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Pavement Maintenance Integration PMS

PCCP MODELS FOR REHABILITATION AND RECONSTRUCTION DECISION-MAKING

by

Jianhua Li
Research Assistant

Stephen T. Muench
Assistant Professor

Joe P. Mahoney
Professor

Department of Civil and Environmental Engineering
University of Washington

Linda M. Pierce
State Pavement Engineer

Nadarajah Sivaneswaran
State Pavement Management Engineer

Washington State Department of Transportation

Washington State Transportation Center (TRAC)
University of Washington, Box 354802
1107 NE 45th Street, Suite 535
Seattle, Washington 98105-4631

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16. ABSTRACT

The majority of the Washington State Department of Transportation (WSDOT) portland cement concrete (PCC) pavements have far exceeded their original design lives and have carried several times the traffic loading originally anticipated. WSDOT is undertaking a major effort to identify both rehabilitation and reconstruction projects to improve its PCC pavements.

This project was performed to estimate WSDOT's concrete pavement performance. The current PCC pavement conditions were thoroughly analyzed. Two major groups of concrete pavement deterioration models were systematically studied: HDM-4 and NCHRP 1-37A. NCHRP 1-37A models proved to be more suitable for WSDOT conditions. The calibrated faulting and roughness models are able to present the typical performance of WSDOT PCC pavements. These models can be used to assist WSDOT in developing a plan for rehabilitating or reconstructing these pavements.

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CONTENTS

EXECUTIVE SUMMARY xi

1: INTRODUCTION 1

2: WSDOT PORTLAND CEMENT CONCRETE (PCC) PAVEMENT DESCRIPTION 4

2.1: Cracking 16
2.1.1: Undoweled PCC Pavements 16
2.1.2: DBR PCC Pavements 17

2.2: Faulting 17
2.2.1: Undoweled PCC Pavements 18
2.2.2: DBR PCC Pavements 18

2.3: Spalling 18
2.3.1: Undoweled PCC Pavements 19
2.3.2: DBR PCC Pavements 19

2.4: Roughness 19
2.4.1: Undoweled PCC Pavements 19
2.4.2: DBR PCC Pavements 20

3: HDM-4 PCC PAVEMENT DETERIORATION MODELS 21

3.1: HDM-4 Models 21
3.1.1: Transverse Cracking Model 21
3.1.2: Faulting Model 22
3.1.3: Spalling Model 22
3.1.4: Roughness Model 22

3.2: Calibration 23
3.2.1: Proposed Calibration Methodology 23
3.2.2: Determination of the Fixed Input Data 25
3.2.3: Calibration Results 25

4: NCHRP 1-37A PCC PAVEMENT DETERIORATION MODELS 27

4.1: Bench test 28

4.2: Preparation of Input Data 29

4.3: NCHRP 1-37A Models 30
4.3.1: Transverse Cracking Model 32
4.3.2: Faulting Model 34
4.3.3: Roughness Model 37

4.4: Calibration 39
4.4.1: Validation 41

4.4.2: Iteration 44
4.4.3: Calibration Results 46
4.4.4: Application to WSDOT PCC Pavement Rehabilitation and Reconstruction ... 52

5: CONCLUSIONS AND RECOMMENDATIONS 54

5.1: Conclusions 54

5.2: Recommendations 55

FIGURES

Figure Page

Figure 1 Percentage of cracked slabs vs. time since original construction as of 2002 (undoweled sections). 7
Figure 2 Percentage of cracked slabs vs. cumulative ESALs since original construction as of 2002 (undoweled sections). 7
Figure 3 Percentage of cracked slabs vs. time since original construction as of 2002 (DBR sections). 8
Figure 4 Percentage of cracked slabs vs. cumulative ESALs since original construction as of 2002 (DBR sections). 8
Figure 5 Percentage of cracked slabs vs. time since DBR as of 2002 (DBR sections). 9
Figure 6 Percentage of cracked slabs vs. cumulative ESALs since DBR as of 2002 (DBR sections). 9
Figure 7 Faulting vs. time since original construction (undoweled sections) as of 2002. 10
Figure 8 Faulting vs. cumulative ESALs since original construction (undoweled sections) as of 2002. 10
Figure 9 Faulting vs. time since DBR (DBR sections) as of 2002. 11
Figure 10 Faulting vs. cumulative ESALs since DBR (DBR sections) as of 2002. 11
Figure 11 Spalling vs. time since original construction (undoweled sections) as of 2002. 12
Figure 12 Spalling vs. cumulative ESALs since original construction (undoweled sections) as of 2002. 12
Figure 13 Spalling vs. time since DBR (DBR sections) as of 2002. 13
Figure 14 Spalling vs. cumulative ESALs since DBR (DBR sections) as of 2002. 13
Figure 15 Roughness vs. time since original construction (undoweled sections) as of 2002. 14
Figure 16 Roughness vs. cumulative ESALs since original construction (undoweled sections) as of 2002. 14
Figure 17 Roughness vs. time since DBR (DBR sections) as of 2002. 15
Figure 18 Roughness vs. cumulative ESALs since DBR (DBR sections) as of 2002. . 15
Figure 19 Default NCHRP 1-37 estimated transverse cracking under varying contraction joint spacings (9–in. undoweled slab, 9–in. granular base, 1.6 million ESALs/year/design lane, Seattle). 33
Figure 20 Percentage of cracked slab by age based on WSDOT data and the default NCHRP 1-37A transverse cracking prediction. 34
Figure 21 WSDOT faulting data and default NCHRP 1-37A prediction of faulting.... 35
Figure 22 Default NCHRP 1-37A estimated faulting vs. base type (9” undoweled slab, 9” base, 15’ joint spacing, 1.6million ESALs/year/designlane, Seatttle). 35
Figure 23 Default NCHRP 1-37A estimated faulting vs. ESALs (9–in. undoweled slab, 9–in. granular base, 15-ft. joint spacing, Seattle). 36

Figure 24 Default NCHRP 1-37A estimated faulting vs. climate (9–in. undoweled slab, 9–in. granular base, 15-ft. joint spacing, 1.6 million ESALs/year/designlane). 36
Figure 25 Default NCHRP 1-37A estimated IRI vs. base type (9–in. undoweled slabs, 9–in. base, 15-ft. joint spacing, 1.6 million ESALs/year/desginlane, Seattle). 37
Figure 26 Default NCHRP 1-37A estimated IRI vs. ESALs (9–in. undoweled slabs, 9–in. granular base, 15-ft. joint spacing, Seattle). 38
Figure 27 Default NCHRP 1-37A estimated IRI vs. climate (9–in. undoweled slab, 9–in. granular base, 15-ft. joint spacing, 1.6 million ESALs/year/designlane). 38
Figure 28 WSDOT IRI data and default NCHRP 1-37A prediction. 39
Figure 29 NCHRP 1-37A calibration methodology flowchart. 45
Figure 30 Calibrated NCHRP1-37A model estimates of transverse cracking for WSDOT undoweled PCC pavements. 48
Figure 31 Calibrated NCHRP1-37A model estimates of transverse cracking for WSDOT DBR pavements. 48
Figure 32 Calibrated NCHRP 1-37A model estimates of faulting for WSDOT undoweled PCC pavements. 49
Figure 33 Calibrated model estimates of roughness for WSDOT undoweled PCC pavements (model uses calibrated cracking and faulting inputs and default roughness model). 50
Figure 34 Calibrated model estimates of roughness for WSDOT DBR pavements (model uses calibrated cracking and faulting inputs and default roughness model). 51
Figure 35 Differences in roughness between calibrated NCHRP 1-37A model and WSPMS data for validation sections; possibly due to studded tire wear. 52

TABLES

Table | Page

Table 1 Calibrated Factors for HDM-4 Models 25
Table 2 Design Parameters Used for Bench Testing 29
Table 3 Calibration Factor Elasticity for NCHRP 1-37A Models 32
Table 4 Design Parameters and Distress Data of Calibration Sections for NCHRP 1-37A Models 41
Table 5 Design Parameters and Distress Data of Undoweled Validation Sections for NCHRP 1-37A Models 43
Table 6 Design Parameters and Distress Data of DBR Validation Sections for NCHRP 1-37A Models 44
Table 7 Final Calibration Factors for NCHRP 1-37A Models 46

EXECUTIVE SUMMARY

A large number of the Washington State Department of Transportation's (WSDOT) portland cement concrete (PCC) pavements are nearing the end of their useful life and will soon require rehabilitation or reconstruction. For WSDOT PCC pavements, this could apply to any of the approximately 2,000 lane-miles of PCC pavement. Given the current condition of these PCC pavements, WSDOT is undertaking a major effort to identify both rehabilitation and reconstruction projects to improve these pavements. This process includes identification of specific candidate projects, type of rehabilitation or reconstruction, and timing.

In order to enhance the prioritization of rehabilitation and reconstruction efforts, the rigid pavement portions of two pavement analysis and design tools (HDM-4 and NCHRP 1-37A) were studied. The basic findings were that (1) the HDM-4 PCC pavement deterioration models cannot be used at this time by WSDOT, and (2) the NCHRP 1-37A models are able to be calibrated with some limited exceptions. This report provides the details associated with these findings.

The calibrated models in NCHRP 1-37A software can be used by WSDOT pavement specialists to better predict future PCC pavement performance.

1: INTRODUCTION

The majority of the Washington State Department of Transportation (WSDOT) portland cement concrete (PCC) pavements were constructed during the late 1950s and 1960s as part of the Interstate construction program. At that time, the pavement design life for these roadways was estimated to be about 20 years. These pavements have far exceeded their original design lives and have carried several times the traffic loading originally anticipated. WSDOT now faces a huge backlog of PCC pavement rehabilitation and reconstruction needs throughout the state, most of which are Interstate system pavements. To date, the amount of pavement preservation (P1) funding applied to PCC pavements has been minimal given the needs.

Pavement rehabilitation and reconstruction is a major process for any state DOT. For WSDOT PCC pavements, this could apply to any of the approximately 2,000 lane-miles of PCC pavement. Given the current condition of these PCC pavements, WSDOT is undertaking a major effort to identify both rehabilitation and reconstruction projects to improve these pavements. This process includes identification of specific candidate projects, type of rehabilitation or reconstruction, and timing.

A key element for estimating WSDOT's PCC pavement rehabilitation and reconstruction needs is the ability to estimate PCC pavement performance. Accurate performance estimates would allow for (1) prediction of future pavement condition so that rehabilitation and reconstruction efforts can be properly scheduled, and (2) determination of the effects and costs of various rehabilitation, reconstruction, and timing options under consideration.

The option of developing a new predictive tool from WSPMS data was briefly considered but discarded because of the anticipated long development time and cost compared to the urgency of the required solution and limited available funds. Therefore, it was decided to use an existing tool and calibrate it to Washington State PCC pavements. While many methods of prediction were available, it was felt that mechanistic approaches would be the most viable because predictions had to be based on measured physical pavement properties as cataloged in the WSPMS. Most empirical approaches, including the *1993 AASHTO Guide for Design of Pavement Structures*, estimate the pavements to be well beyond serviceable life or do not include a future performance prediction feature. On the basis of some promising early use (Al-Yagout et al., 2005), currently two of them are of special interest to WSDOT: (1) the Highway Development and Management System (HDM-4), and (2) the software associated with the *2002 Guide for the Design of New and Rehabilitated Pavement Structures* (NCHRP 1-37A project). Other methods considered but not chosen were as follows:

- **Embedded models in WSPMS.** PCC pavement performance prediction curves already exist in WSPMS; however, they are empirical, have simplistic power functions, and are generally inadequate for the types of decisions needed.
- **Highway Economic Requirements System, State Version (HERS-ST).** Although generally accepted and used by the Federal Highway Administration (FHWA), HERS-ST pavement performance models are based on roughness alone and are unable to predict detailed cracking and faulting behavior, which is essential for PCC pavement performance prediction on a project level.

- **Advanced models (e.g., EverFE, ILLISLAB 2000, ABACUS).** These programs can provide detailed analysis (such as stresses, strains, or deflections), which can be tied to performance through transfer functions. The lack of embedded transfer functions is major impediment to use.

Neither HDM-4 nor NCHRP 1-37A software can be used directly without calibration. To study the usability of the two sets of models for WSDOT, a two-step approach was undertaken: (1) calibrate existing HDM-4 PCC models, and (2) calibrate NCHRP 1-37A models. The calibration process used WSDOT-specific data.

The UW research team calibrated the HDM-4 models first, in part, because of the availability of needed data. Furthermore, some HDM-4 PCC models are quite similar to the 1-37A models, so experience gained on the HDM-4 work benefited the NCHRP 1-37A work.

Starting with the HDM-4 work, this two-step process resulted in the most efficient expenditure of research effort and the highest likelihood of success (Muench and Mahoney, 2004).

2: WSDOT PORTLAND CEMENT CONCRETE (PCC) PAVEMENT DESCRIPTION

The Washington State Pavement Management System (WSPMS) is a historical archive of WSDOT highway pavement condition data. The data are organized into analysis units and project units: analysis units contain homogeneous pavement sections that are structurally uniform (same type of materials and thicknesses); project units are established according to similar pavement performance criteria and made up of one or more analysis units. WSDOT schedules pavement preservation efforts on the basis of project units, so this study also analyzed PCC pavements in the same units. The section lengths range from 0.07 to 22 miles, the average being 2.5 miles. Bridges were excluded, and the WSPMS contains no significant bridge-related information.

WSDOT has over 2,000 lane miles of PCC pavements that vary in age between 1 and 78 years, with the bulk (68 percent) being between 25 and 45 years old. All but a few hundred lane-feet are jointed plain concrete pavements (JPCP), with 99 percent originally constructed without dowels. Older WSDOT PCC pavements are generally 8 to 9 inches thick and built on a granular or asphalt treated base of 3 to 10 inches. PCC pavements built within the last 10 years tend to be about 12 to 13 inches thick on a dense, graded hot mix asphalt base of 3 to 5 inches. Joint spacing on all pavements is typically about 15 feet or less.

About 78 percent of WSDOT PCC pavements have never been rehabilitated. Rehabilitation that has occurred has generally been limited to isolated diamond grinding projects, dowel bar retrofits (DBR) in severely faulted areas, or hot mix asphalt (HMA) overlays. Most of the severely faulted, undoweled PCC pavement (about 230 lane-miles) was retrofitted with dowel bars from 1994 to the present. These DBR pavements are

located on I-5 near Bellingham and Olympia, on I-90 between Snoqualmie Pass and Ellensburg, and on I-82 between Ellensburg and Yakima. A typical DBR project involves retrofitting three to four dowel bars in each wheelpath and then diamond grinding the slabs (Pierce, 1999). This serves to restore load transfer between slabs and eliminate accumulated faulting and other roughness. In general, DBR pavement sections remain relatively smooth; however, some slabs have recently exhibited large longitudinal cracks from dowel slot to dowel slot. The suspicion is that DBR may have contributed to these cracks, but nothing definitive has been uncovered; however, this DBR performance issue will be studied to determine the failure mode.

This study was mainly focused on undoweled and dowel bar retrofitted (DBR) sections with high and median level traffic (measured by equivalent single axle loads (ESALs)). Therefore, two categories of PCC pavement were analyzed:

- **216 undoweled sections**: PCC pavements that were originally built without dowel bars and that were not rehabilitated as of 2002.
- **58 DBR sections**: PCC pavements that were dowel bar retrofitted before 2002. They are located on I-5, I-82, and I-90 (WSDOT, 2003).

To investigate the characteristics of WSDOT pavement performance data, slab cracking, faulting, spalling, and roughness for these sections were graphed versus slab age or the cumulative ESALs. The annual ESAL growth rate was assumed to be 1.6 percent (WSDOT, 2003). The slab age and ESALs were a function of either the original construction year or the year of the last rehabilitation. Age and ESALs are the primary factors that influence PCC pavement deterioration conditions for both the HDM and

NCHRP 1-37A models (Odoki et al., 2000). The sections in the WSPMS that had the following conditions were considered to be outliers and were excluded from the database:

- **For undoweled sections,**
  - Age > 60 years. Because the sections are old and not rehabilitated, the data are questionable.
  - Age < 5 years, and cracking > 50 percent of total slabs, or faulting > 0.25 inches. The deterioration is likely due to construction quality issues, which are not considered in either the HDM-4 or NCHRP 1-37A models.
  - IRI > 5 m/km. WSDOT's trigger of International Roughness Index (IRI) for rehabilitation is 3 m/km, so the sections with large IRIs are considered non-representative of the WSDOT system.
- **For DBR sections.**
  - Age since DBR < 5 years, and faulting > 0.25 inches. The high faulting is likely due to construction quality issues, which the HDM-4 and NCHRP 1-37A pavement deterioration models do not consider.

Figures 1 to 18 (sorted by the pavement deterioration types) show pavement deterioration condition in 2002 according to WSPMS 2003. Each figure is discussed in the text that follows.

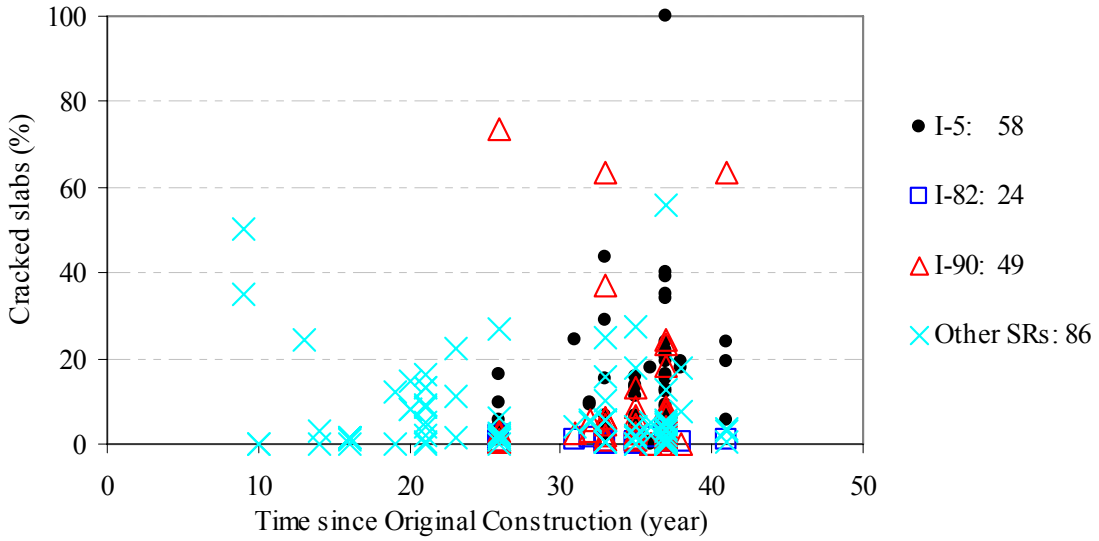

Figure 1 Percentage of cracked slabs vs. time since original construction as of 2002 (undoweled sections).

Note: 1. The number following each state route is the number of sections for that route in the figure.
2. 'Cracked slabs' means the total percentage of slabs having all types of cracking.
3. 'Other SRs' means all other state routes except I-5, I-82 and I-90.

