FACOLTÀ DI INGEGNERIA CIVILE E INDUSTRIALE
Dipartimento di Ingegneria Civile, Edile e Ambientale

Dottorato di Ricerca in
Infrastrutture e Trasporti

Identification of Cost-Effective Pavement Management Systems Strategies
A Reliable Tool to Enhance Pavement Management Implementations

Tutor:
Prof. Ing. Giuseppe Loprencipe

Dottorando:
Antonio Pantuso Romero

XXXI Ciclo – ANNO ACCADEMICO 2017/2018

Abstract

Modeling asset deterioration is a key business process within Transportation Asset Management. Road agencies should budget a large amount of public money to reduce the number of accidents and achieve a high level of service of the road system. Managing and preserving those investments is crucial, even more in the actual panorama of limiting funding. Therefore, roadway agencies have to increase their efforts on monitoring pavement networks and implementing data processing tools to promote cost-effective Pavement Management System (PMS) strategies. A comprehensive PMS database, in fact, ensures reliable decisions based on survey data, and sets rules and procedures to analyze data systematically. However, the development of adequate pavement deterioration prediction models has proven to be difficult, because of the high variability and uncertainty in data collection and interpretation, and because of the large quantity of data information from a wide variety of sources to be processed.

This research proposes a comprehensive methodology to design and implement pavement management strategies at the network level, based on road agency local conditions.

Such methodology includes the identification of suitable indexes for the pavement condition assessment, the design of strategies to collect pavement data for the agency maintenance systems, the development of data quality and data cleansing criteria to support data processing and, at last, the implementation spatial location procedures to integrate pavement data involved in the comprehensive PMS.

This work develops network level pavement deterioration models, and reviews road agency preservation policies, to evaluate the effectiveness of maintenance treatment, which is essential for a cost-effective PMS. It is expected that the resulting methodology and the developed applications, product of this research, will constitute a reliable tool to support agencies in their effort to implement their PMS.

Acknowledgments

First, I want to express my sincere gratitude to Professor Giuseppe Loprencipe and Professor Paola Di Mascio for your consistent guidance, your encouragement to reach new goals and for being trustful giving many opportunities for my professional growth during the Ph.D. I am grateful for your kindness, friendship, and sincerity which makes me feel confident and comfortable ready to face new challenges.

Secondly, I want to express my highest appreciation to Professor Dr. Gerardo W. Flinstch to invite me as a Visiting Research Scholar at the Center for Sustainable Transportation (CSTI) group at Virginia Tech Transportation Institute (VTTI) and let me to participate in working meetings, attend classes, help on testing campaigns, and meet international professors and researchers. Thank you for giving me a unique and valuable opportunity to improve the quality of my entire Ph.D. research. I appreciate your support, your kindness, guidance, consistent feedback, and help before, during, and after my stay. I extend my gratitude to Dr. Samer W. Katicha for helping me to increase my capacities in statistical data analysis and programming skills. My gratitude is special to Dr. Edgar de León Izeppi for your feedback and for your interest in my research offering me help when I was working after-hours, and more especially for those moments when I felt home with your family. Thank you for inviting me to celebrate Easter with you and your wife Bonnie, for inviting me to barbecues with your friends and to introduce me to the Hokie Nation. I appreciate all your kindness and your familiarness.

I am grateful to faculty members at Sapienza University of Rome, in special, to Professor Giuseppe Cantisani, Professor Laura Moretti, Professor Nicola Fiore, and Professor Antonio D'Andrea. Thank you for your sincere feedback and kind guidance what has helped me learn a lot during the Ph.D.

I want to thank all my Ph.D. Colleagues at Sapienza University of Rome, Pablo Zoccali, Lorenzo Puzzo and Giulia Del Serrone for your help, to suggest me new experimental approaches to enhance the quality of my research and for your friendship. Especially, I want to acknowledge my friend and colleague Claudio Mineo to share with me your culture, your kindness and your friendship in the last year of the Ph.D.

I extend my gratitude to my friends and colleagues in Blacksburg, VA, to be there always when I needed it, specially to Kenneth X. Velez to be like a brother and to Shivesh Shrestha for your friendship during my stay in Blacksburg. Thank you all for sharing with me breakfasts, lunches, dinners, at home and outside. Because of you all, I felt home in Virginia.

Furthermore, I want to express my gratitude to Dr. Guido Bonin to collaborate in my research, giving me useful and consistent feedback adding value to my research in practical terms. Thank you for your suggestions and encouragement to develop new applications. I appreciate your friendship and sincerity.

I want to express all my respect and appreciation to Professor Silvia Caro for giving enlightenment and new horizons to my research. You gave me valuable and useful tips that were an essential input for the critical lecture of the challenges of my research and their opportunities. You gave me a lot in a very short time during your stay as a Visiting Professor. Thank you for your sincerity, your natural happiness and your closeness, also after your departure.

I would also like to special thanks to Professor Germán Alfonso Ríos García, my Master's Thesis director for your help, expertise, encouragement, and engineering judgment. You gave me input and engineering criteria. Your rigor and pragmatism fundamental to become me an engineer. Thank you for your friendship and generosity sharing with me your culture.

Finally, I want to thank my family who is responsible for who I am and because without your support I would not have been able to complete the Ph.D. To my parents and my sister because you are the perfect family somebody can have. To my uncles and cousins in Lebrija and Roma. To my Italian grandparents for sharing their love with me every day. To the memory of my grandmother Dolores for everything given to me. You all helped me every day heeding me with my work and worries, giving me something that cannot be explained with words but fills my life and my heart. I also want to thank you, Delia, my friend, my English teacher, for sharing with me your superlative taste in fine arts, humanities, and broad aspects of life. You gave me a learned perspective on life, and even now, you inspire me to learn more every day. Especially, I want to thank you, Angela, my friend and companion, to follow me in my adventures, helping me to be myself and to improve me every day. Your light, your support, and love made possible this work.

Table of Contents

Chapter 1. Introduction, objective and background 1

1. Introduction, objective and background 1

1.1 Introduction 1
1.2 Problem definition and research approach (problem statement) 2
1.3 Research Objective 3
1.4 Results and significance 3
1.5 Dissertation organization 3

Chapter 2. Literature review 7

2. Literature Review 7

2.1 Pavement Management Systems 7
2.2 Pavement Condition Data Collection 9
2.2.1 Roughness / Longitudinal profile 13
2.2.2 Transverse profile: rutting 16
2.2.3 Vehicle Road Interaction Surface Characteristics 17
2.2.3.1 Surface Texture 17
2.2.3.2 Skid Resistance 20
2.2.4 Pavement surface distresses 21
2.2.5 Pavement structure evaluation 25
2.2.6 Incorporating Structural Capacity Performance Measures 30
2.3 Pavement Condition Assessment 34
2.3.1 ASTM D6433 Standardized Procedure: Pavement Condition Index 34
2.3.2 Virginia DOT: Critical Condition Index (CCI) 37
2.3.3 National Park Services: Pavement Condition Rating (PCR) 38
2.3.4 Wisconsin DOT: Pavement Surface Evaluation and Rating (PASER) 40
2.3.5 Pavement Condition Assessment in the European Scene 41
2.3.6 Pavement Management Systems in Urban Networks 41
2.4 Pavement Deterioration Modeling at The Network Level 44
2.5 Pavement Preservation Strategies Experiences 46
2.6 Pavement Data Management Issues: Network Level Considerations 47
2.6.1 Linear Referencing Systems and Spatial Positioning Systems 48
2.6.2 Sampling Intervals and Sectioning, Survey Frequency 49
2.7 Summary of Literature Review 49

Chapter 3. Pavement Distress Identification Guidelines and Condition Assessment in Urban Areas 53

3. Pavement Distress Identification Guidelines and Condition Assessment in Urban Areas 53

3.1 Introduction 53
3.2 Objective and Framework 53

3.3 The PCI Method: Visual Survey on Valentano Road Network ... 55
3.4 Pavement Distress Distribution Analysis ... 56
3.5 Algorithm for the Computerization of PCI Calculation ... 59
3.5.1 VBA PCI Calculation Standardized Procedure ... 59
3.5.2 Deduct Value - Polynomial Interpolating Curve Hermite Procedure ... 60
3.6 Definition of New Distress Identification Catalogue for Asphalt Concrete Pavements in Urban Areas ... 66
3.7 Validation of the Computerization of PCI Calculation ... 70
3.8 Discussion ... 72
3.9 Conclusions ... 75

Chapter 4. Pavement Management System Considering the Vehicle Operating Costs: A Case of Study in Urban Areas ... 77

4. Pavement Management System Considering the Vehicle Operating Costs: A Case of Study in Urban Areas ... 77

4.1 Introduction ... 77
4.2 Methodology for PMS Implementation in Urban Areas ... 78
4.3 Results of a Pilot PMS Implemented in Urban Road Network ... 80
4.3.1 Repair and Maintenance Planning ... 80
4.3.2 Life-Cycle Cost Analysis—Vehicle Operating Costs Estimation ... 85
4.4 Discussion ... 88
4.5 Conclusions ... 89

Chapter 5. Pavement Management Data Strategies: Guidelines to Enhance Data Processing and Interpretation ... 91

5. Pavement Management Data Strategies: Guidelines to Enhance Data Processing and Interpretation ... 91

5.1 Pavement Data Management Methodologies Studied During the Ph.D. Program ... 91
5.2 Spatial Integration Tools ... 92
5.2.1 Implementation of LRS System from Pavement Survey Data ... 93
5.2.2 Enhancing Data Integration using Spatial GPS Matching Procedures ... 94
5.2.3 Pavement Database Integration – Merging Data From Different Sources ... 96
5.3 Pavement Historical Data Management ... 98
5.4 Data Visualization Tools ... 100
5.5 Discussion and Conclusions ... 102

Chapter 6. Analysis of Pavement Condition Survey Data for Effective Implementation of a Network Level Pavement Management Program For a Developing Country ... 105

6. Analysis of Pavement Condition Survey Data for Effective Implementation of a Network Level Pavement Management Program For a Developing Country ... 105

6.1 Introduction ... 105
6.2 Pavement Survey Data Available ... 106
6.3 Pavement Data Analysis and Processing ... 107
6.3.1 Analysis of Pavement Functional Evaluation: PAvement Condition Rating ... 107

6.3.2 Analysis of Traffic Data 108
6.3.3 Analysis of Deflection Data: Required Overlay Parameter 109
6.4 Methodology for Determining Homogeneous Sections and a Decision-Making Process Based on Survey Data111
6.4.1 Definition of Pavement Homogeneous Section Criteria 111
6.4.2 Definition of the Decision-making Process 112
6.5 Conclusions 115

Chapter 7. Development of Pavement deterioration curves of flexible pavements using the Linear Empirical Bayesian Approach at the Network Level 117

7. Development of Pavement Deterioration Curves of Flexible Pavements using the Linear Empirical Bayesian Approach at the Network Level 117

7.1 Introduction 117
7.2 Overview Data Analysis and Methodology Used 118
7.2.1 Analysis of CCI data: data cleansing and filtering process considerations 119
7.2.2 Data Cleansing and Filtering Process 119
7.2.3 The Empirical distribution of Pavement Condition: Negative Binomial (Poisson-Gamma model) 122
7.2.4 Negative Binomial model Development and Virginia DOT Pavement deterioration models 126
7.2.5 Empirical Bayesian Approach 126
7.3 Deterioration Model Development 130
7.4 Validation of modelling procedure 132
7.5 Discussion and Conclusions 137

Chapter 8. Design Pavement Management Strategies: Performance Assessment for Network Level Pavement Preservation 139

8. Design Pavement Management Strategies: Performance Assessment for Network Level Pavement Preservation 139

8.1 Introduction 139
8.2 Methodology 140
8.3 Results 142
8.3.1 Pavement Performance Models 142
8.3.2 VDOT Pavement Maintenance Policies Effectiveness 148
8.3.2.1 Performance Jump 149
8.3.2.2 Maintenance Treatment Service Life 151
8.3.2.3 Cost-Benefit Analysis 151
8.4 Discussion of Results 154
8.5 Recommendations for Future Research 155

Chapter 9. Conclusions, recommendations and Future Research 157

9. Conclusions, recommendations and Future Research 157

9.1 Summary of Dissertation 157
9.2 Findings 157
9.3 Conclusions 159

9.4 Recommendations for Future Research 160

Appendix A. Distress Identification Manual for Urban Road Pavements 169

Appendix B. Software calculation PCI documentation 179

Appendix C. Code used in Chapter 6 181

Appendix D. Code used in Chapter 7 185

Appendix E. Empirical Bayes Application 193

List of Figures

Figure 2-1. Pavement Life Cycle: Consequences of Poor Maintenance Timing (after Shahin (2005)). 8

Figure 2-2. Components of pavement management systems (after (AASHTO 2012)) 9

Figure 2-3. Information Quality Level (after Bennett and Paterson (2003))....................................... 10

Figure 2-4. Quarter-car model for the calculation of the IRI (after Cantisani and Loprencipe (2010))14

Figure 2-5. High speed profiler participants at Surface Properties Round up (source: TPF-5(141) Final Report).. 15

Figure 2-6. Proposal for new speed-related IRI thresholds, considering the a_{WZ} limits established by technical standards (after Cantisani and Loprencipe (2010))... 16

Figure 2-7. Pavement Surface Characteristics Influence Wavelength (after Cantisani, D'Andrea et al. (2016)).. 17

