WISCONSIN HIGHWAY RESEARCH PROGRAM # 0092-05-06

EFFECTS OF HEAVY LOADING ON WISCONSIN'S CONCRETE PAVEMENTS

FINAL REPORT

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16. Abstract

Wisconsin DOT District 7 filed a Report of Early Distress for a 6.5-mile stretch of USH 8 and an 8-mile stretch of USH 51 near Rhinelander in 2001. An investigation of the causes for the premature failures concluded that overloaded logging trucks were a key factor leading to the premature failure of the doweled jointed plain concrete pavements (JPCP). Consequently, a recommendation was made to develop design guidelines for heavy truck loading on concrete pavements in Wisconsin.

To develop the guidelines, JPCP design guides were solicited from several agencies, specifically, agencies located in Federal Highway Administration (FHWA) Climate Regions III and VI. A review of the design guides indicated that the 1993 AASHTO guide and the Portland Cement Association method are the two most popular state-of-the-art methods that attempt to address overloading, either using load safety factors or probabilistic concepts such as reliability. The two methods were further evaluated in terms of their ability to provide a transition to the AASHTO 2002 mechanistic-empirical design and allow a range of rehabilitation options for old JPCP. Based on the evaluation, the 1993 AASHTO guide was recommended for consideration in the design of JPCP in Wisconsin. The 1993 AASHTO guide was evaluated using data from one logging truck corridor along USH 8. The results indicated that a high-end reliability combined with modified rigid ESAL factors has the greatest potential to address overloading on Wisconsin's concrete pavements.

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EXECUTIVE SUMMARY

This report consists of a research study that developed design guidelines for heavy truck loading in Wisconsin. The study was to address concerns expressed in a 2001 Report of Early Distress for a 6.5-mile stretch of USH 8 and an 8-mile stretch of USH 51 near Rhinelander, Wisconsin.

A literature review was conducted to find all relevant information to rigid pavement design practices. In addition, concrete pavement design guides were solicited from U.S. states and Canadian provinces within solid frost regions, similar to Wisconsin. A review of the design guides found that a majority of agencies (80%) use AASHTO procedures for jointed plain concrete pavement (JPCP) design. Only the state of Iowa and the Canadian province of Quebec use the PCA method. Despite the detrimental effects of overloaded trucks on pavement performance, the state-of-the-art regarding overloading considerations in design is not very advanced. Only the state of Iowa indicated using a traffic load factor of 1.2 to account for overloading potential. Other agencies simply referred to contacting the pavement research division if conditions warrant. The results indicated that pavement engineers recognize the methods for dealing with overloading, yet there were no documented procedures in their design manuals to facilitate the design process.

Based on the review of the design guides, two feasible state-of-the-art pavement design methods were investigated for their applicability to Wisconsin pavement design, including the 1993 AASHTO guide and the PCA method. The two methods were evaluated based on their ability to address overloading, provide a transition to the AASHTO 2002 mechanistic-empirical design, and provide guidelines for rehabilitation of existing jointed plain concrete pavements. Based on this investigation, the 1993 AASHTO guide was recommended as the preferred design method for complete evaluation. Consequently, guidelines on the selection of design inputs for use of the 1993 AASHTO guide for new thickness design and structural overlay design for old JPCP were developed. Significant among the guidelines is the modification and extension of the existing WisDOT rigid ESAL factors to cover more Federal Highway Administration (FHWA) vehicle classes, particularly, vehicle classes 7 and 10. Wisconsin currently does not have ESAL factors for FHWA vehicle classes 7 and 10. However, a review of truck data from the pavements that prematurely failed, and also from a cross-section of traffic count sites suggest that these classes have surfaced on Wisconsin's highways, and need to be considered in the traffic loading estimation process.

The 1993 AASHTO guide was evaluated using data from one logging truck corridor along USH 8. The results indicated that a high-end reliability combined with modified rigid ESAL factors has the greatest potential to address overloading on Wisconsin's concrete pavements.

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1 INTRODUCTION

1.1 Background and Problem Statement

In July 2001, the WisDOT District 7 Soils Engineer filed a Report of Early Distress for a 6.5-mile stretch of USH 8 and an 8-mile stretch of USH 51 near Rhinelander [1]. These new pavements were constructed in the early 1990s as 9-inch doweled jointed plain concrete pavements (JPCP) with randomly spaced and skewed joints (unsealed), centerline parting strip, and six-inch dense graded base course. The pavements, however, exhibited significant early longitudinal cracking within two to four years of initial construction. An investigation of the causes for the premature failures concluded that, overloaded trucks and lower flexural strength values were the key factors leading to the premature failure of the pavements. Consequently, a recommendation was made to implement construction specification for PCC pavement flexural strength. Current Wisconsin DOT design procedures are based on AASHTO Interim Guide for Design of Pavement Structures, 1972, Chapter III Revised, 1981 [2], and do not provide criteria for designs involving overloaded pavements.

1.2 Objective

The objective of this research study is to recommend the most economical and efficient design procedure for concrete pavements subject to heavier than normal truck loading.

1.3 Significance of Work and Potential Benefits

The work contained in this research is significant since it will provide a better guide to WisDOT, and possibly other highway agencies, in making well-informed design decisions regarding heavy loading corridors.

The potential benefits to be derived from this study include:

- A design guide for concrete pavements subject to heavy loadings.
- Longer service lives for concrete pavements.
- Decreased maintenance costs and better utilization of resources.

CHAPTER 2 LITERATURE REVIEW

A literature review was conducted to find all relevant information to rigid pavement design practices within U.S. states and Canadian provinces within solid frost Regions III and VI. According to the Federal Highway Administration's (FHWA) classification, Region III has characteristics of a wet, hard freeze, and spring thaw [3]. Wisconsin is entirely included in Region III. Region VI is for areas having a dry, hard freeze, and spring thaw, and generally is west of Minnesota and Iowa. These regions are further defined as Climate Zones 01, 04, and 07 by the FHWA based on the relative amount of moisture. In these climate zones and regions, a long winter is experienced, with temperatures below freezing for extended periods. The potential for a slowly advancing freezing front into the subgrade is extremely high. Frost damage is to be expected accompanied with other low temperature problems [3]. The FHWA Regions are further depicted in Figure 2.1.

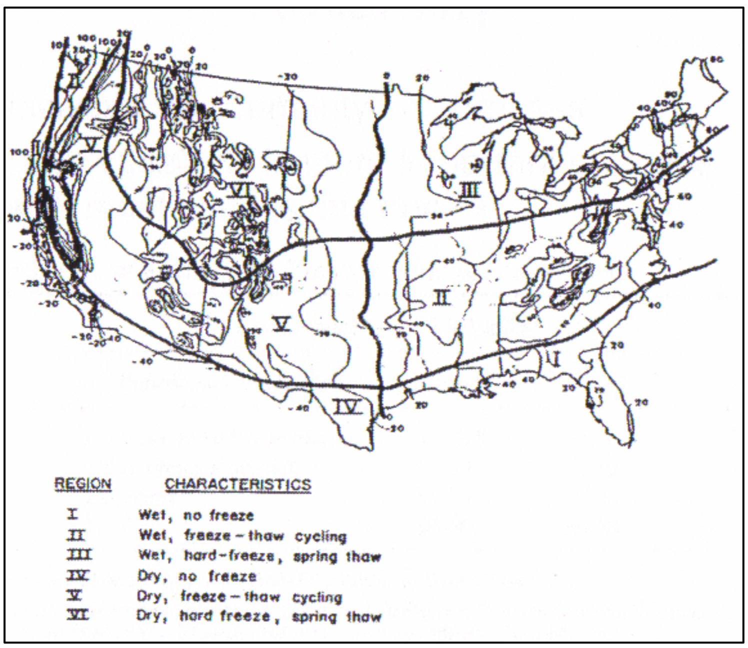

Figure 2.1 FHWA Climate Regions

2.1 Rigid Pavement Design Methods

The thickness design of rigid pavements follows one of two approaches namely, empirical and mechanistic. The latter method is often referred to as mechanistic-empirical because it tends to incorporate both empirical and mechanistic aspects.

Empirical methods are developed based on experience or results from experiments, often without any consideration to pavement system behavior or theory. They can be simple or complex in form. The simplest forms are purely based on experience (i.e., they use standard structural sections that yielded "adequate" performance in the past). The standard sections are provided for various design conditions with the hope that the recommended sections will again provide adequate performance. The major limitation with this approach involves the extrapolation of experience to future conditions such as variations in traffic volumes, different subgrades, new materials and construction methods. The complex forms of empirical methods mostly use statistical regression models to capture the relationship between observed performance and design inputs (loads, material properties, slab thickness, geographic location, and climatic conditions). The design in this case estimates the pavement thickness or the number of load applications to failure or some performance output (e.g., distress, serviceability index) using the design inputs. The main drawback for the complex forms is their inability to describe phenomena that occur outside the range of data or conditions used in their development. According to McGhee [4], The Federal Highways Administration's 1995-97 National Pavement Design Review found that approximately 80% of states make use of either the 1972, 1986 or 1993 AASHTO Pavement Design Guides. The 1972 version of the guide was empirically developed based on findings of the AASHO Road Test conducted in the late 1950s in northern Illinois [5]. The 1986 version indirectly incorporated mechanistic procedures for evaluating seasonal damage and to establish drainage coefficients for drainage and load transfer [6].

Mechanistic design methods, unlike empirical methods, seek to explain specific pavement responses (stresses, strains, and deflections) based on the physical causes including loads (traffic and/or environment) and material properties of the pavement structure. In addition, empirical elements are used to define values of the responses that result in failure. Failure is manifested as some magnitude of unacceptable pavement distress or roughness. The potential benefits from a successful application of mechanistic design procedures have been outlined in the 1993 AASHTO Guide [6], and include the following:

a. Estimates of the consequences of new loading conditions can be evaluated. For example, damaging effects of increased loads, high tire pressures, multiple axles, etc. can be modeled using mechanistic procedures.
b. Better utilization of available materials can be estimated. For example, the use of stabilized materials in both rigid and flexible pavements can be simulated to predict future performance.
c. Improved procedures to evaluate premature distress can be developed or conversely to analyze why some pavements exceed their design expectations. In effect, better diagnostic techniques can be developed.
d. Aging can be included in estimates of performance.
e. Seasonal effects such as thaw weakening can be induced in estimates of performance.
f. Consequences of subbase erosion under rigid pavements can be evaluated.
g. Methods can be developed to better evaluate the long-term benefits of providing improved drainage in the roadway section.

McGhee [4] indicates that, unlike empirical procedures, mechanistic methods do not become outmoded with changes in construction materials, traffic patterns, vehicle types, or tire types and configurations. Thus, they allow a full range of future enhancements to be readily developed and implemented. Although the mechanistic methods are much more rational than empirical methods, they are more technically demanding. Most analysis models associated with mechanistic methods employ finite element matrix and differential equations solutions, which generally involve significant computations and time. Hence, almost all mechanistic design procedures require some type of computer hardware and software to perform the extensive computations [7]. Examples of mechanistic methods include the Portland Cement Association (PCA) method for the thickness design of concrete highway pavements [8] and the Zero-Maintenance design procedure developed by Darter and Barenberg [9].

The key input parameters in any concrete pavement design procedure has been outlined by Hall [10] and include, traffic over a selected design period, subgrade support, base course properties, environment (temperature and moisture effects), concrete material properties (elastic modulus and flexural strength), performance criteria, and design reliability or a safety factor.

A review of concrete pavement design manuals obtained through email correspondence with Regions III and VI agencies, and other website searches, yielded the summary design practices shown in Table 2.1. Table 2.1 indicates that the majority of agencies (80%) use AASHTO procedures for JPCP design. Only the state of Iowa and the Canadian province of Quebec use the PCA method. Pavement Serviceability Index (PSI) is the chosen performance criterion among those states designing pavements with the AASHTO methods. A few states also include the International Roughness Index (IRI) or the Distress Index (DI) as an added performance criterion. Table 2.1 also indicates that one of two main parameters is used to represent the traffic input in thickness design, equivalent 18-kip single-axle load (ESAL) or direct axle load spectra. The former is associated with the AASHTO methods, while the latter is associated with the PCA design procedure. A database query of a 1999 national survey of U.S. State Department of Transportation (DOT) concrete pavement practices also indicated that approximately 93% of the states use some version of the AASHTO guide, while four use the PCA method [11].

The concrete strength parameters used in thickness design are also summarized in Table 2.2 for Regions III and VI agencies. Table 2.2 shows two main concrete strength parameters that are used, flexural and compressive strengths. Table 2.3 summarizes statistics of Table 2.2. Among the flexural group, a mid-point or a third-point test method is used to determine the flexural strength. Nearly all agencies use the third-point method to determine flexural strength. Flexural strength values range from 440 psi (for Pennsylvania) to 750 psi (for Quebec province and Wyoming), but 650 psi is the most commonly used value. The mean value of 648 psi is comparable with the current Wisconsin value of 650 psi published in Chapter 14 of its Facilities Development Manual [25]. Only the state of Illinois uses a 14-day test to determine the flexural strength value; all other agencies conduct a 28-day test.

Table 2.1 Rigid Pavement Design Practices for FHWA Regions III and VI States and Canadian Provinces

Agency State or Province	PCC Design Method (JPCP)	Design Input for Traffic		Design Performance Criteria
		ESAL	Axle Load Spectra	
(1)	(2)	(3)	(4)	(5)
Alaska	No PCC Guide			
Colorado	AASHTO 1993	√		PSI and IRI
Idaho	AASHTO 1993	√		PSI
Illinois	AASHTO 1972		√	PSI
Indiana	AASHTO 1993	√		PSI
Iowa	PCA		√	Fatigue and Erosion
Maine	AASHTO 1986	√		PSI
Massachusetts	AASHTO 1972 Modified 1981	√		PSI
Michigan	AASHTO 1993	√		PSI and DI
Minnesota	AASHTO 1972		√	PSI
Montana	AASHT0 1993	√		PSI
Nebraska	AASHT0 1993	√		PSI
Nevada	AASHTO 1993	√		PSI
New Hampshire	No PCC Guide			
New York	AASHTO 1993	√		PSI
North Dakota	AASHTO 1993	√		PSI
Ohio	AASHTO 1993	√		PSI
Oregon	AASHTO 1993	√		PSI
Pennsylvania	AASHTO 1993	√		PSI and IRI
South Dakota	AASHTO 1993	√		PSI
Utah	AASHTO 1993	√		PSI
Vermont	AASHTO 1993	√		PSI
Washington	AASHTO 1993	√		PSI
Wisconsin	AASHTO 1972	√		PSI and IRI
Wyoming	AASHTO 1993	√		PSI
Manitoba	AASHTO 1993	√		PSI
Ontario	AASHTO 1993	√		PSI
Quebec	PCA & AASHTO 1993	√		PSI
Nova Scotia	AASHTO 1993	√		PSI
Canadian Provinces generally not using PCC pavement: Alberta, British Columbia, Saskatchewan, New Brunswick, Prince Edward Island, and Newfoundland.				

Table 2.2 Concrete Strength Parameters Used in JPCP Thickness Design

Agency State or Province (1)	Strength Parameter Used (2)	Flexural Test Method (3)	Strength Value (psi) (4)	Time of Test (days) (5)
Alaska	No PCC Guide			
Colorado	flexural	Third-point	650	28
Connecticut	compressive		3500	28
Idaho	flexural		650	
Illinois	flexural	midpoint	650	14
Indiana	flexural	Third-point	650	28
Iowa	flexural	Third-point	525	28
Maine		Third-point	525-650	28
Michigan	flexural	Third-point	670	28
Minnesota	flexural	Third-point	500 or 675	28
Montana	flexural	Third-point	650	28
Nebraska	flexural	midpoint	710	28
Nevada	flexural	midpoint	600	28
New York	flexural	Third-point	600	28
North Dakota	flexural	Third-point	600	28
Ohio	flexural	Third-point	700	28
Oregon	flexural	Third-point	630	28
	compressive		4000	28
Pennsylvania	flexural		440	
South Dakota	flexural		650	28
Utah	flexural	Third-point	650	28
Washington	flexural	Third-point	650	28
Wisconsin	compressive		3000	28
Wyoming	flexural	Third-point	580 or 750	28
Manitoba	flexural		667	28
Ontario	flexural		740	28
Quebec	flexural		750	28

Table 2.3 Statistical Summary of Concrete Strength Parameters Used in JPCP Thickness Design

Design Parameter (1)	Minimum (2)	Maximum (3)	Mean (4)	Standard deviation (5)	Wisconsin Value (6)
Flexural Strength	440 psi	750 psi	648 psi (N=23)	68.4psi	650 psi
Compressive Strength	3000 psi	4000 psi	3500 psi (N=3)	500 psi	3000 psi

2.2 Traffic Loading Input in Thickness Design

2.2.1 Equivalent Single Axle Load

The concept of equivalent single axle load (ESAL) has traditionally been used by pavement engineers to measure the damaging effects of axle loads on pavements. The concept was originally developed from full-scale road tests conducted near Ottawa, IL, by the American Association of State Highway Officials (AASHO) in the late 1950s [5]. The road tests determined the effect of repeated heavy truckloads on various layered pavement systems. An 18-kip single axle load was established as the standard axle load and was denoted by 1.00 ESAL. Other axle loads were expressed in terms of load equivalency factors (LEF) to denote the relationship between the damage caused by each axle type and load to that caused by the standard axle for a given pavement. The original road test established LEF values for single and tandem axles for both flexible and rigid pavements. In 1986, AASHTO extended the road test results to provide LEF for tridem axles [12].