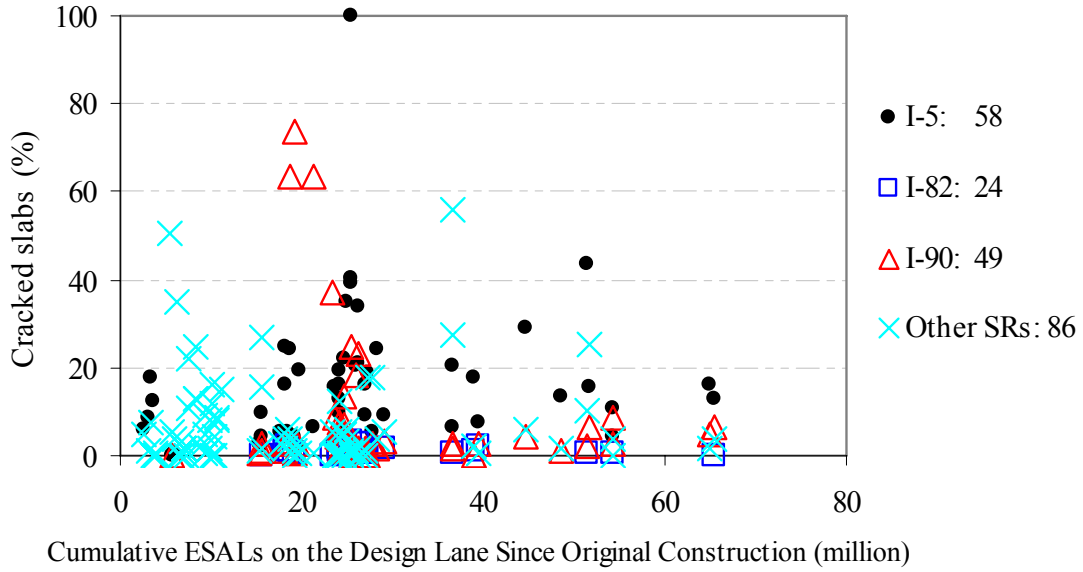

Figure 2 Percentage of cracked slabs vs. cumulative ESALs since original construction as of 2002 (undoweled sections).

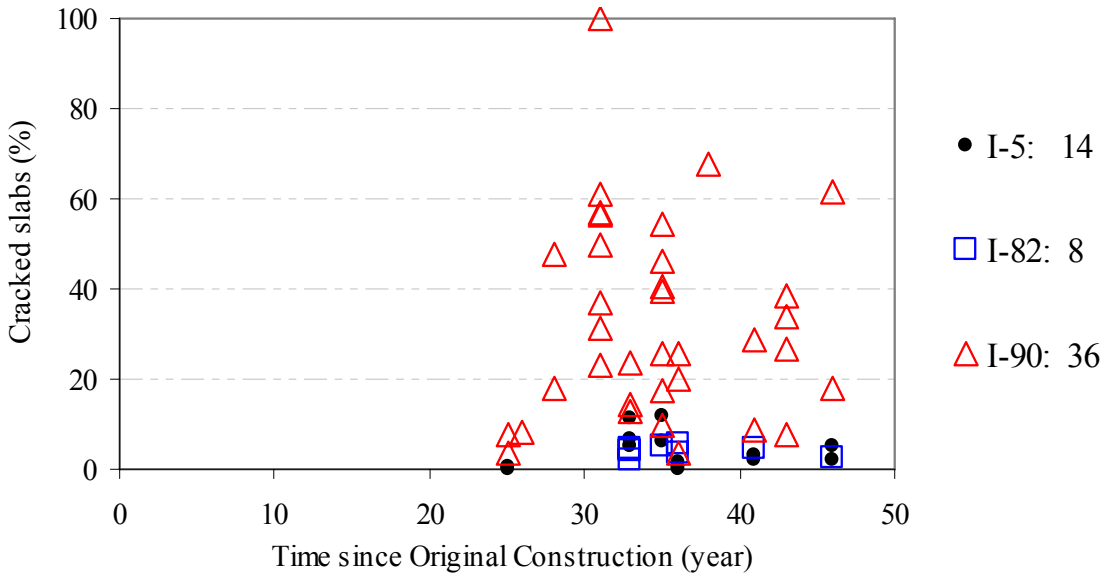

Figure 3 **Percentage of cracked slabs vs. time since original construction as of 2002 (DBR sections).**

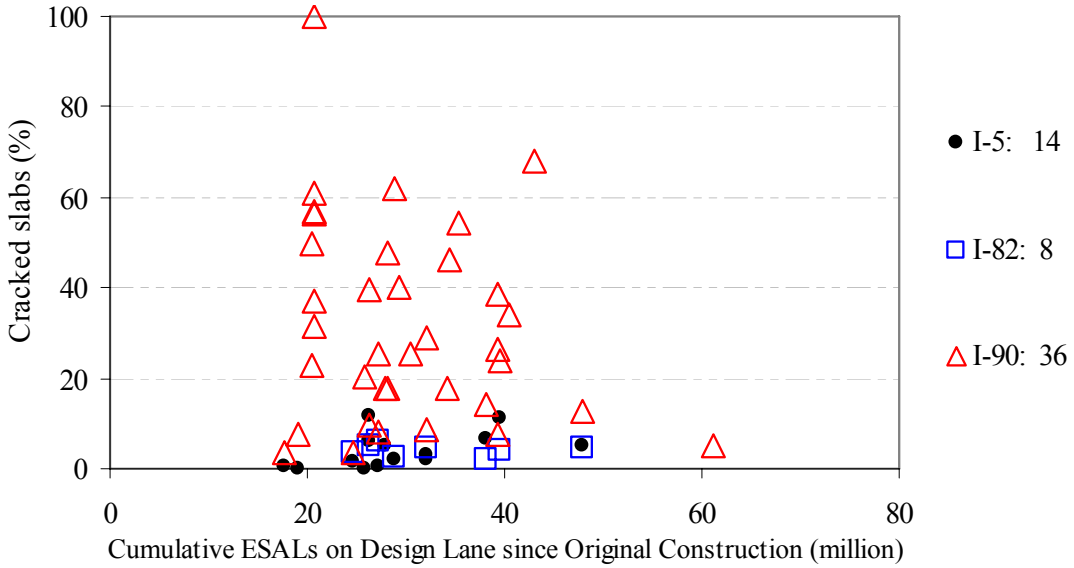

Figure 4 **Percentage of cracked slabs vs. cumulative ESALs since original construction as of 2002 (DBR sections).**

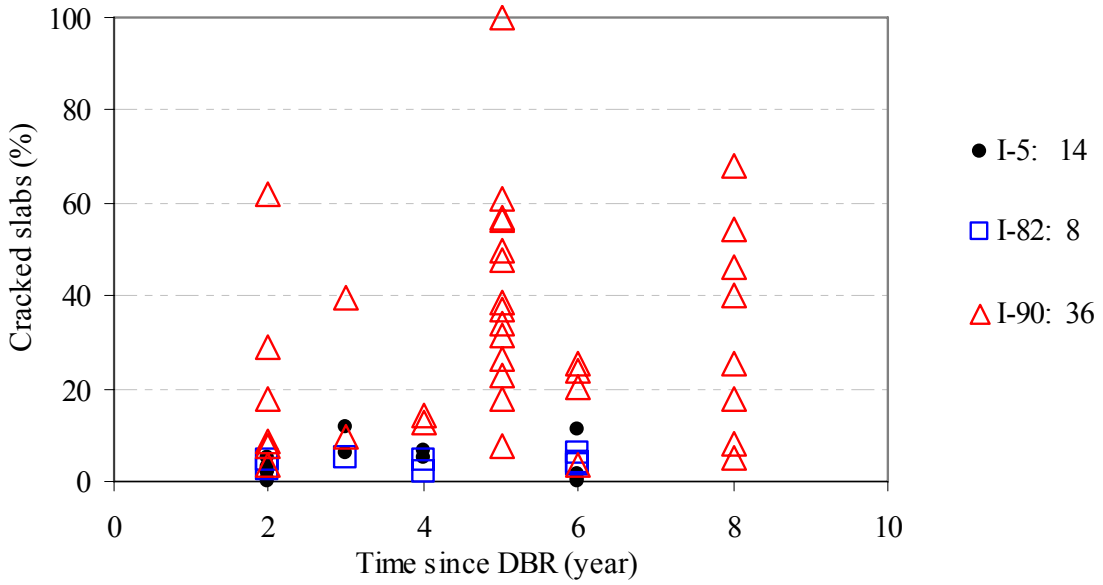

Figure 5 **Percentage of cracked slabs vs. time since DBR as of 2002 (DBR sections).**

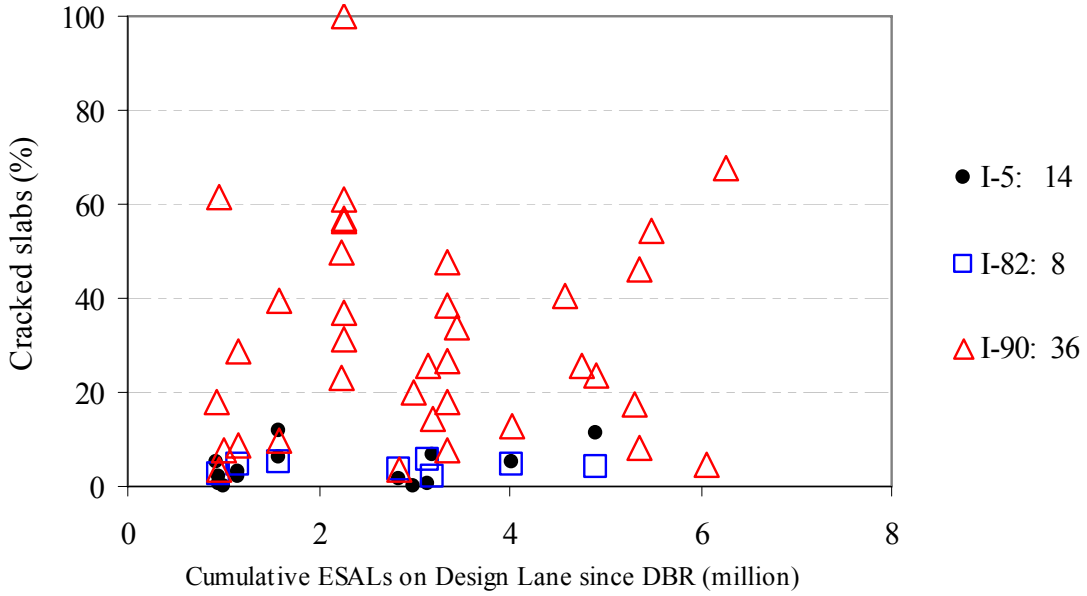

Figure 6 **Percentage of cracked slabs vs. cumulative ESALs since DBR as of 2002 (DBR sections).**

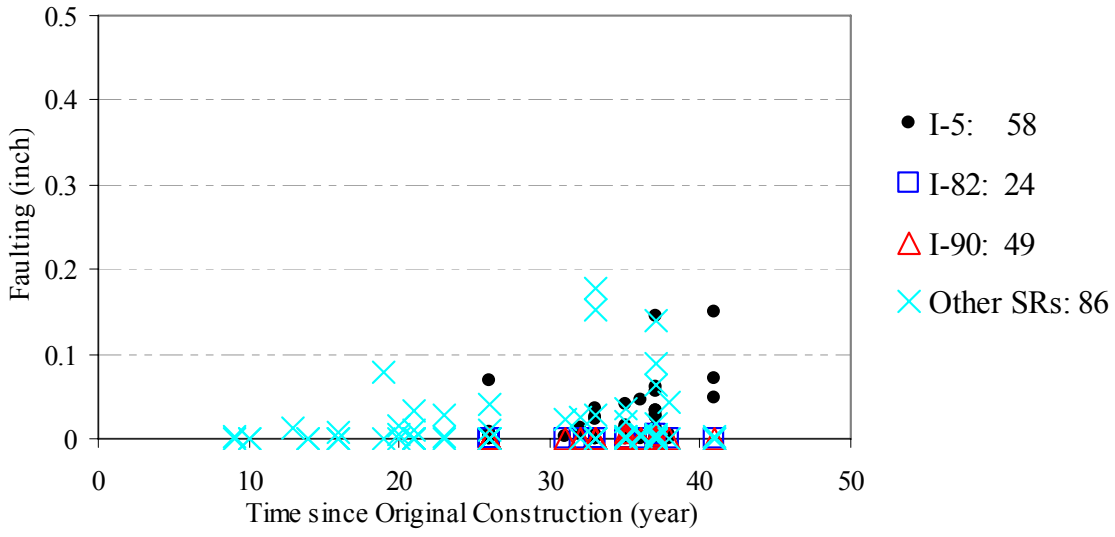

Figure 7 Faulting vs. time since original construction (undoweled sections) as of 2002.

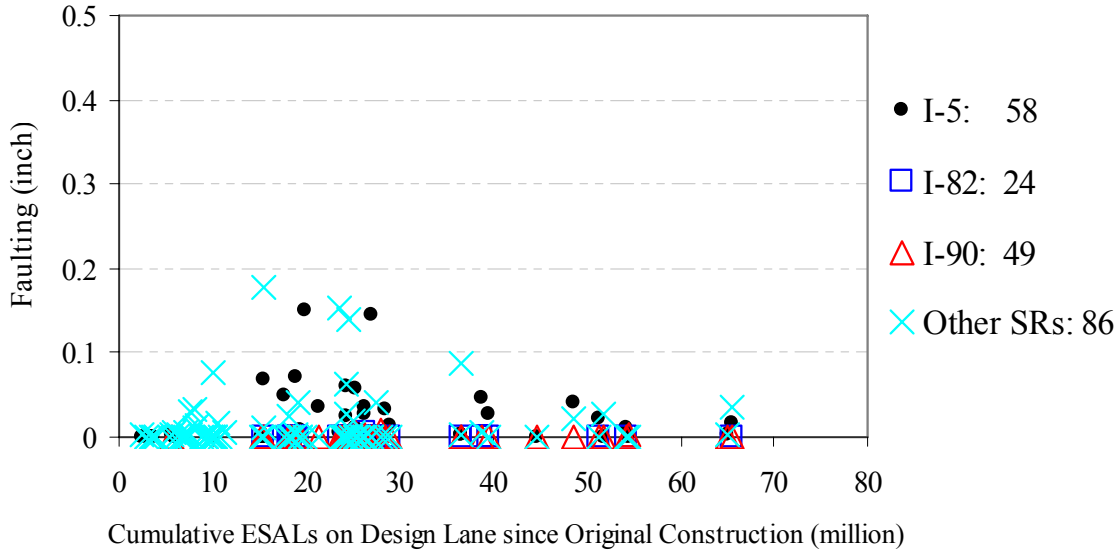

Figure 8 Faulting vs. cumulative ESALs since original construction (undoweled sections) as of 2002.

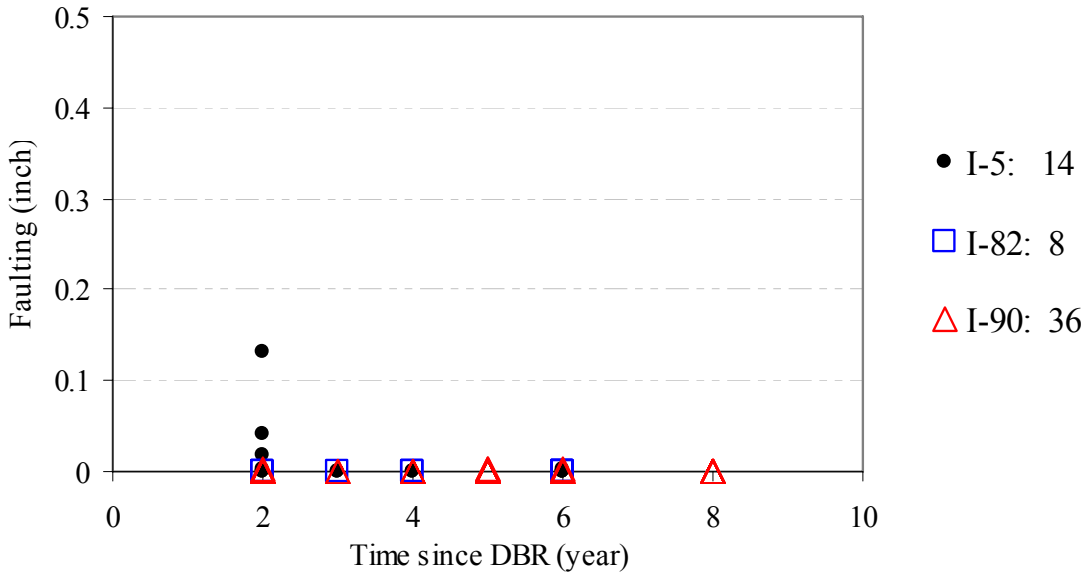

Figure 9 Faulting vs. time since DBR (DBR sections) as of 2002.

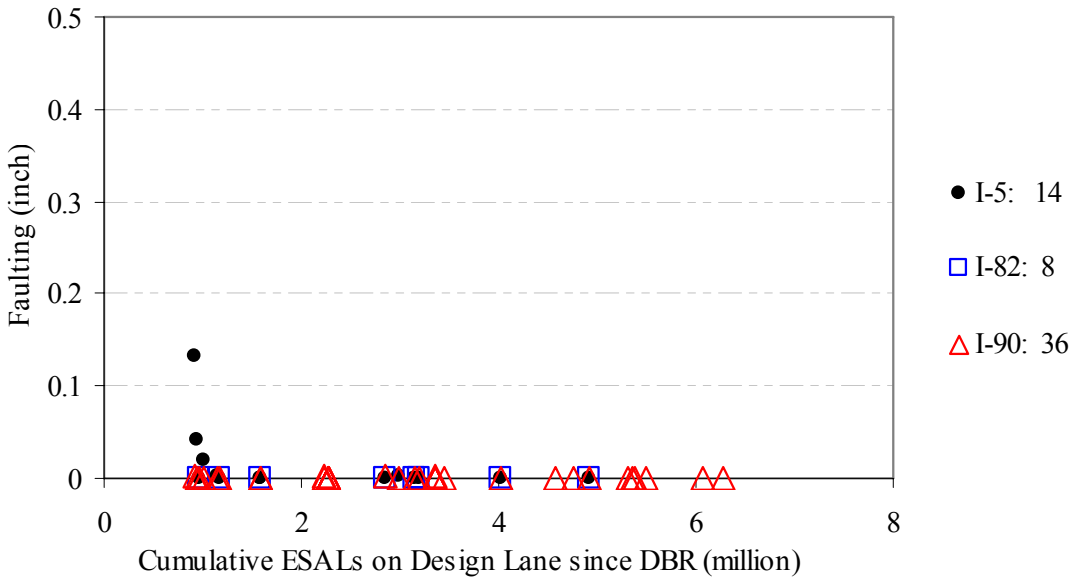

Figure 10 Faulting vs. cumulative ESALs since DBR (DBR sections) as of 2002.