Figure 2-8. Surface Texture Key mechanisms of Pavement Friction (after Hall, Smith et al. (2009)). 18

Figure 2-9. Paved Surface Frictional properties measurement apparatus: a) Circular Meter and b) Dynamic Friction Tester... 19

Figure 2-10. Representation of the profile characteristics for the calculation of MPD (after Flintsch, de Leon et al. (2003)).. 19

Figure 2-11. Virginia SCRIM truck at VTTI Smart Road Testing Facility (Source: www.wdm.co.uk) 21

Figure 2-12. ARAN instrumented vehicle (a) and WiseCrax automated crack detection software (b) (source: Fugro Roadware).. 24

Figure 2-13. LCMS system imaging processing (a) and LCMS sensors mounted on an instrumented vehicle (b) (source: Pavemetrics)... 24

Figure 2-14. FWD testing equipment surface deflection bowl (after Choi, Wu et al. (2009))............. 25

Figure 2-15. Traffic Speed Deflectometer ANAS (Source: ANAS Gruppo FS Italiane) 30

Figure 2-16. Standard and customized PCI rating scale (after Loprencipe and Pantuso (2017))......... 35

Figure 2-17. Distribution of road types on an Italian road network (Loprencipe, Pantuso et al. 2017) 41

Figure 3-1. Urban Pavement Distress Identification and Assessment Methodology based on the PCI (after Loprencipe and Pantuso (2017)).. 54

Figure 3-2. Valentano's Network characterization 55

Figure 3-3. Road pavement observed distresses distribution by Pavement Condition Index (PCI) condition in urban areas. Percentage of observed distress in: (a) Adequate PCI sections; (b) Medium PCI sections; (c) Unsatisfactory PCI sections... 58

Figure 3-4. Logarithm Density- Deduct Value Polynomial Interpolating Curve Hermite Procedure... 60

Figure 3-5. Parametric plane (*u*, *DV*) Polynomial Interpolating Curve Hermite procedure................. 61

Figure 3-6. ASTM D6433 Digitalized DV Curves 65

Figure 3-7. Regressions on 109 SU between PAVER™ PCI and calculated PCI using VBA program71

Figure 3-8. Example of manhole distress identification guidelines, Manholes.................................... 73

Figure 4-1. Comparative of traffic influence on VOC 78

Figure 4-2. Framework of the proposed PMS 79

Figure 4-3. PCI Rating Scale for Urban PMS Applications... 80

Figure 4-4. PCI condition calculated for each strategy 83

Figure 4-5. PCI section rating for each strategy at the considered period of study 85

Figure 4-6. Diagrams to evaluate the effect of roughness on VOC 87

Figure 4-7. Vehicle Operating Costs calculated for each strategy. 88

Figure 5-1. VDOT web-based comprehensive PMS database 92

Figure 5-2. Algorithm GPS procedure LRS implementation through GPS data 95

Figure 5-3. Results of the application of GPS matching procedure for the implementation of LRS 95

Figure 5-4. Algorithm Data Integration between tables within a PMS 96

Figure 5-5. LRS data integration algorithm; get the nearest milepost 98

Figure 5-6. Data Processing Analysis: Typical Pavement Performance Situations 99

Figure 5-7. Use of Geographical Information Systems to support pavement decision-making at the network level 101

Figure 5-8. Web-based API interface uploaded online to present the results of the study 102

Figure 6-1. Pavement Survey Equipment 107

Figure 6-2. Accumulated Difference Centered Deflection along road chainage 111

Figure 6-3. Pavement Condition Rating obtained from Survey along road chainage. 112

Figure 6-4. Pavement Condition Rating and required overlay for Homogeneous Sections along Road Chainage. 114

Figure 7-1. Pavement CCI as function of pavement age of Interstate Network 120

Figure 7-2. Pavement CCI as function of pavement age of Primary Network 120

Figure 7-3. Pavement CCI as function of pavement age of Secondary Network 121

Figure 7-4. Distribution of Pavement Deterioration (DI) of the Interstate Network 122

Figure 7-5. Distribution of Pavement Deterioration (DI) of the Primary Network 122

Figure 7-6. Distribution of Pavement Deterioration (DI) of the Secondary Network 123

Figure 7-7. Empirical Distribution of pavement *CCI* and Negative Binomial Distribution fit along Pavement *age* for the Interstate System 124

Figure 7-8. Empirical Distribution of pavement *CCI* and Negative Binomial Distribution fit along Pavement *age* for the Primary System 124

Figure 7-9. Empirical Distribution of pavement *CCI* and Negative Binomial Distribution fit along Pavement *age* for the Secondary System 125

Figure 7-10. Example of application of the Empirical Bayesian Approach 128

Figure 7-11. Error in estimating the Pavement Condition with the EB Method as a Function of Estimated Error Variance 129

Figure 7-12. Comparison between *CCI* deterioration curves using Virginia DOT default models and Negative Binomial Regression model for the Interstate System 131

Figure 7-13. Comparison between *CCI* deterioration curves using Virginia DOT default models and Negative Binomial Regression model for the Primary System 131

Figure 7-14. Comparison between *CCI* deterioration curves using Virginia DOT default models and Negative Binomial Regression model for the Secondary System 132

Figure 7-15. Illustration of the Approaches to Calculate the Estimate of Future Pavement Condition 134

Figure 7-16. Color Map Plot of the Interstate Network Pavement Family; a) Raw observed pavement condition data; b) Pavement condition modeled using the LEB approach.. 136

Figure 7-17. Color Map Plot of the Primary Network Pavement Family; a) Raw observed pavement condition data; b) Pavement condition modeled using the LEB approach.. 136

Figure 7-18. Color Map Plot of the Secondary Network Pavement Family; a) Raw observed pavement condition data; b) Pavement condition modeled using the LEB approach.. 136

Figure 8-1. Pavement Maintenance Treatment Effectiveness Calculation.. 141

Figure 8-2. Preventive Maintenance Post-Treatment Performance Model for Primary System. 143

Figure 8-3. Corrective Maintenance Post-Treatment Performance Model for Primary System. 143

Figure 8-4. Restorative Maintenance Post-Treatment Performance Model for Primary System. 143

Figure 8-5. Reconstruction Maintenance Post-Treatment Performance Model for Primary System. . 144

Figure 8-6. Post-Treatment Performance Curves Comparison 144

Figure 8-7. Enhanced Post-Treatment Piecewise Performance models 146

Figure 8-8. Enhanced Post-Treatment Corrective Maintenance Performance Model........................ 147

Figure 8-9. Enhanced Post-Treatment Restorative Maintenance Performance Models...................... 147

Figure 8-10. Enhanced Post-Treatment Reconstruction Performance Model 148

Figure 8-11. Pavement Preservation Policies (age, CCI) before M, R&R treatment (a) Marginal Histogram and (b) Boxplot.. 149

Figure 8-12. Pavement Condition CCI Empirical Distributions Before and After the Application of Maintenance Treatments for VDOT Primary System 150

Figure 8-13. Boxplot of Performance Jump by VDOT on the Primary System................................. 150

Figure 8-14. Pavement Treatment Service Life for VDOT Primary System 151

Figure 8-15. Computation of Benefit Area and Do-Nothing Area of Preventive Maintenance Project 152

Figure 8-16. Computation of Benefit Area and Do-Nothing Area of Corrective Maintenance Project 152

Figure 8-17. Computation of Benefit Area and Do-Nothing Area of Restorative Maintenance Project 153

Figure 8-18. Computation of Benefit Area and Do-Nothing Area of Reconstruction Project 153

Figure 8-19. Cost-Effectiveness of Maintenance Treatments Category... 154

List of Tables

Table 2-1. Classification of Information Quality and Detail 10

Table 2-2. Summary of Data Collection Methodologies 12

Table 2-3. Classification Longitudinal Profile measurement devices 13

Table 2-4. Proposed ranges of structural parameters for flexible pavements (adapted from Rada, Nazarian et al. (2016)). 27

Table 2-5. SN required values as a function of Mr and ESAL categories (after Zhang, Manuel et al. (2003)) 32

Table 2-6. SSI Thresholds for Indiana Pavements (after Flora (2009)) 33

Table 2-7. American Society for Testing and Materials (ASTM) D6433 distress identification catalog asphalt concrete list of distresses 36

Table 2-8. VDOT CCI Flexible Pavement Distresses (after McGhee (2002)) 38

Table 2-9. Roughness Condition Indicator, RCI (after FHWA (2009)) 39

Table 2-10. Single Distress Indicator Formulas, SCR (after FHWA (2009)) 39

Table 2-11. PASER Rating Scale Categories and Methodologies (after Walker, Entine et al. (2002) . 40

Table 2-12. Highway vs. Urban Network Configurations 42

Table 2-13. Urban Pavement Distresses considered Chilean Urban Pavement Condition Index (after Osorio, Chamorro et al. (2014)) 43

Table 3-1. Distress Distribution Analysis on Case of Study Urban Network 57

Table 3-2. Road pavement distress distribution by PCI condition in Urban Areas 59

Table 3-3. New distress identification catalogue asphalt concrete pavement for urban areas 67

Table 3-4. Severity levels definitions for urban pavement distresses identification catalogue (after Loprencipe and Pantuso (2017)) 69

Table 3-5. Statistical parameters of automated procedure PCI calculation (ANOVA F-test) 71

Table 3-6. Regression coefficient of the automated procedure for PCI calculation 72

Table 3-7. Comparative analysis between ASTM D6433 and New Distress Identification Manual for Urban Pavements: N_11 Manholes 74

Table 3-8. Comparative analysis between ASTM D6433 and New Distress Identification Manual for Urban Pavements: N_12 Tree roots 74

Table 4-1. Type of M, R & R minor and major treatments 81

Table 4-2. M, R&R strategies at the Project Level for Strategies 3, 4 and 5. 84

Table 4-3. Traffic distribution in network's sections 87

Table 4-4. Vehicle Operating Costs calculated for each strategy 88

Table 5-1. Sample data GPS stations from survey vehicle 94

Table 5-2. Example output LRS implementation table 95

Table 5-3. Pavement condition and pavement inventory datasets 96

Table 5-4. Example of Application of LRS data integration; get nearest LRS milepost 97

Table 6-1. Correspondence between IRI and RCI rating scale 108

Table 6-2. ESAL per vehicle category group Kazakhstan full traffic spectrum 108

Table 6-3. Pavement Maintenance Activities under Different Categories 113

Table 6-4. Maintenance Decisions for the homogeneous sections along road chainage.................... 114

Table 7-1. Sample Size of Pavement Families included in the study.. 121

Table 7-2. Descriptive Statistics and χ^2 – test of Negative Binomial Distribution Fitting 125

Table 7-3. Goodness of fit pavement deterioration models for the defined pavement families.......... 130

Table 7-4. Mean Square Error of prediction of the Estimate of Pavement Condition using different approaches 134

Table 8-1. Virginia Pavement Maintenance Central Office Cost Estimate (Wu, Flintsch et al. 2008) and CCI triggers for Maintenance Treatments (de León Izeppi, Morrison et al. 2015) 139

Table 8-2. Pavement sections used in this study 142

Table 8-3. Post-treatment performance models for VDOT representative preservation maintenance strategies 142

Table 8-4. Enhanced Piecewise Post-Treatment Performance Models Parameters 147

Table 8-5. Average, 25th Percentile, and 75th Percentile of CCI $_{after}$, CCI $_{before}$, CCI $_{PJ}$ 150

Table 8-6. Average, 25th Percentile, and 75th Percentile of Service Life After Maintenance Treatments 151

Table 8-7. Relative Benefit Cost-Effectiveness of VDOT Pavement Preservation Policies.............. 154

List of Acronyms

AASHTO	American Association of State Highway and Transportation Officials
AADT	Annual Average Daily Traffic
ANAS	Ente Nazionale per le Strade
ARAN	Automatic Road Analyzer
ASTM	American Society for Testing and Materials
BPN	British Pendulum Number
CCD	Continuous Deflection Devices
CCI	Critical Condition Index
CM	Corrective Maintenance Treatment
CDF	Cummulative Distribution Function
CDV	Corrected Deduct Value
CSTI	Center for Sustainable Transportation Infrastructure
DBMS	Database Management Systems
DCP	Dynamic Cone Penetrometer
DI	Deterioration Index
DoD	United States Department of Defence
DOT	Department of Transportation
DV	Deduct Value
EB	Empirical Bayes
ESAL	Equivalent Standard Axle Load Applications
ETD	Estimated Profile Depth
FHWA	Federal Highway Administration
FN	Friction Number
FWD	Falling Weight Deflectometer
GIS	Geographical Information Systems
GLM	Generalized Linear Model
GPR	Ground Penetration Radar
GPS	Geographical Positioning Systems
HPMS	Highway Performance Monitoring System
HIS	Interstate Highway System
IFI	International Friction Index
IQL	Information Quality Levels
IRI	International Roughness Index
LCCA	Life-Cycle Cost Analysis
LDR	Load Related Distress Index
LEB	Linear Empirical Bayes
LRS	Linear Referencing Systems
LCMS	3D Laser Measuring Systems
LTPP	Long-Term Pavement Performance Studies
MAP-21	Moving Across Progress -21
MFV	Multi-Functional Vehicle
MPD	Mean Profile Depth
M, R&R	Maintenance, Repair and Rehabilitation
M_R	Resilient Modulus
MS	Mean Square
MSE	Mean Squared Error