In the structural design of rigid pavements, mixed traffic is mostly converted to a single load input, cumulative 18-kip equivalent single axle load (ESAL) using a "simplified" or "rigorous" calculation procedure. The "simplified" procedure uses an average truck factor instead of a class-specific factor. The truck factors are generally developed using LEF values for specific axle configurations and corresponding axle loads. The LEF provides a means of expressing equivalent levels of damage between axles, while the truck factor expresses the total damage from the passing of a particular vehicle of interest.

The "simplified" calculation procedure allows a quick estimate of the design traffic loading to be obtained, but does not necessarily reflect an accurate estimate since it uses an average truck factor. It is used when classification data is unavailable. The "rigorous" calculation procedure requires the use of a specific truck factor for each truck classification. Mathematically, the cumulative ESAL over design period "t" based on the simplified and rigorous procedures are expressed in Equations 2.1 and 2.2, respectively.

$$ESAL = 365 * AADT * G_{Jt} * LDF * DF * TF \text{...}(2.1)$$

$$ESAL = 365 * AADT * G_{Jt} * LDF * DF * \left(\sum_{i=1}^{N} T_i * TF_i\right) \text{..........................}(2.2)$$

Where:

T_i = Proportion of truck type i in the AADT (entered as a decimal)

TF_i = Truck factor for truck type i

TF = Average truck factor for all trucks

LDF = Proportion of trucks in design lane (entered as a decimal)

DF = Directional distribution of truck traffic (entered as a decimal)

G_{Jt} = composite growth factor to account for growth in truck volume and truck factor over design period t.

Aunet [13] points out that ESAL forecasting is a very complex problem. Underestimation results in premature failure of pavements due to overloading, while overestimation results in thicker and longer lasting pavements, yet reduces the number of projects to be constructed from a limited pool of resources. ESAL forecasts generally consist of three major components: (a) traffic growth rate for all vehicles, (b) percent of heavy truck types in total vehicles, and (c) average equivalency factors by truck type. Errors associated with the forecast of any of the three components will compound the final ESAL value. The process is complicated by future uncertainties involving issues such as: (a) potential increases in statutory weight and length limits of trucks, (b) variations in modal shares of freight movement, (c) design of trucks and its impact on truck traffic, and (d) effects of increased tire pressures and uneven axle weight distributions on multiple axle groups.

Agencies have simplified the process by developing standard values and procedures for some variables presented in Equations 2.1 and 2.2. Table 2.4 shows truck or ESAL factor practices for FHWA Region III and VI agencies based on FHWA vehicle classifications 5 through 13. In some states, the factor is adjusted for roadway classification. Colorado ESAL factors are unique, in that the factor is uniform within single unit, single-trailer, and multi-trailer truck configurations. New York applies the same rate of 1.85 across all truck classes. Several states have no ESAL factors for certain truck classes, including Wisconsin, while several states have different factors for a range of roadway classifications. It is not clear how often these values are updated. However, Friedrichs [14] concluded that variations in ESAL factors could appreciably affect pavement life and cost.

Table 2.4 Current Practices for ESAL Factors in Rigid Pavement Design

Agency/State/ Province	FHWA Vehicle Classification								
	Single Unit Trucks			Single-Trailer Trucks			Multi-Trailer Trucks		
(1)	2 Axle-6 tire, Class 5 (2)	3 Axle, Class 6 (3)	4 Axle or more, Class 7 (4)	4 Axle or less, Class 8 (5)	5 Axle, Class 9 (6)	6 Axle or more, Class 10 (7)	5 Axle or less, Class 11 (8)	6 Axle, Class 12 (9)	7 Axle or more, Class 13 (10)
Alaska, 2001 to 2005	0.287	1.51	1.32	1.91	2.01	1.83	2.83	2.88	2.92
Alaska 2005+	0.5	0.85	1.2	1.2	1.55	2.24	1.55	2.24	2.24
Colorado	0.285	0.285	0.285	1.692	1.692	1.692	1.692	1.692	1.692
Indiana	0.90	0.90	0.90	2.0	2.0	2.0	2.0	2.0	2.0
Michigan	0.19	0.59	1.31	0.68	1.27	2.18	1.60	1.20	2.08
Montana – Interstate Routes	0.174	0.715	1.604	0.453	2.211	1.968	1.409	0.768	2.376
Montana – Principal Arterial Routes other than Interstate	0.198	0.799	1.564	0.503	1.989	1.797	1.093	0.812	2.320
Montana – All Other Routes (Minor Arterial)	0.247	1.104	1.916	0.454	2.042	2.028	0.926	1.365	2.338
Nebraska	0.3438	1.0751	1.6566	1.0626	2.3425	2.7508	1.6823	1.4021	1.7694
Nevada – Urban Interstate	0.216	0.980	0.980	0.742	1.983	2.506	2.043	1.247	2.887
Nevada – Urban Freeway/Expressway	0.164	1.248	1.248	0.557	1.877	2.106	1.315	0.733	3.620
Nevada – Rural Interstate	0.148	0.547	0.547	0.705	1.825	1.660	1.619	0.936	1.916
New York	1.85	1.85	1.85	1.85	1.85	1.85	1.85	1.85	1.85
North Dakota	0.20	0.68	1.90	0.45	1.90	1.40	1.80	1.40	2.20
Ohio – Urban Interstate	0.78	0.78	0.78	2.22	2.22	2.22	2.22	2.22	2.22
Ohio – Urban Freeway and Expressway	0.65	0.65	0.65	1.35	1.35	1.35	1.35	1.35	1.35
Ohio – Urban Principal Arterials and All Others	0.71	0.71	0.71	1.60	1.60	1.60	1.60	1.60	1.60
Ohio – Rural Interstate	0.53	0.53	0.53	1.84	1.84	1.84	1.84	1.84	1.84
Ohio – Rural Freeway and Expressway	1.02	1.02	1.02	2.36	2.36	2.36	2.36	2.36	2.36
Ohio – Urban Minor Arterials and All Others	1.59	1.59	1.59	1.45	1.45	1.45	1.45	1.45	1.45
Oregon	0.274	0.740	1.096	1.096	2.603	2.603	2.603	2.603	2.603
Pennsylvania	0.24	1.15	7.0	0.43 or 0.90	1.59	---	2.40	1.42	---
Utah – Rural Interstate	0.5064	0.5064	0.5064	4.8749	4.8749	4.8749	5.2436	5.2436	5.2436
Utah – Rural Principal Arterial	0.2065	0.2065	0.2065	2.9648	2.9648	2.9648	2.2342	2.2342	2.2342
Utah – Rural Minor Arterial	0.3061	0.3061	0.3061	2.7212	2.7212	2.7212	2.6832	2.6832	2.6832
Utah – Rural Major Collector	0.3103	0.3103	0.3103	1.5930	1.5930	1.5930	0.8221	0.8221	0.8221
Utah – Rural Minor Collector	0.3103	0.3103	0.3103	1.5930	1.5930	1.5930	0.8221	0.8221	0.8221
Utah – Rural Local System	0.3103	0.3103	0.3103	1.5930	1.5930	1.5930	0.8221	0.8221	0.8221

Table 2.4 Current Practices for ESAL Factors in Rigid Pavement Design (cont.)

Agency/State/ Province	FHWA VEHICLE CLASSIFICATION								
	Single Unit Trucks			Single-Trailer Trucks			Multi-Trailer Trucks		
(1)	2 Axle-6 tire, Class 5 (2)	3 Axle, Class 6 (3)	4 Axle or more, Class 7 (4)	4 Axle or less, Class 8 (5)	5 Axle, Class 9 (6)	6 Axle or more, Class 10 (7)	5 Axle or less, Class 11 (8)	6 Axle, Class 12 (9)	7 Axle or more, Class 13 (10)
Utah – Urban Interstate	0.3827	0.3827	0.3827	2.8934	2.8934	2.8934	3.6508	3.6508	3.6508
Utah – Urban Freeways and Expressways	0.3827	0.3827	0.3827	2.8934	2.8934	2.8934	3.6508	3.6508	3.6508
Utah – Urban Principal Arterial	0.1993	0.1993	0.1993	3.0104	3.0104	3.0104	3.1270	3.1270	3.1270
Utah – Urban Minor Arterial	0.3827	0.3827	0.3827	4.2476	4.2476	4.2476	5.2762	5.2762	5.2762
Utah – Urban Collector System	0.3827	0.3827	0.3827	4.2476	4.2476	4.2476	5.2762	5.2762	5.2762
Utah – Rural Local System	0.1993	0.1993	0.1993	4.2476	4.2476	4.2476	5.2762	5.2762	5.2762
Vermont – Interstate	0.1565	0.5264	1.725	0.5148	0.9643	1.2112	1.0799	0.7329	2.1179
Vermont – Principal Arterial	0.1352	0.8317	2.4081	0.4957	0.9304	1.9204	0.8415	2.946	4.3645
Vermont – Minor Arterial	0.1352	0.8317	2.4081	0.4957	0.9304	1.9204	0.8415	2.946	4.3645
Vermont – Major Collector	0.1352	0.8317	2.4081	0.4957	0.9304	1.9204	0.8415	2.946	4.3645
Vermont – Minor Collector	0.1352	0.8317	2.4081	0.4957	0.9304	1.9204	0.8415	2.946	4.3645
Vermont – Local System	0.1352	0.8317	2.4081	0.4957	0.9304	1.9204	0.8415	2.946	4.3645
Washington	0.4	0.4	0.4	1.00	1.00	1.00	1.75	1.75	1.75
Wisconsin	0.3	1.2	---	0.6	1.6	---	2.1	---	---
Wyoming	0.2557	0.6554	1.2317	0.539	1.8626	1.7998	1.5888	1.0126	1.847
Connecticut did not respond to email and phone calls. Maine has not constructed PCC pavements since 1985.									

The basic statistical summary of ESAL factors are shown in Table 2.5. Column 6 shows current values used in Wisconsin. Wisconsin has relatively low factors for FHWA Classes 5, 8, and 9 when compared to the average of the states.

Table 2.5 Basic Statistics of ESAL Factors in FHWA Regions III and VI

FHWA Class (1)	Minimum (2)	Maximum (3)	Mean (4)	Standard Deviation (5)	Wisconsin Value (6)
5	0.1352	1.85	0.3992	0.3580	0.3
6	0.1995	1.85	0.7238	0.3830	1.2
7	0.1993	7.0	1.2127	1.1535	-
8	0.43	4.8749	1.5957	1.2018	0.6
9	0.9304	4.8749	2.0584	0.9463	1.6
10	1.0	4.8749	2.2420	0.8599	-
11	0.8221	5.2762	2.0661	1.2538	2.1
12	0.8221	5.2762	2.2019	1.2934	-
13	0.8221	5.2762	2.7330	1.2736	-

2.2.2 Growth Factors

Traffic growth across the design life is an integral factor in the thickness design procedure. A wide range of approaches is used in the estimation of growth factors. Table 2.6 summarizes growth factors collected from state pavement design guides during the study. Inputs include traffic counts for separate classes, ESALs, projected growth percentages, and other regional factors. For example, Colorado uses a unique approach involving projection of land use types. The projected land use values are then used to estimate ESAL.

Table 2.6 Current Practices for Traffic Growth Factors in Rigid Pavement Design

<table>
<tr><th>Agency/State/
Province
(1)</th><th>Method for Computing Traffic Growth Factors

(2)</th></tr>
<tr><td>Colorado</td><td>Designer must request 20 and 30-year traffic projections for rigid pavements from the Traffic Section of DTD. The pavement designer can help ensure that accurate traffic projections are provided by documenting local conditions and planned economic development that may affect future traffic loads and volumes.

Residential Areas:
18k ESAL (20 years) = 62,000 + 80 R
Where, R = number of residential density units served by the roadway.

Commercial Areas:
18k ESAL (20 years) = 62,000 + 80 R + 260,000 CA
Where, CA = acres of commercial property served by the roadway.

Industrial Areas:
18k ESAL (20 years) = 62,000 + 80 R + 260,000 CA + 400,000 IA
Where, IA = acres of industrial property served by the roadway.</td></tr>
<tr><td>Idaho</td><td>Compute Traffic Index (TI) based on anticipated traffic loading for a 20-year design period.

$TI = 9.0\ (ESALs/10^6)^{0.119}$</td></tr>
<tr><td>Illinois</td><td>Calculate Traffic Factor (TF) based on Roadway Class and Vehicle Class.

<table>
<tr><th>Roadway Class
(1)</th><th>Traffic Factor Equation
(2)</th></tr>
<tr><td>Class I</td><td>$TF = DP \times [(0.15xPxPV) + (143.81xSxSU) + (696.42xMxMU] \times 10^{-6}$</td></tr>
<tr><td>Class II</td><td>$TF = DP \times [(0.15xPxPV) + (135.78xSxSU) + (567.21xMxMU] \times 10^{-6}$</td></tr>
<tr><td>Class III</td><td>$TF = DP \times [(0.15xPxPV) + (129.58xSxSU) + (562.47xMxMU] \times 10^{-6}$</td></tr>
<tr><td>Class IV</td><td>$TF = DP \times [(0.15xPxPV) + (127.75xSxSU) + (555.90xMxMU] \times 10^{-6}$</td></tr>
<tr><td colspan="2">Where, PV = Personal Vehicles SU = Single-Unit Trucks MU = Multi-Unit Trucks;
P,S,M = Percent of PV, SU, and MU in the design lane expressed as a decimal;
DP = Design Period, usually 20 years.</td></tr>
</table></td></tr>
<tr><td>Indiana</td><td>Growth Factor = $(\text{Design Year AADT} / \text{Current Year AADT})^{0.05} - 1$</td></tr>
<tr><td>Iowa</td><td>PCA standardized procedure used in traffic projection estimate.</td></tr>
<tr><td>Massachusetts</td><td>Design ADT and Future ADT, usually 20 years beyond the projected opening date, applied to design worksheet.</td></tr>
<tr><td>Michigan</td><td>Design ADT and Future ADT, usually 20 years beyond the projected opening date, applied to design worksheet.</td></tr>
</table>

Table 2.6 Current Practices for Traffic Growth Factors in Rigid Pavement Design (cont.)

<table>
<tr><th>Agency/State/ Province
(1)</th><th>Method for Computing Traffic Growth Factors

(2)</th></tr>
<tr><td>Minnesota</td><td>Three functions provide estimated future truck traffic volumes based on growth projection.

<table>
<tr><th>Function Description
(1)</th><th>Model
(2)</th></tr>
<tr><td>No Growth</td><td>$AADTT_X = 1.0 \text{ x } AADTT_{BY}$</td></tr>
<tr><td>Linear Growth</td><td>$AADTT_X = GR \text{ x } AGE + AADTT_{BY}$</td></tr>
<tr><td>Compound Growth</td><td>$AADTT_X = AADTT_{BY} \text{ x } (GR)^{AGE}$</td></tr>
<tr><td colspan="2">$AADTT_X$ is the annual average daily truck traffic at age X; GR is the traffic growth rate, and $AADTT_{BY}$ is the base year annual average daily truck traffic.</td></tr>
</table>
</td></tr>
<tr><td>Montana</td><td>Growth rate based on a 20-year historical growth rate for total traffic compounded annually. The distribution of truck traffic is taken from the most recent classification count in the area. If a count is not taken in the area, then a statewide average distribution is used for the type of route. A seasonal adjustment is currently made based on total volume. Currently developing a new traffic processing software and will incorporate truck hourly and seasonal factors. Also have the information to process Load Spectrum Data from Weigh-In-Motion locations.</td></tr>
<tr><td>Nebraska</td><td>An annual report for truck weight and vehicle classification count data is issued to provide a basis for estimating the frequency of each truck, give year-to-year changes in axle and gross weights, and allow comparison of characteristics of actual usage with administrative policies.</td></tr>
<tr><td>Nevada</td><td>Design life of 35 years is applied to rigid pavement designs. Specific traffic growth varies by region.</td></tr>
<tr><td>New York</td><td><u>Annual truck weight growth rate</u>. Typical values are between 0% and 2% per year. The annual traffic volume growth rate for the general traffic pattern stream may be used if no values are available for trucks alone. These growth rates are calculated every year based on the previous 15 years of traffic volume and are tabulated by region and functional class. Contact the Regional Planning Group for the annual truck volume growth rate. Where no other information is available use a rate of 1%.

