CAB	175	220
Milwaukee	Chase	1982	8.0	No	4" CAB	175	233
Milwaukee	Sherman	1990	8.0	No	4" CAB	175	287
Milwaukee	Sherman	1989	8.0	No	4" CAB	175	287
Milwaukee	1st St.	1993	8.0	No	4" CAB	175	436
Milwaukee	Center	1987	8.0	No	4" CAB	200	233

WisDOT District 4 - PCC Pavement Performace Data

Segment	RP	Const Year	Hpcc	Base Type	Dowl	k pci	ADT	%T	ESAL	Age	PDI
STH52	7-16	1990	8	4"OGBC	Y	150	1970	10.9	107	2	0
STH52	7-16	1990	8	4"OGBC	Y	150	1970	10.9	107	4	0
STH13	130-143	1989	8	4"OGBC	N	150	6575	10.2	335	1	0
STH13	130-143	1989	8	4"OGBC	N	150	6575	10.2	335	3	0
STH13	130-143	1989	8	4"OGBC	N	150	6575	10.2	335	5	2
STH13	143-146	1987	8	6"CABC	N	150	6575	10.2	335	1	1
STH13	143-146	1987	8	6"CABC	N	150	6575	10.2	335	3	1
STH13	143-146	1987	8	6"CABC	N	150	6575	10.2	335	5	4
STH13	143-146	1987	8	6"CABC	N	150	6575	10.2	335	7	9
STH 13	79-88	1953	8	6"CABC	N	150	10000	7.0	350	33	29
STH 13	79-88	1953	8	6"CABC	N	150	10000	7.0	350	35	38
STH 13	79-88	1953	8	6"CABC	N	150	10000	7.0	350	37	36
STH 13	79-88	1953	8	6"CABC	N	150	10000	7.0	350	39	36
STH 13	79-88	1953	8	6"CABC	N	150	10000	7.0	350	41	32
STH13	114-115	1986	8	6"CABC	N	150	12315	7.0	431	2	0
STH13	114-115	1986	8	6"CABC	N	150	12315	7.0	431	4	3
STH13	114-115	1986	8	6"CABC	N	150	12315	7.0	431	6	9
STH13	114-115	1986	8	6"CABC	N	150	12315	7.0	431	8	11
USH10	256-263	1969	8	6"CABC	Y	150	8000	11.0	440	16	23
USH10	256-263	1969	8	6"CABC	Y	150	8000	11.0	440	18	17
USH10	256-263	1969	8	6"CABC	Y	150	8000	11.0	440	20	21
USH10	256-263	1969	8	6"CABC	Y	150	8000	11.0	440	22	23
USH10	256-263	1969	8	6"CABC	Y	150	8000	11.0	440	24	26
USH10	256-263	1969	8	6"CABC	Y	150	8000	11.0	440	26	57
USH10	247-257	1969	8	6"CABC	Y	150	8000	11.0	440	16	14
USH10	247-257	1969	8	6"CABC	Y	150	8000	11.0	440	18	16
USH10	247-257	1969	8	6"CABC	Y	150	8000	11.0	440	20	14
USH10	247-257	1969	8	6"CABC	Y	150	8000	11.0	440	22	26
USH10	247-257	1969	8	6"CABC	Y	150	8000	11.0	440	24	42
USH10	247-257	1969	8	6"CABC	Y	150	8000	11.0	440	26	65
USH10	242-247	1951	8	4"CABC	N	150	8000	11.0	440	34	51
USH10	242-247	1951	8	4"CABC	N	150	8000	11.0	440	36	76
USH10	242-247	1951	8	4"CABC	N	150	8000	11.0	440	38	75
STH 21	139-156	1988	8	6"CABC	N	150	6550	13.5	442	1	0
STH 21	139-156	1988	8	6"CABC	N	150	6550	13.5	442	3	16
STH 21	139-156	1988	8	6"CABC	N	150	6550	13.5	442	5	12
STH 21	139-156	1988	8	6"CABC	N	150	6550	13.5	442	7	31
STH 21	139-156	1988	8	6"CABC	N	150	6550	13.5	442	8	16
STH 21	137-139	1988	8	6"CABC	N	150	6550	13.5	442	1	0
STH 21	137-139	1988	8	6"CABC	N	150	6550	13.5	442	5	0
STH 21	137-139	1988	8	6"CABC	N	150	6550	13.5	442	7	13
STH 21	137-139	1988	8	6"CABC	N	150	6550	13.5	442	8	31
STH13	119-130	1984	8	6"CABC	N	150	9450	10.2	482	2	0
STH13	119-130	1984	8	6"CABC	N	150	9450	10.2	482	4	3
STH13	119-130	1984	8	6"CABC	N	150	9450	10.2	482	6	13
STH13	119-130	1984	8	6"CABC	N	150	9450	10.2	482	8	23
STH13	119-130	1984	8	6"CABC	N	150	9450	10.2	482	10	43