MSI	Modified Structural Index
MTD	Mean Texture Depth
NB	Negative Binomial
NCHRP	National Cooperative Highway Research Program
NDE	Non-Destructive Evaluation
NDR	Non-Load Related Distress Index
NHS	US National Highway System
NPS	National Park Services
OPI	Overall Pavement Index
PASER	Pavement Surface Evaluation and Rating Methodology
PBM	Performance-based Road Maintenance Contracts
PCI	Pavement Condition Index
PCR	Pavement Condition Rating
PJ	Performance Jump
PM	Preventive Maintenance Treatment
PMS	Pavement Management Systems
PSI	Present Serviceability Index
QA/QC	Quality Assurance and Quality Control
RAMS	Road Assessment Management Systems
RB	Rehabilitation Maintenance Treatment
RC	Reconstruction Treatment
RCI	Roughness Condition Indicator
RL	Remaining Life
RM	Restorative Maintenance Treatment
RMSE	Root Mean Square Error
RTRRMS	Response-Type Road Roughness Measuring Systems
RWD	Rolling Wheel Deflectometer
SCI	Structural Capacity Index
SCR	Surface Condition Rating
SCRIM	Sideway-Force-Coefficient Routine Investigation Machine
SFC	Sideway-Force Coefficient
SN_{eff}	Effective Structural Number
SN_{req}	Required Structural Number
SS	Sum of Squares
SSE	Residual Sum of Squares
SSI	Structural Strength Index
SU	Sample Unit
TAM	Transportation Asset Management
TPM	Transition Probability Matrices
TSD	Traffic Speed Deflectometer
USACE	U.S. Army Corps of Engineers
USDOT	United States Department of Transportation
VBA	Visual Basic For Applications
VDOT	Virginia Department of Transportation
VOC	Vehicle Operating Costs
VTTI	Virginia Tech Transportation Institute

CHAPTER 1. INTRODUCTION, OBJECTIVE AND BACKGROUND

1. INTRODUCTION, OBJECTIVE AND BACKGROUND

1.1 INTRODUCTION

Pavements are one of major asset of roadway infrastructure. Design maintenance and rehabilitation strategies to achieve and maintain an acceptable level of service of roadway pavements is a difficult challenge for pavement managers. Since the 1970s, state highway agencies in the United States and Canada have been applying the Pavement Management Systems (PMS) to manage their assets. Since then PMSs have evolved to be reliable tools for the effective management of interurban pavement networks and their use has been spreading worldwide.

But recently, due to the growing travel demand and the public's expectations for safety, ride quality, and traffic flow, highway agencies are redefining their objectives to focus on activities and strategies to preserve and maintain existing highway systems, instead of on the typical strategy of fixing the "worst first". This requires a change of the maintenance approach which have to be focused on preservation without exceeding budgetary limits and change the current reactive maintenance phylosophy to proactive maintenance. The proactive approach (pavement preservation) cuts the need for costly, time-consuming rehabilitation and reconstruction projects. Besides, it reduces associated traffic disruptions and, as a result, the public is seeing improved mobility, reduced congestion, and safer, longer lasting pavements.

The pavement management process provides a systematic and consistent method for the selection of maintenance and repair needs by evaluating the pavement performance at the network level."The experience with pavement preservation in some States demonstrates this success: each dollar spent now on pavement preservation could save up to six dollars in the future" (ODoherty 2007)

The pavement preservation approach aims to extend a pavement's service life by applying a series of low-cost preventive maintenance treatments. This approach closes the gap between pavement condition and users' expectations introducing performance measures over time in the decision-making process. Therefore, a supporting tool is required to help managers to choose the right maintenance policies to achieve the desired levels of service respecting budget limits. PMSs are potentially a useful tool to reach these goals. However, there are still challenges in their implementation.

In the European scene, the main problems related with the use of PMSs are their low level of implementation in European road agencies, and the lack of a culture of infrastructure maintenance; as only recently, sustainable transportation infrastructure models and policies are being developed in Europe.

Nevertheless, data collection activities are one of the key aspects of the pavement management process (Flintsch, Dymond et al. 2004), and to be effective they should be correctly integrated in a PMS. Data collected include road inventory, pavement structural and functional conditions, traffic, and maintenance and rehabilitation history. An adequate PMS should incorporate a set of tools able to provide a network-level inventory of pavement surface distress and conditions and network management tools, including the prediction of pavement condition, budget planning, inspection scheduling and economic analysis, for determining the most cost-effective maintenance and repair strategy considering the Life-Cycle Cost Analysis (LCCA) (Loprencipe and Pantuso 2017).

This dissertation assembles applications and stand-alone analysis tools studied during the Ph.D. providing knowledge for filling the gap between PMS collected information (input) and cost-effective PMS implementations (actions).

1.2 PROBLEM DEFINITION AND RESEARCH APPROACH (PROBLEM STATEMENT)

The discussion presented in the introduction of the dissertation outlines the main research topics studied during the Ph.D. presenting the limitations of the current-state-of-art which can be summarized as follows:

- There is an extensive number of studies and applications of highway state systems PMS developed by highway agencies in the USA. Most of these systems focus their effort on pavement distress identification guidelines, but still, there are limited applications in the European road agencies and, especially, they are even more limited on urban road agencies.
- Road agencies collect a large amount of pavement condition data through inspections performed along with their network. Pavement data comes from a large variety of sources, spatial integration tools are needed to provide consistency between the geographical positions of survey data and the road network inventory to effectively integrate them on a road agency PMS database.
- Because of uncertainties in the data collection methods and interpretation, and the inherent variability of individual sections, the pavement recorded condition can have a very high variance. Therefore, quality data investigation such as data mining and filtering criteria are needed to address the high variance of pavement data.
- Modeling the pavement deterioration process is essential for a successful PMS able to serve to road agencies in maintaining the managed network into a high level of service.

To achieve this goal, managers need accurate pavement deterioration models able to deal with the uncertainty of pavement condition data with the objective to achieve a more accurate estimate of pavement condition over time.

- Despite the advances in information technologies, there are still difficulties in integrating together the existing database and individual information management systems. Therefore, there is a need to develop guidelines for the implementation of the performance-based decision-making process including multiple variables and criteria at the network level, such as the pavement structural condition, repair and maintenance standards and benefits of pavement preservations.

1.3 RESEARCH OBJECTIVE

The objective of this dissertation is to provide guidelines for cost-effective pavement management implementations. Multiple research topics regarding PMS applications were studied during the Ph.D. Several methodologies covering key aspects in the implementation of pavement management solutions were developed as a kind of umbrella that would be useful to support managers efforts in applying pavement management solutions at the network level.

1.4 RESULTS AND SIGNIFICANCE

This dissertation proposes the development of a comprehensive set of tools to design and implement cost-effective pavement management strategies at the network level. Because of collaborations with road agencies transportation agencies worldwide, specific methodologies had been developed using real pavement condition data integrated into PMSs with different levels of implementation.

It is expected that the resulting methodology of this research will constitute a reliable tool to support agencies in their effort to implement PMS effective strategies in the European scene where there are still many shortcomings for their effective use.

1.5 DISSERTATION ORGANIZATION

The scope of the Ph.D. research widened the effective road pavement management for urban roads networks and interurban highway networks. The limitations of the state of the art and the opportunities for improvement are defined by the characteristics of both systems (see § Literature review 2.3.6), that delimitated the approaches and methodologies developed.

In particular, Ph.D. research activities were developed in three sequential steps:1) perform a wide literature review to provide background to identify the challenges for PMS effective implementation; 2) analyze the opportunities for the development of methodologies and applications for networks in European urban systems (small-mid size networks); 3) expand the range of the research to the international/worldwide context characterized by mature PMS implementations in highway networks (high size networks), different analysis tools, and procurement and funding sources to gain knowledge for making an effective contribute into the European scene. Accordingly, the dissertation document is organized in the following chapters:

1. Introduction, Objective and background: In this first chapter, the background of the PMS implementation is presented. Furthermore, the problem definition and research approach, the research objectives and dissertation organization were described thoroughly.
2. Literature Review: This chapter presents the current state-of-the-art and practice. It includes a review of the PMS components, overall pavement condition indexes and data collection considerations; survey manual, semi-automatic, and automatic data methodologies; data filtering and cleaning procedures; pavement deterioration models; road agencies maintenance preservation strategies; and pavement data management issues are presented.
3. Pavement Distress Identification Guidelines and Condition Assessment in Urban Areas. This Chapter presents the results of a case of study for the development of a specified procedure for the assessment of pavement functional condition in Urban Areas. As a result, a methodology to include new distress definitions were developed to achieve a new distress identification manual and a customized pavement condition index for urban road pavements.
4. Pavement Management System Considering the Vehicle Operating Costs: A Case of Study in Urban Areas. This Chapter presents the results of a case study in which the candidate developed a PMS prototype for an Italian municipality using the PCI method considering the Vehicle Operating Costs (VOC) to perform priority analysis. This research includes the creation of database inventory (gathering and organizing pavement data provided by municipality staff) and pavement condition through a PCI inspection performed in the network. One of the results of this research was to give pavement managers affordable and flexible PMS tools to compare maintenance strategies with minimum costs of implementation.
5. Pavement Management Data Strategies: Guidelines to Enhance Data Processing and Interpretation. This Chapter presents a general overview of pavement management data processing strategies and specific analysis tools developed during the Ph.D. and provides knowledge in the processing of data collection data within extensive roadway networks involving spatial referencing tools and guidelines to get information from the data to support PMS implementations.

6. Analysis of Pavement Condition Survey Data for Effective Implementation of a Network Level Pavement Management Program For a Developing Country. This Chapter presents the result of completed research for the development of a methodology to analyze survey data for the implementation of network level pavement management program for the national highway system of a developing country. This Chapter adds to the state of knowledge a methodology that can be used to design mid-term maintenance strategies to support strategic investment in developing countries funding by International long-term lending institution.
7. Development of Pavement deterioration curves of flexible pavements using the Linear Empirical Bayesian Approach at the Network Level. This Chapter presents the results of a research performed during the stay of the candidate as a Visiting Scholar at the Center for Sustainable Transportation Infrastructure (CSTI) at Virginia Tech Transportation Institute (VTTI). This part of the Ph.D. research provides an innovative approach to model pavement deterioration at the network level accounting for the variance and uncertainties related with data collection using the Linear Empirical Bayesian (LEB) approach.
8. Design Pavement Management Strategies: Performance Assessment for Network Level Pavement Preservation. This Chapter reviews pavement preservation policies along time-series applied by Virginia DOT agency to evaluate the cost-effectiveness of maintenance treatments.
9. Conclusions, recommendations and Future Research. This Chapter overviews the main findings and presents the conclusions and recommendations of the dissertation to assist agencies in the implementation of Pavement Management Systems strategies focusing on the data collection and data processing.
    a. Main Contribution of the Research
    b. Recommendations
    c. Future Research and Developments

Appendix A. Distress Identification Manual for Urban Road Pavements.

Appendix B. Software calculation PCI documentation.

Appendix C. Code used in Chapter 6.

Appendix D. Code used in Chapter 7.

Appendix E. Empirical Bayes Application.

CHAPTER 2. LITERATURE REVIEW

2. LITERATURE REVIEW

Highway agencies have started to go through the benefits of having a decision support system that assists them in finding cost-effective strategies for their pavement management program (Flintsch, Dymond et al. 2004). For this, PMSs have been extensively used as standard tools in most highway state agencies in the USA (Zimmerman 2017).

A comprehensive literature review was conducted to provide background and knowledge about the current state of the practice with a focus on main aspects influencing pavement management decisions, such as (1) Pavement Management Systems, (2) Data Inventory/collection techniques, (3) Pavement Condition Assessment, (4) Pavement structural evaluation and indicators of remaining pavement life, (5) Pavement deterioration models, (6) Pavement Preservation Strategies, and (7) Pavement Data Management Issues: considerations at the network level.

2.1 PAVEMENT MANAGEMENT SYSTEMS

Transportation asset management (TAM) practices are looking to optimize the performance and cost-effective of transportation facilities with the aim of maintaining a high level of service and get the best value of an investment with a limited budget. Pavements are one of the primary assets of transportation systems, so pavement management is a key business process within pavement road agencies. A PMS includes several steps: pavement distress survey, pavement evaluation, Life-Cycle Cost Analysis (LCCA) and, finally, the definition of maintenance strategies (Moretti, Di Mascio et al. 2013, Moretti 2014, Moretti, Cantisani et al. 2016, Bonin, Folino et al. 2017, Moretti, Cantisani et al. 2017, Moretti, Mandrone et al. 2017). A proper definition of PMS allows the reduction of overall road costs (construction and maintenance) as well as traffic disruptions.

Therefore, PMSs should include reliable and sufficient data and tools that can help managers to visualize and govern the use of pavement data and evaluate the impact of the possible solutions considered (Flintsch and McGhee 2009).

PMS have demonstrated to be a reliable tool to help road agencies in the decision-making process providing a systematic and consistent method for selecting Maintenance, Repair and Rehabilitation (M, R&R) needs, determining priorities and the optimal time for repair by predicting the future pavement condition (Tavakoli, Lapin et al. 1992, Shahin 2005, Han and Kobayashi 2013).