<u>Annual truck volume growth rate</u>. It is expressed as a percentage and typically ranges from 0% to 4% per year. A high growth rate can cause a dramatic increase of ESALs over the life of the pavement structure. NYSDOT does not currently have enough data to predict the increase of truck weights. When no other information is available use a rate of 0.5%.</td></tr>
<tr><td>Ohio</td><td>Estimation of ESALs available for locations where historical traffic data is available. This method takes in account for growth rates in numbers of trucks as well as growth rates in the conversion factors associated with the trucks. The method relies on the practice of forecasting the future based on trends of the past. However, trends of the past may not be an indication of future performance. For more information regarding this method, contact the Office of Materials Management, Pavements Section.</td></tr>
<tr><td>Oregon</td><td>Expansion Factor calculated from year of construction to useful life.

$E = [1 + (R/100)]^n$
Where, R = Annual Growth (%)
E= Expansion Factor
N = Number of Years</td></tr>
<tr><td>Pennsylvania</td><td>Annual % Growth in Truck Factor. A 0% growth should be used for the annual percent growth in truck factor, unless historical evidence or loading studies justify otherwise.</td></tr>
</table>

Table 2.6 Current Practices for Traffic Growth Factors in Rigid Pavement Design (cont.)

Agency/State/ Province (1)	Method for Computing Traffic Growth Factors (2)
South Dakota	Growth factors are estimated based on historical data and future trends from the Traffic Unit.
Utah	Growth Rates (for Volume, Classification, and Axle Configuration). The Regional Pavement Management Engineer can give some guidance, when needed, in the areas where there are no growth rates available for volume, classification, and axle configurations. When the truck factors (ESALs/vehicle) are not available, use the Utah 91-93 ESALs.
Washington	Traffic projections are calculated for each individual roadway. Seasonal factors associated with each month for the same time period are tracked. While actual seasonal factors change from year to year, overall the basic seasonal pattern is consistent.
Wisconsin	Traffic information is available from the Division of Transportation Investment Management, Transportation Forecasts and Analysis Section. Data are collected for current ADT, construction year ADT, design year ADT, and Truck classification, by axle configuration. In some cases, the construction year classification may be projected to the design year and both classification counts will be shown. The designer should then use a straight-line average classification between the two counts. Unless otherwise specified, a traffic analysis period of 20 years is used.
Growth Rates were not collected for these states: Connecticut, Maine, North Dakota, Vermont, and Wyoming.	

2.2.3 Uncertainty and Variability in Design Parameters

Uncertainty and variability in design parameters are commonly captured through the use of safety factors or probabilistic design concepts that address the amount of statistical variability associated with parameters in the design process. The reliability concept described by AASHTO [6] is one such tool.

Reliability is a statistical tool used in pavement design that assumes a standard normal distribution exists for all pavement design parameters and allows the designer to account for deviation from the average, equally for all parameters. AASHTO defines reliability as, "the probability that the load applications a pavement can withstand in reaching a specified minimum serviceability level is not exceeded by the number of load applications that are actually applied to the pavement" [6].

The reliability factor gives the designer the option of incorporating a risk reduction factor into the pavement design process. Reliability is not dependent on either type of pavement or type of project. The reliability factor accounts for chance variations in both traffic prediction (ESALs) and performance prediction, therefore providing a predetermined level of assurance or reliability (R) that pavement sections will survive their design life. A higher reliability factor results in a greater chance that the pavement will be above the terminal serviceability at the end of the projected design life.

The standard deviation accounts for both chance variation in the traffic prediction and normal variation in pavement performance prediction for a given number of ESALs. The overall standard deviation is a measure of the spread of the probability distribution for ESALs versus

Serviceability, considering all the parameters used to design a pavement. The standard deviation parameter controls the level of precision for the designs.

Table 2.7 provides reliability factors and standard deviations from design guides reviewed during the study. As shown in the table, the reliability level is dependent on functional classification or a combination of functional classification and the level of traffic volume or load. Standard deviation (So) values generally ranged from 0.34 to 0.40. These values are used for rigid pavement design unless values representative of local conditions can be documented. .

2.3 Effects of Overloaded Trucks on Pavements

The detrimental effects of overloaded trucks on the structural and functional conditions of highway pavements have been well documented. Several factors have been attributed to these effects including, increased truck traffic, increased number of illegally overloaded trucks, increased number of requests for overloaded permits, very liberal policies on permit issuance, the inflexibility of rail transportation systems, construction of improved highway systems, increases in legal truck weight limits, and poor traffic projections.

A national study on the effect of permit and illegal overloads on pavements reported that approximately 15% of all loaded trucks are overloaded with respect to one of the statutory limits (single axle, tandem axle, or gross load). The cost of overloaded trucks suggests that more than $1 billion of damage is done yearly to all federal-aid highways [15].

The 1977-78 congressional hearings on the impact of overloads on the Highway Trust Fund revealed that the Interstate system was deteriorating 50% faster than it could be replaced. Overloaded trucks were cited as one of the key factors causing this rapid deterioration. The pavement damage cost per year due to overloaded truck axles was estimated to vary from $160 million to $670 million [12].

Taylor et al. [16] reported on a study regarding the economic benefits of increased pavement life that results from port of entry in Idaho. The study indicated that the average overload on a truck was 12% in excess of the legal limit. The study also found that 32% of all trucks observed were overloaded for the route studied. The corresponding damage to the pavement was estimated to be 20% higher than originally projected.

Table 2.7 Reliability Factors in Rigid Pavement Design

Agency/State Province (1)	Functional Classification: Interstate (2)	Principal Arterial (3)	Minor Arterial (4)	Major Collector (5)	Local (6)	Standard Deviation (7)
Colorado – Urban	85-95	80-95	70-95	50-90	50-80	-
Colorado - Rural	80-95	70-95	60-90	50-85	50-75	Same as Urban
Connecticut	N/A					
Idaho	90	85	85	80	---	0.34
Illinois	N/A					
Indiana	98	95	90	90	90	0.35
Maine	No PCC for 20 years					
Massachusetts	N/A					
Michigan	95	95	95	95	95	0.39
Minnesota	N/A					
Montana	85-95	75-95	75-95	50-80	---	0.39
Nebraska	85-95	80-95	75-95 ADT> 3,000	75-95 ADT> 3,000	70-95 ADT< 3,000	0.35
Nevada	95 > 54 Million ESALs	90 < 54 Million ESALs	---	---	---	0.35
New York	90	90	90	90	90	---
North Dakota	90	90	---	---	---	0.39
Ohio – Urban	95	90	90	90	90	0.39
Ohio – Rural	90	85	85	85	85	0.39
Oregon – Urban	90	90	85 Minor Collector	85	75	0.39
Oregon – Rural	90	85	80 Minor Collector	85	75	0.39
Pennsylvania	97-99	90-99	90-99	85-95	70-90	0.35
Utah	95	90	90	90	90	0.35
Vermont – Urban	99	95	90	90	75	0.35
Vermont – Rural	95	90	85	85	75	0.35
Washington	75-95	75-95	75-95	75-95	75-95	0.40
Wisconsin	N/A					
Wyoming	90-95	85	80	75-80	75	0.35
Canada - Quebec	90-95	80-90	80-90	66-80	50-66	---

Figure 2.2 illustrates the relationship between pavement damage cost and travel distance for various levels of truck overloads as reported by Casavant and Lenzi [17] for Washington State. The damage cost increases exponentially with travel distance for various truck-overload levels.

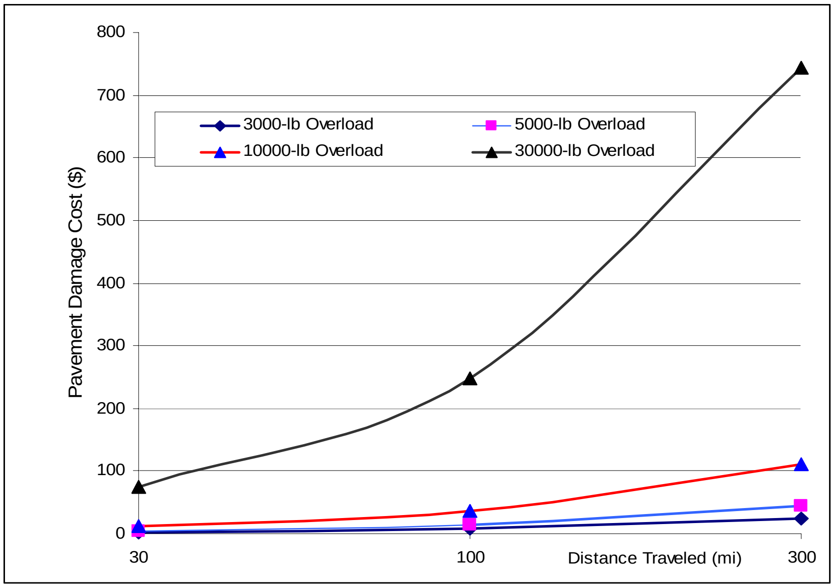

Figure 2.2 Pavement Damage Cost versus Overloaded Truck Traveled Distance (Adopted from [17])

Eriksen and Casavant [18] examined the impacts of the North American Free Trade Agreement (NAFTA) on the state of Washington's transportation infrastructure. They specifically evaluated highway investment requirements to sustain industry capacity in trade flows. It was determined that NAFTA truck ton-miles on the state's highways were approximately $10.4 billion in 1994 and projected to increase by 29% by 2005. The study also indicated that a total of $9.1 million was spent in the maintenance of three major routes to sustain highway usage associated with NAFTA. The maintenance cost was projected to increase to $22.6 million.

2.4 Overloading Considerations in Design

Despite the detrimental effects of overloaded trucks on pavement performance, the state-of-the-art regarding overloading considerations in design is not very advanced. A summary of the review of FHWA Regions III and VI agencies is shown in Table 2.8. Only the state of Iowa indicated using a traffic load factor of 1.2 to account for overloading potential. Other agencies simply referred to contacting the pavement research division if conditions warrant.

A 1987 nationwide survey of pavement engineers regarding methods to incorporate truck overloading in state pavement design indicated a wide range of approaches to dealing with overloaded trucks in design. The approaches are summarized in Table 2.9. The most common approach reported involved the use of conservative safety factors in design. However, the specific factors were not reported in the study. It is interesting to note that for states that indicated using historical data and regression models to predict the magnitude of truck overloading, the elements deemed important in the regression model included legal weight limits, effectiveness of enforcement program, and severity of penalty for violation. It would be expected that some correlation exists between the results reported in Table 2.8 (most recent data from design manuals and email correspondence) and Table 2.9 but that is not the case. The results seem to suggest that pavement engineers recognize the methods for dealing with overloading, yet there are no documented procedures in their design manuals to facilitate the design process.

Table 2.8 Methods for Addressing Overloading in Pavement Design

Agency State or Province (1)	Methods for Addressing Overloading (2)
Colorado	Div. of Trans. Development should be notified of special traffic situations.
Idaho	None
Illinois	If heavy trucks are 10% of truck traffic, contact Bureau of Materials and Physical Research. Give consideration to the potential impacts of heavily loaded vehicles, especially in areas near mines, grain elevators, factories, landfills, and river ports.
Indiana	None
Iowa	Load Factor of 1.2
Massachusetts	None
Michigan	None
Minnesota	Truck volumes and loadings characterized in terms of the volume of heavy trucks applied over the pavements design life and axle load spectra for single, tandem, tridem, and quad axles.
Montana	None. Design all PCC pavements with 9-inch slab thickness and believe this thickness provides additional capacity for unknowns such as overloading.
Nebraska	None
Nevada	None
New York	None
Ohio	None
Oregon	None
Pennsylvania	None
South Dakota	None
Utah	None
Vermont	Review effects caused by several traffic generators (recreation, school, logging) and apprise Traffic Research in the ESAL Request
Washington	None
Wisconsin	None
Manitoba	None
Ontario	None
Quebec	None
Nova Scotia	None

Table 2.9 Methods to Incorporate Truck Overloading in the Pavement Design (Adopted from [15])

Method (1)	Number of States (2)
Assume a fixed percentage for overloading	2
Based on historical truck weight data and regression model to estimate extent and magnitude of overloading	9
Obtain violation data from enforcement agency and compute seasonal overloading truck population	1
Use very conservative safety factors in design	14
Not considered	9
Use state shifting procedure to shift truck weight distribution curve and then compute ESAL.	2
Use W-4 Table, which contains certain percentage of overloaded trucks	9

2.5 Summary and Conclusions

A review of rigid pavement design practices was conducted, with particular attention to practices in FHWA Regions III and VI. The purpose of the review was to determine practices addressing overloaded trucks in the pavement design process, and assess their applicability to Wisconsin pavement design. Based on the review, the following conclusions are reached:

- The most dominant and complex empirical method used by agencies for the design of rigid pavements is specified in the Guide for Design of Pavement Structures published in 1993 by AASHTO.
- The PCA method is the preferred mechanistic method for rigid pavement design among pavement engineers.
- Although agencies recognize the high cost and damaging effects of overloaded trucks on pavements, there is zero to very little documentation in agency design manuals regarding how overloading is addressed in the pavement design process.
- Although the AASHTO and PCA methods are popular methods among engineers, there is considerable non-uniformity in pavement design practices among agencies on key input parameters including flexural strength, ESAL factors, and traffic projection methods. Flexural strength values range from 440 psi (for Pennsylvania) to 750 psi (for Quebec province and Wyoming). The mean value of 648 psi is comparable to the current Wisconsin value of 650 psi published in Chapter 14 of its Facilities Development Manual [25]. ESAL factors, on the other hand, had a wide range for the various FHWA vehicle classes. Pennsylvania had an extremely high factor for FHWA Class 7, with a value of 7.0. Wisconsin had relatively low factors for FHWA Classes 5, 8, and 9 when compared to the average of the states.

CHAPTER 3 FEASIBLE RIGID PAVEMENT DESIGN METHODS FOR WISCONSIN

Two popular and state-of-the-art feasible pavement design methods were initially investigated for their applicability to Wisconsin pavement design, in terms of their potential to address overloaded truck impacts in the design process and provide a transition to the AASHTO 2002 mechanistic-empirical design guide. In addition, for a procedure to be considered feasible it should allow the designer to consider a range of rehabilitation options for existing pavement structures. A basic comparison of existing WisDOT procedure with the two selected methods, the 1993 AASHTO guide and PCA, are presented in Table 3.1 and discussed in the following sections.

Table 3.1 Preliminary Comparison of Feasible JPCP Design Methods with Current WisDOT Design

Design Method (1)	Extent of Usage by Pavement Engineers (2)	Addresses Overloading (3)	Provides a Transition to AASHTO 2002 (4)	Addresses Overlay/rehabilitation Design (5)
WisDOT (AASHTO 1972)	Seldom	No	No	No
AASHTO 1993	Dominant	Yes	Yes	Yes
PCA	Moderate	Yes	Yes	No

3.1 Feasible Pavement Design Methods and Overloading Considerations

The 1993 AASHTO design method uses a reliability factor input as a safety factor for which a pavement is designed. The reliability reflects the degree of risk of premature failure associated with a given design by accounting for chance variations in both traffic and performance predictions. It therefore, provides a predetermined level of assurance that the pavement sections will survive the design period [6]. Higher reliability levels are associated with high volume roads and roadways of higher functional classification.

Besides the uncertainty in ESAL estimating, uncertainty also exists in all other variables that go into the thickness design of the pavement. According to Hall [10], the reliability factor is a function of the overall standard deviation associated with the AASHTO model and reflects errors associated with each of the design inputs (ESAL, subgrade K-value, concrete strength, serviceability, etc). In addition, the reliability factor is generally applied to the traffic ESAL input, and when that is done, Hall [10] points out that only average values should be considered for the remaining inputs; that is, no safety factors should be applied to the rest of the inputs. Although the majority of the agencies contacted in the FHWA Regions III and VI indicated using the AASHTO 1993 guide, it was surprising that none of them indicated the importance of the reliability factor input to address overloading (see Table 2.8).