State of Iowa PCC Pavement Database

Rte	Sys	County	Co	Const Year	Length mi	Hpcc in	k pci	Base Type	Base Thick	ADT	ADTT
142	IA	Appanoose	4	1973	4.67	6.00	133			420	50
142	IA	Appanoose	4	1973	5.61	6.00	131			460	53
142	IA	Appanoose	4	1967	4.39	6.00	106			1020	106
102	IA	Mahaska	62	1967	5.73	6.00	58			1520	103
102	IA	Marion	63	1967	1.82	6.00	89			2820	327
21	IA	Keokuk	54	1965	11.01	6.00	101			1050	173
243	IA	Sac	81	1964	0.56	6.00	59			310	18
3	IA	Franklin	35	1976	1.12	7.00	108			3250	577
330	IA	Marshall	64	1974	3.17	7.00	65			2260	268
210	IA	Boone	8	1967	5.94	7.00	136			2320	169
4	IA	Greene	37	1979	1.86	7.50	178			1780	154
9	IA	Kossuth	55	1978	6.04	7.50	199			1410	228
17	IA	Hamilton	40	1978	7.17	7.50	100			3220	422
17	IA	Hamilton	40	1978	3.21	7.50	190			2830	408
9	IA	Winnebago	95	1978	0.94	7.50	191			2050	228
2	IA	Page	73	1975	1.81	7.50	193			2200	92
44	IA	Audubon	5	1981	0.81	8.00	76			600	63
17	IA	Boone	8	1980	11.13	8.00	211			2220	369
21	IA	Benton	6	1980	8.13	8.00	124			1650	274
17	IA	Hamilton	40	1980	3.78	8.00	134			2440	369
21	IA	Benton	6	1980	6.84	8.00	129			1740	274
175	IA	Hamilton	40	1980	1.75	8.00	124			2460	190
52	US	Jackson	49	1979	1.15	8.00	171			1420	129
2	IA	Lee	56	1979	5.41	8.00	200			2400	265
20	US	Buchanan	10	1979	6.45	8.00	179	CTB	100mm	5100	654
20	US	Buchanan	10	1979	6.45	8.00	192	CTB	100mm	5100	654
21	IA	Benton	6	1979	4.98	8.00	144			1980	164
2	IA	Lee	56	1979	3.39	8.00	196			2940	274
20	US	Buchanan	10	1979	6.06	8.00	189	CTB	100mm	4730	645
93	IA	Bremer	9	1978	0.91	8.00	99			1460	161
20	US	Buchanan	10	1979	6.06	8.00	161	CTB	100mm	4730	645
13	IA	Clayton	22	1978	13.46	8.00	96			2180	311
44	IA	Guthrie	39	1978	6.06	8.00	108			1070	130
44	IA	Audubon	5	1978	7.51	8.00	183			720	127
2	IA	Page	73	1978	3.20	8.00	173			2200	92
44	IA	Guthrie	39	1978	6.11	8.00	104			710	127
2	IA	Taylor	87	1978	0.91	8.00	157			1700	183
38	IA	Delaware	28	1978	2.46	8.00	145			2250	274
2	IA	Taylor	87	1978	11.86	8.00	172			1230	92
9	IA	Winnebago	95	1977	7.53	8.00	198			1430	170
25	IA	Adair	1	1977	0.75	8.00	177			1950	213
9	IA	Winnebago	95	1977	8.54	8.00	205			1640	213
67	US	Clinton	23	1978	5.68	8.00	99			2260	143
69	US	Decatur	27	1977	1.43	8.00	92			2320	150