From its first applications, the main objective of PMSs was to provide an indicator of the present pavement condition regarding the integrity and surface operational condition, as well as a warning system for the early identification or projection of significant repair needs'.. Shahin (2005) settled the basis of the importance of the optimal timing in the selection of M, R&R (interventions at early stages of deterioration will save a 50 % of repair costs). Figure 2-1 illustrated the consequences of poor maintenance timing of M, R&R.

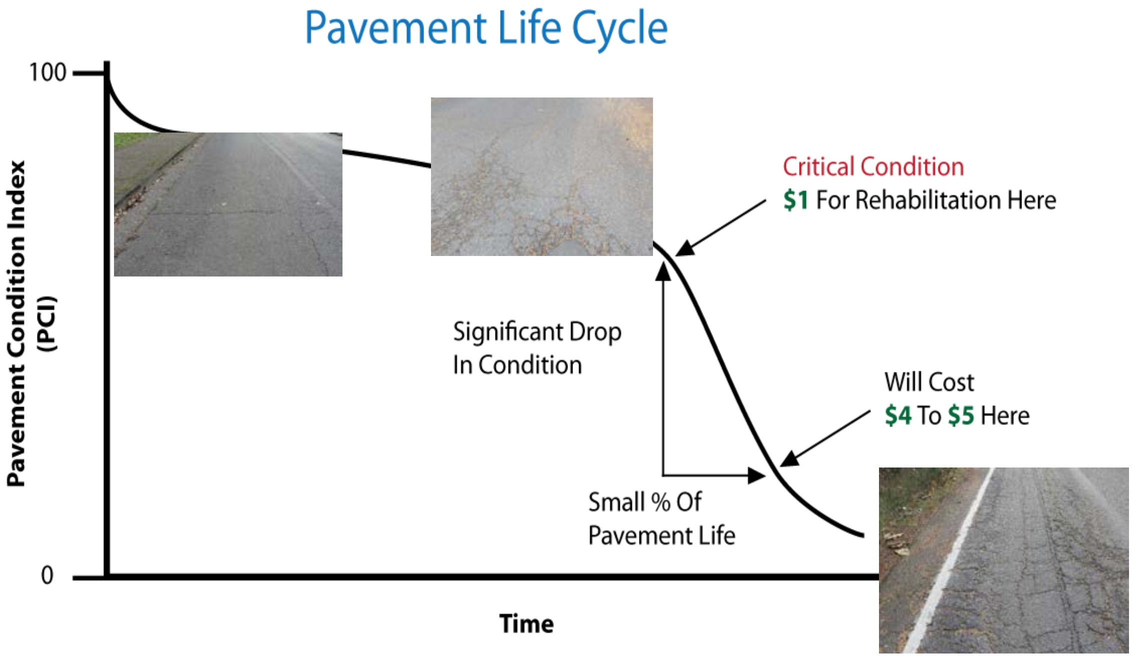


Figure 2-1. Pavement Life Cycle: Consequences of Poor Maintenance Timing (after Shahin (2005))

In the literature available, there are a high level of understanding about different PMS level of analysis and details that can be distinguished as strategic level, network level, and project level. Therefore, PMS methodologies and tools are used to support decisions based on those levels of analysis. They can be defined as:

1. The Strategic analysis is done on the entire road network for long-term budget planning to optimize the maintenance strategies across different networks based on pavement performance.
2. Network-level analysis tools support planning and programming decisions. This kind of analysis considers the entire network for inspecting, planning, budgeting, scheduling and selection of M, R&R works.
3. Project-level analysis. Singular projects/works are considered in more detail for the selection of alternatives/final project considering the in-site specific conditions. In particular, it involves pavement type selection, LCCA, pavement analysis and structural design.

Accordingly, the structure of the PMS implemented in each agency would vary depending on the type of analysis and the intended use of the data. In general, PMS components are depicted in Figure 2-2 and are composed of different modules/standalone tools developed for different purposes which are interrelated between them.

However, pavement data involves a high variety of data sources such as inventory data, traffic, structural condition, pavement condition, and M, R&R events. Therefore, their cost-effective use requires databases to support pavement management, that may range from spreadsheets and tabular data (appropriate for small – mid-size agencies) to comprehensive and well-structured database with specific retrieval procedures integrated on it (appropiate for big size agencies). The evolution and level of use of supporting databases would depend on multiple factors related to the network extension, survey collection methodologies available, software used, and previous staff experience in the use of PMS. Nevertheless, PMS implementations support different types of analyses through the data, analysis, and reporting components.

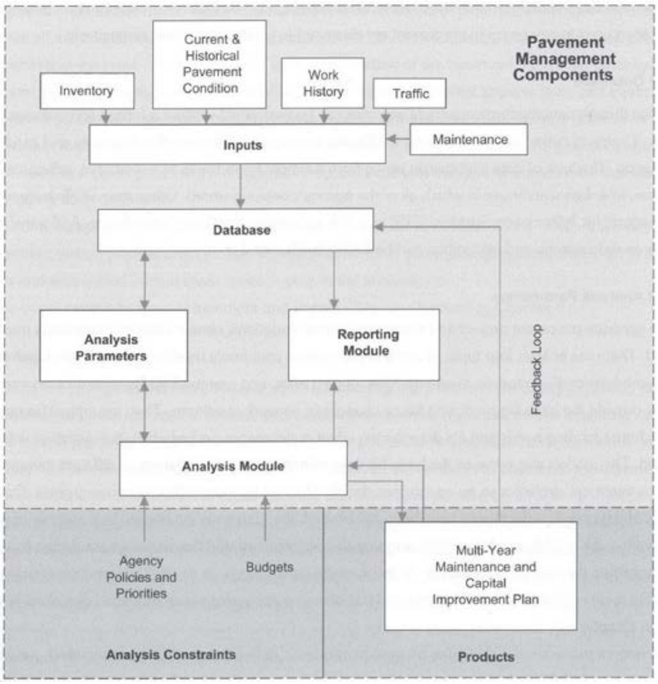


Figure 2-2. Components of pavement management systems (after (AASHTO 2012))

2.2 PAVEMENT CONDITION DATA COLLECTION

As introduced before, PMSs are defined regarding the use of the data and the level of decisions being supported, so pavement agency data collection practices should be designed according to the final use of collected data. However, PMS data collection requirements can be very costly for the agency. For this reason, agencies should design their data collection practices to collect only the data that they need (Bennett, De Solminihac et al. 2006).

Paterson and Scullion (1990) introduced the concept “Information Quality Levels” (IQL) for road management to assess the data collection practices and use. IQL methodology provides an objective reference to help agencies in making appropriate choices about the information needed for decision making, and the appropriate methods for collecting and processing it. Following this guide, the information can be classified into 4 levels of quality and detail (Table 2-1).

Bennett and Paterson (2003) defined this approach for the development and calibration of the HDM-4 system, and it became known and used worldwide, ranged from level IQL-5 (high level data) appropriate for executive decision making to very detailed and project detail of data - IQL-1 (low-level data). Pavement performance measurements at the network level can be associated with the level IQL-3, see Figure 2-3.

Table 2-1. Classification of Information Quality and Detail

IQL Level	Amount of Detail
1	Most comprehensive level of detail, such as that which would be used as a reference benchmark for other measurement methods or in fundamental research. Would also be used in detailed field investigations for an indepth diagnosis of problems and for high-class project design. Normally used at project level in special cases; unlikely to be used for network monitoring. Requires high-level skill and institutional resources to support and utilize collection methods.
2	A level of detail sufficient for comprehensive programming models and for standard design methods. For planning, would be used only on sample coverage. Sufficient to distinguish the performance and economic returns of different technical options with practical differences in dimensions or materials. Standard acquisition methods for project level data collection. Would usually require automated acquisition methods for network surveys and use for network level programming. Requires reliable institutional support and resources.
3	Sufficient detail for planning models and standard programming models for full network coverage. For project design, it would suit elementary methods such as catalog type with manager data needs and low-volume road/bridge design methods. Can be collected in network surveys by semi-automated methods or combined automated and manual methods.
4	The basic summary statistics of inventory, performance, and utilization that are of interest to providers and users. Suitable for the simplest planning and programming models, but for projects is suitable only for standardized designs of very low-volume roads. The simplest, most basic collection methods, either entirely manual or entirely semiautomated, provide direct but approximate measures and suit small or resource– poor agencies. Alternatively, the statistics may be computed from more detailed data.

Source: Paterson and Scullion (1990)

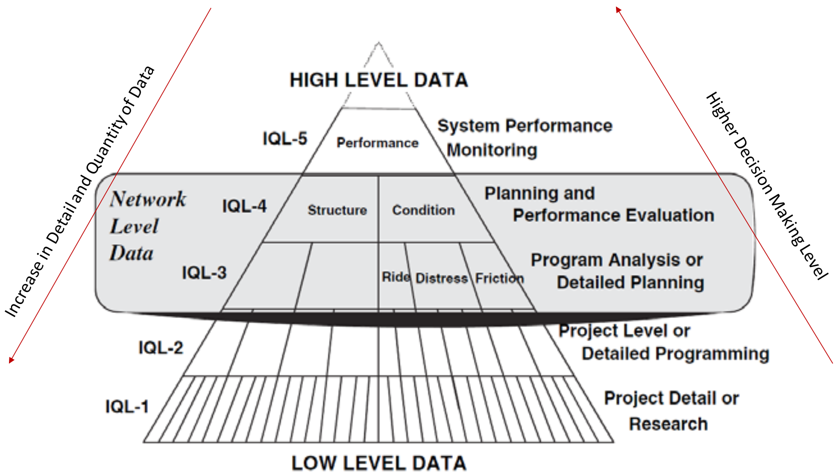


Figure 2-3. Information Quality Level (after Bennett and Paterson (2003))

Data quality and data collection methodologies depend on the type of analysis to be performed:

- Network-level data collection includes large quantities of pavement condition data, affected by the high variance of data. It is composed by data belonging of different sources with different levels of detail. These levels of detail should be able to have a level of detail sufficient to distinguish the performance and economic returns of different M, R&R covering the entire network, (sample coverage and inventory). Network-level data collection survey requires automated pavement condition acquisition methods and windshield surveys at highway speed to minimize traffic disruptions.
- Project-level requires a more comprehensive level of detail, and specific inspection data which are typically collected using semi-automated or even manually. These techniques involve data collection of structural condition, friction data, and specific measures of distress quantity and severity, even from data collection methodologies based on walking surveys. This detailed information would be used to support the decision-making process about pavement preservation and repair treatments including the design of pavement treatment and the estimation the project level cost estimate.

The collected data have itself a significant impact both on the level of investment and on the choice of M, R&R treatments. Therefore, they must be reliable (Pierce, McGovern et al. 2013). Besides, data collection methodologies can be very expensive and can have a high impact on the usability of data and quality of data. Therefore, the definition of adequate data collection campaigns and the use of the stored data are essential for a cost-effective PMS. Pavement data collection methodologies should fit the agency required use of the data and data collection equipment available (Flintsch and McGhee 2009).

Table 2-2 aims to summarize data collection methodologies currently in use in the state-of-art and would serve as a guide for the entire chapter. The characteristics of data collection methodologies and pavement agencies use of data have been analyzed. A comprehensive review of pavement condition indicators and pavement performance issues have been performed, focusing the study on the identification of pavement surface distresses and agency data processing at the network level.

Table 2-2. Summary of Data Collection Methodologies

Pavement Survey Type	**Pavement Characteristic observed**	**Equipment used**	**Most common Pavement Condition Indicators**
Profiling	Evenness Longitudinal Profile/Smoothness	ASTM Class I precision profilers: Manually operated profilers Walking Dipstick ASTM Class II: Non-contact Laser Profilers ASTM Class III: *Response-Type Road Roughness Measuring Systems* (RTRRMS), accelerometers	International Roughness Index (IRI)
	Evenness Transverse Profile/Rutting	Profilometers: Laser Rut Depth Measurement - Non-contact sensors Straight Edge	Rut-Depth
Static Measurement and Dynamic Measurements of Pavement Surface Characteristics	Vehicle Road Interaction Surface Characteristics: Surface texture (macrotexture and microtexture)	Surface texture: Static: Volumetric patch/sand patch method, Circular Track (CT) meter and imaging technologies used to assess road surface texture Dynamic Measurements: profilometer method and WDM High-Speed Texture System.	Mean Profile Depth (MPD), Estimated Profile Depth (ETD)
	Skid resistance	Skid Resistance: Dynamic Measurements (GripTester – Trailer system locked/partially locked wheel and Scrim-Vehicle mounted system) and Static Measurements (British Pendulum Tester)	Friction Number (FN) or Skid Number (SN), British Pendulum Number (BPN) and International Friction Index (IFI)
Manual Data Collection: Visual Survey and/or Windshield Pavement Survey. Automated Data Collection using imaging technologies and/or other sensor equipment.	Surface Distresses	Walking metric ruler, straightedge Distress Identification manual and paper forms; manual or computerized (Tablet, Smartphone) 3D Laser Measuring Systems (LCMS) and digital Processing of pavement images	Pavement Condition Index (PCI), Critical Condition Index (CCI), Pavement Condition Rating (PCR), Overall Pavement Index (OPI),
Pavement Structural Evaluation	Structural Capacity	Non-Destructive Evaluation (NDE): Falling Weight Deflectometer (FWD), Rolling Wheel Deflectometer (RWD), Traffic Speed Deflectometer (TSD), Ground Penetration Radar (GPR). Destructive Evaluation: Pavement Coring/Boring, Dynamic Cone Penetrometer (DCP)	Deflections, Resilient Modulus, Structural Condition Index (SCI), Structural Number (SN)

2.2.1 ROUGHNESS / LONGITUDINAL PROFILE

According to ASTM E867-06 (ASTM 2012), pavement roughness can be defined as "the deviation of a pavement surface from a true planar surface with characteristic dimensions that affect vehicle dynamics, ride quality, dynamic loads and pavement drainage. (Sayers and Karamihas 1998)"

Table 2-3. Classification Longitudinal Profile measurement devices

Equipment Class	**Longitudinal Sampling**	**Vertical Measurements Resolution**	**Precision (1 SD)**	**Bias**
Class I	less than or equal to 25 mm (1 in.)	less than or equal to 0.1 mm (0.005 in.)	0.38 mm (0.015 in.)	1.25 mm (0.050 in.)
Class II	greater than 25 mm (1 in.) to 150 mm (6 in.)	greater than 0.1 mm (0.005 in.) to 0.2 mm (0.010 in.)	0.76 mm (0.030 in.)	2.50 mm (0.100 in.)
Class III	greater than 150 mm (6 in.) to 300 mm (12 in.)	greater than 0.2 mm (0.010 in.) to 0.5 mm (0.020 in.)	2.50 mm (0.100 in.)	6.25 mm (0.250 in.)
Class IV	greater than 300 mm (12 in.)	greater than 0.5 mm (0.020 in.)		