The PCA thickness design method attempts to limit the number of load repetitions based on both fatigue and erosion analyses. It uses a cumulative damage concept for the fatigue analysis to prevent the first crack initiation due to critical edge stresses. The erosion analysis, on the other hand, focuses on prevention of pavement failures such as pumping, erosion of foundation, and joint faulting due to critical corner deflections [19]. To account for unpredicted truck overloads, a load safety factor (LSF) ranging from 1.0-1.3 is applied to the expected axle loads for the thickness design. An LSF value of 1.2 is typically applied to interstate and other multilane projects associated with high volumes of truck traffic, while a value of 1.1 is applied to highways and arterials with moderate volumes of truck traffic. The higher value of 1.3 is reserved for very busy facilities (e.g., urban freeways) with no alternate detour routes for the traffic [8].

3.2 Feasible Pavement Design Methods and Transition to AASHTO 2002 Mechanistic Design

Any proposed design method to be used by WisDOT is expected to provide a transition to the AASHTO 2002 design guide. The AASHTO 2002 design procedure, when it becomes fully implemented, will be the ultimate state-of-the-art practice tool for the design of new and rehabilitated pavement structures. It is based on mechanistic-empirical principles and concepts and includes methodologies for calibration, validation, and adaptation to local conditions.

A design method was considered to provide a transition if it uses design inputs and concepts similar to those outlined in the AASHTO 2002 design guide. Table 3.2 provides a summary comparison of inputs for the design of jointed plain concrete pavements based on the WisDOT, PCA, AASHTO 1993, and AASHTO 2002 design methods. From Table 3.2, WisDOT is lacking in design considerations that include uncertainty or reliability, base/subbase effects, load transfer, and broader environmental effects.

Table 3.2 Design Input Considerations for JPCP Design Methods

Design Parameter (1)	Design Method			
	WisDOT (2)	AASHTO 1993 (3)	PCA (4)	AASHTO 2002 (5)
Traffic	ESAL (derived from axle load spectra)	ESAL (derived from axle load spectra)	Axle load spectra	Axle load spectra
Performance criteria	Serviceability loss	Serviceability loss	Fatigue damage and erosion	*At least one of the following*: • Transverse cracking • Transverse joint faulting • Pavement Smoothness (IRI)
Roadbed strength indicator	Modulus of subgrade reaction, K	Effective Modulus of subgrade reaction, K	Modulus of Subbase-subgrade reaction	Effective Modulus of subgrade reaction, K
Unbound Subbase/base considerations	Standard thickness commonly used; thickness effects are not directly considered in the design	Subbase thickness, resilient modulus, and base erodibiity are used to determine effective K.	Subbase type and thickness used in overall K determination	*a. Data for Critical Response Computations* Layer thickness, resilient modulus, Poisson's ratio, and coefficient of lateral earth pressure. *b. Additional Data for Distress/Transfer Functions* Gradation parameters and base erodibility *c. Additional Data for Climatic Modeling* Gradation parameters, effective grain-size, specific gravity, and hydraulic conductivity. [23]
PCC material properties	Elastic modulus, working stress	Elastic modulus, modulus of rupture	Modulus of rupture	a. *Data for Critical Response Computations* Elastic modulus, coefficient of thermal expansion, unit weight, and Poisson's ratio. *b. Additional Data for Distress/Transfer Functions* Modulus of rupture, split tensile strength, compressive strength, cement type and content, water-to-cement ratio, ultimate shrinkage *c. Additional Data for Climatic Modeling* Thermal conductivity, heat capacity, and surface shortwave absorptivity. [23]
Environment/ Climatic factors	A regional factor, R=3 is used throughout the state	Moisture effects are evaluated through the use of a drainage layer coefficient	Not directly considered in the design	Temperature and moisture variations are evaluated.
Uncertainty	Not considered	Reliability applied to load input	Load safety factor	Reliability applied to performance criterion
Load transfer considerations	Not directly considered in design	Doweled and non-doweled joint effects evaluated	Doweled and non-doweled joint effects evaluated	Effects of dowel bar size on expected faulting can be analyzed
Shoulder	Standard sections used; impacts are not considered in the design.	Asphalt and PCC shoulder effects considered	PCC and non-PCC shoulder effects considered	Effects of AC, widened PCC slab, and tied PCC shoulders can de analyzed.

3.3 Feasible Pavement Design Procedures and Rehabilitation of Existing JPCP

Rehabilitation involves work done to significantly extend the service life of existing pavements through the principles of resurfacing, restoration, and/or reconstruction [6]. It is a critical component in the life cycle cost analysis of pavement options. A design method was considered feasible if it provided guidelines on the design and/or selection of rehabilitation options for correcting deficiencies in an existing pavement. The 1993 AASHTO guide provides design and selection guidelines for two broad rehabilitation categories, namely overlay and non-overlay methods. The PCA method on the other hand, does not contain any rehabilitation guidelines. Existing WisDOT rehabilitation guidelines are also very limited in the number of options and do not provide design guidelines for structural overlays.

3.4 Summary and Recommended Design Method

Two main feasible pavement design methods were investigated for their applicability to Wisconsin pavement design, including the 1993 AASHTO guide and the PCA method. The two methods were evaluated based on their ability to address overloading, provide a transition to the AASHTO 2002 mechanistic-empirical design, and provide guidelines for rehabilitation of existing pavements. The PCA method addresses overloading using a load safety factor varying from 0 to 20 percent with pavement functional classification. The factor is applied to each axle load range prior to determining slab stresses. The use of inappropriate factors can lead to over design or under design. A more realistic approach to addressing uncertainty is the direct use of probabilistic design concepts or the use of safety factors that reflect the amount of statistical variability associated with each of the parameters in the design process. The 1993 AASHTO guide captures the latter approach better than the PCA with the reliability concepts. The reliability concepts are also used directly in the AASHTO 2002 mechanistic-empirical design. In addition, the 1993 AASHTO guide, unlike the PCA method, provides guidelines on rehabilitation design. Hence, the 1993 AASHTO guide is recommended as the preferred design method for complete evaluation.

CHAPTER 4 AASHTO 1993 GUIDE INPUTS FOR JPCP DESIGN

The 1993 AASHTO guide presents an equation that requires a number of inputs related to loads, pavement structure, performance, and subgrade support. The general form of the design equation is as presented in Equation 4.1.

$$Log_{10}W_{18} = Z_R * S_O + 7.35*\log_{10}(D+1) - 0.06 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.5-1.5}\right]}{1+\frac{1.624*10^7}{(D+1)^{8.46}}} + (4.22-0.32P_t)*A \quad (4.1)$$

$$A = \log_{10}\left[\frac{S'_C * C_d\left[D^{0.75}-1.132\right]}{215.63*J\left[D^{0.75}-\frac{18.42}{\left(E_c/k\right)^{0.25}}\right]}\right]$$

Where:

W_{18} = predicted number of 18-kip ESALs
Z_R = standard normal deviate
S_o = combined standard error of the traffic prediction and performance prediction
D = slab thickness (inches)
P_t = terminal serviceability
ΔPSI = difference between the initial design serviceability index, Pi, and the design terminal serviceability index, Pt
S'_c = modulus of rupture of PCC
C_d = drainage coefficient
J = load transfer coefficient
E_c = elastic modulus of PCC (psi)
K = modulus of subgrade reaction (psi/in)

4.1 General Input Variables

The following subsections describe input variables for the 1993 AASHTO guide. The variables include traffic, reliability, performance criteria, drainage coefficient, load transfer, and modulus of subgrade reaction (K).

4.1.1 Traffic

The design procedure is based upon cumulative expected 18-kip equivalent single axle loads (ESAL). ESAL estimates can be made using the rigorous or simplified procedure discussed under Section 2.2.1 of this report.

4.1.2 Reliability

The reliability of a pavement design-performance process is the probability that a pavement section designed using the process will perform satisfactorily over the traffic and environmental conditions for the design period [6]. The standard normal deviate, Z_R, and the overall standard deviation, S_o, variables account for reliability. The range of S_o specified by AASHTO for rigid pavements is 0.30-0.40, while the recommended values for reliability for various functional classes are as shown in Table 4.1.

Table 4.1 Suggested Levels of Reliability for Various Functional Classifications (Adopted from [6])

Functional Classification	Recommended Level of Reliability, %	
	Urban	Rural
(1)	(2)	(3)
Interstate and Other Freeways	85-99.9	80-99.9
Principal Arterials	80-99	75-95
Collectors	80-95	75-95
Local	50-80	50-80

4.1.3 Performance Criteria

Performance is defined in terms of initial and terminal serviceability. The primary measure of serviceability is the Present Serviceability Index (PSI), which ranges from 0 (very poor road) to 5 (an excellent road). The initial serviceability, P_i, is the PSI value immediately after construction, while the terminal serviceability, P_t, is the lowest PSI value that prompts the need for rehabilitation, resurfacing, or reconstruction of the existing pavement. AASHTO suggested values for P_i and P_t are respectively 4.5 and 2.5 or higher for rigid pavements. Agencies are however, encouraged to develop their own values since construction methods and maintenance practices vary from agency to agency.

The elastic modulus of the PCC can be estimated using the following relationship in Equation 4.2. For PCC Modulus of Rupture, the mean value determined after 28 days using third-point loading is used in design.

$$E_c = 57{,}000 * \sqrt{f'_c} \quad \text{............(4.2)}$$

Where:

E_c = PCC elastic modulus (psi)
f'_c = PCC compressive strength in psi as determined using AASHTO T22, T140, or ASTM C39

4.1.4 Drainage Coefficient

The expected level of drainage for rigid pavements is considered through the use of a drainage coefficient, C_d. The C_d value represents the relative loss of strength due to the pavement's drainage characteristics and total time it is exposed to near-saturation moisture conditions. The quality of drainage rating is based on the guidelines in Table 4.2. Using the drainage quality rating, the C_d value can be determined from Table 4.3.

Table 4.2 Drainage Quality Rating (Adopted from AASHTO 1993 [6])

Quality of Drainage (1)	Water Removed Within (2)
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very Poor	Water will not drain

Table 4.3 Recommended Values of Drainage Coefficient, C_d for Rigid Pavement Design (Adopted from AASHTO 1993 [6])

Drainage Quality	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
(1)	Less than 1% (2)	1-5% (3)	5-25% (4)	Greater than 25% (5)
Excellent	1.25-1.20	1.20-1.15	1.15-1.10	1.10
Good	1.20-1.15	1.15-1.10	1.10-1.00	1.00
Fair	1.15-1.10	1.10-1.00	1.00-0.90	0.90
Poor	1.10-1.00	1.00-0.90	0.90-0.80	0.80
Very Poor	1.00-0.90	0.90-0.80	0.80-0.70	0.70

4.1.5 Load Transfer

The load transfer coefficient, J, is a factor used in rigid pavement design to account for the ability of a concrete pavement structure to transfer load across discontinuities, such as joints or cracks [6]. The lower the J factor the better the load transfer. Table 4.4 provides recommendations for ranges of load transfer coefficients for JPC pavements for different conditions. The AASHO Road Test conditions represent $J = 3.2$ as all joints were doweled and no tied shoulders. The lower J-value for the tied shoulders assumes that traffic is not permitted to run on the shoulder.

Table 4.4 Recommended Load Transfer Coefficients for Doweled JPCP and Design Conditions

Shoulder Type (1)	Load Transfer Coefficient, J (2)
Asphalt	3.2
Tied PCC or widened outside lanes	2.5-3.1

4.1.6 Modulus of Subgrade Reaction, K

The modulus of subgrade reaction, K, is used to estimate the support of the PCC slab by the underlying layers. An effective K-value is calculated to reflect subbase and subgrade contributions as well as the loss of support that occurs over time due to erosion and/or differential vertical movement of the underlying layers.

4.2. Suggested Design Inputs for use in the 1993 AASHTO Guide for JPCP Design in Wisconsin

There cannot be a fixed set of design inputs for use in the 1993 AASHTO Guide that can be applied to WisDOT rigid pavement designs; however, some guidance is offered as a starting point in the design process. Further knowledge about these inputs will improve and undoubtedly change over time. Listed below are suggested design inputs for consideration in the use of the 1993 AASHTO Guide for the design of JPCP in Wisconsin. Key elements that warrant consideration include rigid ESAL factors for loading estimation, effect of shoulder type and thickness in the JPCP thickness design process, load transfer device considerations, and reliability to address uncertainties in the design.

4.2.1 Traffic

ESAL estimates for Wisconsin's rigid pavements are based on site-specific truck distributions using standard rigid pavement ESAL factors established by WisDOT. Equation 4.3 is used to determine the design lane ESALs. Rigid ESAL factors currently in use are shown for various vehicle classifications in Table 4.5. According to Friedrichs [14], the current ESAL factors for rigid pavements were introduced in 1987. In 1989, Aunet [13] further pointed out that Wisconsin's traffic program at the time collected data on nine different truck types but only the five listed in Column 3 of Table 4.5 were common enough to be used for pavement design purposes.

A review of recent (2001-2003) Wisconsin vehicle classification data from several classification count sites suggests that FHWA vehicle classifications 7 and 10 in particular, are common on Wisconsin's roadways, and thus require considerable attention in the traffic load estimation process. A closer look at the traffic count data for the failed pavements in the Report of Early Distress (R.E.D.) further reveals the presence of all FHWA light and heavy truck classes including 7 and 10 (see Table 4.6 and Appendix A), which currently are not addressed in the load estimation process.

Several efforts, including emails and repeated phone calls were made to contact the Wisconsin State Patrol Officers in charge of the USH 8 and USH 51 segments to determine the feasibility of setting up portable weigh stations at these sites. This was to enable actual loading data to be acquired for estimating load factors for the suspected overloaded logging trucks. However, no responses were received from the patrol offices regarding the emails and the phone messages.

The research team visited the USH 8 and 51 project segments on March 24-26, 2005, to randomly observe logging truck configurations, and longitudinal crack distresses present on USH 51. Logging truck configurations observed were FHWA Classes 6, 7, 9, and 10. Figures in Appendix B provide documentary evidence of these truck classes on, or adjacent to, the failed pavement segments. These observations further support the need to account for traffic loading across all truck classifications.

Table 4.5 (Column 4) further shows the rigid ESAL factor means for all FHWA Regions III and VI surveyed, while Column 5 shows the mean values for states in the same geographic region as Wisconsin. It is reasonable to assume that truck traffic patterns will be similar within a geographic region compared to patterns between regions. Based on this assumption, the ESAL factor values in Column 6 of Table 4.5 are recommended until Wisconsin establishes specific values from a detail truck weight study.

$$DDESAL = \frac{ADT_C + ADT_P}{2} * DF * LDF * Efactor_i \text{..............} (4.3)$$

Where :

DDESAL = design daily ESALs

ADTc = current or construction year average daily traffic

ADTp = average daily traffic projected for the design year

DF = directional factor (usually 0.5)

LDF = lane distribution factor

Efactori = rigid esal factor for truck type i

Table 4.5 ESAL Factors for Rigid Pavement Design

FHWA Class (1)	Wisconsin Designation (2)	Current WisDOT ESAL Factors (3)	FHWA Regions III & VI Means (4)	WI, OH, MI, IN Means (5)	Recommended Values (6)
5	2D	0.3	0.3992	0.570	0.57
6	3SU	1.2	0.7238	0.893	0.89
7	-	-	1.2127	1.03*	1.03
8	2S-1, 2S-2	0.6	1.5957	1.270	1.27
9	3S2	1.6	2.0584	1.668	1.67
10	-	-	2.2420	1.990*	1.99
11	2-S1-2	2.1	2.0661	1.875	1.88
12	-	-	2.2019	1.667*	1.67
13	-	-	2.7330	1.960*	1.96
* does not include Wisconsin					

Table 4.6 Average Percent of Traffic by Vehicle Classification 2003

Year	Days	AADT	FHWA CLASS								
			5	6	7	8	9	10	11	12	13
Site 430157 - USH 51 South of CTH Y Hazelhurst Township											
2003	2	7196	2.16	2.89	0.44	3.22	2.16	0.41	0.02	0.02	0.40
Site 430652 - USH 8 / STH 47 West of CTH G - Rhinelander											
2003	2	4485	2.76	2.00	0.27	2.31	4.59	0.38	0.01	0.01	0.01
2003	40	4485	2.75	1.62	0.12	2.72	3.38	0.16	0.00	0.00	0.01
Source: 2003 Wisconsin Vehicle Classification Data, Wisconsin Dept. of Transportation, Madison, Wisconsin, March 2004 [26].											

4.2.2 Design Serviceability Loss

Design Serviceability Loss, ΔPSI, involves the selection of an initial PSI (P_i) and terminal PSI (P_t). Current WisDOT jointed rigid pavement thickness design procedures allow P_i and P_t values of 4.5 and 2.5 respectively [25]. This results in design serviceability loss of 2.0.

4.2.3 Modulus of Rupture and Elastic Modulus

WisDOT fixes design inputs for Modulus of Rupture (S'_c) and Elastic Modulus (E_c) at 650 psi and 4,200,000 psi, respectively [25].