Note 1: Joint Spacing for all pavements is 20"
Note 2: Pavements 8" or greater have dowels (1.25" diameter and 12" spacing)

State of Iowa PCC Pavement Database

Rte	Sys	County	Co	Const Year	Length mi	Hpcc in	k pci	Base Type	Base Thick	ADT	ADTT
62	IA	Jackson	49	1976	1.48	8.00	127			1320	86
44	IA	Audubon	5	1976	1.00	8.00	191			750	127
52	US	Dubuque	31	1976	2.43	8.00	148			5700	376
9	IA	Winnebago	95	1976	2.41	8.00	203			2590	162
169	US	Madison	61	1976	3.06	8.00	114			1060	146
2	IA	Taylor	87	1976	3.46	8.00	192			1140	130
2	IA	Fremont	36	1975	10.36	8.00	214			1570	159
141	IA	Monona	67	1976	1.50	8.00	62			1630	224
2	IA	Taylor	87	1976	4.00	8.00	185			1060	130
9	IA	Winneshiek	96	1975	9.88	8.00	152			2280	353
2	IA	Ringgold	80	1976	2.49	8.00	206			670	114
2	IA	Taylor	87	1976	5.91	8.00	189			750	98
52	US	Winneshiek	96	1975	11.77	8.00	194			2530	293
9	IA	Winneshiek	96	1975	4.46	8.00	146			2470	369
169	US	Union	88	1975	9.41	8.00	204			990	146
17	IA	Hamilton	40	1974	2.64	8.00	121			3940	512
9	IA	Howard	45	1974	8.47	8.00	200			2460	435
38	IA	Jones	53	1974	3.30	8.00	120			2540	267
44	IA	Guthrie	39	1974	5.62	8.00	91			2720	186
333	IA	Fremont	36	1974	0.73	8.00	181			1660	117
175	IA	Hardin	42	1975	4.22	8.00	98			2850	330
145	IA	Fremont	36	1974	2.68	8.00	139			660	94
44	IA	Dallas	25	1974	1.70	8.00	190			3410	178
2	IA	Page	73	1973	1.65	8.00	146			1370	112
38	IA	Delaware	28	1974	5.17	8.00	153			2190	274
2	IA	Appanoose	4	1972	10.95	8.00	95			2280	236
175	IA	Grundy	38	1973	4.95	8.00	166			2070	331
2	IA	Wayne	93	1972	7.12	8.00	91			1430	194
71	US	Page	73	1972	3.37	8.00	125			3200	314
71	US	Montgomery	69	1972	5.41	8.00	133			1950	309
71	US	Page	73	1972	2.18	8.00	101			2120	265
71	US	Page	73	1972	6.50	8.00	141			2140	314
71	US	Page	73	1972	10.64	8.00	121			1600	293
2	IA	Wayne	93	1971	2.50	8.00	106			3550	237
8	IA	Benton	6	1971	5.19	8.00	137			970	233
150	IA	Fayette	33	1971	4.88	8.00	189			1690	189
150	IA	Fayette	33	1971	4.37	8.00	100			1800	208
141	IA	Carroll	14	1972	5.78	8.00	154			1900	292
141	IA	Carroll	14	1971	2.64	8.00	205			1220	221
187	IA	Fayette	33	1970	1.23	8.00	218			1470	218
150	IA	Fayette	33	1970	4.13	8.00	119			2720	374
127	IA	Harrison	43	1969	0.72	8.00	215			1000	99
316	IA	Marion	63	1968	2.57	8.00	86			1440	124
173	IA	Cass	15	1968	5.57	8.00	182			1140	182

Note 1: Joint Spacing for all pavements is 20"
Note 2: Pavements 8" or greater have dowels (1.25" diameter and 12" spacing)