Pavement roughness measuring devices can be classified by the ASTM (2018) E 950-94 (Table 2-3) in different groups according to their accuracy and methodology to calculate the pavement roughness:

- Class I profilometers measure the *true profile* or absolute profile. Dipstick Road Profiler (ASTM 2015), contact device that is operated manually walking along the pavement section, belongs to this group. Dipstick data collection operation was developed by Long-Term Pavement Performance Studies (LTPP) in the USA (Perera, Kohn et al. 2008). The Dipstick succeeds in measuring the highest wavelengths of the road and in the detection of the longitudinal and / or transversal slope of the road. Class I Dipstick profile measurements are affected by low speed of measurements (150 m/h), traffic disruption and operator safety hazards because should be operated manually walking along the surface.

- Class II profilometers consider the dynamic profile. These measuring methods can determine profile elevations by summarizing statistics calculated from elevation data. This kind of profilometers can measure the *relative profile* and requires the calibration with Class I profilometer. Class II non-contact laser profilers are usually mounted on vehicles able to measure the distance between the road surface and a reference on the vehicle, coupled with an accelerometer to correct the profile for the inertial movement of the vehicle. Therefore, the measurement results are independent of the dynamics of the measuring vehicle, and can be measured at higher speed (30-40 km/h), minimizing the traffic disruption and operator safety hazards. The difference between the true profile and the related profile consists, as mentioned previously, in the ability or otherwise of the profilometer used for the measurement of keeping an absolute reference to the planar surface. Therefore, in the case of use of a Class II instrumentation, it is not possible to detect the highest wavelengths.

- Class III equipments include mechanical or electronic devices that indirectly evaluate pavement profiles; these are usually denominated *Response-Type Road Roughness Measuring Systems* (RTRRMS). The most common of these are vertical acceleration profile accelerometers. Measures obtained using these devices require calibration though correlations with standardized roughness thresholds (Cantisani and Loprencipe 2010, Loprencipe and Zoccali 2017, Loprencipe and Zoccali 2017).
- Class IV is the least accurate of them, and the roughness profile is calculated based on subjective evaluation produced by traveling along the pavement section or conducting a visual inspection.

The most worldwide used index for the measurement of road roughness is the International Roughness Index (IRI), presented by the World Bank (Hall, Smith et al. 2009). IRI is a standardized calculated response of a mathematical model of *a quarter of a passenger car* or *full car* (Figure 2-4) driving over a longitudinal profile, the mechanical parameters of the mathematical model have been standardized by the ASTM E1926-08 (ASTM 2015) . IRI is calculated as the cumulated vertical suspension movement divided by the distance travelled, and it is expressed in m/km or in/mi.

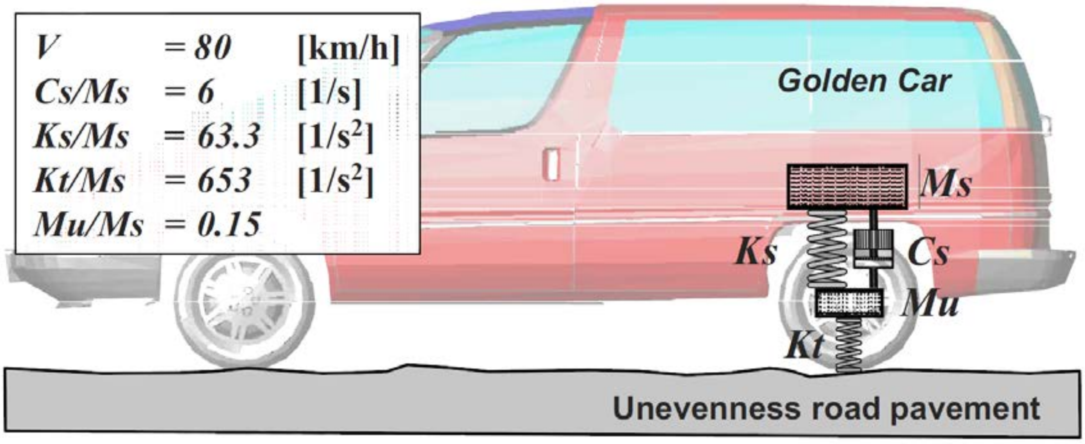


Figure 2-4. Quarter-car model for the calculation of the IRI (after Cantisani and Loprencipe (2010))

Although IRI is the most used worldwide pavement roughness index, there are not fully understanding about the limit of applicability of IRI worldwide, and there is different threshold of acceptability varying significantly from one country to another. There is still work to be done to improve the measurement accuracy and repeatability.

For this purpose, Federal Highway Administration (FHWA) promoted the development of the Long-Term Pavement Performance Studies (LTPP) which constitutes a large research program that includes several specific pavements related researches that are critical to pavement performance. LTPP program consisted on the collection and maintenance of pavement information database of more of 2500 sections located on in-service highways throughout USA. The objective of this research project was to compare the results on different US State DOT agencies performance regarding pavement roughness for the calibration and repeatability analysis of current in use data collection practices. Likewise, FHWA is founding other projects involving US State DOT Agencies for the measurements of pavement surface properties that organized a yearly round-up for testing measurement equipment to evaluate the repeatability and reproducibility of these. One of these is the Pavement Surface Properties Consortium that hosts an annual event at the Virginia Smart Road at Virginia Tech Transportation Institute (VTTI) in Blacksburg, VA, USA, in order to evaluate and compare various types of highway friction and profile measuring equipment, the event is called "the Surface Properties Round up" (Figure 2-5), (Flintsch, McGhee et al. 2012).

Figure 2-5. High speed profiler participants at Surface Properties Round up (source: TPF-5(141) Final Report)

Another worldwide accepted use of the IRI is the quality control of new roads. This application is being widely used by establishing incentives or sanctions to the construction and/or motorway concession companies if they are meeting roughness IRI thresholds requirements. Nevertheless, IRI has become one of the primary indicators of pavement performance for the Highway Performance Monitoring System (HPMS) which contains administrative and extent of system information on all public road as a result of Moving Across Progress -21 (MAP-21) act which is transforming the US Federal-aid highway program by establishing new requirements for performance management (USDOT 2017).

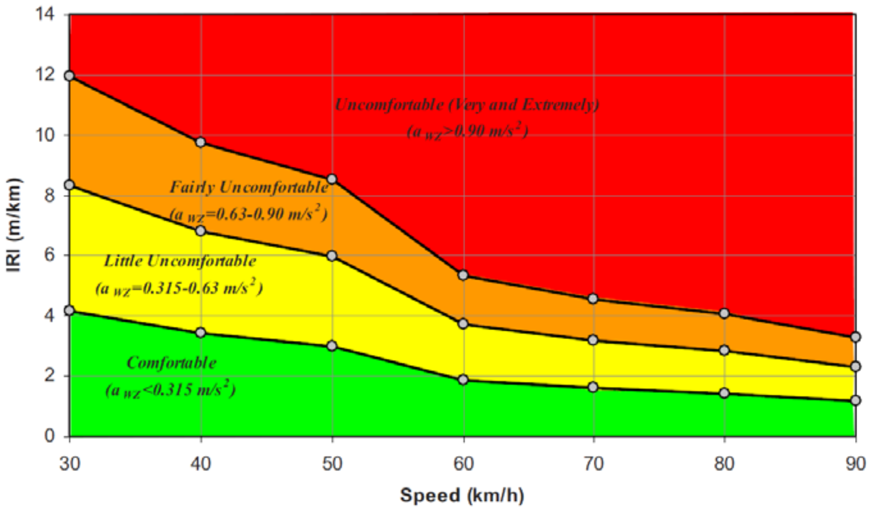


Figure 2-6. Proposal for new speed-related IRI thresholds, considering the a_{WZ} limits established by technical standards (after Cantisani and Loprencipe (2010))

Recently, some researchers were working to develop correlations with IRI and acceptability limits of frequency-weighted vertical acceleration a_{wz} according to ISO 2631 (1997) using low-cost technologies (Cantisani and Loprencipe 2010).

2.2.2 TRANSVERSE PROFILE: RUTTING

The transverse evenness is a measure of the variation within a transverse profile along pavement tested. The leading indicator for express the transverse evenness is the rut depth, defined as "Maximum perpendicular distance between the bottom surface of a straightedge and the contact area of the gauge with the road surface at a specified location, usually measured in the wheel tracks." (Puzzo, Loprencipe et al. 2017). The Rut-depth is expressed in mm or inches.

Rut depth can be measured manually using a 6 ft to 12 ft (1.80 m to 3.60 m) straightedge and a gauge graduated to 1 mm (ASTM 2015). This procedure is very time consuming and, for this reason, only a small number of the section on the roadway can be evaluated. Besides, manual rut-depth measurement causes traffic disruption, and operators are exposed to hazard traffic. For this reason, many agencies in the USA use automated procedures to measure rut-depth at the network level, eliminating traffic exposure to operators, traffic disruption. As a consequence, higher number of sections are being measured in the shorter amount of time.

Automated transverse profile measurements are done by a vehicle traveling over the pavement at highway speeds, using different the measurements of sensors mounted on the profilometric bar. According to ASTM (2013), the transverse profile “is determined on the basis of the vertical distance between an imaginary string line run across the traffic lane from the shoulder to the lane line”. Recently, pavement data collection vendors have developed technologies with high-powered pulse lasers mounted on the back of the van to project a line across the pavement providing “a lateral resolution of more than 1280 points across the width of the pavement (4 m)” at normal traffic speed.

2.2.3 Vehicle Road Interaction Surface Characteristics

2.2.3.1 Surface Texture

Pavement surface characteristics are essential for the safety and the comfort of drivers because they affect vehicle/road interaction processes such as rolling resistance, tire friction, and pavement noise levels significantly. Pavement surfaces should provide adequate friction and maintain a good level of ride quality to ensure satisfaction of driving public (Najafi, Flintsch et al. 2017).

Pavement texture has been categorized into three categories or subgroups based on the wavelength of the irregularities: microtexture, macrotexture, and mega texture (Flintsch, McGhee et al. 2012). Wavelength greater than the upper limit of megatexture are defined as roughness, smoothness, or evenness (Cantisani, D’Andrea et al. 2016), see Figure 2-7.