4.2.4 Load Transfer Coefficient

Doweled transverse joints are specified for all rigid pavement designs regardless of thickness to ensure proper load transfer across joints. For doweled JPCP with asphalt shoulders a J-factor of 3.2 is recommended. This is similar to the AASHO Road Test conditions, where all joints were doweled and no tied shoulders existed [6]. For the standard WisDOT design of doweled joints and widened outer lanes, a J-factor of 3.0 is to be used. The range provided by AASHTO is 2.5-3.1. The lower end of the range assumes that traffic is not permitted to run on the shoulders. However, Sawan et al. [24] indicated that the proportion of truck encroachment on the shoulder varies between 1 to 8% of the adjacent outer lane truck traffic. In addition, shoulders may be used by traffic during maintenance of the mainline pavement or used as a parking location for disabled vehicles.

4.2.5 Drainage Coefficient

Drainage coefficient, C_d, value of 1.15 is recommended for JPCP sections with permeable Open Graded Base Courses (OGBC) associated with edge drains. For sections without OGBC, a C_d-value of 1.0 is recommended.

4.2.6 Modulus of Subgrade Reaction

The modulus of subgrade reaction, K, is currently provided in the soils report by WisDOT. In lieu of the report, standard correlations between K-values and other soil strength indicators (soil support value, design group index, resilient modulus) are described in WisDOT's Facilities Development Manual Procedure 14-1-1 [25]. These correlations, however, do not consider the effects of subbase type and thickness, and the effects of a shallow rigid layer, if present. The 1993 AASHTO guide adjusts K-values to account for the potential loss of subbase support. A supplement to the guide published in 1998 [20], however, indicated that substantial loss of support existed for many sections at the AASHO Road Test, which led to increased slab cracking and loss of serviceability. Hence, the performance data used in developing the AASHO Road Test performance model already reflected the effect of considerable loss of support. It has also been argued by the American Concrete Pavement Association (ACPA) that the AASHTO procedure for estimating K-values does not produce realistic results. The values deviate significantly from historical and theoretical values [21]. Figure 4.1 was therefore, derived from values developed by the ACPA to account for the thickness of untreated granular subbases under rigid pavements. Figure 4.1 is recommended for use in adjusting the subgrade K-value for design, in order to give credit to the subbase under the JPCP layer.

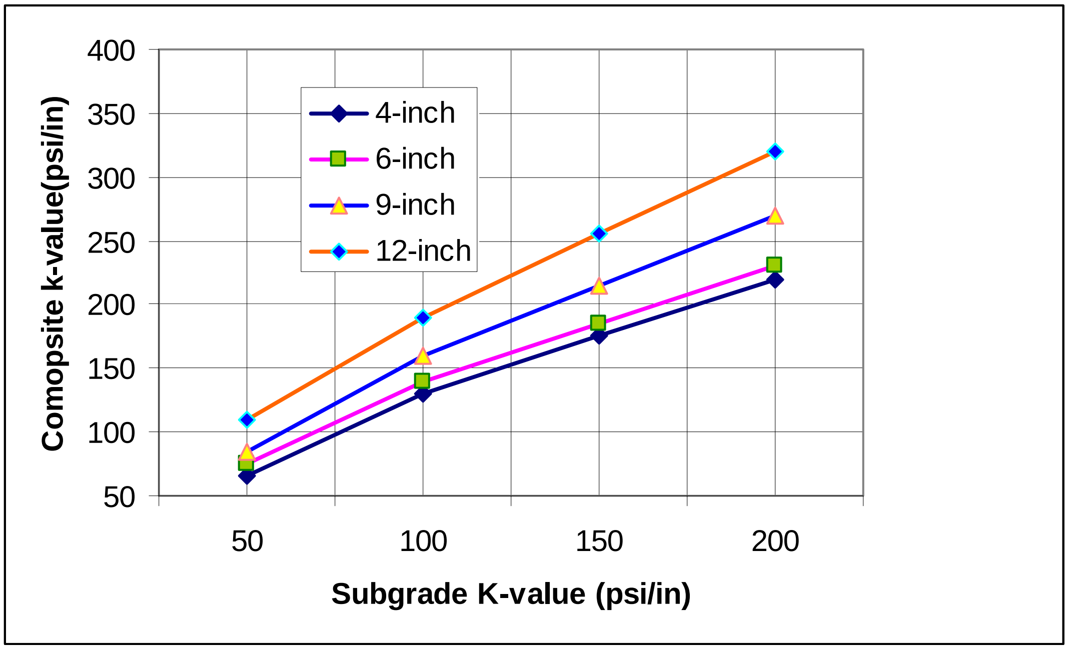

Figure 4.1 Adjusted K-values Based on Presence of Untreated Granular Base Thickness Under JPCP

4.2.7 Reliability

In an earlier study, Aunet [13] proposed the reliability values in Table 4.7 for use on Wisconsin's Highways based on highway functional classification and design AADT. The proposed reliability values developed 15 years ago were not implemented, and hence, were never evaluated. A review of current reliability values from FHWA Regions III and VI seems to suggest that modifications to the Wisconsin reliability values are needed. Table 4.8 shows proposed values to be used on Wisconsin's highways based on a comparison with states in the same geographic region (MI, OH, IN) as Wisconsin, and also based on the FHWA regions surveyed. The proposed values in Table 4.8 are not stratified by AADT compared to the one developed by Aunet [13]. Stratification can be done by assembling data on existing WisDOT pavements and backcalculating the reliability that are inherent in their original designs (see Chapter 6). The study by Aunet [13] did not recommend values for the overall standard deviation, but suggested appropriate values for local conditions must be determined. The proposed value range using the 1993 AASHTO guide is set to 0.30-0.40 for rigid pavements. A midrange value of 0.35 is to be considered for use in Wisconsin until values for local conditions are established.

Table 4.7 Reliability Values for Wisconsin's Highways (Adopted from [13])

Highway Type (1)	Functional Class (2)	Design AADT (3)	Reliability, % (4)
RURAL	Interstate	Less than 12,000	80
		12,000-20,000	85
		20,000-30,000	90
		Greater than 30,000	95
	Other Principal Arterial	Less than 3,000	70
		3,000-6,000	75
		6,000-12,000	80
		Greater 12,000	85
	Minor Arterial	Less than 1,000	65
		1,000-3,000	70
		Greater 3,000	75
	Major Collector	--	60
	Minor Collector		55
	Local		50
URBAN	Interstate	Less than 50,000	85
		50,000-100,000	90
		100,000-150,000	95
		Greater than 150,000	98
	Other Freeway and Expressway	Less than 25,000	80
		25,000-50,000	85
		50,000-100,000	90
		Greater than 100,000	95
	Other Principal Arterial	Less than 10,000	75
		10,000-25,000	80
		Greater than 25,000	85
	Minor Arterial	--	70
	Collector		60
	Local		50

Table 4.8 Proposed Reliability Values for Wisconsin's Highways

Highway Type (1)	Functional Class (2)	Reliability Values (%)			
		Michigan	Ohio	Indiana	Wisconsin*
RURAL	Interstate	95	90	98	90-95
	Principal Arterial	95	85	95	85-95
	Minor Arterial	95	85	90	85-95
	Major Collector	95	85	90	85-95
	Local	95	85	90	85-90
URBAN	Interstate	95	95	98	95-98
	Principal Arterial	95	90	95	90-95
	Minor Arterial	95	90	90	90-95
	Major Collector	95	90	90	90-95
	Local	95	90	90	90
* Proposed reliability values based on typical values from states in the same geographic region as Wisconsin.					

CHAPTER 5 OVERLAY DESIGN FOR EXISTING JPCP

Overlays are rehabilitation methods for correcting structural and functional deficiencies of existing pavements. Structural deficiencies affect the load-carrying capacity of the existing pavement, while functional deficiencies relate to ride quality perceived by highway users. The overlay design methods for the structural deficiency of JPCP, as outlined in the 1993 AASHTO guide, are summarized in the following sections with suggested design parameter values for WisDOT overlay design consideration.

5.1 General Considerations for Overlay Design

Design of structural overlays requires consideration of several items, including overlay type feasibility, pre-overlay repair requirements, traffic loading estimates, existing pavement evaluation, overlay thickness determination, and life cycle cost evaluation.

5.1.1 Overlay Type Feasibility Evaluation

The feasibility of any overlay type is dependent on several major considerations, including:

a) Availability of adequate funding to construct the overlay;

b) Construction feasibility of the overlay as related to traffic control, traffic disruptions and user delay costs, material and equipment availability, climatic conditions, construction problems (e.g. noise, subsurface utilities, overhead bridge clearance, and shoulder thickness); and

c) Required future design life of the overlay, which generally depends on factors such as the existing pavement and subgrade conditions, future traffic loading, subdrainage conditions, and local climate.

5.1.2 Pre-Overlay Repair

The type and quantity of pre-overlay repair depends on the type of distresses observed as well as their extent and severity. Distresses in the existing pavement with the potential to affect the performance of the overlay in the short term ought to be repaired prior to the placement of the overlay. In addition, cost tradeoffs of pre-overlay repair and overlay type should be considered. For example, selecting an overlay type that is less sensitive to severely deteriorated pavement condition may be more cost-effective than doing extensive pre-overlay repair.

5.1.3 Traffic Loading Estimates

Traffic loading estimates are based on procedures described under Section 2.2.1, except that the load equivalency category to use in the determination of ESALs is based on the type of overlay selected. Table 5.1 shows the feasible overlay types and recommended load equivalency categories for JPCP.

Table 5.1 Feasible Structural Overlays and Load Equivalency Factor Categories for JPCP (Adapted from [6])

Existing JPCP Type (1)	Overlay Type (2)	Load Equivalency or ESAL Factor Category (3)
Fractured JPCP (rubblized, crack-and-seat)	AC	Flexible
Non-Fractured JPCP	AC	Rigid
Non-Fractured JPCP	PCC	Rigid

5.1.4 Existing JPC Pavement Evaluation

The main objective of evaluating the existing pavement is to determine its effective structural capacity in order to determine additional capacity to handle the estimated future traffic. Three main procedures are recommended in the 1993 AASHTO guide for structural capacity evaluation and include visual survey and material testing, nondestructive deflection testing (NDT), and remaining life or fatigue damage from traffic. It is further indicated that because of uncertainties associated with the determination of structural capacity, the three methods cannot be expected to provide equivalent values. Hence, it is recommended to use all three methods whenever possible and select the "best" estimate based on judgment [6].

The *Visual Survey Method* involves the assessment of the existing pavement condition based on key distress types and their severity and extent. Particular attention is given to distresses indicating structural deficiency. For JPCP, such key distresses include, deteriorating (spalling or faulting) transverse or longitudinal cracks, corner breaks at transverse joints or cracks, localized failing areas showing disintegration of the JPCP with spalls and potholes. In addition to the visual survey, coring and testing may be necessary to verify or identify the causes of the observed distresses.

The *NDT Method* is used to estimate the in situ effective modulus of subgrade reaction, the elastic modulus of the pavement slab, and examine load transfer efficiency at the joints and cracks.

The *Remaining Life Method* is based on a fatigue damage concept. This concept indicates that repeated loads gradually damage the pavement and reduce the number of additional loads the pavement can carry to failure. Knowledge of the actual amount of traffic the pavement has carried to date as well as the total amount of traffic the pavement could be expected to carry to "failure" are required to estimate the remaining life of the existing JPCP.

5.1.5 Overlay Thickness Determination

The feasible types of overlay for an existing JPCP include: a) asphalt concrete (AC) overlay on an existing non-fractured JPCP, b) AC overlay on an existing fractured JPCP, c) bonded JPC overlay of the existing JPCP, and d) unbonded JPC overlay of the existing pavement. The thickness design for each of these categories of overlays is presented in the following sections.

5.1.5.1 AC Overlay of an Existing Non-fractured JPCP

The basic equation in the 1993 AASHTO Guide is:

$$D_{OL} = A*(D_f - D_{eff}) \quad (5.1)$$

$$A = 2.2233 + 0.0099*(D_f - D_{eff})^2 - 0.1534*(D_f - D_{eff}) \quad (5.2)$$

Where,
D_{OL} = Required thickness of AC overlay, inches
A = Factor to convert JPCP thickness deficiency to AC overlay thickness
D_f = Slab thickness to carry future traffic
D_{eff} = Effective thickness of existing JPCP slab

The 1993 AASHTO Guide procedure is summarized as follows:

i). Determine the slab thickness to carry the future traffic (D_f). This is done as it would be in the normal AASHTO design process for a new pavement described in chapter 4. Material properties used in the determination of this thickness pertain to those of the overlay material and not the existing JPCP.

ii) Determine the effective thickness of the existing JPCP slab (D_{eff}) by one of the following two methods:

5.1.5.1.1 Visual/condition survey

Equation 5.3 is used to adjust the actual existing slab thickness based on condition survey results.

$$D_{eff} = F_{jc} * F_{dur} * F_{fat} * D \quad (5.3)$$

Where,
D_{eff} = Effective thickness of existing JPCP slab
D = existing JPCP slab thickness, inches
F_{jc} = Joints and cracks adjustment factor, which accounts for the extra loss in serviceability caused by deteriorated reflection cracks in the overlay that will result from any unrepaired distresses in the existing JPCP slab. If all deteriorated areas are repaired prior to overlay, Fjc = 1.00, otherwise Figure 5.1 is used.
F_{dur} = durability adjustment factor accounting for the extra loss in serviceability caused by any durability problems (e.g. "D" cracking) in the existing JPCP. Table 5.2 shows AASHTO recommended values of F_{dur} based on JPCP condition.
F_{fat} = fatigue damage adjustment factor accounting for past fatigue damage in the existing JPCP slab. AASHTO recommended values are shown in Table 5.3.

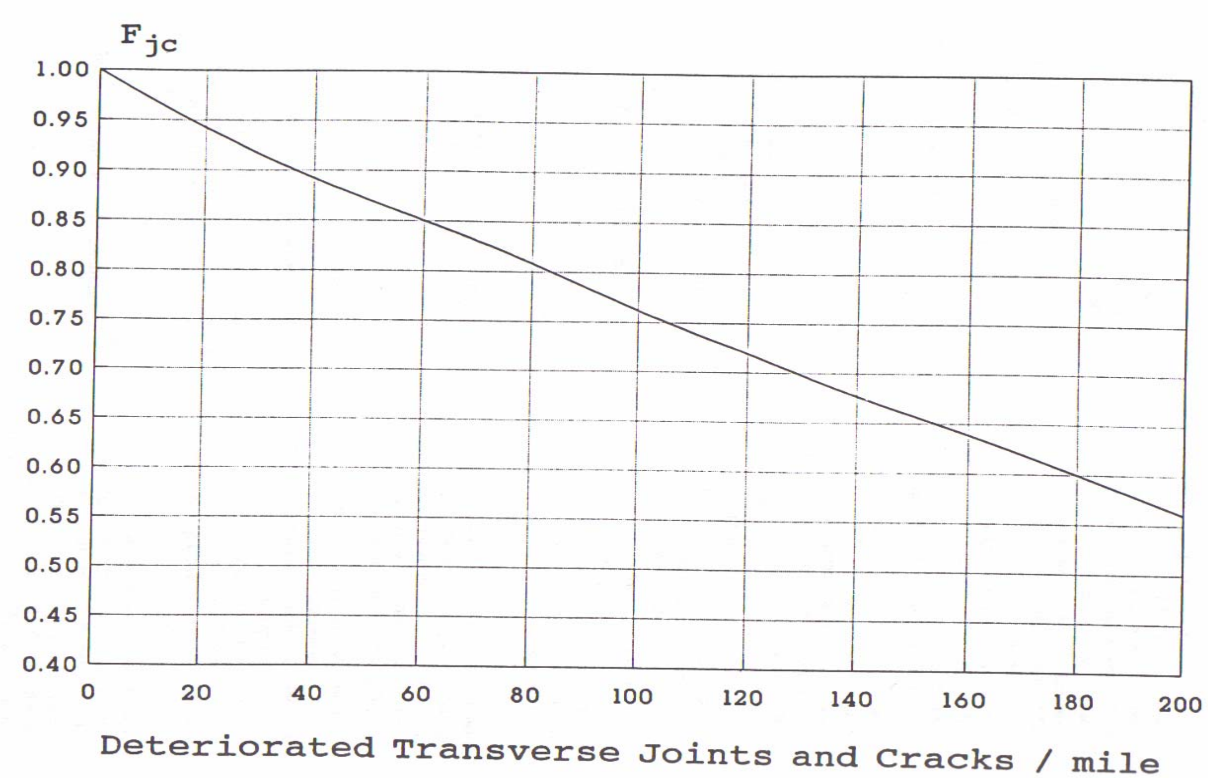

Figure 5.1 Fjc Adjustment Factor (Reproduced from [6])

Table 5.2 F_{dur} Adjustment Factors [6]

Existing JPCP Condition (1)	F_{dur} Factor (2)
No sign of JPCP durability problems	1.00
Durability cracking exists, but no spalling	0.96-0.99
Substantial cracking and some spalling exists	0.88-0.95
Extensive cracking and severe spalling exists	0.80-0.88

Table 5.3 F_{fat} Adjustment Factors [6]

Existing JPCP Condition (1)	F_{fat} Factor (2)
Few transverse cracks exist (none caused by “D” cracking): *< 5% slabs are cracked*	0.97-1.00
A significant number of transverse cracks (none caused by “D” cracking): *5-15% slabs are cracked.*	0.94-0.96
A large number of transverse cracks (none caused by “D” cracking): *>15% slabs are cracked*	0.90-0.93

The value of D_{eff} determined in Equation 5.3, is substituted in Equation 5.1 for the overlay thickness.