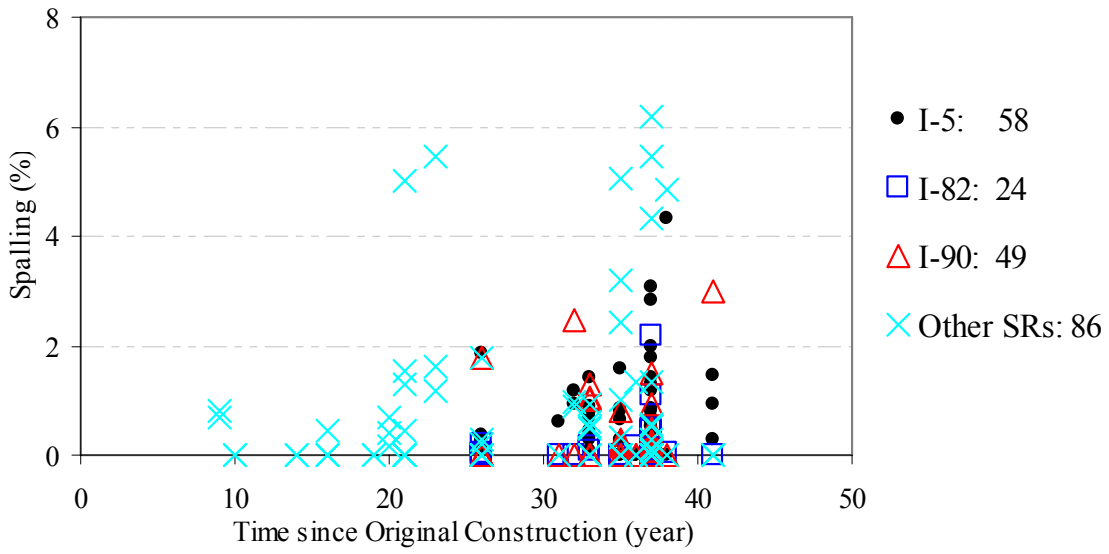

Figure 11 Spalling vs. time since original construction (undoweled sections) as of 2002.

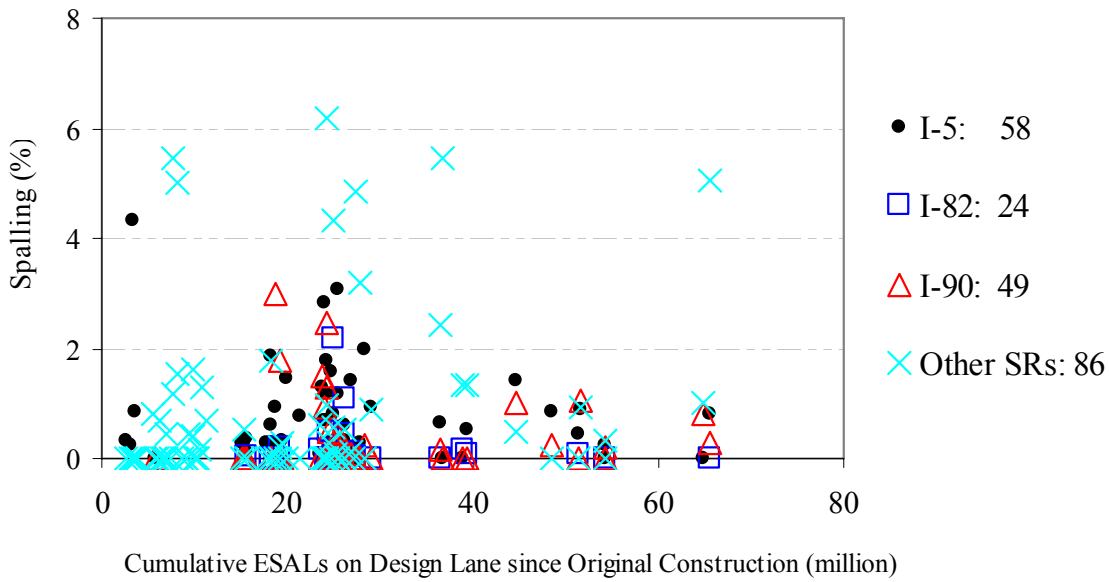

Figure 12 Spalling vs. cumulative ESALs since original construction (undoweled sections) as of 2002.

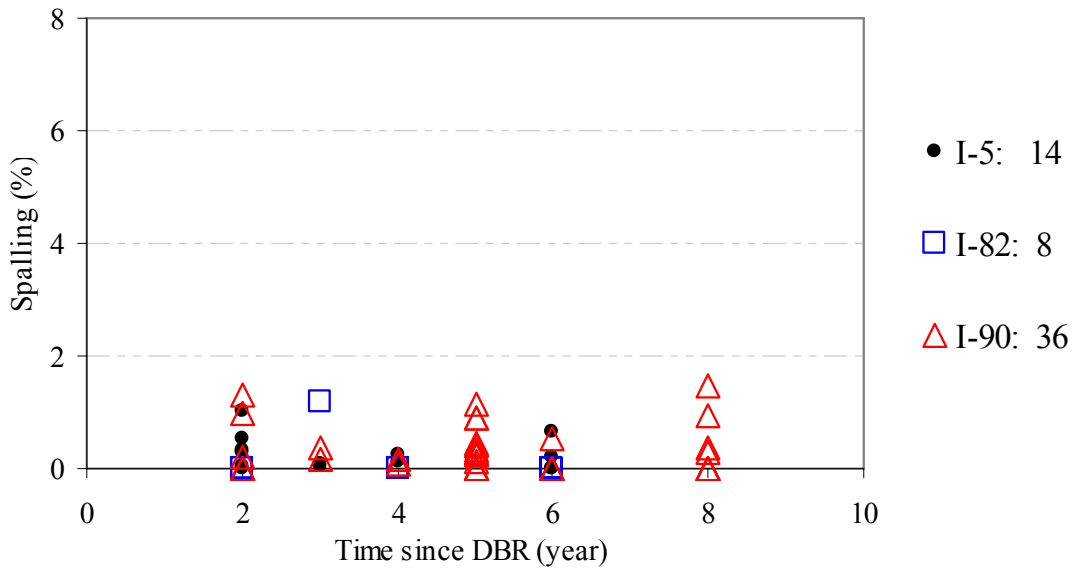

Figure 13 Spalling vs. time since DBR (DBR sections) as of 2002.

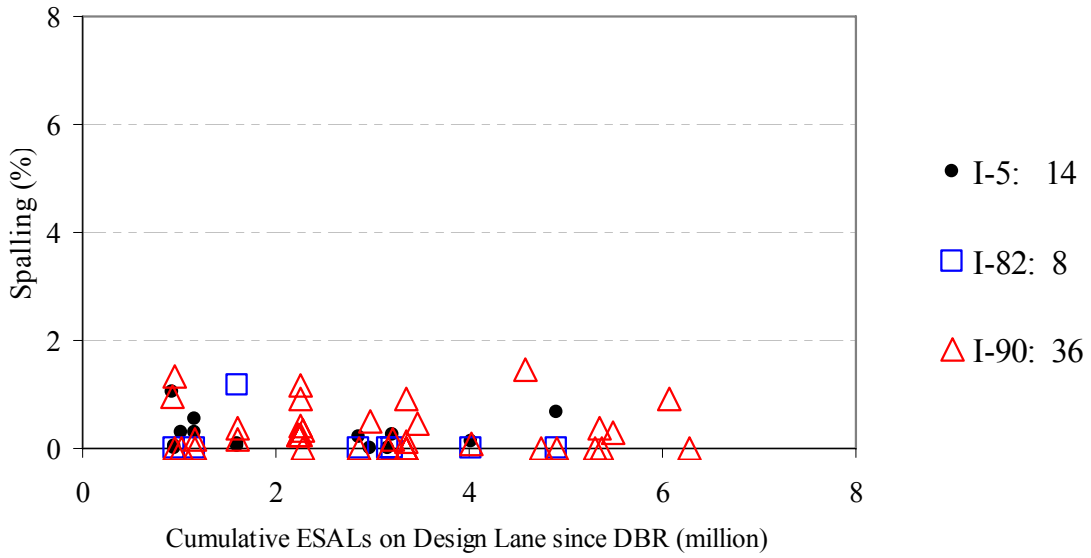

Figure 14 Spalling vs. cumulative ESALs since DBR (DBR sections) as of 2002.

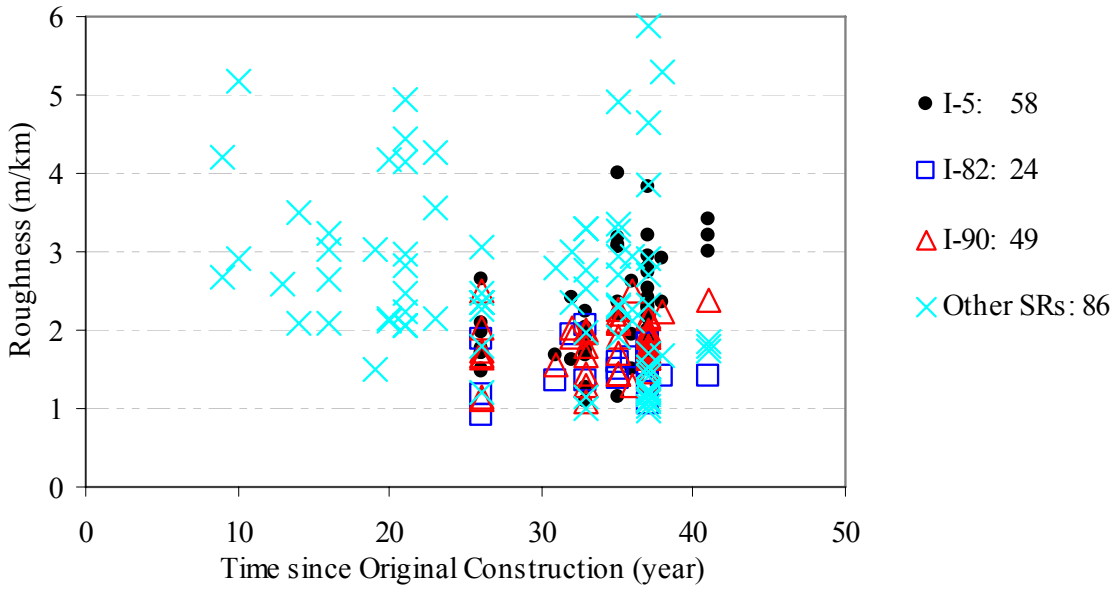

Figure 15 Roughness vs. time since original construction (undoweled sections) as of 2002.

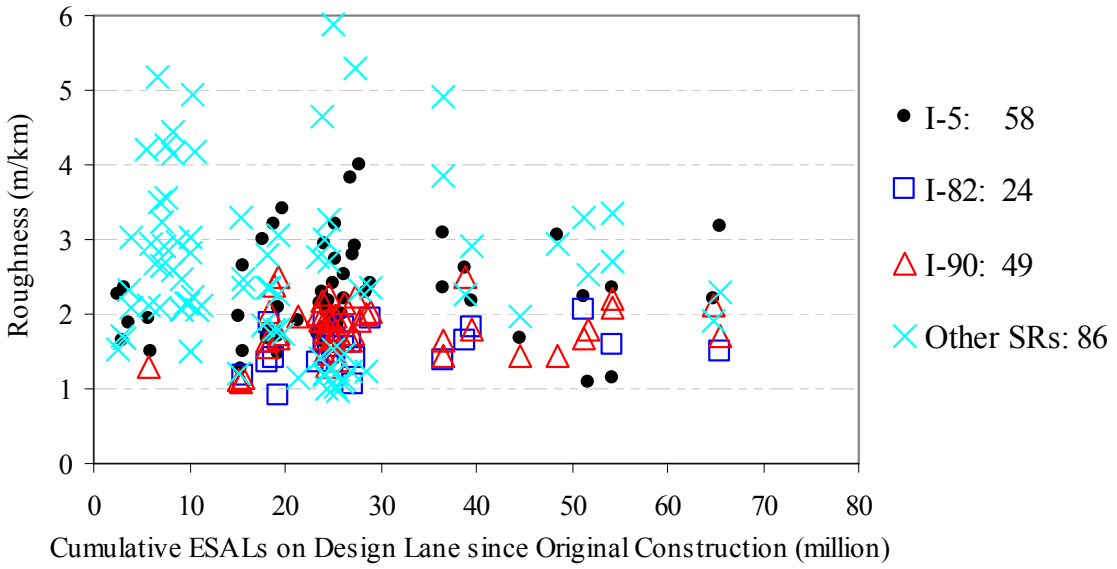

Figure 16 Roughness vs. cumulative ESALs since original construction (undoweled sections) as of 2002.

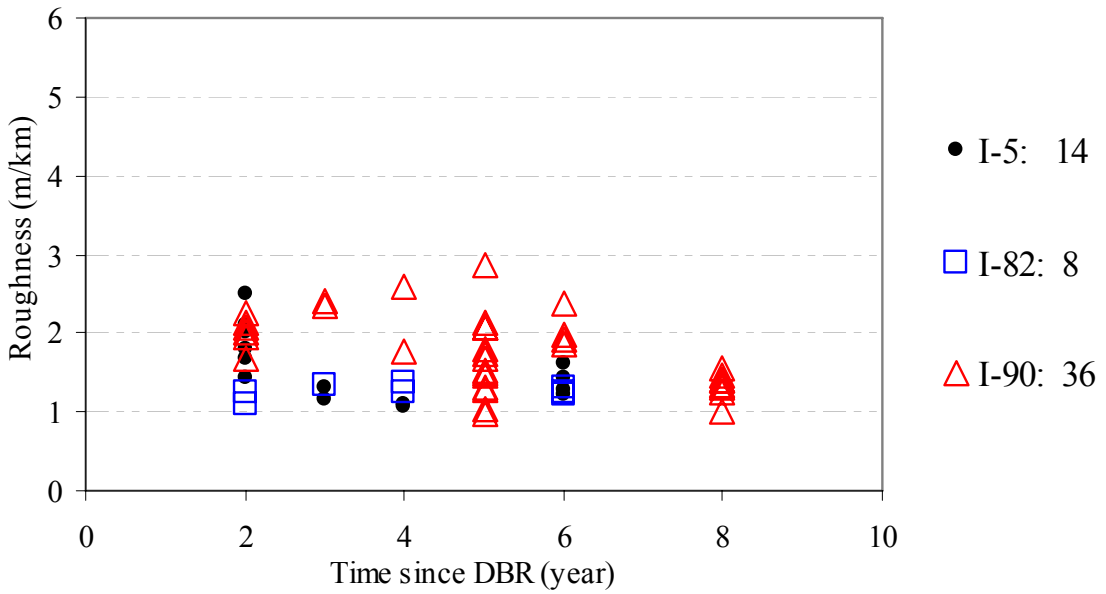

Figure 17 Roughness vs. time since DBR (DBR sections) as of 2002.

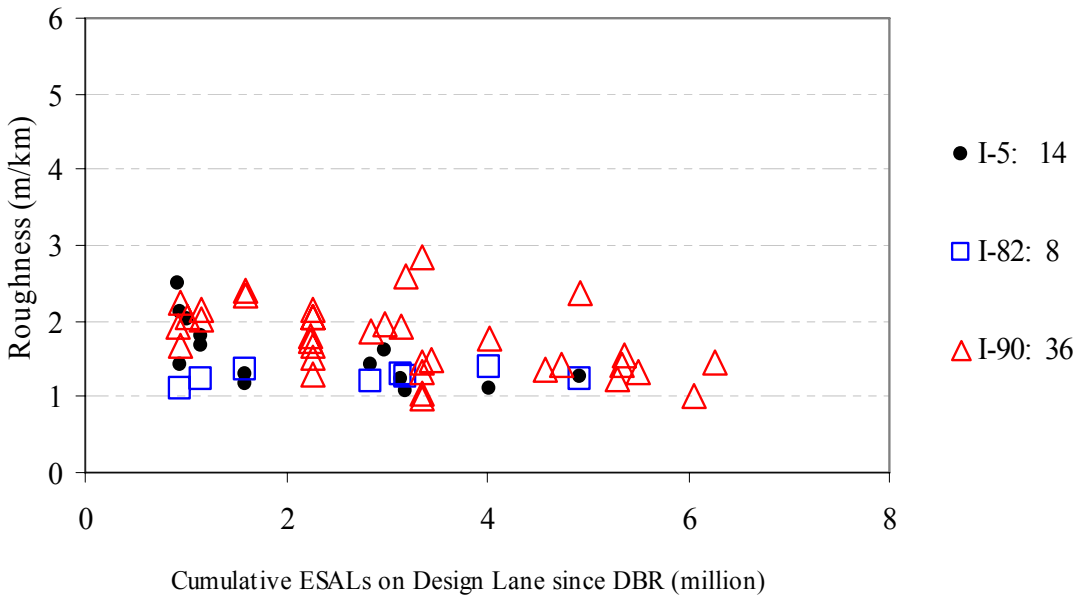

Figure 18 Roughness vs. cumulative ESALs since DBR (DBR sections) as of 2002.

2.1: CRACKING

The WSPMS does not differentiate between longitudinal and transverse cracks. However, extensive observation indicates that a large majority of cracks (especially in the Tacoma-Seattle-Everett I-5 corridor) are longitudinal. This is as expected, given the typically short transverse joint spacing that would tend to preclude transverse cracks.

HDM-4 and NCHRP 1-37A only model transverse cracking. However, WSDOT measures all types of cracking with three severity levels:

- CR1 = percentage of slabs with 1 crack per panel
- CR2 = percentage of slabs with 2 or 3 cracks per panel
- CR3 = percentage of slabs with 4 or more cracks per panel

To define WSDOT PCC pavement cracking, CR1+CR2+CR3 was used to present the total percentage of slabs that are cracked (see figures 1 to 6). The graphs are not able to show the transverse cracking performance trend of the WSDOT PCC slabs, but the transverse cracking cannot be greater than CR1+CR2+CR3.

2.1.1: Undoweled PCC Pavements

Undoweled PCC pavements were defined as pavements that had not been retrofitted with dowel bars as of 2002. Figures 1 and 2 show the following:

- The amount of slab cracking appears to be relatively independent of the cumulative ESALs since the original construction year.
- The amount of slab cracking is somewhat correlated to the slab age. For slabs younger than 20 years, few (project) sections were cracked.
- The cracking for all I-82 sections was lower than 5 percent of the total slabs.

- The cracking for most I-5 sections was greater than 10 percent of the total slabs.

2.1.2: DBR PCC Pavements

For the PCC pavements that were dowel bar retrofitted before 2002, figures 3 and 4 show the following:

- Only projects on I-5, I-82, and I-90 were dowel bar retrofitted.
- All sections were 25 years old or more before receiving DBR rehabilitation.
- Most DBR sections on I-90 had significant amounts of cracking; however, it appears that most of the slab cracking on I-90 occurred prior to DBR.
- All cracking on I-5 and I-82 was lower than 10 percent of the total number of slabs.