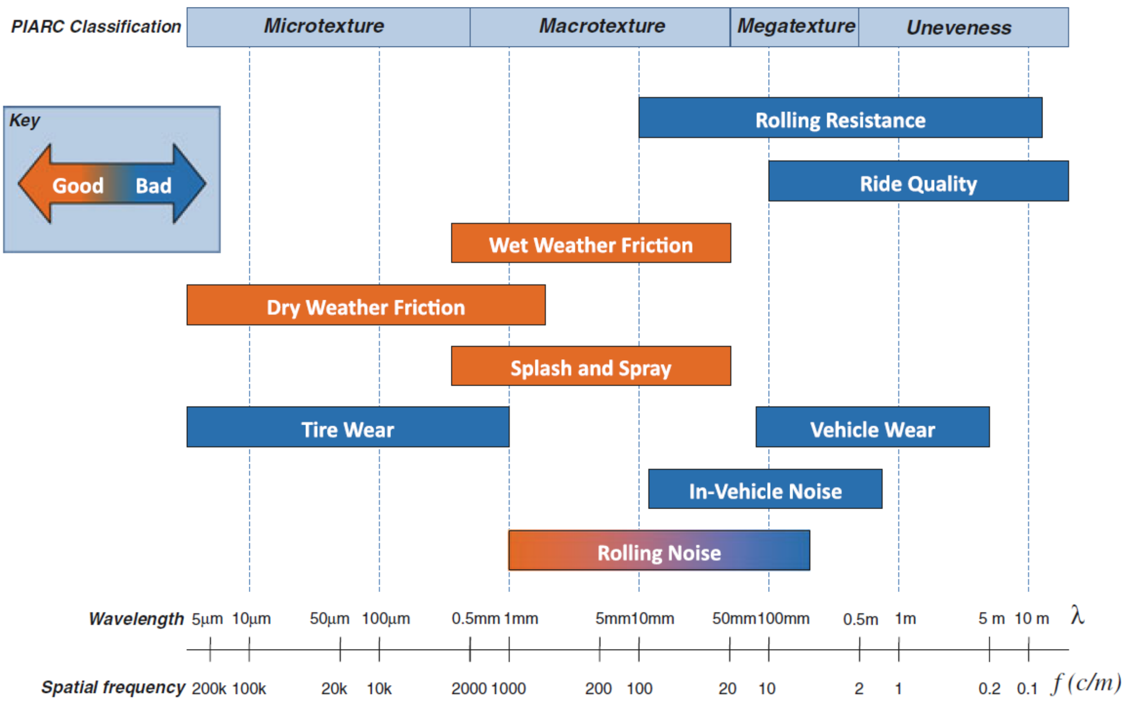


Figure 2-7. Pavement Surface Characteristics Influence Wavelength (after Cantisani, D’Andrea et al. (2016))

Pavement microtexture and macrotexture affect mostly the resistance to skidding. Pavement macrotexture provides the hysteresis component of the friction and allows for the rapid drainage of water from the pavement (Hall, Smith et al. 2009). Microtexture offers direct tire-pavement contact and contributes to the adhesion component of friction (Hall, Smith et al. 2009), see Figure 2-8. Enhanced drainage improves the contact between the tire and the pavement surface helping to reduce the probability of hydroplaning (Najafi, Flintsch et al. 2017). This increased texture may sometimes increase the level of discomfort for exterior noise, vibration, fuel consumption, and tire wear. However, the decreasing of surface texture has as negative aspect the potential increase of the number of accidents due to inadequate levels of pavement friction (Flintsch, McGhee et al. 2012, Najafi, Flintsch et al. 2013, Fernandes and Neves 2014, Najafi, Flintsch et al. 2017)

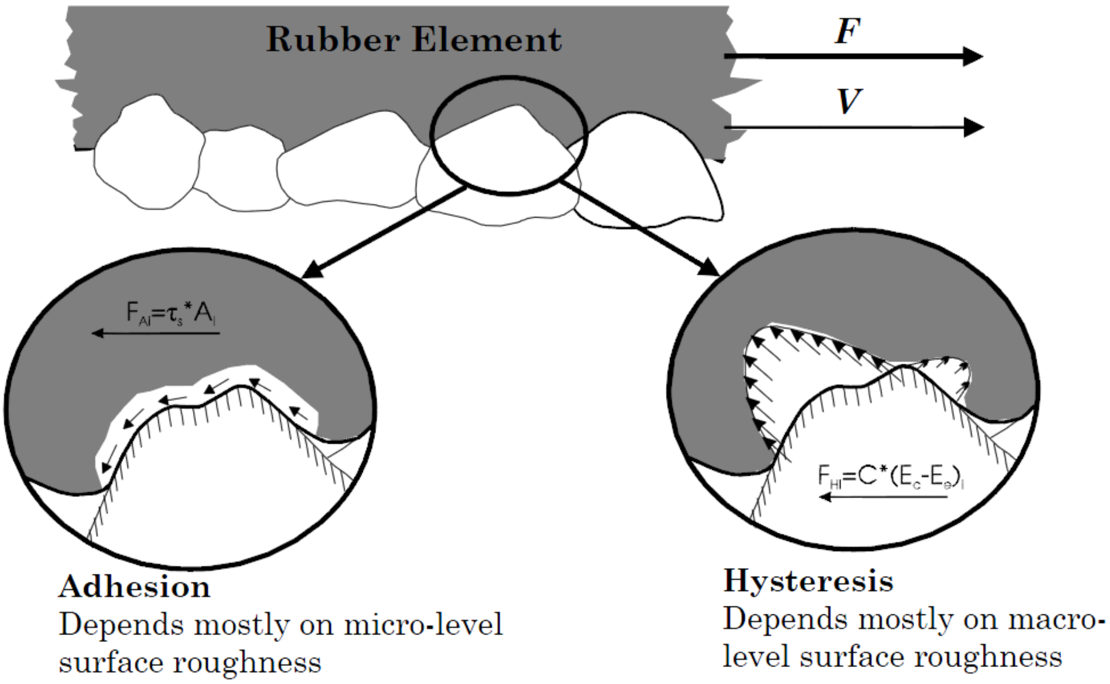


Figure 2-8. Surface Texture Key mechanisms of Pavement Friction (after Hall, Smith et al. (2009))

The macrotexture of a pavement surface results from the large aggregate particles in the mixture. NCHRP synthesis (Flintsch, McGhee et al. 2012) gathered up a list of different devices for measuring pavement surface texture. Macrotexture measurements can be divided into two main classes: static analysis and dynamic measurements.

The most common static measurements is the sand patch and Circular Texture Meter. Sand patch is standardized manual method ASTM E-965 (ASTM 2015), to measure the Mean Texture Depth (MPD) consisting on pouring a known volume of sand on a cleaned road surface and spread out to form a circular patch, the diameter of this patch is measured and the MPD is calculated using a mathematical formula, so with low-cost equipment a direct measure of surface texture is provided with good accuracy and repeatability. CTMeter (Figure 2-9a) has a laser displacement sensor mounted on a circumference with a 142-mm radius and measures the texture with a sampling interval of approximately 0.9 mm. The Dynamic Friction Tester (Figure 2-9b) consists of a horizontal spinning disk fitted with three spring loaded rubber sliders which contact the paved surface as the disk rotational speed decreases due to the friction generated between the sliders and the paved surface. A water supply unit delivers water to the paved surface being tested. The torque generated by the slider forces measured during the spin down is then used to calculate the friction as a function of speed (ASTM 2009).


(a)


(b)

Figure 2-9. Paved Surface Frictional properties measurement apparatus: a) Circular Meter and b) Dynamic Friction Tester

Vehicle-mounted laser devices used to measure macrotexture without traffic disruption (dynamic measurements). ASTM standard E-1845 (ASTM 2015) provides a method to calculate the mean profile depth (MPD) of pavement texture.

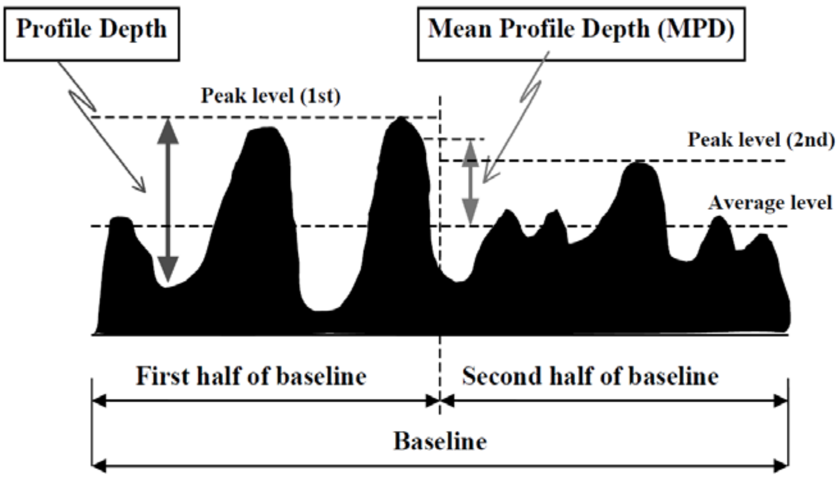


Figure 2-10. Representation of the profile characteristics for the calculation of MPD (after Flintsch, de Leon et al. (2003))

According to ASTM E-1845, the measured profile of the pavement macrotexture is divided for the analysis purposes into segments, having a base length of 100 mm. The segment is splited in half, and the highest peak in each half-segment is found. MPD is calculated as the difference between the resulting height and the average level of the segment and the average of both halves computed, see Figure 2-10. When MPD is used to estimate the Mean Texture Depth (MTD) using a transformation equation, the calculated value is called the estimated texture depth (ETD). ETD=0.2+0.8*MPD, both measures expressed in mm. Recently, some researchers have presented a method to estimate MTD measurements from digital images, taking photos from different angles of a pavement surface (Puzzo, Loprencipe et al. 2017).

2.2.3.2 SKID RESISTANCE

Surface texture influence on water drainage into the pavement-tire contact and its effect on skid resistance and partial or total hydroplaning have been discussed extensively in the literature review (Cairney 1997). In the context of roadway safety management, skid resistance measurement and assessment are essential to analyze the risk of wet crash and the potentially chances of hydroplaning (de León Izeppi, Katicha et al. 2015).

The measurement methods of skid resistance can be grouped into two categories: high-speed equipment and low speed or stationary equipment. The decision of which method to use may depend on the size of the network, the availability of material and the purpose of the measurement.

The most known static device for measuring skid resistance is the British Pendulum Tester, standardized by ASTM E303 (ASTM 2013). The British Pendulum test is based on the slid of a known weight pendulum over wet pavement. As a result of the energy loss caused by the arm friction with the pavement, the British Pendulum Number (BPN) is obtained. The BPN is correlated with a skid resistance coefficient. This static measurement does not require higher cost of implementation but the result accomplished can vary from the operator and operation is very slow. The high-speed pavement friction measurements can be divided into four groups: locked-wheel longitudinal friction force), fixed-slip (longitudinal friction force), sideway-force (sideway “lateral” friction factor), and variable slip (Hall, Smith et al. 2009). Most of the US DOT State agencies used the locked-wheel skid tester, standardized by ASTM E274 (ASTM 2015).

Locked-wheel devices are installed on a trailer which is towed at a typical speed of 60 km/h. A water film, typically of 0.5 mm thickness, is applied in front of the test tire during the measurement, test tire is towed if necessary and a braking system is forced to lock the tire. The resistance drag is measured and average for 1 to 3 second after the wheel is fully locked. These measurements are averaged over this time interval to provide a single measurement called a Skid Number (SN) ASTM E274 (ASTM 2015).

In the European panorama and in some states of the US, sideway-force measurement equipments are used to measure the sideway-force coefficient (SFC), also referred as lateral friction, moving to continuous measurement friction equipment. The commonly used sideway-force equipment is the Sideway-Force-Coefficient Routine Investigation Machine (SCRIM). Pavement side friction equipment measure to the direction of the travel or one or two skewed tires. Water is sprayed on the pavement surface, and one or two skewed free rotating wheels are pulled over the surface. Side force, tire load, distance, and vehicle speed are recorded (Figure 2-11). In Italy, SCRIM friction measurements are used with the aim to establish performance-based measures for the quality control of pavement mixing (Drusin 2014).

Figure 2-11. Virginia SCRIM truck at VTTI Smart Road Testing Facility (Source: www.wdm.co.uk)

2.2.4 PAVEMENT SURFACE DISTRESSES

Pavement surface distress data collection is one of the main elements of a cost-effective PMS. The development of pavement condition data as a significant factor in PMS applications (Wolters and Zimmerman 2010) implied that several methods have been proposed worldwide to support the pavement management process. Therefore, researchers and highway state agencies around the United States have developed pavement condition indicators to measure the overall condition of the pavement by aggregating and summarizing several distress types aiming to develop agency-specific pavement condition data collection procedures and distress rating protocols. Each agency distress rating manual is unique and may contain additional information helpful for data collection (Pierce, McGovern et al. 2013).

Surface distress measurements cover a broad range of distresses that are defined according to the specific pavement agencies protocols. Distress data collection includes the identification of the type of distress, severity, and quantity of surface distresses. Mainly, they can be classified by means: the distress is observed (manual survey and automated survey), and by the level of definition of the data collected (estimated pavement condition data, windshield survey, and measured pavement condition data, detailed measured inspection).

Manual Distress Data Collection

Manual distress data collection is based on visual observation of pavement distress and recording the type, severity, and quantity of distress in paper forms; manual or computerized (tablet, smartphone) using standard distress definitions and rating guidelines.

Distress identification guidelines includes distress type description, representative photographs of distress types helping operators to correctly identify distress type and severity, and a set of guidelines for the measurement of distress quantity. This supporting document is usually named distress identification manual.

Manual distress data collection can be performed walking along the pavement section recording detailed pavement distress quantities for a small length of pavement. This technique can be very accurate and does not require high-cost equipment, but it is slow and requires the closure of the road because of the hazard of the operator. However, the consistency and repeatability of this methodology depend on the operator experience. For this, manual distress data collection requires experienced operators able to identify distress type and severity correctly.

Another kind of manual inspection is the windshield/windscreen rating. In this kind of distress data collection, distress recording is made travelling by truck/car over the entire length of pavement, usually slower than traffic speed, and the estimated pavement condition data are recording on paper forms or using a computer using pre-defined set of criteria for the overall condition of pavement surface surveyed (Walker, Entine et al. 2002).

Automated Distress Data Collection

Automated distress data collection is mainly based on the collection of pavement distress data using images or videos previously collected over the entire length of the test section processed using automated or semi-automated procedures. A detailed analysis of these procedures had been presented by McGhee (2004).