5.1.5.1.2 Remaining Life Approach

The remaining life of the existing pavement is estimated using Equation 5.4.

$$RL = 100*\{1-(N_P/N_{1.5})\} \quad (5.4)$$

Where,
RL = Remaining life of existing JPCP, as percent of total life
N_P =Total ESALs to date
$N_{1.5}$= Total ESALs to "failure" which is assumed to occur at PSI = 1.5

Using the RL value, the D_{eff} is determined by Equation 5.5.

$$D_{eff} = CF *D \quad (5.5)$$

Where,
CF = Condition factor determined from Figure 5.2, and

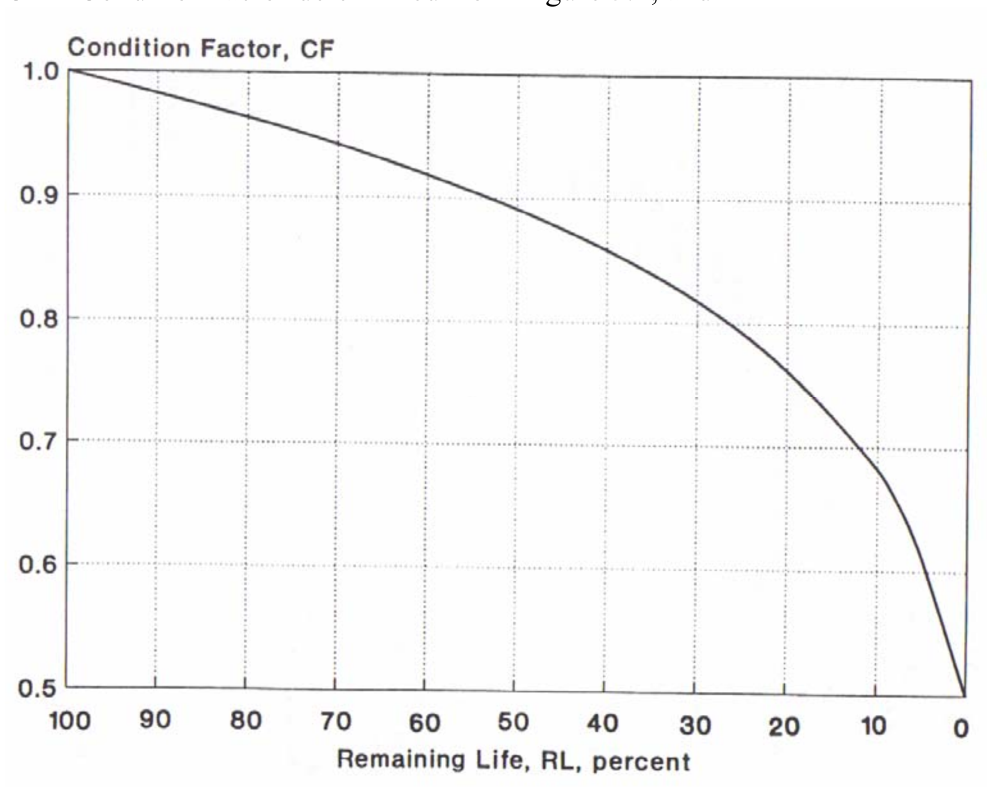

Figure 5.2 Relationship Between Condition Factor and Remaining Life [6]

5.1.5.2 AC Overlay of an Existing Fractured JPCP

Fracture methods of JPCP include rubblization and crack-and-seat. Both methods involve breaking up the existing distressed JPCP into smaller size pieces and overlaying the resulting surface with a flexible pavement. Crack-and-seat particle sizes range from 12 to 36 inches per side, while rubblized pieces are less than 12 inches per side.

The basic equation in the 1993 AASHTO Guide is:

$$D_{OL} = \frac{SN_{OL}}{a_{OL}} . = \left(\frac{SN_f - SN_{eff}}{a_{OL}} \right) \quad (5.6)$$

Where,
SN_{OL} = Required overlay structural number
a_{OL}= Structural coefficient for AC overlay
D_{OL} = Required AC overlay thickness, inches
SN_f = Structural number to carry the estimated future traffic
SN_{eff} = Effective structural number of the existing fractured pavement

The structural number to carry the estimated future traffic, SN_f, is determined based on the 1993 AASHTO design equation represented by Equation 5.7.

$$Log_{10} W_{18} = Z_R * S_O + 9.36 * \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 * \log_{10} .M_R - 8.07 \quad (5.7)$$

Where:
W_{18} = predicted number of 18-kip ESALs
Z_R = standard normal deviate
S_o = combined standard error of the traffic prediction and performance prediction
SN = structural number to carry estimated ESALs
ΔPSI = difference between the initial design serviceability index, Pi, and the design terminal serviceability index, Pt
M_R = subgrade resilient modulus, psi

The range of S_o specified by AASHTO for flexible pavements is 0.40-0.50, while the recommended values for reliability for various functional classes are as previously shown in Table 4.1.

The effective structural number of the existing fractured JPCP, SN_{eff}, can be estimated by a component analysis based on the structural number equation represented by Equation 5.8:

$$SN = a_2 D_2 m_2 + a_3 D_3 m_3 \quad (5.8)$$

Where,
a_2, a_3 = corresponding structural layer coefficients

m_2, m_3 = drainage coefficients for fractured JPCP and granular base under the fractured JPCP

AASHTO recommended values for structural layer coefficients are shown in Table 5.4 for fractured JPCP and base or subbase layers, while the drainage layer coefficients for the bases and subbases are presented in Table 5.5.

Table 5.4 Suggested Layer Coefficients for Fractured JPCP [6]

Material Type (1)	Condition (2)	Coefficient (3)
Crack-and-seat JPCP	Fractured pieces size range: 12 in- 36in.	0.20 to 0.35
Rubblized JPCP	Completely fractured slab pieces less than 12 in.	0.14 to 0.30
Granular base/subbase	No evidence of degradation or intrusion of fines	0.10 to 0.14
Granular base/subbase	Some evidence of degradation or intrusion of fines	0.00 to 0.10

Table 5.5 Recommended Values for Modifying Structural Layer Coefficients of Untreated Base and Subbase Materials in Flexible Pavements AASHTO 1993 [6]

Drainage Quality	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
(1)	Less than 1% (2)	1-5% (3)	5-25% (4)	Greater than 25% (5)
Excellent	1.40-1.35	1.35-1.30	1.30-1.20	1.20
Good	1.35-1.25	1.25-1.15	1.15-1.00	1.00
Fair	1.25-1.15	1.15-1.05	1.00-0.80	0.80
Poor	1.15-1.05	1.05-0.80	0.80-0.60	0.60
Very Poor	1.05-0.95	0.95-0.75	0.75-0.40	0.40

5.1.5.3 JPCP Overlay of an Existing Non-fractured JPCP

There are two major types of JPCP overlays, namely, bonded and unbonded overlays.

5.1.5.3.1 Unbonded JPCP Overlays

An unbonded JPCP overlay consists of a new JPCP layer over an existing non-fractured JPCP layer. Bonding between the existing pavement and overlay is prevented by using an AC bond breaking interlayer, which helps prevent distresses in the existing pavement from reflecting through into the overlay. They can be applied over badly deteriorated pavements without much surface preparation.

The basic equation in the 1993 AASHTO Guide is presented as Equation 5.9:

$$D_{OL} = \sqrt{D^2{}_f - D_{eff}{}^2} \quad \text{……}(5.9)$$

Where,

D_{OL} = Required thickness of unbonded JPCP overlay, inches
D_f = Slab thickness to carry future traffic, inches
D_{eff} = Effective thickness of existing slab, inches

The D_{eff} can be estimated using the remaining life approach described in section 5.1.5.1 or the visual/condition survey method. The latter method is based on the number of deteriorated transverse joints and cracks per mile. The D_{eff} based on the visual/condition survey results is presented as Equation 5.10.

$$D_{eff} = F_{jcu} * D \qquad (5.10)$$

Where,
D = existing JPCP thickness, inches (maximum D for use in unbonded overlay design is 10 inches)
F_{jcu} = joints and cracks adjustment factor accounting for the extra loss in serviceability caused by deteriorated transverse joints and cracks in the existing pavement. This adjustment factor is determined using Figure 5.3.

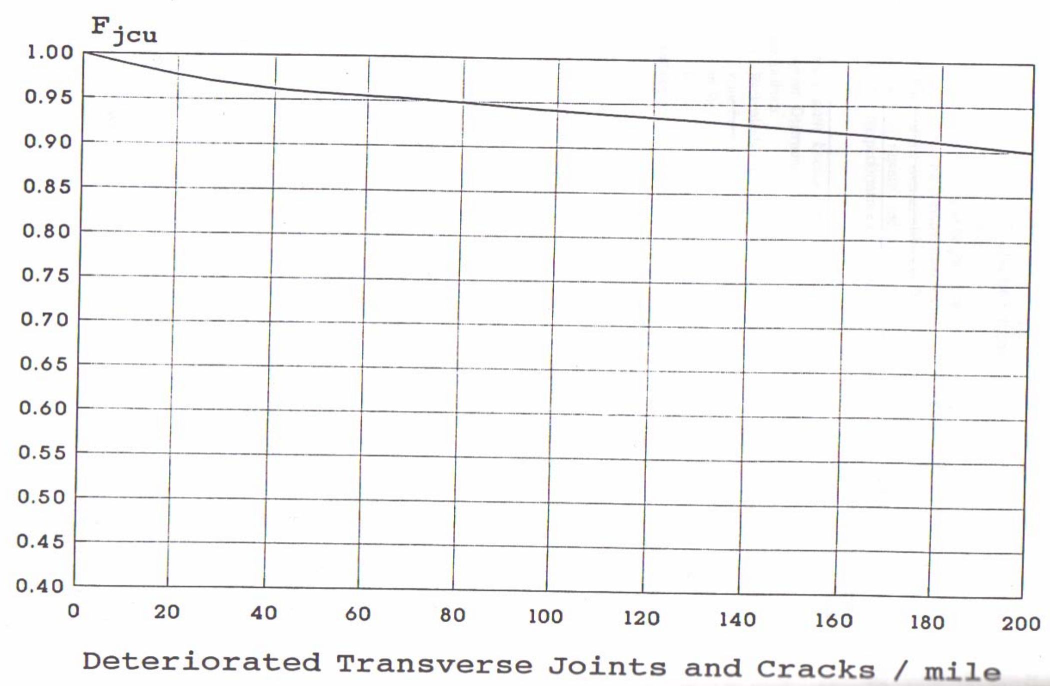

Figure 5.3 F_{jcu} Adjustment Factor for Unbonded JPCP [6]

5.1.5.3.2 Bonded JPCP Overlays

A bonded JPCP overlay consists of a new JPCP layer over an existing JPCP. The overlay is bonded to the existing pavement by means of a PCC slurry or grout, resulting in a composite pavement section. Its use is restricted to existing rigid pavements that have little deterioration. The overlay thickness is determined using Equation 5.11.

$$D_{OL} = D_f - D_{eff} \quad \text{...} (5.11)$$

Where,

D_{OL} = Required thickness of bonded JPCP overlay, inches

D_f = Slab thickness to carry future traffic, inches

D_{eff} = Effective thickness of existing slab, inches

$$= F_{jc} * F_{dur} * F_{fat} * D \quad \text{...} (5.12)$$

D = existing JPCP slab thickness, inches

F_{jc} = Joints and cracks adjustment factor, which accounts for the extra loss in serviceability caused by deteriorated reflection cracks in the overlay that will result from any unrepaired distresses in the existing JPCP slab. If all deteriorated areas are repaired prior to overlay, Fjc = 1.00, otherwise Figure 5.1 is used.

F_{dur} = durability adjustment factor accounting for the extra loss in serviceability caused by any durability problems (e.g. "D" cracking) in the existing JPCP. (Table 5.6 shows AASHTO recommended values of F_{dur} based on JPCP condition).

F_{fat} = fatigue damage adjustment factor accounting for past fatigue damage in the existing JPCP slab. (AASHTO recommended values are shown in Table 5.7.)

Table 5.6 F_{dur} Adjustment Factors [6]

Existing JPCP Condition (1)	F_{dur} Factor (2)
No sign of JPCP durability problems	1.00
Durability cracking exists, but no spalling	0.96-0.99

Table 5.7 F_{fat} Adjustment Factors [6]

Existing JPCP Condition (1)	F_{fat} Factor (2)
Few transverse cracks exist (none caused by "D" cracking): *< 5% slabs are cracked*	0.97-1.00
A significant number of transverse cracks (none caused by "D" cracking): *5-15% slabs are cracked.*	0.94-0.96
A large number of transverse cracks (none caused by "D" cracking): *>15% slabs are cracked*	0.90-0.93

5.2 Suggested JPCP Overlay Design Inputs for use in Wisconsin

There cannot be a fixed set of design inputs for use in the 1993 AASHTO Guide that can be applied to WisDOT JPCP overlay designs; however, some guidance is offered as a starting point in the design process. Further knowledge about these inputs will improve and undoubtedly change over time. Table 5.8 shows suggested design inputs for consideration in the use of the 1993 AASHTO Guide for the design of JPCP structural overlays in Wisconsin.

The suggested structural layer coefficients for new AC (a_{OL}=0.44) and the rubblized JPCP (a_2 = 0.20-0.24) are based on existing WisDOT design standards. For the granular layer underneath the fractured slab, the suggested layer coefficient, (a_3) corresponds to midrange values of those proposed by AASHTO (see Table 5.4). The mid-range values are to be considered for use in Wisconsin until values for local conditions are established. For drainage layer coefficients (m_2 and m_3), the use of edge drains are expected to significantly improve the quality of drainage and reduce the percent time the pavement structure will be exposed to moisture levels approaching saturation. A value of 1.1 is recommended based on good quality drainage and 5-25% exposure time of pavement to moisture levels approaching saturation. The value of 1.0 is used for all other conditions. This value corresponds to current WisDOT flexible pavement design, as well as represents drainage conditions at the AASHO road test.

The visual/condition survey method for estimating the effective thickness of the JPCP slab is recommended based on ease of obtaining field data for design. The approach is distress dependent; WisDOT personnel routinely measure distresses, and distress characteristics are very well understood. This method therefore, requires WisDOT to examine individual distresses as opposed to the combination of distresses as denoted by the pavement distress index (PDI). The other two approaches, namely the remaining life and NDT can be quite sensitive to unknown conditions and thus require knowledgeable and experienced personnel. For example, the remaining life approach requires knowledge of the total traffic to-date for the existing pavement. Since traffic data are not gathered routinely for most roadway segments, estimates could vary significantly than actual. This could result in a wrong estimate of the effective thickness of the existing slab.

For the visual/condition survey method, the key distresses to be observed include deteriorating (spalling or faulting) transverse or longitudinal cracks, corner breaks at transverse joints or cracks, localized failing areas showing disintegration of the JPCP with spalls and potholes.

Table 5.8 Suggested Input Guide for JPCP Overlay Design in Wisconsin.