Figure 5 shows the percentage of slab cracks versus time since DBR, and Figure 6 shows the percentage of slab cracks versus cumulative ESALs since DBR. The graphs indicate that DBR has had very little effect on slab cracking.

2.2: FAULTING

Both the HDM-4 and NCHRP 1-37A models account for faulting with an average depth in mm. The WSPMS uses the percentage of slabs within a given range of faulting. To reconcile these differences, the following rules were used to convert the faulting from the WSPMS to the HDM and NCHRP 1-37A models:

Faulting = faulting % for low severity * 4.7625 + faulting % for medium severity * 9.525 + faulting % for high severity * 12.7 (mm)

In 2002, WSDOT faulting ranged from 0 to about 0.5 inches, with the majority being less than 0.1 inches. Many of the most severely faulted pavements have been retrofitted with dowel bars.

2.2.1: Undoweled PCC Pavements

For the undoweled PCC pavements, figures 7 and 8 show the following:

- The faulting seems to be independent of ESAL loading; however, sections that had exhibited significant faulting had received a DBR rehabilitation.
- Those projects showed that faulting fell within a range of 0 to 0.2 inches, regardless of age or ESALs. Ninety-five percent of faulting was in the 0 to 0.05 inches range.
- Most I-82 and I-90 sections showed little faulting.

2.2.2: DBR PCC Pavements

Figures 9 and 10 show that DBR PCC slabs exhibited little faulting. The time since construction spanned two to eight years. Fifty-eight project units were dowel bar retrofitted, and only four of them showed measurable faulting. Certainly this will change, but the existing data provide no indication of when faulting will occur for DBR sections.

The above observations lead to the conclusion that, given the input variables, the HDM-4 and NCHRP 1-37A models will have difficulty predicting future faulting for DBR actions.

2.3: SPALLING

HDM-4 assumes that joint spalling is the percentage of joints that are spalled, and the spalling is assumed to be 75 to 100 mm wide. Therefore, the percentage of spalled

joints corresponds to the WSPMS high severity category (3 inches or more). The NCHRP 1-37A software does not provide spalling outputs, so the spalling model was not calibrated.

2.3.1: Undoweled PCC Pavements

Figures 11 and 12 show spalled joints for the undoweled PCC sections.

- Most slabs were not spalled. Up to 93 percent of the slabs had neglectable spalling (2 percent or less of the joints were spalled).
- The amount of spalling does not appear to be related to the cumulative ESALs.
- According to Figure 11, spalling is somewhat correlated to slab age. Slabs younger than 20 years had little spalling.

2.3.2: DBR PCC Pavements

The DBR PCC pavements (figures 13 and 14) either were not spalled or had very small amounts of spalling.

2.4: ROUGHNESS

HDM-4 uses m/km as the units of roughness, the same as WSDOT. NCHRP 1-37A uses inches/mile.

2.4.1: Undoweled PCC Pavements

Figures 15 and 16 show the following observations:

- Roughness on I-82 was generally lower than 2 m/km.
- Roughness on I-5 and I-90 showed an increasing trend with slab age.
- Roughness on the other state routes showed no discernable pattern.

2.4.2: DBR PCC Pavements

In figures 17 and 18, roughness after DBR is generally at moderate levels, regardless of ESALs and time since DBR. All pavements are smaller than 3 m/km.

3: HDM-4 PCC PAVEMENT DETERIORATION MODELS

The Highway Development and Management System (HDM-4), originally developed by the World Bank for international use, is a software tool for systematically addressing flexible and concrete pavement performance and rehabilitation issues. Currently, critical program errors render the PCC pavement portion of the program (Version 1.3) essentially non-functional (Li et al., 2005). However, all models (as listed in Appendix A) are given, and all variables are available or transferable from WSPMS or other reasonable sources. Therefore, by using the given models, variables, and condition data from WSPMS, the calibration factors can be regressed (calibrated) via econometric software. LIMDEP was chosen to estimate the calibration factors for this study.

3.1: HDM-4 MODELS

HDM-4 models four types of distress: transverse cracking, faulting, spalling, and roughness. The first three are modeled independently, and then the estimated results are incorporated into the roughness model. Doweled and undoweled pavements are modeled separately for transverse cracking and faulting.

3.1.1: Transverse Cracking Model

The default HDM model estimated almost no transverse cracking for WSDOT. In addition, when the slab joint spacing increased, the HDM model estimated less transverse cracking. This is unreasonable. Furthermore, because WSDOT does not record transverse cracking, the cracking model could not be effectively calibrated.

3.1.2: Faulting Model

The major factors considered in the faulting model are ESALs, slab thickness, joint spacing, base type, freezing index, annual average precipitation, and number of hot days (greater than 90°F) per year. For doweled pavements, additional factors are included, such as slab age, load transfer between joints, dowel support modulus, dowel diameter, dowel modulus of elasticity, and monthly temperature range.

Most WSDOT PCC sections exhibit less than 0.05 inches of faulting, but the HDM faulting models tended to predict more faulting than the actual WSDOT data.

3.1.3: Spalling Model

Major factors such as slab age, joint spacing, type of dowel corrosion protection, number of hot days per year, and freezing index are included in the spalling model.

By using WSDOT condition data, the default model estimated negative spalling values for Western Washington, which is unrealistic.

3.1.4: Roughness Model

The HDM roughness model uses faulting, spalling, transverse cracking, patching, and initial roughness after original construction. It does not account for studded tire wear, which is considered one of the primary factors affecting roughness on WSDOT pavements. The use of studded tires during the winter in Washington State, which averages about 10 percent of vehicles in Western Washington and 32 percent of vehicles in Eastern Washington (WSDOT, 2005), seems to be the primary contributor to wheelpath wear in PCC pavements. Wear depths range from barely measurable up to about 0.75 inches, depending upon pavement age and location.

3.2: CALIBRATION

The key input data used in the HDM PCC pavement deterioration models are related to the conditions of climate and environment, dowel use, traffic, pavement history, pavement geometry, pavement structural characteristics, and material properties. As undoweled and doweled (DBR) pavements are modeled separately for faulting and roughness in NCHRP 1-37A, they were calibrated independently. Furthermore, the authors found that one group of calibration factors was not able to estimate the different performances of pavement in Western and Eastern Washington. Thus, the two climate zones had to be calibrated independently. Accordingly, the calibration was performed in four categories:

- Undoweled:
  1. Western Washington
  2. Eastern Washington
- DBR:
  3. Western Washington
  4. Eastern Washington

3.2.1: Proposed Calibration Methodology

The general expression used for the HDM PCC pavement deterioration models is:

Predicted Distress**:** $Y' = K'_y * f\,(Y_a, a_0, a_1, a_2, \ldots, a_n, X_1, X_2, \ldots, X_m)$ (1)

where:

K_y' default calibration factor of distress type Y given by HDM-4 (all default values are initially set at 1.0)

Y' predicted value of distress type Y by HDM-4

a_i default coefficient values given by models, which are determined by factors of climate and environment, traffic, pavement history, pavement geometry, pavement structural characteristics or material properties

X_i pavement conditions of climate and environment, traffic, pavement geometry, pavement history, pavement structural characteristics and material properties.

For any specific type of pavement distress, the best calibration factor was obtained by following these steps:

1. Use default value of 1.0 given by HDM-4 as the calibration factor.
2. Input formula (1) and related independent variables into econometric software. Forecasted distress values (*Y'*) in 2002 are obtained.
3. WSPMS 2003 provides the actual distress values, Y, in 2002.
4. Reject outliers of *Y'* and *Y*.
5. The optimal K_y is obtained by regressing Equation 2 in the econometric software on the basis of inputs of *Y* and *Y'*:

$$Y = K_y * Y' \tag{2}$$

where:

Y Value of distress type Y in WSPMS 2003.

Y' Predicted value of distress type Y by using default calibration factors.

K_y Calibration factor of distress type Y.

LIMDEP was used to estimate the calibration factors (Greene, 2002).

3.2.2: Determination of the Fixed Input Data

Some input data are fixed for different WSDOT PCC pavements. They are as follows:

- Erodibility index: Erosion Resistant (3).
- Subgrade k static modulus of reaction: 54 MPa/m (200pci).
- Modulus of elasticity of concrete (Ec): 27500 MPa (4,000,000 psi).
- Modulus of rupture (flexural strength) of concrete: 5 MPa (725 psi).
- Thermal coefficient of concrete: 0.0000063 (/F^{o}) for gravel aggregate type.
- Shrinkage coefficient: 0.00045 m/m.
- Dowel diameter: 38 mm (1.5 inches).
- Joint seal material: Asphalt.
- Dowels corrosion coated or not: Yes, because WSDOT dowel bars are epoxy coated or stainless steel.

3.2.3: Calibration Results

According to the HDM models and WSPMS data, the calibration factors were regressed, and they are shown in Table 1.

Table 1 Calibrated Factors for HDM-4 Models

Section	Undoweled[a]		DBR[a]	
Climate	*WW*	*EW*	*WW*	*EW*
Cracking	3806	3806	14006	19501
Faulting	0.097	0.001	0.15	0.034
Spalling	0	0.076	0	0.04
Roughness	1.368	1.089	0.859	1.070

Note:

a: All default calibration factors are 1.0

Transverse Cracking: As defined by HDM-4, the calibration factors must be in the range of 0 to 20, but all calibrated factors ranged from 3,806 to 19,521 (Table 1). Therefore, these factors cannot be used.

Faulting: All calibrated factors had R-squared values smaller than 0.01. Some were negative. The calibrated faulting model predicts substantially larger faulting than actual values. Thus, the factors are not suitable for WSDOT use.

Spalling: The model estimated negative spalling values for Western Washington. Such errors are not able to be solved via model calibration. Thus, the spalling models are not suitable for WSDOT use.

Roughness: The calibration factors listed in Table 1 were based on actual faulting and spalling measurements, as well as estimated transverse cracking by using default calibration factors. Most calibrated factors had R-squares smaller than 0.1. Only the calibration category of DBR for Western Washington had an R-squared of 0.55; however, there were only 14 sections. The roughness model requires estimated values of transverse cracking, faulting, and spalling, but these models are not able to generate suitable results for WSDOT conditions. In addition, the model does not consider studded tire wear, which is a major factor for Washington State. Therefore, the model's estimation is marginal.

In conclusion, the HDM-4 PCC models are not able to reasonably predict WSDOT pavement performance.

4: NCHRP 1-37A PCC PAVEMENT DETERIORATION MODELS

In choosing the NCHRP 1-37A software as the preferred predictive tool, it is understood that there may be issues with particular model specifications, software bugs and predictive abilities. Many of these questions should be answered by the pending NCHRP 1-40A project, which will provide an independent review of these items with recommendations for improvement. Despite potential shortcomings, the NCHRP 1-37A software is currently the only major design tool able to predict pavement deterioration and the progression of that deterioration over time for a wide range of pavements. This calibration effort did not duplicate the NCHRP 1-40A work.

The NCHRP 1-37A models can not be systematically calibrated in the same manner as HDM-4, since most of the major independent variables required in the NCHRP 1-37A pavement distress models are not available for WSDOT. For example, the transverse cracking model requires the monthly applied number of load applications for each axle type, load level, and temperature difference. The faulting model needs accurate incremental changes for each month. (Appendix B lists all NCHRP 1-37A PCC pavement performance models.) WSDOT does not have such detailed data.

Most of the software design inputs are different from the model variables. The NCHRP 1-37A software allows three levels of design inputs: level 1 is the most precise, with data obtained from comprehensive laboratory and field tests; level 2 inputs are based on a limited number of laboratory or field measurements; level 3 inputs are based on experience with little or no testing. In this study, the input values were taken from typical WSDOT values or level 3 estimations.

Currently, the only way to calibrate the models is to use the software: that is, to change the calibration factors manually and run the software iteratively until the estimated pavement distress conditions achieve a reasonable match with the actual data. This calibration process is a trial and error calibration approach.

This study of the NCHRP 1-37A models involved four major tasks: a bench test, data input preparation, model analysis, and calibration.

4.1: BENCH TEST

Bench testing describes the process used to check the NCHRP 1-37A software for run-time issues and model prediction reasonableness, as well as identification of calibration needs.

Although the software had a few problems with unexpected crashes, this did not present significant difficulties. The reasonableness of the models was checked by varying the primary design parameters of traffic loading, climate, slab thickness, joint spacing, dowels, base type, and soil type (as shown in Table 2) and then comparing the results with generally accepted PCC pavement performance. Key observations from the bench testing were as follows:

- Transverse cracking was most influenced by joint spacing. When joint spacing was set at 15 feet (typical for WSDOT), results showed very little cracking (as expected).
- Dowel bar use heavily influenced the development of faulting and related roughness (as expected).
- Base type, traffic loading, and climate had significant impacts on faulting and roughness predictions (as expected).

- With a few exceptions (Kannekanti et al., 2005) predicted performance and its relation to input values matched well with consensus pavement knowledge.

Table 2 Design Parameters Used for Bench Testing

Design Parameters	Varied Values
Traffic loading (million ESALs)	2, 1, 0.5, 0.2, 0.05
Climate	WW, EW, mountain pass, Minnesota, Alaska, Florida
Slab thickness (inch)	14, 12, 9, 5
Joint spacing (feet)	21, 19, 17, 15, 13, 11
Dowels	yes or no
Base type	Granular, ATB, CTB
Soil type	SM, SC, ML, A-4…

These findings correlate well with previous studies and indicate that the NCHRP 1-37A software predicts reasonable PCC pavement performance (Kannekanti et al., 2005). In comparing the NCHRP 1-37A software output with actual WSDOT data, several calibration issues were identified. First, the default models tended to (1) over-predict transverse cracking, (2) predict significantly different faulting trends, and (3) under-predict roughness.

4.2: PREPARATION OF INPUT DATA

Loadings, materials, climate, and design features are required inputs in the NCHRP 1-37A pavement deterioration models. The accuracy of the performance prediction models depends on a process of calibration and validation on independent data sets. Therefore, how well the data inputs represent local conditions is critical.

Input values were generally taken from typical WSDOT values or default software values in level 3. Specific input categories source data references were as follows:

- **Traffic**. Previous work (Al-Yagout et al., 2005) established a standard load spectrum that provides reasonable results for Washington State. This load spectrum was used in calibration.
- **Materials**. Typical values for Washington State were used with specific values from previous WSDOT studies (as summarized in the WSDOT *Pavement Guide Interactive* (Muench et al., 2003)). Input values not available from WSDOT research were assigned typical nationwide values or default values from the JPCP example included with the software.
- **Climate**. The default climate data for weather stations located in Washington State built in the software were tested, inspected, and judged acceptable for this study.
- **Design details**. Details such as joint spacing, dowel, and tie bar details were taken from standard WSDOT design practices during the period in which a particular PCC pavement was constructed.

The DBR sections had no dowel bars before they were retrofitted. It is known that the sections were faulted when they were about 23 to 32 years old. This study assumed 3.3 percent of slabs had transverse cracking (1/3 of 10 percent of all types of cracking in the WSPMS), faulting was 0.25 inches, and the IRI was 3.5 m/km. Generalizations were based on historical WSPMS data (Pierce, 1999).

4.3: NCHRP 1-37A MODELS

Three primary NCHRP 1-37A software models for JPCP need to be calibrated: transverse cracking, faulting, and roughness. The transverse cracking and faulting models are independent of one another, while the roughness model incorporates cracking and

faulting model outputs as well as a spalling model output. The software does not give calibration access to the spalling model. The order of calibration is important: transverse cracking and faulting must be calibrated before roughness because they serve as inputs to the roughness model.

There are 16 calibration factors to consider in the three models. To evaluate the relative impact of each factor on model estimation, elasticity was adopted and defined as follows:

$$E_{distress}^{C_i} = \frac{\partial(distress)/distress}{\partial(C_i)/C_i} \tag{3}$$

where,

$E_{distress}^{C_i}$	Elasticity of factor Ci for the associated distress condition.
$\partial(distress)$	Change in the estimated distress associated with a change in the factor Ci.
$\partial(C_i)$	Change in the factor Ci.
distress	Estimated distress using default calibration factors.
Ci	Default value of Ci.

Elasticity can be zero, positive, or negative. Zero means the factor has no impact on the model; positive means the estimation increases as the factor increases; negative means the estimation decreases as the factor increases. The larger the absolute value of elasticity, the greater impact the factor has on the model (Greene, 2003). Table 3 shows the elasticity of each calibration factor. An elasticity of 1.0 or more is significant (Greene, 2003).

Table 3 Calibration Factor Elasticity for NCHRP 1-37A Models

Calibration Factor		Elasticity	Related Variables
Cracking	C_1	-7.579	PCC modulus of rupture and stress
	C_2	-7.079	PCC modulus of rupture and stress
	C_4	0.658	traffic loading and effective temperature difference though PCC slab
	C_5	-0.579	traffic loading and effective temperature difference though PCC slab
Faulting	C_1	0.42	EROD, PCC corner deflection
	C_2	0.08	base freezing index, EROD, PCC corner deflection, C_5, C_6, percent soil passing #200 sieves, annual wet days, subgrade load
	C_3	0.07	deformation energy, EROD, C_5, C_6, C_7, PCC corner deflection, percent soil passing #200 sieves, annual wet days, subgrade load, etc.
	C_4	0.01	base freezing index, deformation energy, EROD,C5, C6, C7, PCC corner deflection, percent soil passing #200 sieves, annual wet days, etc.
	C_5	0.07	EROD, PCC corner deflection, percent soil passing #200 sieves, annual wet days, subgrade load, etc.
	C_6	0.57	EROD, C5, average annual number of wet days, percent soil passing #200 sieves, and subgrade load
	C_7	0.55	deformation energy, PCC corner deflection, EROD, C_5, C_6
	C_8	0	dowel deterioration
Roughness	C_1	0.011	transverse cracking
	C_2	0.003	spalling
	C_3	0.077	faulting
	C_4	0.003	Site factor

4.3.1: Transverse Cracking Model

The structure of the NCHRP 1-37A transverse cracking model is the same as that of the HDM model. The major difference from the HDM cracking model is that the estimated cracking increases as the slab joint spacing increases, as shown in Figure 19. The figure also indicates that shorter joint spacings result in less transverse cracking.

Transverse cracking is the only type of cracking modeled by NCHRP 1-37A;

however, WSDOT records all types of cracking by severity levels instead of types, and the major cracking type in Washington State is longitudinal. Figure 20 shows WSDOT cracking of all types and the default NCHRP 1-37A transverse cracking estimation. WSDOT cracking data were averaged in each 10-year period. The averaged values were used to develop the cracking progression trend. The trend is similar to the default NCHRP 1-37A estimation.

Using the typical WSDOT design parameters, the default NCHRP 1-37 software model always overestimated transverse cracking (Figure 20). The transverse cracking model needs to be roughly calibrated to 1/3 of the actual cracking of all types in the WSPMS because the longitudinal cracking was approximately 2/3 of all types of cracking, according to the historical WSDOT PCC pavement images.