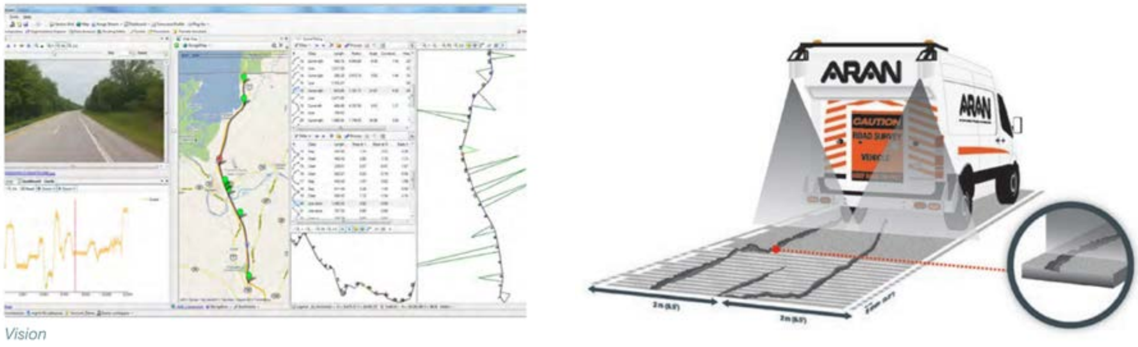
Pavement imaging is collected continuously using high resolution cameras mounted on testing vehicles. This methodology presents the advantage of being less disruptive with traffic because testing vehicle travel over the pavement surface at normal traffic speed, reduce the safety hazard of operators. However, the use of automated distress data collection technologies requires high initial investments and higher costs on the training of agency personnel who should make distress data processing and analysis. The quality of the analysis is influenced by the quality of the collected images, the functionality and validity of the analysis of the image processing software, and the experience of personnel doing data analysis.

Automated distress data collection has the main advantage that is not influenced by human judgement, that is, while performing visual inspections, different inspectors may not agree on survey results. However, each of automated distress system available for automated crack detection is based on crack detection proprietary algorithm, which can be an important limitation for its implementation on certain types of pavements. For this reason, validation and/or calibration of these systems should be performed on their implementation for new networks.

Nevertheless, there is still limitations in the current state of art of automated distress procedures because they can severely influence the assessment of cracking severity level and quantity. They are mainly related with pavement imaging environmental conditions, lighting levels and exposition of pavement images: measuring grey level variations, shadowing, luminance and viewing angle. Flintsch and McGhee (2009) did a complete review about the sources of variability in pavement condition data collection.

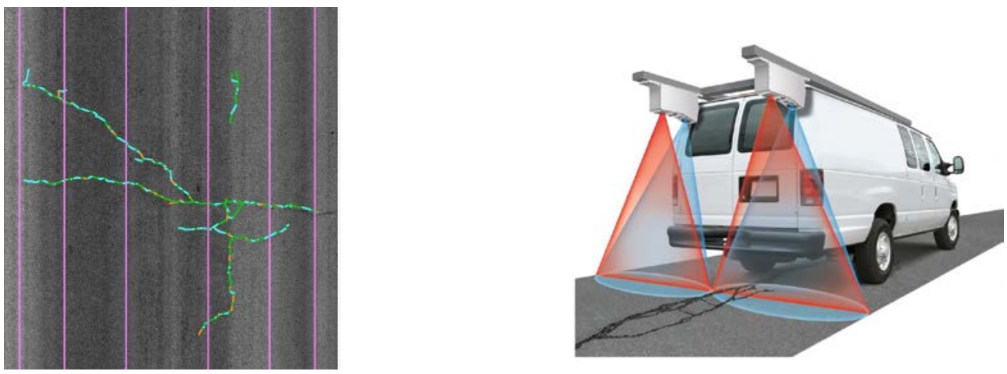
However, automated detection of cracking is a promising topic and researchers are working in the development of procedures for the identification of cracking by the processing of digital images and 3D pavement data point clouds using algorithms to detect pavement cracks with high precision (Sollazzo, Wang et al. 2016).

Pavement data collection vendor, Fugro Roadware (2018) a firm that manufactures and operates equipment that is used to measure the pavement conditions on State and municipal networks, had developed WiseCrax software for automated crack detection and analysis, reporting for each crack, type, severity, extent and location, mounted on Automatic Road ANalyzer (ARAN) instrumented vehicle fully integrated with other profiling sensors in order to measure longitudinal and transverse profile, macrotexture, geometrics (crossfall, gradient, and radius of curvature) and crack measurements, see Figure 2-12.



(a) (b)

Figure 2-12. ARAN instrumented vehicle
(a) and WiseCrax automated crack detection software (b) (source: Fugro Roadware)



(a) (b)

Figure 2-13. LCMS system imaging processing
(a) and LCMS sensors mounted on an instrumented vehicle (b) (source: Pavemetrics)

Dynatest consulting firm (2018) markets an extensive range of instrumentation and pavement data distress and structural capacity instrument at network-level and project-level. The Dynatest Multi-Functional Vehicle (MFV) combines the functionality of the Road Surface Profiler (RSP) able to measure profiling sensors to measure longitudinal and transverse profile, macrotexture and geometrics, with the LCMS manufactured by Pavemetrics (2018), see Figure 2-13.

LCMS system measure transverse profiles with the automatic detection and measurement of cracking with the following characteristics:

- 4 meters wide transverse profile.
- 4000 points per profile.
- One profile every 1 to 5 mm at 100 km/h
- The accuracy of 0.5 mm in elevation.

LCMS software processes real-time pavement profiles and images for the automated detection crack type, severity, extent, and location.

2.2.5 PAVEMENT STRUCTURE EVALUATION

Pavement structural condition affects pavement performance and knowledge of the structural condition of a highway pavement is important for pavement management at both network level and project level (Zhang, Claros et al. 2003, Zhang, Manuel et al. 2003, Flora 2009, Bryce, Flintsch et al. 2012, Bryce, Flintsch et al. 2016, Katicha, Ercisli et al. 2016). Therefore, Highway State agencies in the USA are working to include pavement structural condition for PMS applications at the network level.

Bryce, Flintsch et al. (2012) did a complete overview of structural capacity indicators derived from pavement deflection measurements used as pavement structural condition indicators.

Pavement structural capacity of a pavement is typically obtained by using non-destructive techniques based on the surface deflection measurements, such as Falling Weight Deflectometer (FWD), Rolling Weight Deflectometer (RWD) and Traffic Speed Deflectometer (TSD), or destructive techniques based on extraction of material in situ (coring) and testing of extracted materials.

FWD is the most widely non-destructive testing device accepted for structural evaluation of pavement based on their deflection responses (Rada, Nazarian et al. 2016). FWD test is performed by dropping a weight on the pavement that creates a load pulse which simulates the load produced by a rolling vehicle wheel. Deflection sensors (geophones) are mounted radially from the center of the load point (plate) to measure the deflection response of the pavement (Figure 2-14). Usually, a total number of 3 drops are done on each pavement tested, and the deflection values recorded by the effect of the final drop (for which the result represents the "final" pavement response) is typically used.

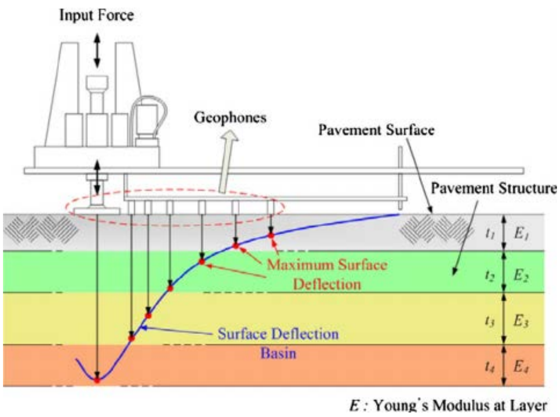


Figure 2-14. FWD testing equipment surface deflection bowl (after Choi, Wu et al. (2009))

FWD data collection technologies are usually independent of surface distress data collection, and sampling data is obtained at individual points. Pavement structural thickness and material data are necessary to perform analysis of FWD deflection bowl or deflection basin obtained from the application of the load under the plate.

Pavement structural properties of pavement and subgrade layers can be provided from the coring, and other technologies as Ground Penetration Radar (GPR) and Dynamic Cone Penetrometer (DCP) that are used in conjunction of deflection data for improving the accuracy of the elastic modulus estimates (Bennett, De Solminihac et al. 2006). This kind of data is typically used to detect patterns of variability in pavement support conditions or to estimate the strength of pavement and subgrade layers. GRP is a pulse-echo technique that uses radio waves to penetrate the pavement using a wave energy transmission from a moving antenna. As energy traveled through the pavement structure, echoes are created at boundaries of dissimilar materials. The strength of this echoes and the time it takes them to travel through the pavement can be used to determine the pavement layer thickness and other properties (Bennett, De Solminihac et al. 2006).

DCP is a low-cost equipment that provides quick information about in-situ structural properties of pavement unbound material (ASTM 2015). This test consists of manually raised and dropped to the bottom of a handle located at the top of the device, used to hold DCP shafts plump and to limit the upward movement of the 8 kg hammer and then dropped (allowed to free fall) from a height of 575 mm. The lower part of the shaft is a 16-mm diameter steel 900-1200 mm long, marked in 5-mm increments for recording the penetration of 60º degrees standardized steel cone. DCP testing is usually performed after pavement coring of bound materials.

Representing in a chart the number of blows against penetration distance, the boundaries of pavement unbound layers can be determined. Relationships between widely known indicators of pavement unbound strength such as California Bearing Ratio (CBR), and DCP values (Livneh 1989, Paige-Green and Du Plessis 2009) are available in literature. Although these indicators are material properties dependent, some authors recommended their use with extreme caution, especially if that relationship was developed outside of the area and/or on different soils in which the DCP penetrations have been performed (Jones and Harvey 2005). However, the lecture of the number of blows against the distance of DCP test can provide an overall indication about unbound layers' material thicknesses and relative strengths.

This information can be used to calculate the pavement Structural Number of existing pavement, defined as effective Structural Number (SN_{eff}) (AASHTO 1993), and elastic modulus of the different layers performing back calculation analysis. Proposed ranges of the structural parameter for flexible pavements can be summarized in Table 2-4.

Therefore, FWD deflection data can be used to predict SN_{eff} as a strength index for flexible pavements containing both pavement and a subgrade contribution. A range of elastic modulus of typical pavement layer materials can be found in the literature (Von Quintus, Rao et al. 2015).

Table 2-4. Proposed ranges of structural parameters for flexible pavements (adapted from Rada, Nazarian et al. (2016)).

Input	Layer type	Default	Minimum Value	Maximum Value
Modulus (MPA)	AC	3445	2067	4823
	Base	344.5	172.25	1722.5
	Subgrade	68.9	34.45	206.7
Thickness (mm)	AC	127	25.4	228.6
	Base	304.8	152.4	609.6

Back-calculation is a mechanistic evaluation of pavement surface deflection basins generated by the deflection device (Ullidtz 1987). There are many programs and methods worldwide used to estimate in situ elastic moduli from the analysis of deflection data:

Back-calculation methodology widely used worldwide was outlined by Von Quintus, Rao et al. (2015), the general method consists of taking a deflection lecture and attempt to match it with a calculated surface deflection generated from an identical pavement structure using assumed layer stiffness (seed moduli). The assumed layer moduli in the model are adjusted until they produce a surface deflection that closely matches the measured one. The combination of assumed layer stiffness that results in this match is then expected to be near the actual in situ moduli of the various pavement layers.

Computationally, it can be done by programing subroutines to match the measured deflection for a set of layer moduli (*iteration search methods*):

- searching in a database of calculated deflection basins generated for specific pavement structure to find the calculated deflection basin that best matches the measured deflection basin (*database search methods*) such as implemented in Texas Modulus software (Rohde and Scullion 1990);
- using the Odemark method of equivalent thickness (*equivalent thickness methods*);
- reducing the multilayer elastic model to reduced number of layers (three or less). This approach, developed by Ullidtz and Stubstad (1985), is implemented in ELMOD® software and commercially distributed worldwide by Dynatest.

Rohde (1994) simplified the method for the determination of pavement SN from falling weight surface deflections was used, without performing back calculation analysis. Therefore, the remaining pavement structural capacity and service life can be calculated, and with this information design rehabilitation and/or improvement pavement structural capacity. Agency use of structural condition analysis can lead to cost-effective methods for rehabilitating roads by means of the evaluation of long-term properties of new rehabilitation techniques. This method assumed the pavement structure as a two-layered structure: a top Hot Mix Asphalt (HMA) Concrete layer of H_p total thickness and a subgrade layer at the bottom modeled as a semi-infinite space.

SN_{eff} is obtained directly from FWD measurements as follows:

1. Normalize pavement FWD deflection data to standard 40kN load deflections, Eq. 2-1:

$$D_k^{40kN} = \frac{L_{40kN}}{L_P} \cdot D_k^p \qquad \text{Eq. 2-1}$$

   where D_k^a = deflections at the fixed sensors at 40 kN standard load; L_{40kN} = stress in the plate for the standard load ; L_p = measured stress in the plate for the tested peak load.

2. Determine the deflection at an offset of 1.5Hp. This will require the interpolation among deflection measured at the fixed sensors positions Eq. 2-2:

$$D_x = \frac{(R_x - R_B)\cdot(R_x - R_C)}{(R_A - R_B)\cdot(R_A - R_C)} \cdot D_A + \frac{(R_x - R_A)\cdot(R_x - R_C)}{(R_B - R_A)\cdot(R_B - R_C)} \cdot D_B + \frac{(R_x - R_A)\cdot(R_x - R_B)}{(R_C - R_A)\cdot(R_C - R_B)} \cdot D_C \qquad \text{Eq. 2-2}$$

   where Dx=Deflection at of Rx; D_A, D_B, D_C = deflection at the closest fixed sensor position; R_A, R_B, R_C = offset of closest sensors to Point X; X = point for which deflection is determined. Hp = pavement depth – thickness of all layers above the subgrade.