Feasible Overlay Type (1)	Overlay Thickness Design Equations (2)	ESAL factors for ESAL Estimation. (3)	Structural Layer Coefficients, ai (4)	Drainage layer coefficients, m_i (5)
AC over rubblized JPCP	$$D_{OL} = \frac{SN_f - (a_2 D_2 m_2 + a_3 D_3 m_3)}{a_{OL}}$$	ESAL factors for flexible pavement required, but not developed as part of this study. Use proposed rigid ESAL factors (Table 4.5) to estimate ESALs and use 67% of the resulting value [6].	a. *New AC layer* $a_1 = a_{OL} = 0.44$ b. *Rubblized JPCP*: $a_2 = 0.20\text{-}0.24$ c. *Granular base or subbase under rubblized layer* $a_3 = 0.12$, if no evidence of degradation or intrusion of fines $a_3 = 0.05$, if there is evidence of intrusion of fines	$m_2 = m_3 = 1.10$ if edge drains are installed and there is no evidence of degradation or intrusion of fines; otherwise use $m_2 = m_3 = 1.00$
AC over non-fractured JPCP	$D_{OL} = A*(D_f - D_{eff})$ $D_{eff} = F_{jc} * F_{dur} * F_{fat} * D$ $A = 2.2233 + 0.0099*(D_f - D_{eff})^2 - 0.1534*(D_f - D_{eff})$ Fjc (see Figure 5.1) F_{dur} (see Table 5.2) F_{fat} (see Table 5.3)	Use proposed rigid ESAL factors (Table 4.5)	Not Applicable	Drainage coefficient, C_d, needed for D_f estimate. Use, $C_d = 1.2$ for sections with OGBC; otherwise use $C_d = 1.0$
Bonded JPCP over non-fractured JPCP	$D_{OL} = D_f - D_{eff}$ $D_{eff} = F_{jc} * F_{dur} * F_{fat} * D$ Fjc (see Figure 5.1) F_{dur} (see Table 5.6) F_{fat} (see Table 5.7)	Use proposed rigid ESAL factors (Table 4.5)	Not Applicable	Drainage coefficient, C_d, needed for D_f estimate. Use, $C_d = 1.2$ for sections with OGBC; otherwise use $C_d = 1.0$
Unbonded JPCP over non-fractured JPCP	$D_{OL} = \sqrt{D^2{}_f - D_{eff}{}^2}$ $D_{eff} = F_{jcu} * D$ F_{jcu} (see Figure 5.3)	Use proposed rigid ESAL factors (Table 4.5)	Not Applicable	Drainage coefficient, C_d, needed for D_f estimate. Use, $C_d = 1.2$ for sections with OGBC; otherwise use $C_d = 1.0$
D_{OL}= overlay thickness; D_f = JPCP thickness to carry future traffic, D_{eff}= effective thickness of existing slab, D= existing slab thickness.				

CHAPTER 6 APPLICATION OF THE 1993 AASHTO GUIDE TO WISCONSIN JPCP DESIGN

Chapters 4 and 5 outlined design inputs for using the 1993 AASHTO Guide in the design of new JPCPs and rehabilitation of old JPCPs in Wisconsin, with particular attention to heavy loading corridors. In this chapter, the AASHTO method is demonstrated using data from one logging truck corridor along USH 8. A previous investigation of the performance of pavement segments along USH 8 and USH 51 indicated that overloading was a primary cause for the premature failure of the pavements [1]. The USH 8 segment has undergone a new design phase because of the premature failure. Hence, it was appropriate to use the data to demonstrate the 1993 AASHTO guide design process.

6.1 Reliability Inherent in Current WisDOT Design for New JPCP

Reliability is the key variable used in the 1993 AASHTO guide for addressing uncertainties such as overloading in the thickness design process. It was therefore, necessary to at least, investigate the current reliability inherent in the WisDOT pavement design process. This was done using the original design data from the existing USH 8 and the USH 51 pavements. The project attributes obtained from as-built plans are shown in Table 6.1 and 6.2. Detail traffic data during the early life of the pavements are also presented in Tables 6.3 and 6.4. Tables 6.3 and 6.4 provide respective comparisons of USH 8 and USH 51 data. Generally, there was an increase in truck traffic for Classes 5 through 8, and a decrease for Classes 9 through 13. Email correspondence with District 7 pavement engineers suggested that most logging trucks were a 3-axle configuration (Class 6); this particular class had increases of 131.4% and 92.7% for USH 8 and 51, respectively. These data, along with those in Table 2.5, highlight the need to provide ESAL factors for all FHWA truck classes, in particular Classes 7, 10, 12, and 13. An estimate of ESALs across all classes will yield a more accurate estimate of truck loading throughout the design life of the pavement. In addition, logging truck configuration for classes 6, 7, 9, and 10 were observed in March 2005 on the failed pavement segments.

This information was used in conjunction with the AASHTO 1972 Portland Cement Concrete design equation (currently used by WisDOT in the WisPave program) to backcalculate the estimated loading to reach a terminal serviceability, P_t of 2.5 set forth by WisDOT. The estimated load was then used in the WinPAS software to determine the reliability inherent in the current WisDOT design process. WinPAS is a pavement analysis software developed based on the 1993 AASHTO design procedure [21].

Table 6.1 Project Attributes for JPCP Projects in District 7 from Plan Cover Sheet

Attribute (1)	USH 8 (2)	USH 51 (3)	
Limits	STH 47 to USH 8 (Bus.)	USH 8 to CTH K	
Counties	Oneida	Lincoln and Oneida	
Project I.D.	1593-01-74	1173-04-74	
Centerline length, miles	6.472	7.741	
Year Constructed	1991	1992	
A.D.T. 1990	4400	South of CTH L = 3675	North of CTH L = 5165
A.D.T. 2010	6500	South of CTH L = 4815	North of CTH L = 6785
D.H.V. 2010	845	1300	
D.	60/40	90/10	
T.	9.5%	13%	
V.	65 mph	65 mph	

Table 6.2 Structural Design Data for USH 8 and 51

Attribute (1)	USH 8 (2)	USH 51 (3)
Soil Support Value	5.4	5.4
Subgrade Modulus, K (pci)	275	300
Transverse Joints	Random spacing; Skewed, Unsealed.	Random spacing; Skewed, Unsealed.
Pavement Base	CABC 6-inch thick	CABC 6-inch thick
Pavement	PCC 9-inch thick	PCC 9-inch thick
Dowels	1-1/4” diameter, 18-long, 12-in O.C., epoxy coated.	1-1/4” diameter, 18-long, 12-in O.C., epoxy coated.
Centerline parting strips	Yes	Yes
Lane width	15 feet	15 feet
Direction Factor	---	0.5
Lane Distribution Factor	---	1.0
Growth Rate	---	3%

Table 6.3 USH 8 Comparison of Truck Traffic Data during Early Design Life

Truck Type from Pavement Type Selection Report (1)	Truck Category from FDM 14-1-5 (2)	FHWA Class Conversion (3)	1990 ADT, % (4)	2001 ADT, % (5)	2003 ADT, % (40-day sample) (6)	1990 to 2003 Arithmetic Change, % (7)	1990 to 2003 Percent Change, % (8)
2D	2 axles, 6 tires	4 and 5	4.4	2.5	0.61+2.75 = 3.36	-1.04	-23.6
3ax	3 axles	6	0.7	1.4	1.62	+0.92	+131.4
2-S1	Single trailer, 3 axles	8	0.3	---	---	---	---
2-S2	Single trailer, 4 axles	8	0.5	1.6*	2.72*	+1.92	+240.0
3-S2	Single trailer, 5 axles and above	9 and 10	3.6	4.7	3.38+0.16 = 3.54	-0.06	-1.7
Double Bottom	Two Trailers	11, 12, and 13	---	0.3	---	---	---
		Total =	9.5	10.5	11.24	+1.74	+18.3
* 2-S1 and 2-S2 data combined							

Table 6.4 USH 51 Comparison of Truck Traffic Data during Early Design Life

Truck Type from Pavement Type Selection Report (1)	Truck Category from FDM 14-1-5 (2)	FHWA Class Conversion (3)	1990 ADT, % (4)	2003 ADT (2-day sample), % (5)	2004 ADT , % (6)	1990 to 2003 Arithmetic Change, % (7)	1990 to 2003 Percent Change, % (8)
2D	2 axles, 6 tires	4 and 5	2.5	0.38 + 2.16 = 2.54	2.5	+0.04	+1.6
3SU	3 axles	6	1.5	2.89	3.3	+1.39	+92.7
2-S1 & 2-S2	Single trailer, 3 axles	8	0.5	3.22	3.2	+2.72	+544.0
3-S2	Single trailer, 4 axles	9 and 10	8.0	2.16 + 0.41 = 2.57	2.6	-5.43	-67.9
Double Bottom	Double trailer	11, 12, and 13	0.5	0.02 + 0.02 + 0.4 = 0.44	0.5	-0.06	-12.0
		Total =	13.0	11.66	12.1	-1.34	-10.3

6.1.1 Reliability Associated with USH 8 and USH 51 Pavements

It is worth pointing out that the actual traffic data shown in Tables 6.1, 6.3, and 6.4, generated thicknesses of 7.5 inches and 8.5 inches respectively for the USH 8 and 51 pavements based on WisDOT design procedure. This contradicted the as-built cross-section of 9 inches listed in Table 6.2. Further investigations revealed that the 9-inch thicknesses were constructed based on addenda to the original pavement design reports. It was therefore, necessary to determine the actual loading corresponding to the 9-inch thicknesses for both pavements. Other design inputs used for the USH 8 segment include the original subgrade modulus value of 275psi/in and the following WisDOT default values: concrete elastic modulus of 4,200,000 psi and concrete working stress of 490 psi. This resulted in an estimated loading of 906 ESALs/day or 6,613,800 ESALs for the 20-year design. The latter value was used in WinPAS to arrive at 92.95% reliability as shown in Figure 6.1 with the other inputs.

For the USH 51 project, the back-calculated loading was 938 ESALs/day or 6,847,400 ESALs for the 20-year design period. With the given subgrade modulus of 300 psi/in and the other WisDOT default inputs, a reliability of 92.96% shown in Figure 6.2 was obtained.

To confirm the estimated reliability values, a new set of data from field measurements and lab tests performed in October 2001 for USH 8 and 51 as reported by reference [1] were used. The key variables included the modulus of rupture and the elastic modulus. The average moduli of rupture based on flexural strength tests on sawn beams were reported to be 530 psi and 710 psi respectively for USH 8 and 51. The reported elastic modulus was 5,200,000 psi for both highways. The analyses were repeated using these key input variables. The resulting reliability value was 92.88% for both USH 8 and 51 as shown in Figures 6.3 and 6.4. This value is not significantly different from the 92.95% reliability values based on the original design data. Thus, the reliability associated with the current WisDOT rigid pavement design appears to be stable at an approximate value of 93% based on the results of this evaluation.

If both pavements failed prematurely by loading as indicated in reference [1], then it would be assumed that the average loading to date on the existing pavements should be at least equal to the back-calculated values. Using the 1990 and 2003 traffic volume data and the truck distribution data (from Tables 6.3 and Table 6.4) for the two segments yielded 187 ESALs/day and 513 ESALs/day respectively for USH 8 and USH 151. These values are significantly less than the back-calculated values based on the thickness. If failure, however, occurred under these loading conditions, then it could have been probably caused by a specific axle load rather than the combined effect indicated by ESALs/day. This highlights a weakness underlying the use of AASHO Road Test load equivalency factors to estimate loading effects on pavement. This weakness is addressed in the AASHTO 2002 guide that considers axle load spectra rather than ESAL in the design process.

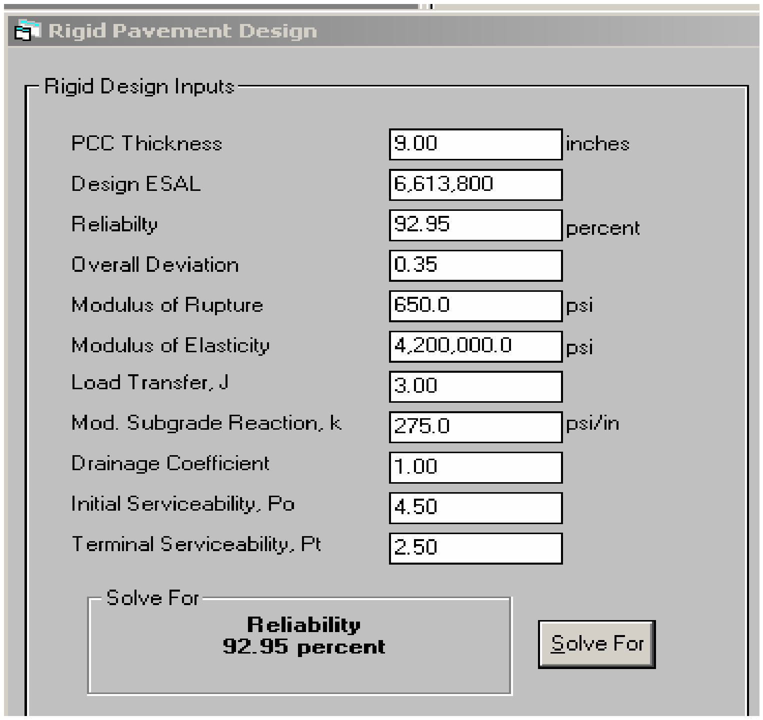

Figure 6.1 Reliability associated with USH 8 Original Design using WinPAS Software

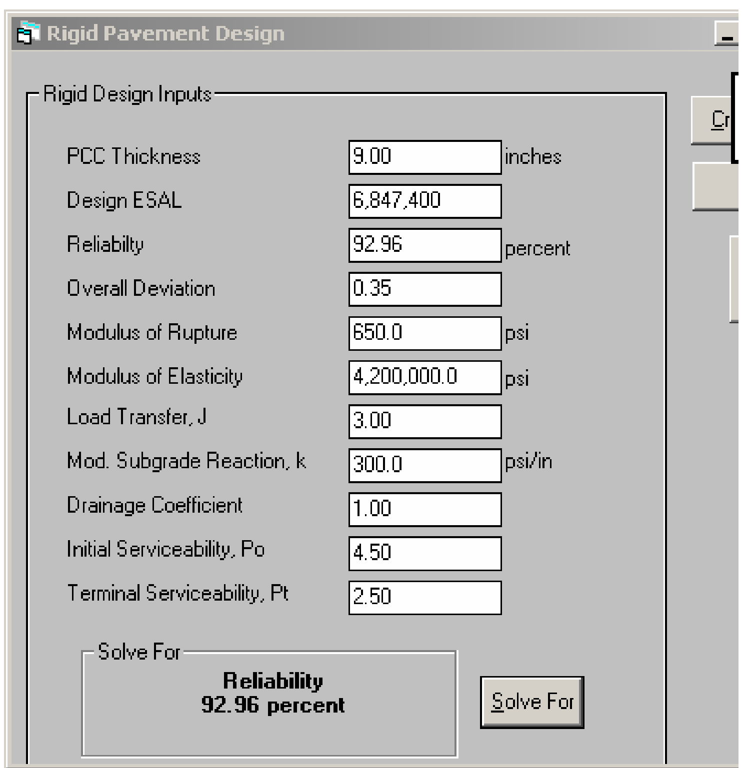

Figure 6.2 Reliability associated with USH 51 Original Design using WinPAS Software

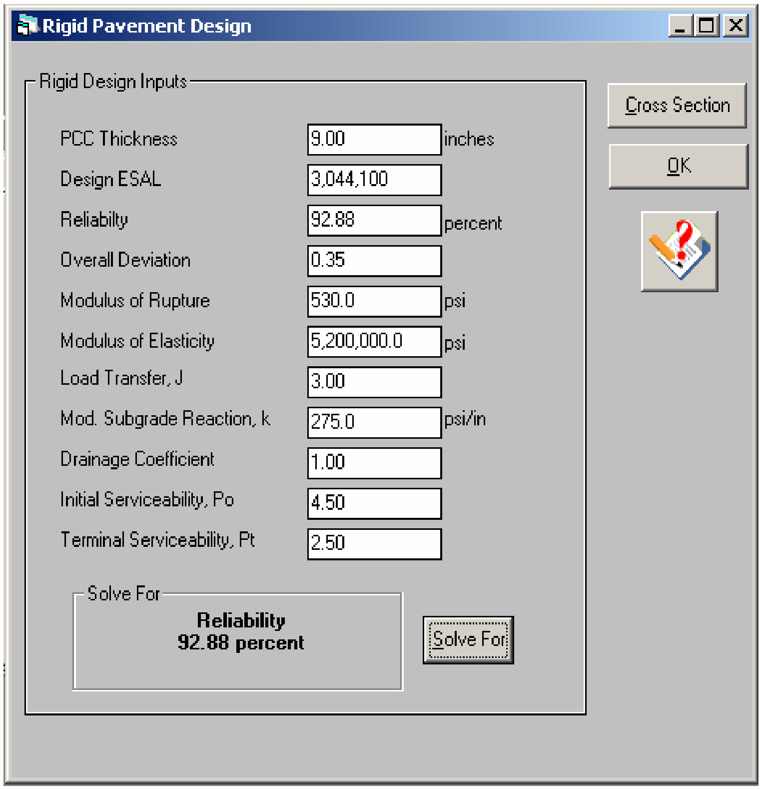

Figure 6.3 Reliability associated with USH 8 based on field data.