State of Iowa PCC Pavement Database

Rte	Sys	County	Co	Const Year	Length mi	Hpcc in	k pci	Base Type	Base Thick	ADT	ADTT
214	IA	Grundy	38	1969	5.04	8.00	118			950	115
316	IA	Warren	91	1968	0.97	8.00	64			1440	124
25	IA	Greene	37	1967	1.06	8.00	179			780	92
25	IA	Guthrie	39	1967	0.99	8.00	129			860	92
25	IA	Ringgold	80	1965	5.96	8.00	187			630	71
86	IA	Dickinson	30	1964	7.68	8.00	146			4090	232
25	IA	Ringgold	80	1964	6.00	8.00	179			760	88
13	IA	Clayton	22	1983	3.13	8.50	160			2890	401
363	IA	Benton	6	1984	0.89	8.50	199			2060	306
20	US	Hamilton	40	1979	3.99	8.50	195	ATB	100mm	5300	1009
20	US	Hamilton	40	1979	3.99	8.50	207	ATB	100mm	5300	1009
52	US	Clayton	22	1978	13.84	8.50	129			2040	317
20	US	Woodbury	97	1978	2.73	8.50	106	PCB	100mm	7700	590
71	US	Buena Vista	11	1977	4.14	8.50	136			3390	433
20	US	Woodbury	97	1978	2.73	8.50	107	PCB	100mm	7700	590
141	IA	Polk	77	1976	8.43	8.50	132	ATB	100mm	13200	1233
141	IA	Polk	77	1976	8.43	8.50	138	ATB	100mm	13200	1233
20	US	Woodbury	97	1976	1.48	8.50	162	ATB	100mm	14500	703
20	US	Woodbury	97	1976	1.48	8.50	156	ATB	100mm	14500	703
141	IA	Dallas	25	1976	0.70	8.50	119	ATB	100mm	7600	801
63	US	Davis	26	1975	10.34	8.50	91			2040	396
61	US	Clinton	23	1975	3.14	8.50	179	CTB	100mm	5300	768
61	US	Clinton	23	1975	3.14	8.50	142	CTB	100mm	5000	703
30	US	Clinton	23	1975	8.60	8.50	168	CTB	100mm	5800	991
30	US	Clinton	23	1975	7.78	8.50	204	CTB	100mm	5900	1039
34	US	Union	88	1973	8.08	8.50	113			4440	431

Note 1: Joint Spacing for all pavements is 20"
Note 2: Pavements 8" or greater have dowels (1.25" diameter and 12" spacing)

APPENDIX C

PDI CONVERSION PROCEDURES

Iowa Pavements

The pavement performance data provided within the Iowa database includes transverse cracking (# / 0.5 mi), patching (ft^2 / 0.5 mi), and transverse joint faulting. To develop PDI values based on this information, the following rules were applied:

Transverse Cracking

1. Transverse cracking values were divided by 5 to estimate the average number of cracks per PDI segment (0.1 mile) .
2. The number of transverse cracks per 0.1 mile were divided by 53 to estimate the percent of cracked slabs per PDI segment.
3. Where % Slabs Cracked was 40% or less, all cracked slabs were assigned a PDI designation of 2-3 lg.
4. Where % Slabs Cracked exceeded 40%, the first 40% were assigned a PDI designation of 2-3 lg and the remaining were designated as Additional.
5. Where % Slabs Cracked exceeded 100% (1 section), all slabs were assigned a PDI designation of Fragmented.

Patching

1. Patching values were divided by 5 to estimate the average patched area per PDI segment (0.1 mile).
2. The patched area per PDI segment was divided by 36 to estimate the number of patches per segment.
3. Where the patching extent was 6 or less per segment, a PDI designation of Good Severity was used.
4. Following the 25% rule, where the patching extent was between 7 and 20 per segment, a PDI designation of Fair Severity was used.
5. Following the 25% rule, where the patching extent exceeded 20 per segment, a PDI designation of Poor Severity was used.

Faulting

1. The severity of faulting was directly assigned based on the established PDI criteria.
2. The extent of faulting was assigned as GT 3 for all sections where the average faulting exceeded 1/8" (0.125").