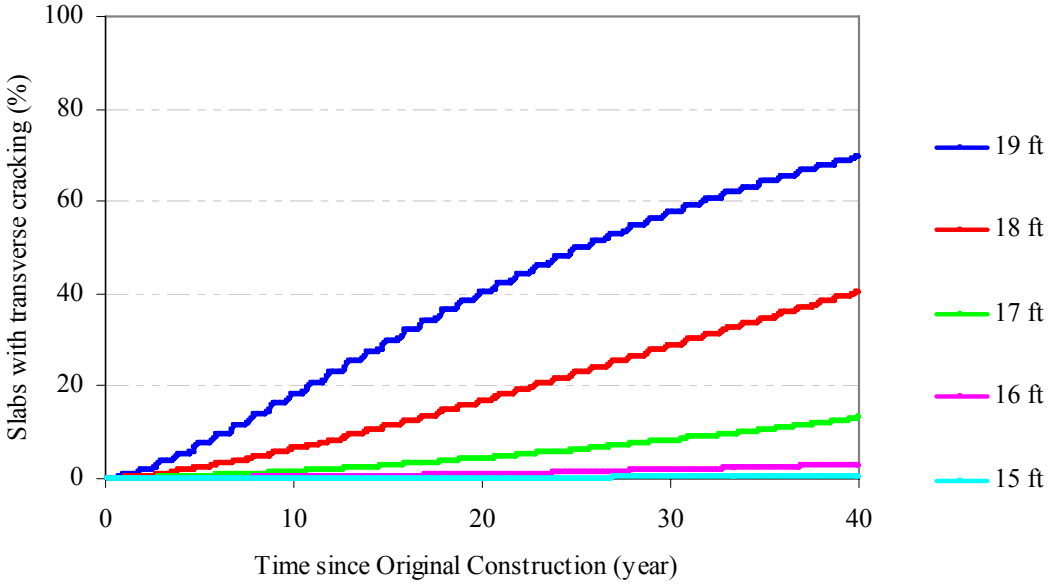

Figure 19 Default NCHRP 1-37 estimated transverse cracking under varying contraction joint spacings (9–in. undoweled slab, 9–in. granular base, 1.6 million ESALs/year/design lane, Seattle).

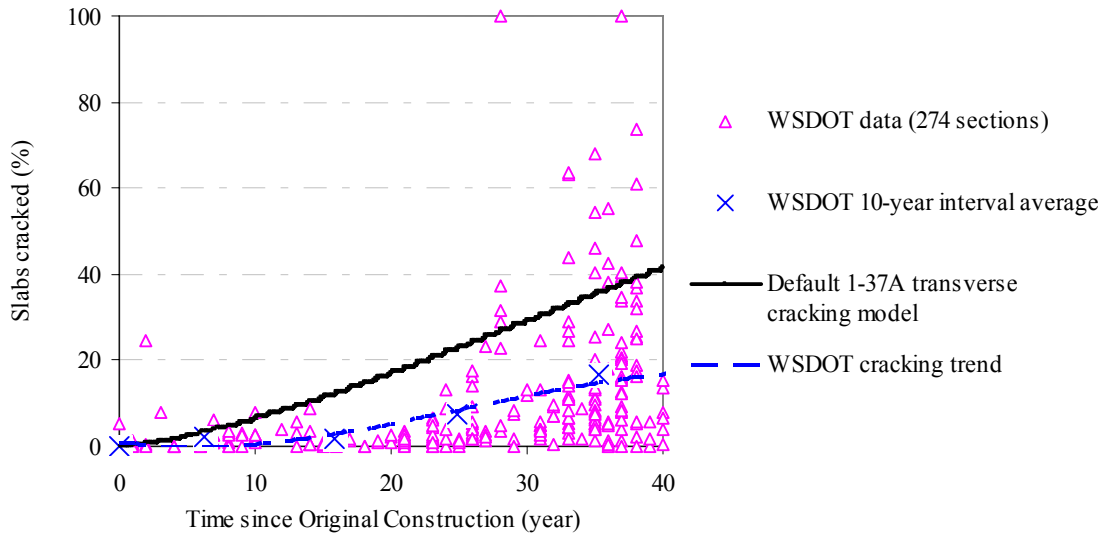

Figure 20 Percentage of cracked slab by age based on WSDOT data and the default NCHRP 1-37A transverse cracking prediction.

4.3.2: Faulting Model

All PCC slabs that have experienced significant faulting have been dowel bar retrofitted. These sections were originally built without dowels, thus, the condition data just before DBR were included in the undoweled group. This study assumed that the sections had 0.25 inches of faulting and 3.5 m/km IRI just before DBR (Pierce, 1999). Other undoweled WSDOT PCC slabs had substantially less faulting. Figure 21 shows undoweled WSDOT PCC pavement faulting data and the NCHRP 1-37A faulting estimation. WSDOT faulting data were averaged in each 10-year period, and the resulting trend was plotted. The trend is different from the default NCHRP 1-37A estimation both in trend shape and values.

By inputting typical WSDOT design parameters, it was found that the most critical factors of the model were base type, traffic load, and climate. Figures 22 to 24 indicate that slabs with asphalt treated base had better performance than those with a

granular base; slabs with light traffic loads had better performance than those with heavy traffic; and the slabs in Western Washington had better performance than those in Eastern Washington. (All of these trends were as expected).

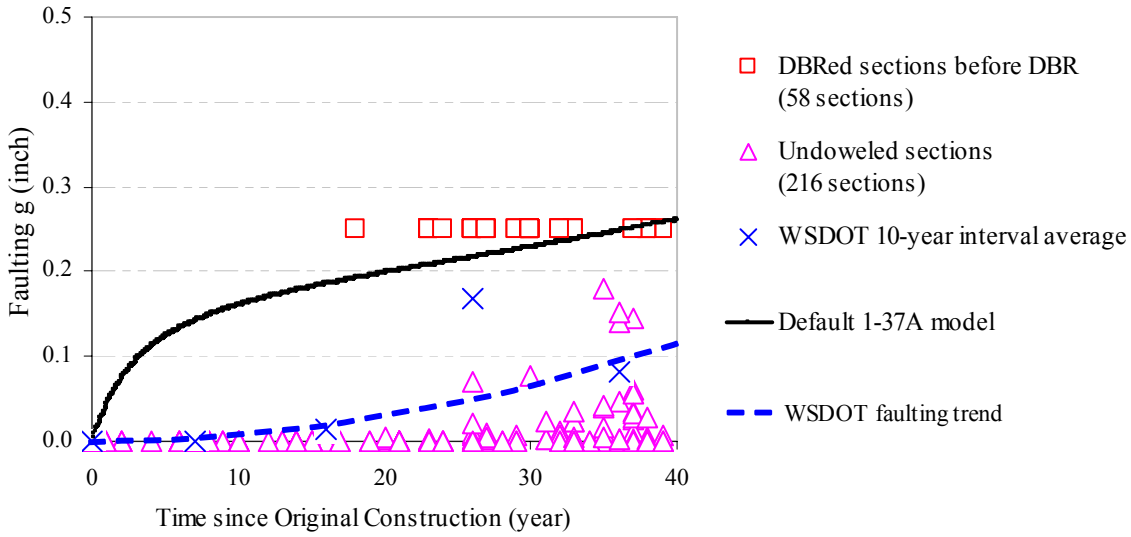

Figure 21 WSDOT faulting data and default NCHRP 1-37A prediction of faulting.

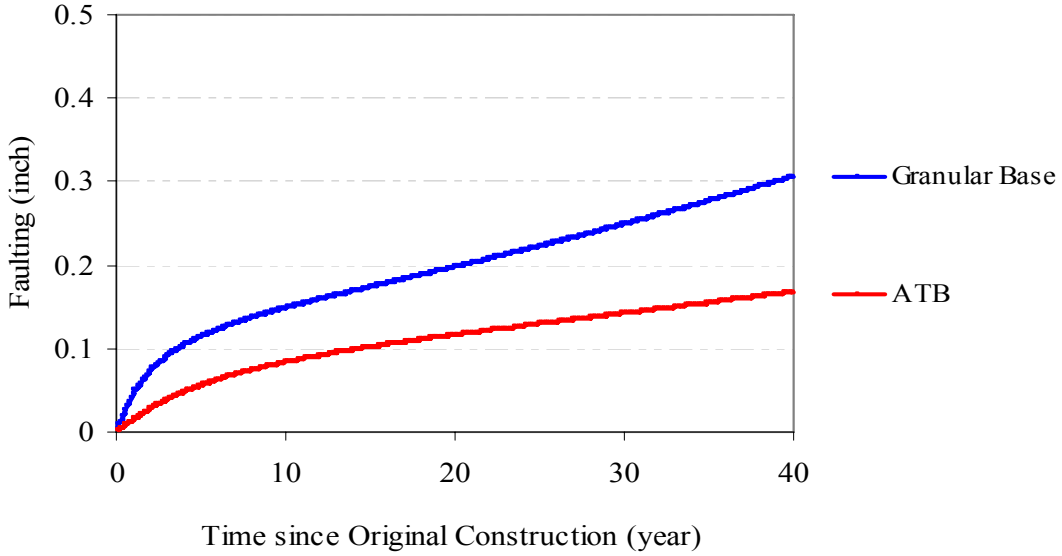

Figure 22 Default NCHRP 1-37A estimated faulting vs. base type (9" undoweled slab, 9" base, 15' joint spacing, 1.6million ESALs/year/design lane, Seattle).

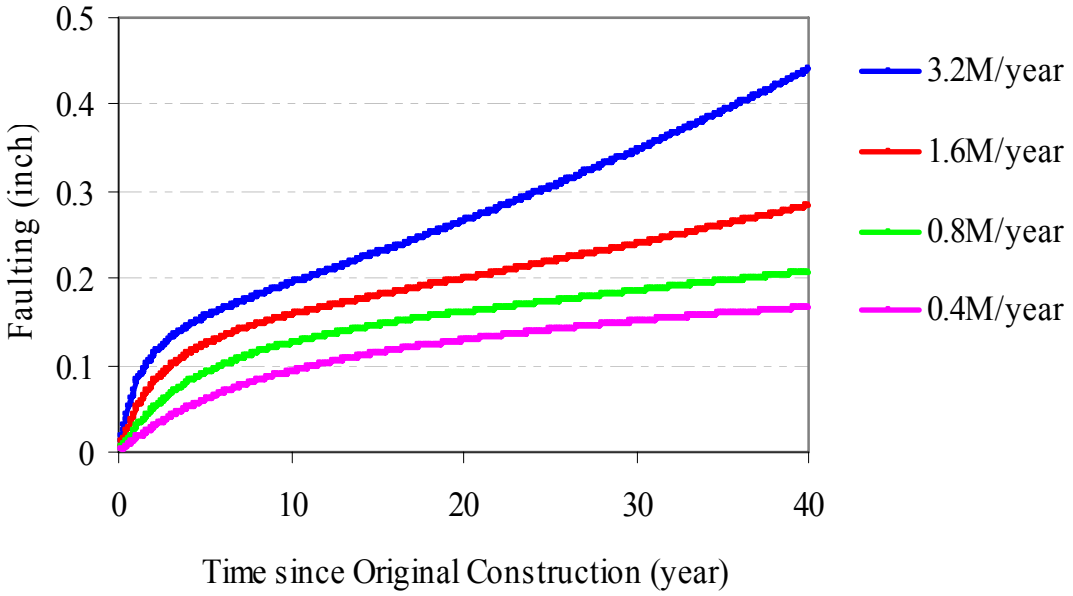

Figure 23 Default NCHRP 1-37A estimated faulting vs. ESALs (9–in. undoweled slab, 9–in. granular base, 15-ft. joint spacing, Seattle).

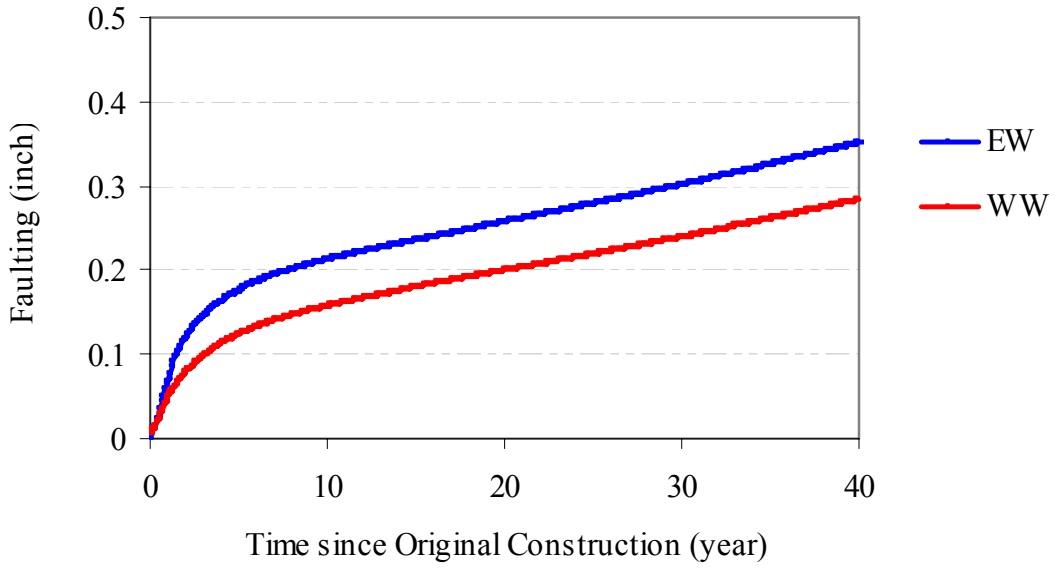

Figure 24 Default NCHRP 1-37A estimated faulting vs. climate (9–in. undoweled slab, 9–in. granular base, 15-ft. joint spacing, 1.6 million ESALs/year/design lane).

4.3.3: Roughness Model

The NCHRP 1-37A roughness model does not consider studded tire wear. The model only considers inputs of transverse cracking, spalling, faulting, and a related site factor (based mostly on local climate). The elasticity for each factor is 0.011 for cracking, 0.003 for spalling, 0.077 for faulting, and 0.003 for the related site factor, where faulting has a much larger elasticity than other factors. On the basis of elasticity, the roughness condition is mainly dependent on faulting.

As with the faulting model, the most critical input factors for roughness were base type, traffic load, and climate (figures 25, 26 and 27). The differences among inputs were quite similar to those of faulting, and the progression curves also had the same trend.

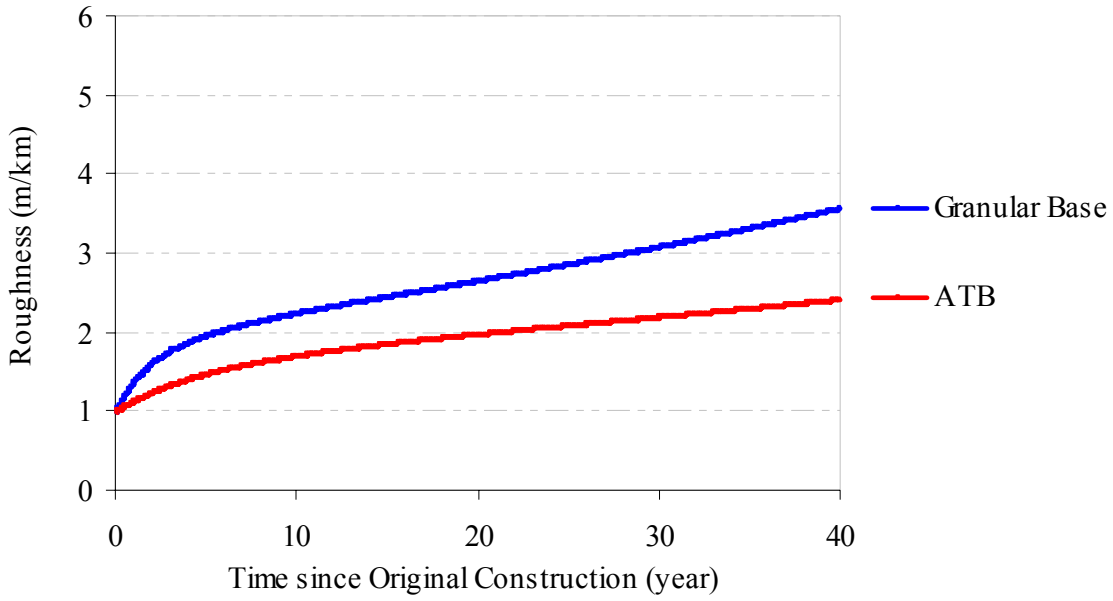

Figure 25 Default NCHRP 1-37A estimated IRI vs. base type (9–in. undoweled slabs, 9–in. base, 15-ft. joint spacing, 1.6 million ESALs/year/designate, Seattle).

Figure 26 Default NCHRP 1-37A estimated IRI vs. ESALs (9–in. undoweled slabs, 9–in. granular base, 15-ft. joint spacing, Seattle).

Figure 27 Default NCHRP 1-37A estimated IRI vs. climate (9–in. undoweled slab, 9–in. granular base, 15-ft. joint spacing, 1.6 million ESALs/year/design lane).

The default NCHRP 1-37A roughness model, along with inputs from the calibrated transverse cracking and faulting models, were used to estimate roughness. The trend is shown in Figure 28 along with WSDOT IRI data. DBR sections just before DBR were included. WSDOT IRI data were averaged in each 10-year time interval, and the resulting trend was plotted. The model estimates were smaller than the actual WSDOT data, and the estimated trend was similar to the calibrated faulting trend.

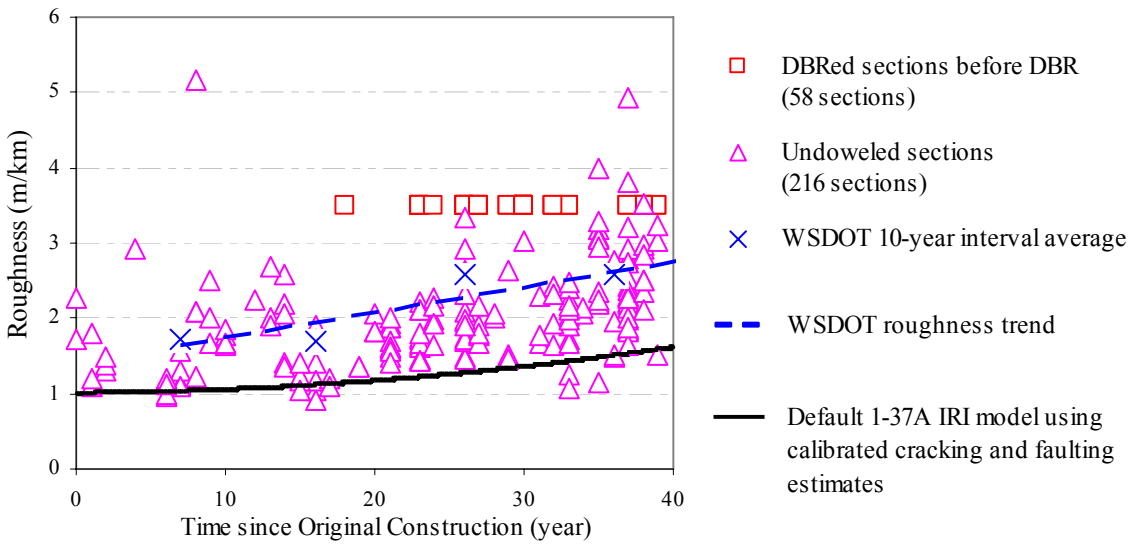

Figure 28 WSDOT IRI data and default NCHRP 1-37A prediction.