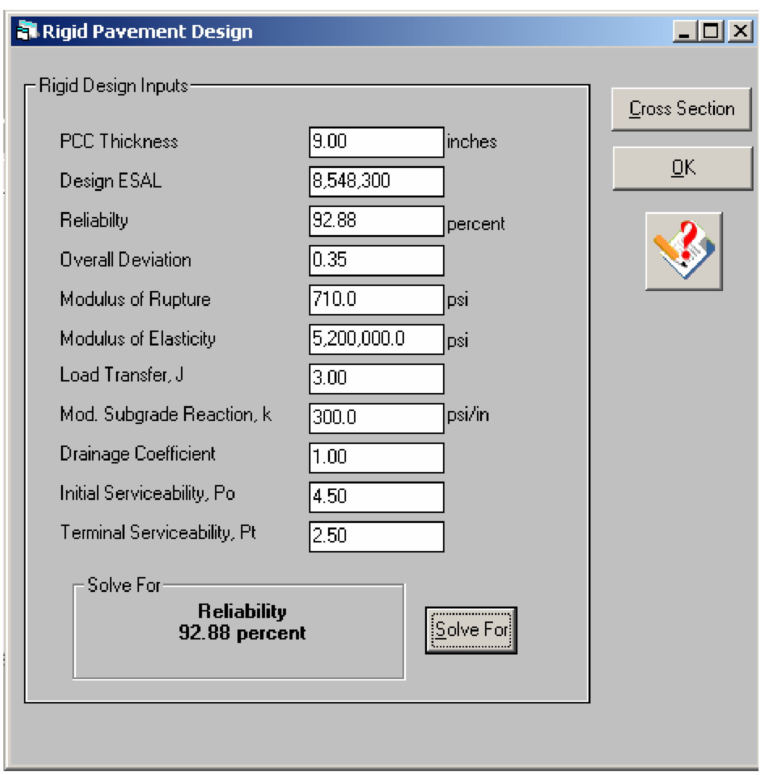

Figure 6.4 Reliability associated with USH 51 based on field data.

6.1.2 Increased Reliability Effects on Slab Thickness

If both pavements failed under the estimated reliability values shown in Figures 6.1 through 6.4, then higher values are needed where overloading is anticipated. The reliability values are identical for the two pavements; hence, the effect of reliability on slab thickness change is examined for only one of the failed pavements (USH 51) and shown in Figure 6.5. Figure 6.5 suggests that a 4.3% increase in reliability (i.e., 97% reliability) can result in a 5.7% increase in slab thickness. Thus, a 9-inch thickness will require an additional half-inch thickness to protect the pavement from premature failure due to the estimated loading. Thus, for overloading design, higher than normal recommended reliability values should be considered. Where a range is specified, the high end of the range must be used if overloading is anticipated.

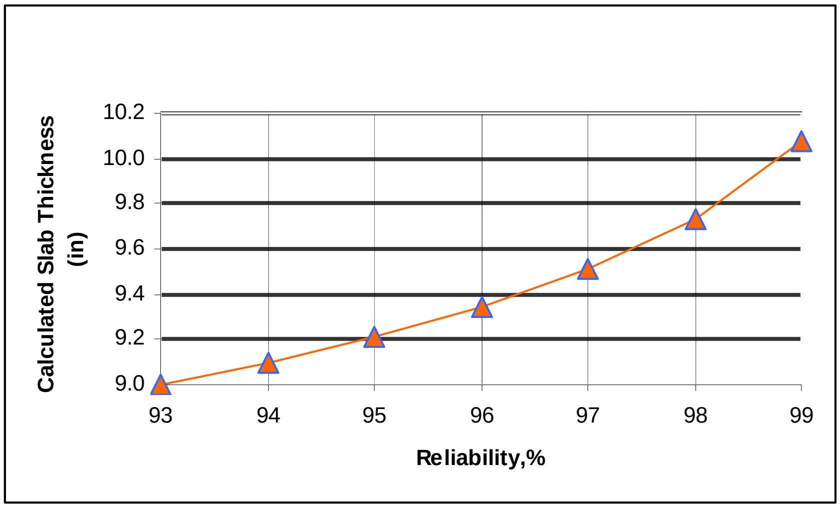

Figure 6.5 Reliability Effects on Slab Thickness

6.2 Design Options and Analysis for USH 8

Two main design options were considered for the failed USH 8 pavement segment. The options were:

a) Option 1: total reconstruction involving a new JPCP
b) Option 2: rubblizing the existing JPCP and overlaying with asphalt concrete

6.2.1 Option 1 Design Scenarios

This option was analyzed under four main scenarios including:

a) New JPCP design using WisDOT method with no modifications to ESAL factors.
b) New JPCP design using WisDOT method with proposed ESAL factors.
c) New JPCP design using AASHTO method with current WisDOT ESAL factors.
d) New JPCP design using AASHTO method with proposed ESAL factors.

These design options were analyzed based on the 2003 traffic counts for USH 8 shown in Table 6.5, and the WisDOT supplied design data shown in Table 6.6 for overlay of the existing USH 8 pavement. Based on the WisDOT 2000 and 2020 AADT values, a growth rate of 1.7% was estimated. Applying this value to the 2003 AADT value (i.e., 4485veh/day) shown in Table 6.5 resulted in a design year (2023) AADT of 6,279. All design analyses were based on these values and the traffic mix shown in Table 6.5.

Table 6.5 Average Percent of Traffic by Vehicle Classification 2003

Year	Days	AADT	FHWA CLASS								
			5	6	7	8	9	10	11	12	13
Site 430652 - USH 8 / STH 47 West of CTH G - Rhinelander											
2003	2	4485	2.76	2.00	0.27	2.31	4.59	0.38	0.01	0.01	0.01
2003	40	4485	2.75	1.62	0.12	2.72	3.38	0.16	0.00	0.00	0.01
Source: 2003 Wisconsin Vehicle Classification Data, Wisconsin Dept. of Transportation, Madison, Wisconsin, March 2004 [26].											

Table 6.6 New Design Data for USH 8

Attribute (1)	USH 8 (2)
Soil Support Value	5.3
Subgrade Modulus, K (pci)	275
2000 ADT	4,000
2020 ADT	5,600
% Trucks	10.5
Flexible Pavement Daily ESALs	180
Rigid Pavement Daily ESALs	278
Frost Index	F-2
Design Group Index	4

For the AASHTO method, the following design inputs were considered:

- Concrete elastic modulus: 4,200,000 psi
- Modulus of rupture: 650 psi
- Load transfer coefficient: 3.0
- Drainage coefficient, Cd: 1.0
- Subgrade effective k (adjusting original k-value of 275 psi/in for the presence of a 6-inch CABC): 310 psi/in
- Reliability: 95%

A summary of the design thickness values based on Design Option 1 scenarios is shown in Table 6.7. The calculated thickness values do not seem to indicate any significant difference between the current WisDOT method and the 1993 AASHTO based on the current WisDOT rigid ESAL factors (see Options 1A and 1C). This may be because a high reliability factor is inherent in the present WisDOT design as estimated for the two projects. Application of the modified ESAL factors in conjunction with the AASHTO method produced a 5.5% increase in thickness over the current WisDOT design (see Options 1A and 1D). In general, the AASHTO method will trigger a higher calculated thickness to prompt the designer to use a higher rounded thickness for construction.

Table 6.7 Calculated Thickness Based on Design Method and Selected ESAL Factors

Design Option (1)	Description (2)	Design ESALs (3)	Calculated Thickness (in.) (4)	Rounded Thickness for Construction (in.) (5)
1A	WisDOT design with current WisDOT ESAL factors	271 per day	7.3	7.5
1B	WisDOT design with modified ESAL factors	338 per day	7.5	7.5
1C	AASHTO design with current WisDOT ESAL factors	1,978,300	7.4	7.5
1D	AASHTO design with modified WisDOT ESAL factors	2,467,400	7.7	8.0

6.2.2 Design Option 2 for USH 8 Based on the 1993 AASHTO Method

Design Option 2 involved the rubblization of the existing JPCP and overlaying with asphalt concrete. This involved determining the structural number to carry the estimated future traffic, SN_f, as well as the effective structural number, SN_{eff}, of the existing rubblized JPCP. The structural number for the future traffic requires determining ESALs based on flexible ESAL factors. This study however, did not look at flexible ESAL factors. It therefore, uses 67% of the total rigid ESALs to represent the total ESALs for the design of the asphalt overlay. This is based on a correlation provided by AASHTO [6]. Thus, the total flexible ESALs based on the modified rigid ESAL factors, for example, is 0.67*338 = 227 ESALS/day (1,657,100 total ESALs over a 20-year period). The main inputs used to determine the structural number to carry the estimated traffic, SN_f, are as follows:

- 20-year design ESAL: 1,657,100 and 1,328, 600 respectively, for the modified and unmodified WisDOT rigid ESAL factors
- Design reliability: 95%
- Overall standard deviation: 0.45
- Subgrade resilient modulus: 7,047 psi.
- Initial and terminal serviceability values: 4.2 and 2.5 respectively.

In order to use the AASHTO design, a conversion between the WisDOT soil support value and the soil resilient modulus (M_R) was needed. Based on the nomograph originally developed by Van Til, et al. [22], the following simple correlation equation was developed:

$$\text{Log}_{10}(M_R) = 3 + 0.16*SSV \qquad (6.1)$$

The inputs used to determine the effective structural number, SN_{eff} of the existing JPCP were as follows:

- Structural layer coefficient of new asphalt concrete: 0.44
- Structural layer coefficient of rubblized JPCP, a_2: 0.22 (WisDOT specified range = 0.20-0.24)
- Structural layer coefficient of 6-inch CABC under rubblized layer, a_3: 0.10
- Drainage layer coefficients: $m_2 = m_3 = 1.0$

With the aid of the WinPAS software, the estimated overlay thicknesses are as summarized in Table 6.8.

Table 6.8 Asphalt Overlay Design of Rubblized JPCP

Flexible Design ESAL (1)	SN_f (2)	$SN_{eff} = a_2D_2m_2 + a_3D_3m_3$ (3)	$D_{OL} = (SN_f - SN_{eff})/a_{OL}$ (4)	Rounded Overlay Thickness for Construction (in.) (5)
1,657,100 (Based on modified ESAL factors)	4.10	0.22*9*1+0.10*6*1= 2.58	(4.10-2.58)/0.44 =3.46	3.5
1,328, 600 (unmodified ESAL factors)	3.96	0.22*9*1+0.10*6*1= 2.58	(3.96-2.58)/0.44 =3.14	3.25

The calculated overlay thickness using the loading based on the modified ESAL factors is approximately 10% more than the thickness based on the unmodified ESAL factors.

CHAPTER 7 CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

This research study reviewed design methods for jointed plain concrete pavement (JPCP) with the purpose of recommending a design method for addressing pavement overloading in Wisconsin. In addition, methods were evaluated in terms of their ability to provide a transition to the AASHTO 2002 mechanistic-empirical design, and provide structural rehabilitation options for old JPCPs. The design methods reviewed were from Canadian provinces, and the Federal Highway Administration (FHWA) Regions III and VI. Region III has characteristics of a wet, hard freeze, and spring thaw, while Region VI is characterized by dry, hard freeze, and spring thaw. The main design methods identified were the Portland Cement Association (PCA) method and versions of the AASHTO design guide. The 1993 version of the AASHTO guide was chosen as the preferred guide for Wisconsin. The 1993 AASHTO guide was evaluated using data from a logging truck corridor along USH 8 near Rhinelander, Wisconsin. A new thickness design comparison was made between the 1993 AASHTO guide and the current method used by the Wisconsin Department of Transportation (WisDOT).

Based on the review and analysis, the following conclusions are reached:

- Although agencies recognize the high cost and damaging effects of overloaded trucks on pavements, there is zero to very little documentation in agencies' design manuals regarding how overloading is addressed in the pavement design process.
- The PCA method addresses overloading by applying load safety factors to each axle load range prior to determining slab stresses. The safety factor varies from 0-20%.
- The 1993 AASHTO guide addresses uncertainty such as overloading, through probabilistic design concepts that reflect the amount of statistical variability associated with parameters in the design process. The ultimate parameter is the design reliability.
- The 1993 AASHTO design guide is the dominant method for the design of JPCPs; it is used by 80 percent of agencies. However, there is considerable variation among agencies regarding key input parameters, including rigid pavement ESAL factors for FHWA vehicle classifications, flexural strength values, and traffic projection methods.
- Wisconsin's rigid ESAL factor values were found to be relatively low for FHWA vehicle classes 5, 8, and 9, when compared to the average of the surveyed agencies.

7.2 Recommendations

- The 1993 AASHTO method is recommended for design of JPCP in Wisconsin. It addresses overloading using reliability. In addition, it uses more concepts and terminology that provide a transition to the AASHTO 2002 mechanistic-empirical design compared to the PCA method. Furthermore, it outlines procedures for designing structural overlays for existing or old JPCPs.
- The guidelines for the selection of new design input values for the 1993 AASHTO method recommended in this study should be followed until Wisconsin develops appropriate values for local conditions. These include reliability, modified rigid ESAL factors, and effective K-values.

- Where overloading is anticipated, higher than normal reliability values should be considered in conjunction with the modified rigid ESAL factors.
- For structural overlay design, the procedure outline and suggested input parameters for AC overlay of JPCP, AC overlay of fractured (rubblized, crack-and-seat) JPCP, bonded, and unbonded JPCP overlays should be used.
- To allow for a full range of design options to be considered, there is the need to conduct a similar research for flexible pavements, particularly, regarding the ESAL factors to use for flexible pavement design if the 1993 AASHTO guide is to be implemented.

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APPENDIX A

CURRENT TRUCK TRAFFIC TRENDS FOR A CROSS SECTION OF WISCONSIN TRAFFIC COUNT SITES

Table A.1 Current Truck Traffic Trends for a Cross Section of Wisconsin Traffic Count Sites

Site ID	Location	County	Year	AADT	% FHWA Vehicle Class in AADT								
					Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
3400001	USH 45-0.4mi N of CTH N-Antigo	Lang-lade	2001	7996	1.95	0.83	0.32	1.40	2.53	0.14	0.00	0.02	0.02
			2002	8256	1.78	0.82	0.22	1.42	2.48	0.17	0.00	0.03	0.01
			2003	8376	1.93	0.81	0.31	1.56	2.46	0.15	0.00	0.02	0.01
346104	USH45-STH 47-South of CTH G		2002	8385	1.81	3.60	0.60	2.53	3.19	0.75	0.01	0.02	0.67
350002	USH 51-0.4 mi S of River Rd-Pine River	Lincoln	2001	16122	1.55	0.60	0.27	2.34	3.90	0.23	0.06	0.02	0.01
			2002	17102	1.59	0.54	0.22	2.62	4.20	0.19	0.07	0.03	0.01
			2003	18188	1.53	0.51	0.20	2.34	4.51	0.20	0.07	0.03	0.01
350110	STH 64-East of CTH E North		2001	2456	2.07	2.31	0.40	1.06	2.22	0.33	0.00	0.00	0.29
360002	I-43-0.5 mi S of Brown County-Cooperstown	Mani-towoc	2001	17522	1.74	0.63	0.08	1.21	8.95	0.38	0.32	0.06	0.09
			2002	18298	1.84	0.56	0.08	1.34	8.78	0.35	0.25	0.07	0.03
			2003	19664	1.44	0.54	0.11	1.00	9.73	0.28	0.26	0.05	0.05
430001	STH 17-2.3 mi S of CTH A East Sugar Camp	Oneida	2003	6036	1.96	0.68	0.31	1.46	1.94	0.21	0.03	0.00	0.07
436113	STH 17-North of Lincoln CO line		2003	3330	1.28	0.92	0.14	1.47	2.55	0.42	0.00	0.00	0.64
440109	STH 47-1.0 mi North of CTH O East	Outa-gamie	2002	6856	1.68	2.52	0.43	1.77	2.89	0.36	0.07	0.01	0.02
440154	STH 96 W of STH 76 Greenville Tnshp		2002	13652	2.08	1.35	0.11	2.02	4.56	0.41	0.12	0.01	0.10

APPENDIX B

LOGGING TRUCK CONFIGURATIONS OBSERVED FOR USH 8 AND 51 PROJECT SEGMENTS (MARCH 24-26, 2005)

Figure B.1 FHWA Class 6 Logging Truck along USH 8 (March 26, 2005)

Figure B.2 FHWA Class 7 Logging Truck along USH 8 (March 26, 2005)

Figure B.3 FHWA Class 9 Logging Trucks along USH 8 (March 24, 2005)

Figure B.4 FHWA Class 10 Logging Truck along USH 51 (March 24, 2005)