4.4: CALIBRATION

The NCHRP 1-37A software is designed to evaluate one pavement design at a time: the user provides a set of input values, and the damage over time is estimated. On the basis of the acceptability of these results, the user modifies input values until an acceptable damage progression over time is estimated. Because this process only allows for the evaluation of one pavement section at a time, a full econometric calibration of all WSDOT PCC pavements (which allows simultaneous calibration of multiple pavement

sections) is not possible. Rather, single sections of PCC pavement must be chosen, run through the NCHRP 1-37A software, and the resulting damage estimates compared to actual pavement condition. This method requires that these "calibration sections" be carefully chosen to represent typical design parameters and pavement condition data for a larger group of PCC pavements.

Test runs indicated that three representative calibration sections were needed: (1) undoweled pavements, (2) undoweled mountain pass pavements, and (3) DBR pavements. These three general groupings behaved significantly different from one another for at least one of the three distress modes (transverse cracking, faulting, or roughness). For each of these three groups, design input values and distress condition data from WSPMS data were averaged, then a section with values similar to the average was chosen as the representative section. This section was then used for calibration. Table 4 shows key design parameters and pavement condition data from these three representative calibration sections.

Table 4 Design Parameters and Distress Data of Calibration Sections for NCHRP 1-37A Models

Characteristic	Design Parameters and Distress Data		
Calibration Section Name	"Undoweled"	"Undoweled – MP"[a]	"DBR"
Dowel Type	Undoweled	Undoweled	DBR[b]
Base Type	Granular	Granular	Granular
Traffic Level	High	High	High
Climate	WW	Mountain Pass	WW
Route	I-5	I-90	I-5
Milepost	164.37 - 165.32	90.68 - 91.66	255.36 - 258.00
Direction	Northbound	Westbound	Southbound
Weather Station	Seattle (Boeing Field)	Stampede Pass	Bellingham
ESALs (per year per lane)	1,354,000	604,000	584,000
Age (years)	35	30	2
Soil Type[c]	SC	SM	SC
Slab Thickness (inches)	9	9	9
Base Thickness (inches)	11	9	7
2002 Cracking[d] (%)	6.4	25.5	3.3
2002 IRI (inches/mile)	196	220	88
2002 Faulting (inches)	0.054	0.25	0.001

Notes:

a. Mountain pass climate

b. Dowel bar retrofitted

c. From the Unified Soil Classification system

d. All Types of cracking

4.4.1: Validation

Calibration results were validated by using PCC pavement sections typical of several subgroups within each of the three calibration groups. Subgroups were formed by using the most critical input factors determined during bench testing:

- **Traffic level:** Traffic was divided into three categories on the basis of equivalent single axle loads (ESALs) in the design lane: high (>500,000 ESALs,), medium (>50,000 to 500,000 ESALs), and low (≤ 50,000 ESALs).

- **Base type.** Although there were a few isolated cement treated bases, most were either granular or asphalt treated base.
- **Climate.** Designated as either Eastern Washington (EW), Western Washington (WW), or mountain pass. For each validation section, tables 5 and 6 list the actual weather station data used.

This resulted in 18 possible validation subgroups for each calibration group. Because many of these subgroup populations were zero, there were far fewer actual validation subgroups. The following list shows each calibration group, followed by the calibration section listed as # 1, and then the validation sections (see tables 5 and 6):

- Undoweled (no low traffic level):
  1. High traffic, granular base in Western Washington (calibration section)
  2. High traffic, granular base in Eastern Washington
  3. Medium traffic, granular base in Eastern Washington
  4. High traffic, asphalt treated base in Western Washington
- Undoweled mountain pass (all were high traffic, granular base so another section with similar characteristics was chosen for validation):
  1. High traffic, granular base, mountain pass (calibration section)
  2. High traffic, granular base, mountain pass
- DBR (all had high traffic and granular base):
  5. High traffic, granular base, Western Washington (calibration section)
  6. High traffic, granular base, Eastern Washington
  7. High traffic, granular base, mountain pass

Table 5 Design Parameters and Distress Data of Undoweled Validation Sections for NCHRP 1-37A Models

Characteristic	Design Parameters and Distress Data			
Related Calibration Section	"Undoweled"	"Undoweled"	"Undoweled"	"Undoweled"
Dowel Type	Undoweled	Undoweled	Undoweled	Undoweled
Base Type	Granular	Granular	ATB[a]	Granular
Traffic Level	High	Medium	High	High
Climate	Eastern Washington	Eastern Washington	Western Washington	Mountain Pass
Route	I-82	US 82	I-5	I-90
Milepost	71.01 - 75.37	54.17 - 61.3	215.06 - 217.66	72.03 - 73.20
Direction	Southbound	Northbound	Northbound	Westbound
Weather Station	Ellensburg	Pullman /Moscow	Everett	Stampede Pass
ESALs (per year per lane)	516,000	394,000	727,000	604,000
Age (years)	21	23	26	35
Soil Type[b]	ML	ML	SC	SM
Slab Thickness (inches)	9	9	9	9
Base Thickness (inches)	6	6	4.2	9
2002 Cracking[c] (%)	2.6	1.3	2.6	25.5
2002 IRI (inches/mile)	101	101	129	220
2002 Faulting (inches)	0.025	0	0	0.25

Notes:

a. Asphalt treated base

b. From the Unified Soil Classification system

c. All Types of cracking

Table 6 Design Parameters and Distress Data of DBR Validation Sections for NCHRP 1-37A Models

Characteristic	Design Parameters and Distress Data	
Related Calibration Section	"DBR"	"DBR"
Dowel Type	DBR[a]	DBR[a]
Base Type	Granular	Granular
Traffic Level	High	High
Climate	Eastern Washington	Mountain Pass
Route	I-82	I-90
Milepost	3.29 - 10.31	58.59-60.00
Direction	Northbound	Westbound
Weather Station	Ellensburg	Stampede Pass
ESALs (per year per lane)	500,000	692,000
Age (years since DBR[a])	5	5
Soil Type[b]	ML	SM
Slab Thickness (inches)	9	9
Base Thickness (inches)	9	9
2002 Cracking[c] (%)	4	22.9
2002 IRI (inches/mile)	79	95
2002 Faulting (inches)	0	0

Notes:

a. Dowel bar retrofitted

b. From the Unified Soil Classification system

b. All Types of cracking

4.4.2: Iteration

Because the NCHRP 1-37A software only allows for the analysis of one pavement section at a time, calibration is an iterative process, as described in Figure 29. A set of calibration factors is chosen and then the design software is run on a calibration section. On the basis of results, the calibration factors are changed in order of high to low elasticity, and the design software is run again. When this process converges on an acceptable set of calibration factors, it is essentially repeated for the validation sections.

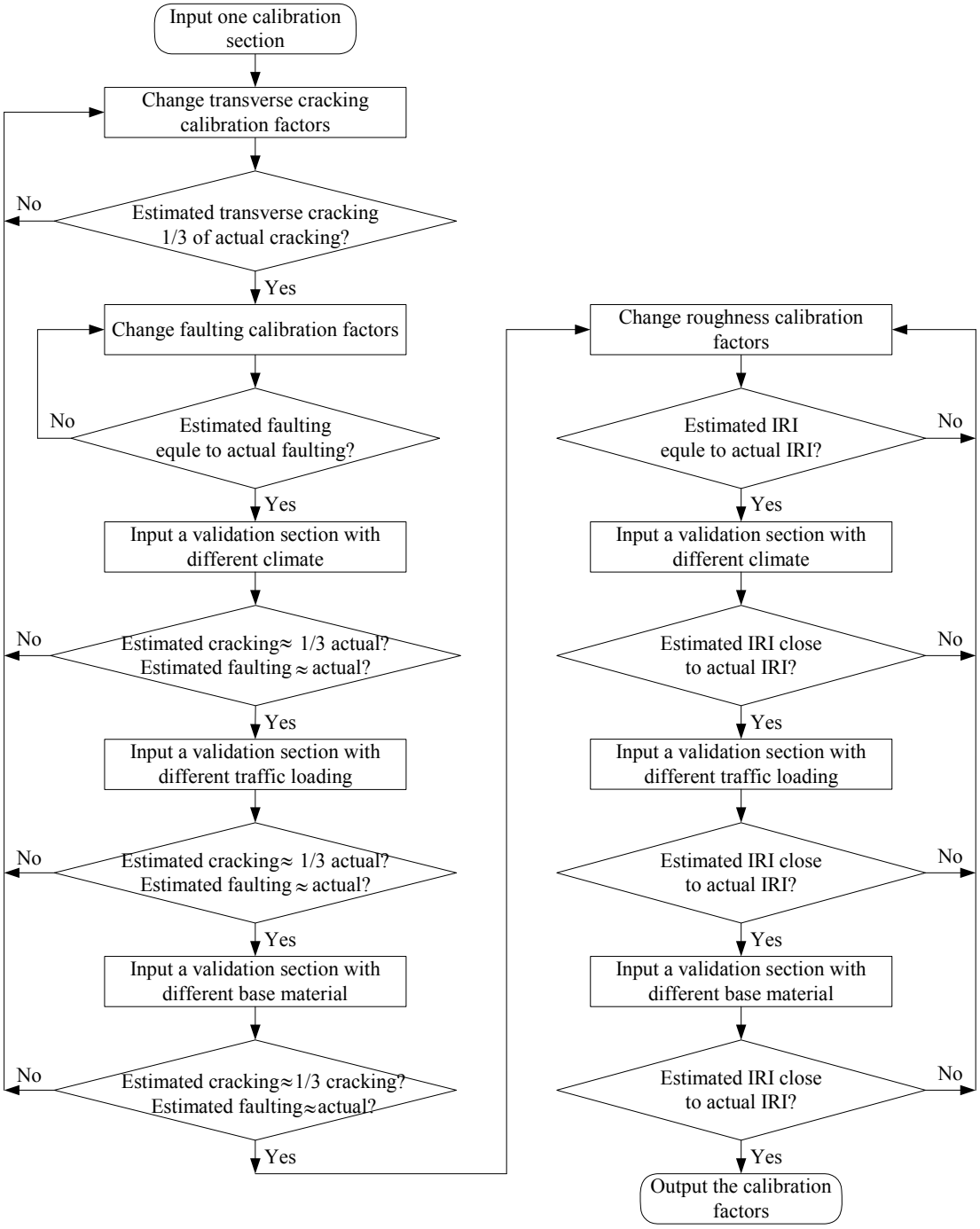

Figure 29 NCHRP 1-37A calibration methodology flowchart.

4.4.3: Calibration Results

This section discusses the calibration of each model. For each model, the calibration results are presented along with a description of WSDOT data and key assumptions and observations. For each calibration group, WSPMS data were averaged for each 10-year age interval (0 – 10 years, 10 – 20 years, 20 – 30 years, and 30 – 40 years). These averaged data points were used to generate a plot that the calibrated model should approximate. Table 7 shows default and final calibration factors for the three calibration groups.

Table 7 Final Calibration Factors for NCHRP 1-37A Models

Calibration Factor		Default for New Pavements	Undoweled	Undoweled – MP[a]	DBR[b,c]
Cracking	C_1	2	2.4	2.4	2.4
	C_2	1.22	1.45	1.45	1.45
	C_4	1	0.13855	0.13855	0.13855
	C_5	-1.68	-2.115	-2.115	-2.115
Faulting	C_1	1.29	0.4	0.4	0.934
	C_2	1.1	0.341	0.341	0.6
	C_3	0.001725	0.000535	0.000535	0.001725
	C_4	0.0008	0.000248	0.000248	0.0004
	C_5	250	77.5	77.5	250
	C_6	0.4	0.0064	0.064	0.4
	C_7	1.2	2.04	9.67	0.65
	C_8	400	400	400	400
Roughness[d]	C_1	0.8203	0.8203	0.8203	0.8203
	C_2	0.4417	0.4417	0.4417	0.4417
	C_3	1.4929	1.4929	1.4929	1.4929
	C_4	25.24	25.24	25.24	25.24

Notes:

a. Mountain pass climate
b. Dowel bar retrofitted
c. DBR faulting calibration factors are the same as default "restoration" values
d. Roughness calibration factors are the same as the default values

Transverse Cracking Model

Calibration results: The calibrated estimates for undoweled pavements are shown in Figure 30, and estimates for DBR sections are shown in Figure 31. Results showed very small amounts of transverse cracking, which match well with WSPMS data.

WSDOT data: WSPMS data do not distinguish between transverse and longitudinal cracking. Instead, it is the total of cracking of all types. Therefore, the NCHRP 1-37A model's predictions of transverse cracking should have been lower than or equal to WSPMS data. Attempts at direct comparison were confounded by WSPMS's inclusion of longitudinal cracking. Despite this, the NCHRP 1-37A estimated transverse cracking curve showed the same trend as the WSPMS data-generated curve shown in Figure 20.

Key assumptions: On the basis of observation and analysis for WSDOT-recorded PCC pavement images, it was assumed that 2/3 of all cracks were longitudinal. Therefore, the estimated transverse cracking was calibrated to 1/3 of WSPMS measured values.

Key observations: Longitudinal cracking is significant in WSDOT PCC pavements but is not modeled in the NCHRP 1-37A software. To accurately predict PCC pavement performance, especially in urban areas where high levels of longitudinal cracking are observed, a longitudinal cracking model is needed.

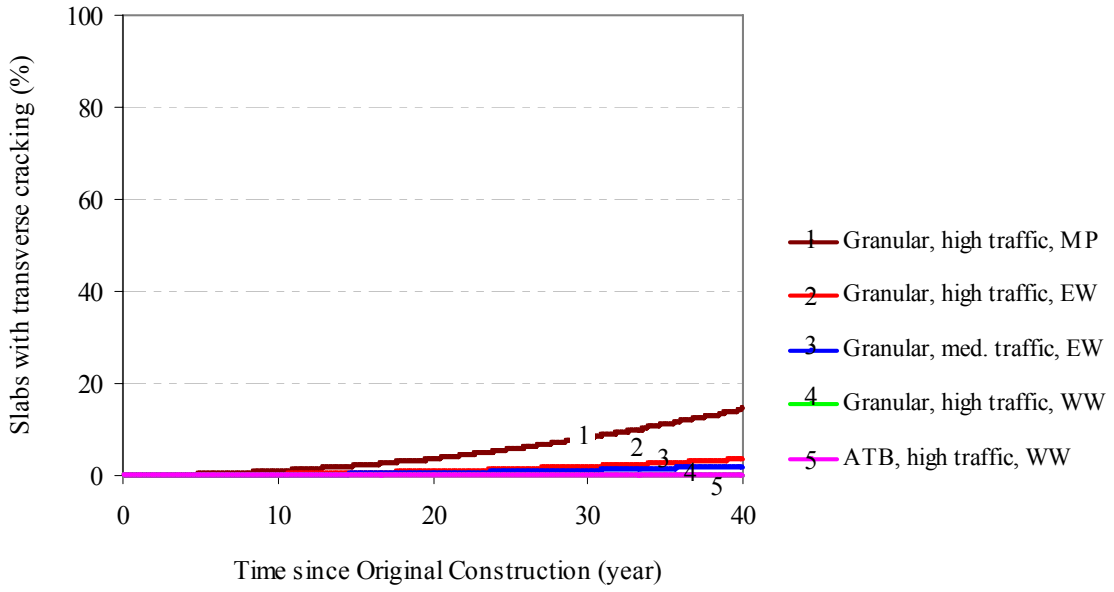

Figure 30 Calibrated NCHRP1-37A model estimates of transverse cracking for WSDOT undoweled PCC pavements.

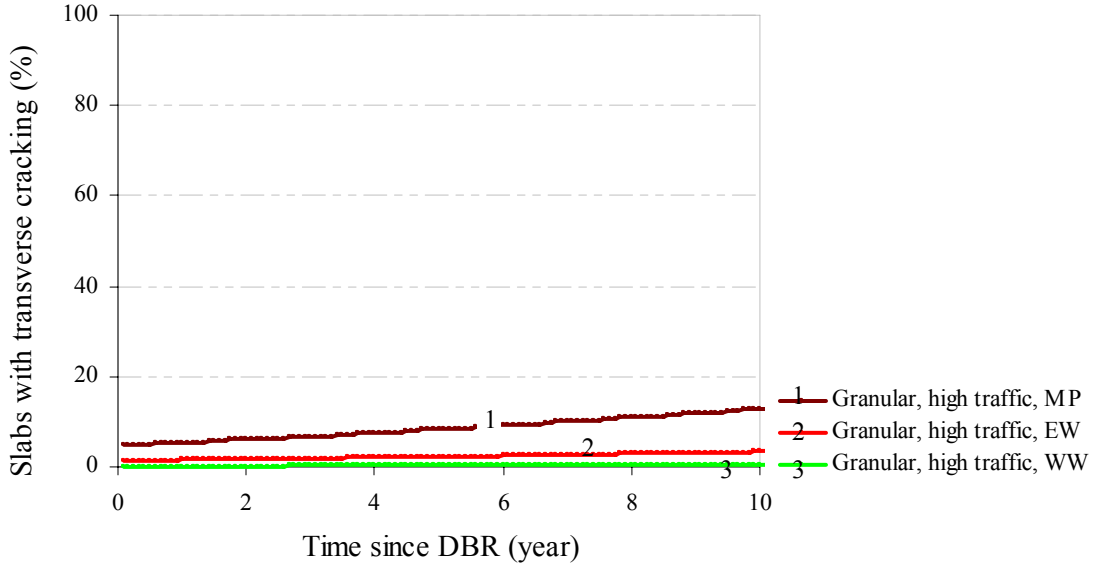

Figure 31 Calibrated NCHRP1-37A model estimates of transverse cracking for WSDOT DBR pavements.

Faulting Model

Calibration results: The calibrated estimates are shown in Figure 32. The faulting model was calibrated in three groups: undoweled, undoweled for mountain passes, and DBR. Results showed calibration factors significantly different from default values and a general agreement in level and progression with the WSPMS data for undoweled and undoweled mountain pass groups shown in Figure 21. All DBR sections are less than 10 years old, and the current faulting values are all very small. The default calibration factor for restored pavements ("restored" is a term used in the NCHRP 1-37A software to define any rehabilitated pavement) estimated very small amounts of faulting for the DBR group. This matched well with the actual conditions. Thus, the default calibration factors were used.

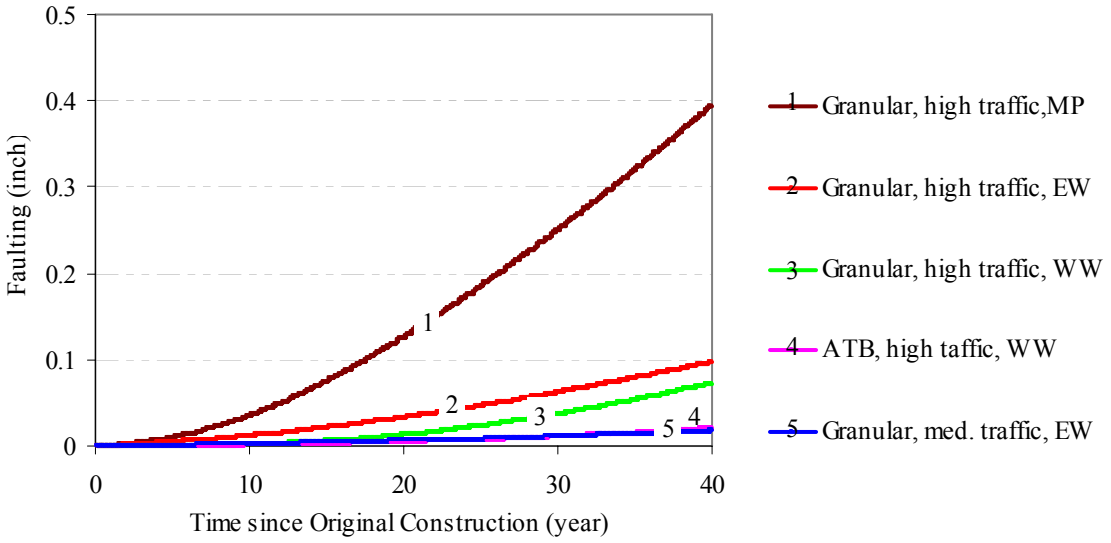

Figure 32 Calibrated NCHRP 1-37A model estimates of faulting for WSDOT undoweled PCC pavements.

WSDOT data: WSPMS data show generally low levels of faulting throughout the state (Figure 21). Most of the severely faulted PCC pavement has been dowel bar

retrofitted along with diamond grinding to remove the differential fault height. Figure 21 shows that both faulting values and progression for WSDOT PCC pavements are markedly different than the default NCHRP 1-37A model estimates.

Key assumptions: To accurately represent faulting, DBR pavements should be represented as undoweled PCC pavement with age and average fault height at the time of their retrofit. On the basis of WSDOT DBR criteria of (1) faulting greater than 0.25 inches or (2) IRI greater than 3.5 m/km, this calibration effort assumed that DBR sections had faulting of 0.25 inches at the time of DBR.

Roughness Model

Calibration results: The roughness model was calibrated in three groups: undoweled, undoweled for mountain passes, and DBR. The calibrated curves for undoweled pavements are shown in Figure 33. Figure 34 shows the estimation for DBR sections.

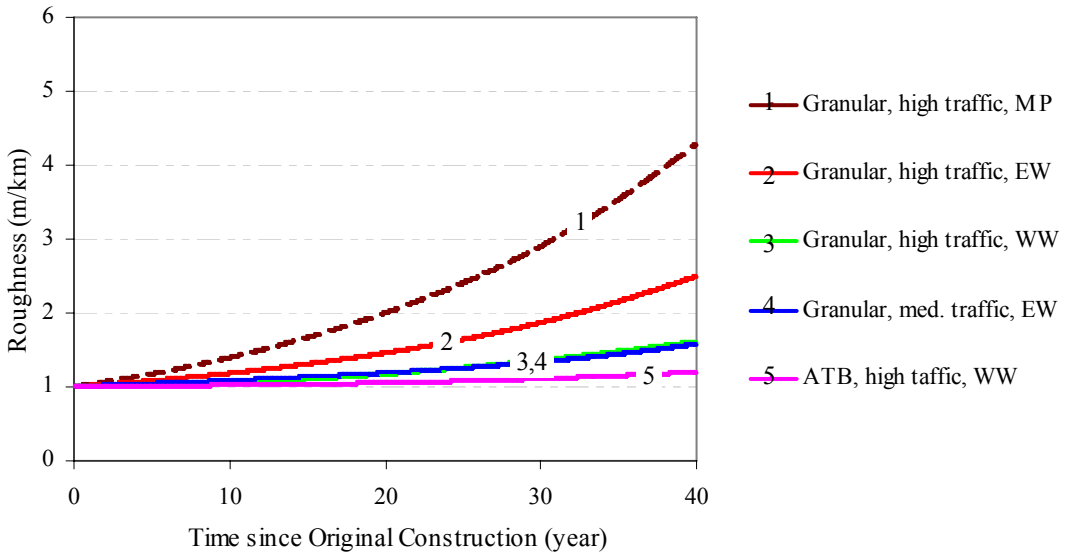

Figure 33 Calibrated model estimates of roughness for WSDOT undoweled PCC pavements (model uses calibrated cracking and faulting inputs and default roughness model).

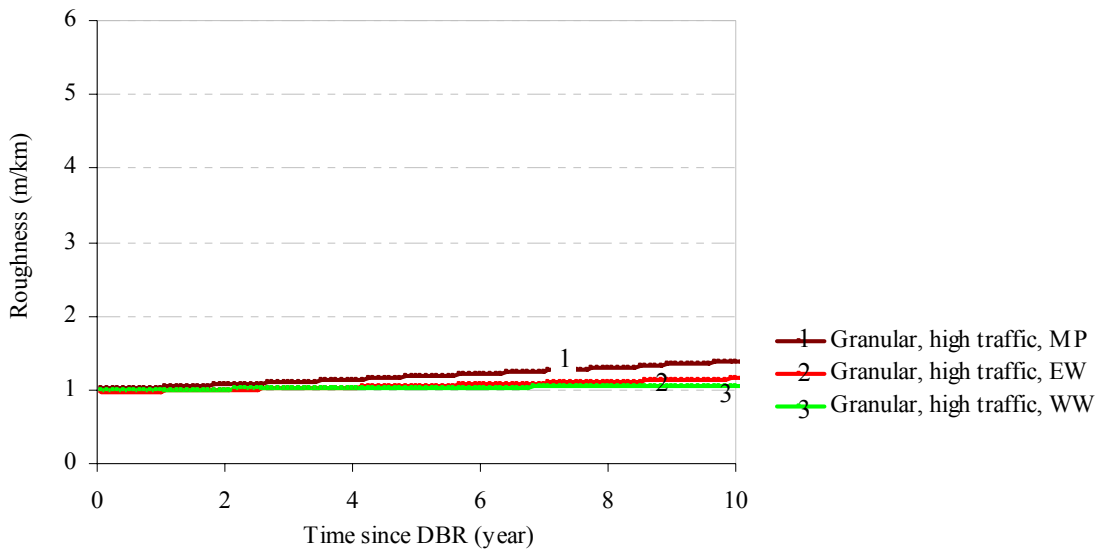

Figure 34 Calibrated model estimates of roughness for WSDOT DBR pavements (model uses calibrated cracking and faulting inputs and default roughness model).

WSDOT data: WSPMS data show that over half the IRI values are between about 1.5 and 3 m/km. These roughness data include the effects of studded tire wear, which may be significant. Because the DBR sections were diamond ground during DBR, the sections showed no significant roughness as of 2002.

Key assumptions: To reasonably represent faulting, DBR pavements were represented as undoweled PCC pavement with age and average roughness at the time of their retrofit. For calibration, the WSDOT DBR criteria were (1) faulting greater than 0.25 inches or (2) IRI greater than 3.5 m/km.

Key observations: NCHRP 1-37A software understandably does not model studded tire wear. As a result, WSDOT PCC pavements tended to be rougher than default roughness model predictions that used calibrated cracking and faulting estimates. When calibrated cracking and faulting estimates were used, the default roughness calibration

factors always underestimated actual WSDOT roughness, except for mountain passes and DBR sections. The differences between NCHRP 1-37A model predictions and actual data were too large to be accommodated by roughness model calibration. However, these differences were reasonably consistent for most representative sections (see Figure 35). Thus, it is suggested that this difference can be attributed to studded tire wear.

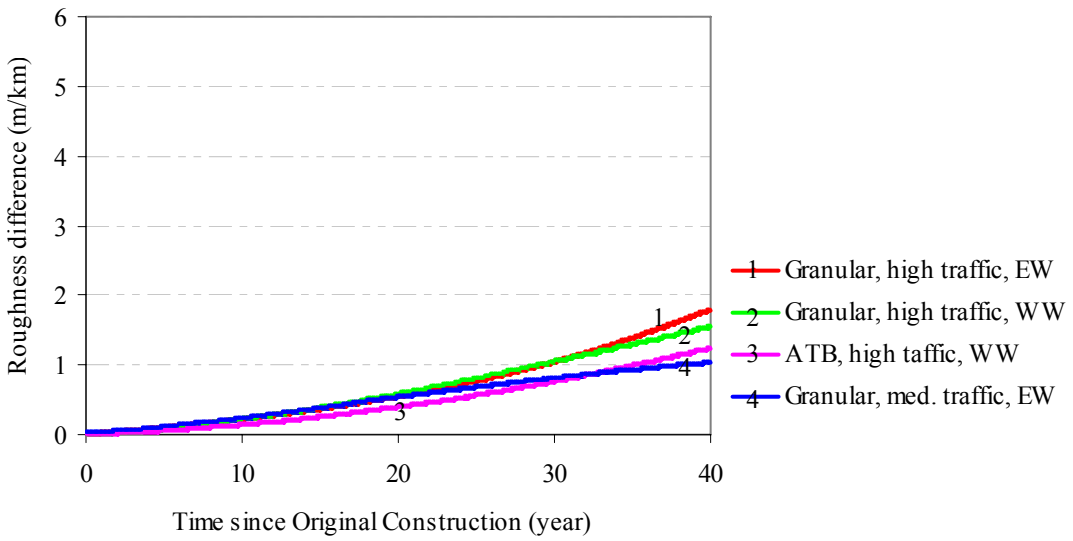

Figure 35 Differences in roughness between calibrated NCHRP 1-37A model and WSPMS data for validation sections; possibly due to studded tire wear.

4.4.4: Application to WSDOT PCC Pavement Rehabilitation and Reconstruction

Results from the calibration can be used to assist WSDOT in predicting PCC pavement performance, which will aid in making informed rehabilitation and reconstruction decisions. However, at this point in the calibrated software's development, it is not recommended for use as a design tool for WSDOT.

Transverse cracking. Predicted trends are likely to be accurate, although individual values may not be due to the assumed distribution of transverse vs.

longitudinal cracking on WSDOT PCC pavements. In areas where longitudinal cracking dominates (e.g., the Tacoma-Seattle-Everett I-5 corridor), WSDOT still does not have the ability to accurately predict crack progression or ultimate slab failure.

Faulting. Predicted trends and values seem fairly accurate.

Roughness. For undoweled pavements, predicted trends are reasonable, but actual values are under-predicted. While studded tire wear is believed to cause this, the hypothesis remains unproven. For DBR pavements, predicted trends and values are reasonable; however, the young age of these pavements (generally less than 13 years old) may indicate that studded tire wear has not had sufficient time to contribute significantly to roughness. Shortcomings in roughness prediction are less critical because, in general, PCC pavement failures are caused by excessive cracking and faulting. Roughness measurements serve as a secondary performance measure in Washington State.

In using the NCHRP 1-37A software, the lack of a longitudinal crack prediction model appears to be the most significant deficiency in WSDOT's ability to predict PCC pavement deterioration and ultimate failure. Although some initial work on longitudinal cracking has been done (Heath et al., 2003), to date there is no generally accepted model.

5: CONCLUSIONS AND RECOMMENDATIONS

5.1: CONCLUSIONS

A large portion of WSDOT PCC pavements are nearing the end of useful life and will soon require rehabilitation or reconstruction. In order to prioritize rehabilitation and reconstruction efforts, the rigid pavement portions of HDM-4 and NCHRP 1-37A software were studied. Significant findings are as follows:

1. The HDM PCC pavement deterioration models cannot be used by WSDOT for the following reasons:
    - The cracking model only considers transverse cracking, however, the main type of cracking in Washington State is longitudinal.
    - The estimated transverse cracking (percentage of slabs with transverse cracking) is much smaller than transverse cracking observed in Washington State.
    - The faulting model over-predicts actual faulting, and the calibration process cannot handle the large differences.
    - The spalling model estimates negative values for Western Washington, which is unrealistic.
    - For the HDM-4 roughness model, estimated transverse cracking, spalling, and faulting are main inputs. These calibrated models are not suitable for WSDOT conditions, so the roughness estimation is not suitable either. Furthermore, the roughness model does not consider studded tire wear.

2. The NCHRP 1-37A models were calibrated in an effort to predict future PCC pavement performance and the time of ultimate failure.
    - The WSDOT pavement network requires calibration factors different than the default NCHRP 1-37A values.
    - Pavement distress models can be calibrated for PCC pavements.
    - For WSDOT pavements, it is not advisable to apply one set of calibration factors to the entire network. Climate differences must be considered.
    - In general, the NCHRP 1-37A calibrated models can be used to predict deterioration of existing PCC pavements with the following exceptions:
        - NCHRP 1-37A software does not model longitudinal cracking, which is prominent in WSDOT PCC pavements.
        - The roughness model does not consider studded tire wear. This could conceivably be overcome by applying a standard studded tire wear offset based on pavement age; however, this method has not been adequately proven.

5.2: RECOMMENDATIONS

The current calibration results for NCHRP 1-37A PCC pavement models are encouraging; however, more work is required.

1. The calibration of transverse cracking needs to be improved by collecting actual transverse cracking data or finding the relationship between transverse cracking and total cracking of all types. This could improve not only the estimation of cracking, but

also that of roughness because the estimated transverse cracking is a component of the roughness model.

2. A method to add studded tire wear in the current roughness model for mountain passes and DBR sections is needed.

3. Input data more accurate than those available via the WSPMS are needed. This might lead to different calibration results.

    - The construction and rehabilitation month has significant effects on pavement performance. This study assumed that all PCC slabs were constructed or rehabilitated during the summer months.
    - Other states are studying the NCHRP 1-37A pavement deterioration models. Their results should be helpful for WSDOT. Of specific interest is the work under way in California and Texas.
    - Vehicle class distribution, hourly and monthly truck distribution, and axle load distribution have notable impacts on pavement performance. The current study used defaults in Level 3.
    - Laboratory test results are needed for more accurate material properties for the surface layer, base, and subgrade.
    - Default climate station data were used in the current study. The accuracy of the data needs further validation.
    - NCHRP requires the input of transverse cracking and roughness conditions before and after pavement rehabilitation. This study assumed that 10 percent of the slabs had transverse cracking, an IRI of 3.5 m/km before DBR, and an

IRI of 1.25 m/km after DBR. More accurate data are necessary for improved calibration results.

Additionally, the software package performs poorly. Sometimes it crashes without any error message. Some software debug work is needed. The current NCHRP 1-37A models are not perfect. They can still be improved by:

- considering the construction quality in the models, because it is a very important factor for pavement performance
- considering studded tire wear in the roughness model (or allowing this type of roughness to be added)
- allowing users to input the historical pavement deterioration conditions for better prediction.

REFERENCES

Al-Yagout, M.A., J.P Mahoney, L.M. Pierce, and M.E. Hallenbeck (2005), "Improving Traffic Characterization to Enhance Pavement Design and Performance: Load Spectra Development." Technical report WA-RD 600.1. Washington State Department of Transportation, Olympia, WA.

Greene, W.H. (2003), "Econometric Analysis." Published in Prentice Hall. Upper Saddle River, NJ.

Greene, W. H. (2002), "LIMDEP Version 8.0 Econometric Modeling Guide." Econometric Software, Inc. Plainview, NY.

Heath, A.C., J.R. Roesler and J.T. Harvey (2003) "Modeling Longitudinal, Corner and Transverse Cracking in Joined Concrete Pavements." The International Journal of Pavement Engineering, Vol. 4 March 2003, pp. 51-58. Hanover, NH.

Kannekanti, V. and J.T. Harvey (2005), "Sensitivity Analysis of 2002 Design Guide Rigid Pavement Distress Prediction Models." Draft report to California Department of Transportation. University of California Davis. Davis, CA.

Li, J., S.T. Muench, J.P. Mahoney, L.M. Pierce, N. Sivaneswaran, and G.C. White (2005) "Calibration and Application of HDM-4 for the Washington State Department of Transportation Road Network." Accepted for publication in Transportation Research Board, pending. Washington D. C.

Muench, S.T., J.P. Mahoney and L.M. Pierce (2003), "WSDOT Pavement Guide." Washington State Department of Transportation (WSDOT) Materials Laboratory, Olympia, WA.

Muench, S.T. and J.P. Mahoney (2004), "PCCP Models for Rehabilitation and Reconstruction Decision-Making," Proposal to WSDOT (unpublished).

Odoki, J.B. and H.G.R. Kerali (2000), "Highway Development and Management (HDM-4) Volume 4: Analytical Framework and Model Descriptions," the World Road Association (PIARC), Paris and the World Bank, Washington, D.C.

Pierce, L.M. (1999), "Dowel Bar Retrofit in Washington State - Summary of Findings." Paper prepared for presentation and publication at the Sixth International Purdue Conference on Concrete Pavements. Washington State Department of Transportation Materials Laboratory. Olympia, WA.

Washington State Department of Transportation (2003), "Washington State Pavement Management System (WSPMS)." Washington State Department of Transportation, Olympia, WA.

Washington State Department of Transportation (2005), "Update: Studded Tire use Illegal until November 2005." Web page on the Washington State Department of Transportation's web site. http://www.wsdot.wa.gov/winter/studtire/. Accessed 18 January 2005.

APPENDIX A: HDM-4 PCC PAVEMENT DETERIORATION MODELS

All WSDOT PCC pavements are Jointed Plain Concrete Pavements (JPCP), so only the following models were studied.

1. Transverse Cracking Model for Undoweled Pavements

$$PCRACK = Kjp_c * \frac{100}{1 + 1.41 * \left(\sum_{tg=1}^{G} \frac{NE4 * FREQ_{tg}}{\left(418.9 - 1148.6 * SR_{tg} + 1259.9 * SR_{tg}^2 - 491.55 * SR_{tg}^3\right) * 10^{2.13 * SR_{tg}^{-1.2}}} \right)^{-1.66}}$$

where,

PCRACK percent of slabs cracked.

Kjp_c calibration factor (default=1).

NE4 cumulative number of ESALs since construction of pavement, in millions 18-kip axles per lane.

$FREQ_{tg}$ frequency of each temperature gradient tg.

tg temperature gradient (tg=1, …, G).

SR_{tg} ratio between combined stress in slab and the Modulus of Rupture of concrete, for temperature gradient tg. Given by

$$\frac{f_{SB} * 100 * 3 * (1+\mu) * P * \left(\ln\left(\frac{E_c * SLABTHK^3}{100 * KSTAT * a_{eq}^4} \right) + 1.84 - \frac{4\mu}{3} + \frac{1-\mu}{2} + 1.18(1+2\mu)\frac{a_{eq}}{\ell} \right)}{(100 + LTE_{sh}) * MR * \pi(3+\mu) * SLABTHK^2}$$

$$* \left(0.454147 + \frac{0.013211 * \ell}{DW} + 0.386201 * \frac{a}{DW} - 0.24565 * \left(\frac{a}{DW}\right)^2 + 0.053891 * \left(\frac{a}{DW}\right)^3 \right)$$

$$+ \frac{0.5 * R_{tg} * f_{SB} * E_c * \alpha * \Delta T_s}{MR} * \left(1 - \frac{2\cos\left(\frac{3\sqrt{2} * JTSPAC}{\ell}\right)\cosh\left(\frac{3\sqrt{2} * JTSPAC}{\ell}\right) * \left(\tan\left(\frac{3\sqrt{2} * JTSPAC}{\ell}\right) + \frac{\sinh\left(\frac{3\sqrt{2} * JTSPAC}{\ell}\right)}{\cosh\left(\frac{3\sqrt{2} * JTSPAC}{\ell}\right)} \right)}{\sin\left(\frac{6\sqrt{2} * JTSPAC}{\ell}\right) + 2\sinh\left(\frac{3\sqrt{2} * JTSPAC}{\ell}\right)\cosh\left(\frac{3\sqrt{2} * JTSPAC}{\ell}\right)} \right).$$

where,

N_{tg} maximum number of 18 kip equivalent standard axle load repetitions during temperature gradient tg before flexural failure occurs (ESALs per lane).

ΔT_s adjusted difference in temperature at the top and bottom of the slab (^oF).

ΔT difference between the temperature measured at the top and bottom of the slab (^oF).

SLABTHK	slab thickness (inches).
a0 and a1	model coefficients based on climate zones; Use a0=7.68, a1=436.36 for EW, and a0=6.66, a1=218.18 for WW.
MR	modulus of Rupture of concrete (psi). Use $43.5(E_c/10^6) + 488.5 = 706$.
μ	Poisson's ratio. Use 0.15.
P	total load applied by each wheel of a single-axle dual wheel (lb). Use 9000.
SLABTHK	slab thickness (inch).
E_c	modulus of elasticity of concrete (psi). Use 5,000,000.
KSTAT	modulus of subgrade reaction (pci). Use $M_R/19.4$ (Pavement Guide).
a	load application radius for a single-wheel axle, in inches. Use $\sqrt{P/(\pi * p)} = \sqrt{9000/(3.14*100)} = 5.354$.
p	tire pressure (psi). 60 ~ 120 psi; Use 100.
SP	spacing between central wheels of dual wheel single axle (inches). Use 4.
LTE_{sh}	efficiency of load transfer between slab and edge support (for example, shoulder), (%)

Default: =20, if concrete shoulders are placed during initial construction

=10, if concrete shoulders are placed after initial construction

Use 0, assuming all shoulder are flexible.

DW average wheels location, given by the average distance of the exterior wheel to slab edge (inch). Use 22.

α thermal coefficient of concrete. Use $6*10^{-6}/{}^{o}F$.

λ intermediate parameter expressed in sexagesimal degrees.

JTSPACE average transverse joint spacing (ft). Use 15.

E_{base} modulus of elasticity of stabilized base (psi). Use 28,000 for Granular base, 400,000 for asphalt treatment base, and 1000,000 for cement treated base.

E_c modulus of elasticity of concrete (psi). Use 5,000,000.

f_{SB} adjustment factor for stabilized bases, and given by

$$\frac{2*\left(SLABTHK-\dfrac{0.5*SLABTHK^2+\dfrac{E_{base}}{E_c}*BASETHK*\left(SLABTHK+0.5*BASETHK\right)}{SLABTHK+\dfrac{E_{base}}{E_c}*BASETHK}\right)}{\left(SLABTHK^2+BASETHK^2*\dfrac{E_{base}*BASETHK}{E_c*SLABTHK}\right)^{0.5}}.$$

a_{eq} equivalent load application radius for a dual-wheel single axle (inches) radius of relative stiffness of the slab-foundation system (inch), and given by

$$a*\left[\begin{array}{l}0.909+0.339485*\frac{SP}{a}+0.103946*\frac{a}{\ell}-0.017881*\left(\frac{SP}{a}\right)^2-0.045229*\left(\frac{SP}{a}\right)^2*\frac{a}{\ell}\\+0.000436*\left(\frac{SP}{a}\right)^3-0.301805*\frac{SP}{a}*\left(\frac{a}{\ell}\right)^3+0.034664*\left(\frac{SP}{\ell}\right)^2+0.001*\left(\frac{SP}{\ell}\right)^2+0.001*\left(\frac{SP}{a}\right)^3*\frac{a}{\ell}\end{array}\right]$$

$$\left(\text{Limites}:\ 0\le\frac{SP}{a}\le 20,\ 0\le\frac{a}{\ell}\le 0.5\right)$$

ℓ radius of relative stiffness of the slab-foundation system (inch), and given by $\ell=\left[\frac{E_c*SLABTHK^3}{12*(1-\mu^2)*KSTAT}\right]^{0.25}$.

R_{tg} regression coefficient, and given by

$$\begin{aligned}&86.97*Y^3-\left(1.051*10^{-9}*E_c*dT*KSTAT+1.7487*dT\right)*Y^2\\&-\left(1.068-0.387317*dT-1.84*10^{-11}*E_c*dT^2*KSTAT+8.16396*dT^2\right)*Y\\&+\left(1.062-1.5757*10^{-2}*dT-8.76*10^{-5}*KSTAT+\left(1.17-0.181*dT\right)*10^{-11}*E_c*dT*KSTAT\right)\end{aligned}$$

Where,

$$Y=\frac{12*JTSPACE}{100*\ell}$$

$$dT=\alpha*\Delta T_s*10^5=\alpha*\left[\Delta T-a_0-\frac{a_1*(SLABTHK-2)}{SLABTHK^3}\right]*10^5.$$

2. Transverse Cracking Model for Dowel Bar Retrofitted Pavements

$$PCRACK = Kjp_c * \frac{100}{1 + 1.41 * \left(IDMA + \sum_{tg=1}^{G} \frac{NE4 * 10^{2.13 * SR_{tg}^{-1.2}} * FREQ_{tg}}{418.9 - 1148.6 * SR_{tg} + 1259.9 * SR_{tg}^2 - 491.55 * SR_{tg}^3} \right)^{-1.66}}$$

where,

IDMA estimate of past fatigue damage.

3. Faulting Model for Undoweled Pavements

$$\text{FAULT} = \text{Kjpn}_f * \text{NE4}^{0.25} * \left[\begin{array}{l} 0.2347 - 0.1516 * \text{Cd} - 0.00025 * \left(\frac{\text{SLABTHK}^2}{\text{JTSPACE}^{0.25}} \right) \\ -\left(0.0115 * \text{BASE} + 7.78 * 10^{-8} * \text{FI}^{1.5} * \text{PRECIP}^{0.25} \right) \\ -\left(0.002478 * \text{DAYS90}^{0.5} - 0.0415 * \text{WIDENED} \right) \end{array} \right]$$

where,

FAULT average transverse joint faulting (inch).

Kjpn_f calibration factor (default = 1).

NE4	cumulative ESALs since pavement construction (millions 18-kip axles per lane).
C_d	drainage coefficient modified AASHTO. Use 1.
SLABTHK	slab thickness (inch)
JTSPACE	average transverse joint spacing (ft). Use 15.
BASE	base type: not stabilized=0; stabilized=1.
FI	freezing index (^{o}F-day), Use 476.04 for EW, and 5.29 for WW.
PRECIP	annual average precipitation. Use 1.102 for EW, and 4.013 for WW.
DAYS90	number of days with mean temperature greater than 90°F. Use 7.73 for EW, and 0 for WW.
WIDENED	widen lane: not widened=0; if widened=1. Use 1

4. <u>Faulting Model for DBR Pavements</u>

$$\mathrm{FAULT} = \mathrm{Kjpn_f} * \mathrm{NE4}^{0.25} * \begin{bmatrix} 0.0628*(1-\mathrm{Cd}) + 3.673*10^{-9} * \mathrm{BSTRESS}^2 \\ +\left(4.116*10^{-6} * \mathrm{JTSPACE}^2 + 7.466*10^{-10} * \mathrm{FI}^2 \mathrm{PRECIP}^{0.5}\right) \\ -\left(0.009503*\mathrm{BASE} - 0.01917*\mathrm{WIDENED} + 0.0009217*\mathrm{AGE}\right) \end{bmatrix}$$

where,

FAULT	average transverse joint faulting (inch).

$Kjpn_f$ calibration factor for faulting (default =1).

NE4 cumulative number of ESALs since construction of pavement, in millions 18-kip axles per lane.

Cd drainage coefficient modified AASHTO. Use 1.

BSTRESS maximum concrete bearing stress, in the dowel-concrete system (psi), and given by

$$\frac{DFAC * P * LT * Kd * \left(2 + 12 * \left(\frac{Kd * DOWEL}{4 * E_s * 0.25 * \pi * \left(\frac{DOWEL}{2} \right)^4} \right)^{0.25} * CON * JTSPACE * \left(\frac{\alpha * TRANGE}{2} + \gamma \right) \right)}{4 * E_s * INERT * BETA}.$$

JTSPACE average transverse joint spacing (ft). Use 15.

FI freezing index (oF-day). Use 476.04 for EW, and 5.29 for WW.

PRECIP annual average precipitation (inch).

BASE base type: not stabilized=0; stabilized=1.

WIDENED widened lane. not widened=0; widened or shoulder provided during initial construction=1; concrete shoulders are placed after initial construction = 0.5. Use 0.

AGE number of years since pavement construction.

DFAC	distribution factor, given by $\frac{24}{\ell + 12}$.
ℓ	radius of relative stiffness of the slab-formulation system (inch).
P	total load applied by each wheel of a single-axle dual wheel (lb). Use 9000.
LT	percentage of load transfer between joint. Use 45.
Kd	modulus of dowel support, (pci). Use 1.5*106.
DOWEL	dowel diameter (inch). Use1.5.
Es	modulus of elasticity of dowel (psi). Use 2.9*107.
CON	adjustment factor due to base/slab frictional restraint. Use 0.8 for non-stabilized base, and 0.65 for stabilized base.
TRANGE	temperature range (the mean monthly temperature range obtained from data on the difference between the maximum and the minimum temperature for each month). Use 12.83 for EW, and 9.26 for WW.
γ	drying shrinkage coefficient of concrete. Use 0.00045.

5. Spalling Model

$$SPALL = Kjp_s * AGE^2 * JTSPACRE * 10^{-6} * \begin{pmatrix} 549.9 - 895.7 * (LIQSEAL + PREFESEAL) \\ +1.11 * DAYS90^3 * 10^{-3} + 375 * DWLCOR \\ +(29.01 - 27.6 * LIQSEAL) * FI \\ -(28.59 * PREFSEAL + 27.09 * SILSEAL) * FI \end{pmatrix}$$

Where,

SPALL	percent of spalled transverse joints
Kjps	calibration factor for spalling (default = 1).
AGE	age since pavement construction (year).
JTSPACE	average transverse joint spacing (ft).
LIQSEAL	presence of liquid sealant in joint: 0, if not present; 1, if present. Use 1.
PREFSEAL	presence of pre-formed sealant in joint: 0, if not present; 1, if present. Use 0.
DAYS90	number of days with temperature greater than 90oF.
DWLCOR	dowel corrosion protection: 0, if no dowels exist, or are protected from corrosion; 1 if dowels are not protected from corrosion. Use 0.
FI	freezing Index (oF-day). Uses 476.04 for EW, and 5.29 for WW.
SILSEAL	presence of silicone sealant in joint: 0, if not present; 1, if present. Use 0.

6. Roughness Model

$$RI_t = Kjp_r * \left(RI_0 + 2.6098 * TFAULT + 1.8407 * SPALL + 2.2802 * 10^{-6} * TCRACKS^3\right)$$

Where:

RIt	roughness at time t (inch/mile).
Kjpr	calibration factor for roughness (default t= 1)
RI0	initial roughness at the time of pavement construction (inch/mile). Use=98.9 as the default.
TFAULT	total transverse joint faulting per mile (in/mile), and given by $\frac{FAULT * 5280}{JTSPACE}$.
JTSPACE	average transverse joint spacing (ft).
SPALL	percentage of spalled joints.
TCRACKS	total number of cracked slabs per mile, and given by $\frac{PCRACK * 5280}{JTSPACE * 100}$.
PCRACK	the percentage of slabs cracked with transverse crack. Because WSDOT has no such data, the transverse cracking estimated by HDM (using the default calibration factor for cracking) is used here.

APPENDIX B: NCHRP 1-37A PCC PAVEMENT DETERIORATION MODELS

1. Transverse Cracking Model for Undoweled Pavements

$$TCRACK = (CRK_{Bottom-up} + CRK_{Top-down} - CRK_{Bottom-up} * CRK_{Top-down}) * 100 - CRK_{\mathrm{Re}\,paired}$$

$$CRK_{TDorBU} = \frac{100}{1 + C_4 FD_{TDorBU}^{C_5}} = \frac{100}{1 + (\sum \frac{n_{i,j,k,l,m,n}}{N_{i,j,k,l,m,n}})^{-1.68}} = \frac{100}{1 + (\sum \frac{n_{i,j,k,l,m,n}}{2.736 * 10^{C_1 (\frac{MR_i}{\sigma_{i,j,k,l,m,n}})^{C_2}}})^{-1.68}}$$

Where,

TCRACK	total cracking (percent).
$n_{i,j,k,l,m,n}$	applied number of load applications at condition i, j, k, l, m, n.
$N_{i,j,k,l,m,n}$	Allowable number of load applications at condition i, j, k, l, m, n.
MR_i	PCC modulus of rupture at age i (psi).
$\sigma_{i,j,k,l,m,n}$	applied stress at condition i, j, k, l, m, n.
i	age (accounts for change in PCC modulus of rupture, layer bond condition, deterioration of shoulder LTE).
j	month (accounts for change in base and effective dynamic modulus of subgrade reaction).
k	axle type (single, tandem, and tridem for bottom-up cracking; short, medium, and long wheelbase for top-down cracking).

l load level (incremental load for each axle type).

m temperature difference.

n traffic path.

C1, C2, C3, C4, C5 Calibration factors.

2. Transverse Cracking Model for DBR Pavements

$$TCRACK = (CRK_{Bottom-up} + CRK_{Top-down} - CRK_{Bottom-up} * CRK_{Top-down}) * 100 - CRK_{\mathrm{Re}\,paired}$$

$$CRK_{TDorBU} = \frac{1}{1 + C_4 FD_{TDorBU}^{C_5}} = \frac{1}{1 + (IDMA_{TDorBU} + \sum \frac{n_{i,j,k,l,m,n}}{N_{i,j,k,l,m,n}})^{-1.68}} = \frac{1}{1 + (IDMA_{TDorBU} + \sum \frac{n_{i,j,k,l,m,n}}{2.736 * 10^{C_1 (\frac{MR_i}{\sigma_{i,j,k,l,m,n}})^{C_2}}})^{-1.68}}$$

Where,

IDMATD estimate of past top-down fatigue damage.

IDMABU estimate of past bottom-up fatigue damage.

C1, C2, C3, C4, C5 Calibration factors.

3. Faulting Model

$$Fault_m = \sum_{i=1}^{m} \Delta Fault_i$$

$$= \sum_{i=1}^{m} C_{34} * (FAULTMAX_{i-1} - Fault_{i-1})^2 * DE_i$$

$$= \sum_{i=1}^{m} C_{34} * (FAULTMAX_0 + C_7 * \sum_{j=1}^{m-1} DE_j * (\log(1 + C_5 * 5^{EROD})^{C_6}) - Fault_{i-1})^2 * DE_i$$

$$= \sum_{i=1}^{m} C_{34} * \{C_{12} * \delta_{curling} * [\log(1 + C_5 * 5^{EROD}) * \log(\frac{P_{200} * WetDays}{P_s})]^{C_6} + C_7 * \sum_{j=1}^{m-1} DE_j * [\log(1 + C_5 * 5^{EROD})^{C_6}] - Fault_{i-1}\}^2 * DE_i$$

$$= \sum_{i=1}^{m} (C_3 + C_4 * FR^{0.25}) * \{(C_1 + C_2 * FR^{0.25}) * \delta_{curling} * [\log(1 + C_5 * 5^{EROD}) * \log(\frac{P_{200} * WetDays}{P_s})]^{C_6} + C_7 * \sum_{j=1}^{m-1} DE_j * [\log(1 + C_5 * 5^{EROD})^{C_6}] - Fault_{i-1}\}^2 * DE_i$$

where,

$Fault_m$ mean joint faulting at the end of month m (inch).

$\Delta Fault_i$ incremental change (monthly) in mean transverse joint faulting during month i (inch).

$FAULTMAX_i$ maximum mean transverse joint faulting for month i (inch).

$FAULTMAX_0$ initial maximum mean transverse joint faulting (inch).

$EROD$ base/subbase erodibility factor.

DE_i differential deformation energy accumulated during month i. Given bu $DE_i = k / 2(\delta^2_{loaded} - \delta^2_{unloaded})$

δ_{loaded} loaded corner deflection (inch).

$\delta_{unloaded}$ unloaded corner deflection (inch).

$\delta_{curling}$ maximum mean monthly slab corner upward deflection PCC due to temperature curling and moisture warping.

P_s overburden on subgrade (lb).

P_{200} percent subgrade material passing #200 sieve.

WetDays average annual number of wet days (greater than 0.1 in rainfall).

FR base freezing index defined as percentage of time the top base temperature is below freezing (32oF) temperature.

C1, C2, C3, C4, C5, C6, and C7 calibration factors.

4. Roughness Model

$$IRI = IRI_I + C1 * CRK + C2 * SPALL + C3 * TFAULT + C4 * SF$$

Where,

IRI predicted IRI (inch/mile).

IRII initial smoothness measured as IRI (inch/mile).

CRK percent slabs with transverse cracks (all severities).

SPALL percentage of joints with spalling (medium and high severities). Given by

$$SPALL = \left[\frac{AGE}{AGE + 0.01}\right]\left[\frac{100}{1 + 1.005^{(-12*AGE+SCF)}}\right]$$

SCF scaling factor based on site-, design-, and climate-related variables. Given by

$$\begin{aligned} SCF = & -1400 + 350 * AIR\% * (0.5 + PREFORM) + 3.4\, fc * 0.4 \\ & - 0.2(FTCYC * AGE) + 43 h_{PCC} - 536 WC\_Ratio \end{aligned}$$

AGE pavement age since construction (year).

AIR% PCC air content (percent).

PREFORM 1 if preformed sealant is present; 0 if not.

fc PCC compressive strength (psi).

FTCYC average annual number of freeze-thaw cycles

HPCC PCC slab thickness (inch).

WC_Ratio PCC water/cement ratio.

TFAULT total joint faulting cumulated per mi (inch).

C1, C2, C3, and C4 calibration factors.

SF site factor. Given by $AGE\left(1+0.5556*FI\right)\left(1+P_{200}\right)*10^{-6}$.

FI freezing index (^{o}F-days).

P200 percent subgrade material passing No. 200 sieve.