3. Determine the structural index SIP of the pavement as follows Eq. 2-3:

$$SIP = D_0 - D_{1.5 \cdot Hp}\ , \qquad \text{Eq. 2-3}$$

   where: SIP = Structural Index of the Pavement defined according to Rodhe et *al.* (1994), D_0 = peak deflection under the standard 40-KN (9000-lb.) FWD (microns), $D_{1.5\ Hp}$ = deflection at 1.5 times the pavement total depth Hp (microns).

4. Determine the existing SN_{eff} Eq. 2-4:

$$SN_{\text{eff}} = k_1 \cdot SIP^{k2} \cdot H_p^{k3} \qquad \text{Eq. 2-4}$$

   where: SNeff = effective structural number expressed in inches (in.); for asphalt pavements k1 = 0.4728; k2 = −0.4810; k3 = 0.7581; SIP = Structural Index of the Pavement in microns; Hp = total pavement thickness (mm).

Kansas DOT State Agency in the USA studies FWD tests on 20% mileage appear to be a valid statistical choice, and three tests per mile are the minimum test frequency required at the network level (Hossain, Chowdhury et al. 2000). However, FWD is a stationary deflection-measuring device that requires traffic control and can only take stationary measurements at discrete points limiting the density of points where data are collected. This makes the device inefficient for network level data collection (Shrestha, Katicha et al. 2018).

In Italy, some researchers proposed techniques to classify pavement FWD deflections without performing back calculation for airfield pavements. The research had the purpose of identifying critical sections or determining localized areas which require further investigations using clustering analysis (Amadore, Bosurgi et al. 2014).

More recently, some researchers are developing moving deflection testing devices that can measure pavement responses at traffic speed (TSD and RWD) represents a more viable alternative to individual point measurements, such as FWD measurements (Rada, Nazarian et al. 2016). TSD and RWD testing equipment are based on the measurement of the curvature of deflection pavement surface response over a force applied through wheels applied continuously on a trailer traveling at normal traffic speed.

Flintsch, Katicha et al. (2013) did a complete review about the current technologies implemented in different types of continuous deflection measuring devices include a catalog of existing continuous deflection measuring technologies and implementation plan for the technology. Continuous Deflection Devices (CCD) can measure pavement deflection traveling at highway speeds, which has the advantage of reducing traffic disruption and operator safety hazard. However, CCD data is contaminated with high noise levels of slope measurements and results on also resulting in a significant amount of data in terms of physical storage and management, so procedures considered for the analysis of the repeatability (Katicha, Flintsch et al. 2012) and specific methodologies for the processing data (Katicha, Flintsch et al. 2014).

Recent studies funded by FHWA with nine participating state highway agencies (Katicha, Flintsch et al. 2017) carried out a field demonstration using the results of TSD testing within PMS at the network level, presenting these promising technologies for incorporate the use of structural evaluation measurements within currently implemented state agencies PMS.

In Italy, national transportation agency (ANAS) evaluates the bearing remaining structural capacity within its network, as a performance-based quality control method using pavement structural evaluation using the 3rd Greenwood Inc.'s TSD (Figure 2-15) in connection with FWD test (Drusin 2014).

Figure 2-15. Traffic Speed Deflectometer ANAS (Source: ANAS Gruppo FS Italiane)

2.2.6 INCORPORATING STRUCTURAL CAPACITY PERFORMANCE MEASURES

Pavement structural condition indicators are based on deflection measurements from FWD testing device in order to develop an index that would provide information about the structural state of the pavement, as a discriminating/screening tool for network-level decisions regarding M, R&R (Zhang, Manuel et al. 2003). The most outstanding experiences are:

A. Texas DOT: Structural Capacity Index (SCI) (Zhang, Claros et al. 2003)

Texas DOT developed the Structural Capacity Index (SCI) (Zhang, Claros et al. 2003) based on the estimation of the AASHTO SN (AASHTO 1993) directly from FWD measurements (Rohde 1994).

$$\mathrm{SCI} = \frac{\mathrm{SN}_{eff}}{\mathrm{SN}_{req}} \qquad \text{Eq. 2-5}$$

SCI (Eq. 2-5) was defined as the ratio between the existing AASHTO SN (SN_{eff}) and the required SN (SN_{req}).

SN_{req} can be calculated for the calculated values of resilient modulus M_R and 20 years (design life) accumulated traffic volumes, estimated Equivalent Axle Load Application (ESAL). ESAL is a concept developed by AASHTO Road Test (AASHTO 1993) to establish a damage relationship for comparing the effect of axles of different loads .

The Design Equivalent Standard Axle Load Applications ($ESAL_d$) is a cumulative traffic load summary statistic which simulate the effect of the traffic during the design period (Watanatada 1987). $ESAL_d$ can be calculated using Eq. 2-6 and Eq. 2-7.

$$ESAL_d = \sum_{i}^{N_d} \left[\left(\sum_{k}^{N_k} Av_k \cdot ESAL_k \right) \cdot G_f \cdot D_f \right] \qquad \text{Eq. 2-6}$$

where $ESAL_d$= Cumulated Equivalent Standard Axle Load Applications over the design period; Av_k = average number of vehicles for each traffic category group k; $ESAL_k$=Equivalent Axle Load factor for each traffic category, in ESAL per vehicle; G_f= growth factor; D_f= distribution factor.

$ESAL_k$ equivalent axle load factor for each traffic category was calculated in Equivalent Standard Axle Load per vehicle using Eq. 2-7.

$$ESAL_k = \sum_{i}^{N_k} \frac{P_{ki}}{100} \cdot \left(\frac{AXL_{kij}}{SAXL_j} \right)^4 \qquad \text{Eq. 2-7}$$

where $ESAL_K$ = equivalent standard axle load factor based on the equivalency exponent of 4.0 for vehicle group k, in Equivalent Standard Axle Load per vehicle; AXL_{kij}=the average load on axle j of load range of vehicle group k; $SAXL_j$ = standard single axle load of axle group type, j; 6.60 t for single-wheel single axle, 8.16 t for dual-wheel single axle, 7.55 t for dual-wheel tandem axle, 7.63 t for dual-wheel triple axle.

$ESAL_d$ was used to combine the damaging effects of the full spectrum of traffic of axle loading (different axle loads, and axle configurations predicted over the design period) that was converted into an equivalent number of single axle load applications summed over that period, according to the AASHTO design guide as follows Eq. 2-8:

$$\log(ESAL) = (Z_R \cdot S_0) + 9.36 \cdot \log(SN_{\mathrm{req}} + 1) - 0.2 + \frac{\log\left(\dfrac{\Delta PSI}{4.2 - 1.5}\right)}{0.4 + \dfrac{1094}{(SN_{\mathrm{req}} + 1)^{5.19}}} + 2.32 \cdot \log(M_r) - 8.07 \qquad \text{Eq. 2-8}$$

where SN_{req} = Required Structural Number; M_R = resilient modulus of the subgrade; ESAL = Predicted number of Equivalent Axle Load Applications (ESALs) that will result in a change in serviceability of ΔPSI, Zr is the standard normal deviate and S0 (0.5) the combined standard error of the traffic prediction and performance prediction, usually the level of reliability considered can be 95 %. For this, the logarithm of ESALs will be decreased by (1.645*0.40). For the calculation of the design period (20-years), an average factor growth factor (G_f) of 1.04 will be used and 0.55 (D_f) as directional distribution factor.

The use of SCI (a ratio of the existing SN to the required SN) is effective in discriminating between pavements that need structural reinforcement and those that are in sound structural condition. Furthermore, the degree of structural deficiency can easily be established when the existing and required SN values are known. Texas DOT proposed SN required values based on subgrade modulus (M_R) and ESAL categories, see Table 2-5.

Boussinesq equations were used to calculate the surface modulus from the surface deflections. The surface modulus, that is, the weighted mean modulus of a semi-infinite space, at a certain distance, r, gives a rough estimate of the modulus at the same equivalent depth. Under the condition that the subgrade behaves as a linear elastic semi-infinitive space, surface modulus should be the same at varying distances. Therefore, the M_R was estimated with the outer deflections as these are almost entirely controlled by the subgrade, Eq. 2-9.

$$M_R = \frac{P \cdot \left(1 - \mu^2\right)}{\pi \cdot D_r \cdot r} \qquad \text{Eq. 2-9}$$

where M_R = elastic modulus of the subgrade (resilient modulus); D_r = deflection measured by the geophone at position r expressed in mm; P = FWD peak load expressed in N; μ = Poisson ratio (usually considered as 0.35); r = offset radius of the measure geophone expressed in mm.

Table 2-5. SN required values as a function of Mr and ESAL categories (after Zhang, Manuel et al. (2003))

	Accumulated traffic (ESALs)				
Mr (psi)	**Very low (50,000–945,000)**	**Low (945,000–1,687,000)**	**Medium (1,687,000–2,430,000)**	**High (2,430,000–3,172,000)**	**Very high (3,172,000–50,000,000)**
Low (1,000–5,400)	4.3	5.2	5.3	5.6	7.1
Medium (5,400–7,500)	3.5	3.9	4.2	4.3	6.0
High (7,500–40,500)	2.3	2.6	2.8	2.8	2.9

B. Indiana DOT: Structural Strength Index (SSI) (Flora 2009)

Indiana DOT (Flora 2009) developed the Structural Strength Index (SSI) Eq. 2-10. This indicator is defined as the probability that homogeneous pavement segment i in a given pavement family (j,k) (pavement type, j and design type, k- e.g. same pavement thickness) have a deflection larger than the given FWD center deflection measurement obtained (1st geophone/sensor in FWD test) -δ_{ijk}. Given center deflection should be considered normalized to a standard load (40 kN-9000lb) and a standard temperature (20ºC-68ºF). SSI obtained is a comparative structural measurement of segment i within the pavement family considered (j,k).

$$\mathrm{SSI} = 100 \cdot \left[1 - \mathrm{F}\left(\delta_{ijk}\right)\right] \qquad \text{Eq. 2-10}$$

Flora (2009) developed a set of thresholds for SSI index for Indiana pavement based as a function of pavement system Table 2-6.

Table 2-6. SSI Thresholds for Indiana Pavements (after Flora (2009))

Pavement Type	System	Measure	Excellent	Good	Fair	Poor
Flexible	Interstate	Deflection (mil)	1.7	2.4	3.1	3.8
		SSI	99.5	74.8	40.2	20.8
	Non-Interstate NHS	Deflection (mil)	2.7	3.8	5.0	6.1
		SSI	95.3	65.1	36.1	21.3
	Non-NHS	Deflection (mil)	3.7	5.8	8.0	10.1
		SSI	96.7	65.2	36.5	21.5
Rigid	Interstate	Deflection (mil)	1.8	2.3	2.9	3.5
		SSI	90.4	66.3	42.9	24.2
	Non-Interstate NHS	Deflection (mil)	2.6	3.1	3.7	4.3
		SSI	94.2	69.5	50.8	15.1
	Non-NHS	Deflection (mil)	3.2	4.9	6.5	8.2
		SSI	89.3	59.9	38.8	24.6

C. Virginia DOT: Modified Structural Index (MSI) (Bryce, Flintsch et al. 2012)

Recently, Virginia DOT incorporated structural information into their Network-level decision making process using trigger values to use with enhanced decision trees to be used in conjunction with VDOT CCI thresholds (see § 2.3.2 Virginia DOT: Critical Condition Index (CCI)) based on to increase or decrease the maintenance action that was decided from the surface distresses (CCI) given traffic, age, and structural inputs as a function of M_R and SN (Diefenderfer, Chowdhury et al. 2009).

Bryce, Flintsch et al. (2012) developed the Modified Structural Index (MSI) as a modified form of the SCI (Eq. 2-5) expressing SN_{eff} in a complete form by substituting Eq. 2-3 in Eq. 2-4, and SN_{req} depending of α and γ parameters in Eq. 2-11, by substituting the fixed values of AASHTO method equation rearranging Eq. 2-8.

$$\mathrm{SN}_{req} = \alpha \cdot \left[\log(\mathrm{ESAL}) - 2.32 \cdot \log(M_R) + 9.07605\right]^{\gamma} \qquad \text{Eq. 2-11}$$

To simplify the application of the MSI at the network level expressed in a generic form depending of α and γ parameters in Eq. 2-12. Bryce et *al.* proposed a mathematical procedure to determine α and γ parameters to apply MSI indicator to different sets of road management systems (e.g. Interstate, Primary and Secondary management networks, or other sets of elements of physical inventory). This procedure consisted on the generation of ESAL and M_R values randomly ranging from the real VDOT data obtained for the set of data considered in the assumption conditions, Eq. 2-12, obtaining by regression from Eq. 2-11 the α and γ parameters derived from the assumption conditions.

$$\mathrm{MSI} = \frac{k_1 \cdot \left(D_0 - D_{1.5 \cdot H_P}\right)^{k_2} \cdot H_p^{k3}}{\alpha \cdot \left[\log(\mathrm{ESAL}) - 2.32 \cdot \log(M_R) + 9.07605\right]^{\gamma}} \qquad \text{Eq. 2-12}$$

This approach will simplify the computational application of the MSI indicators for the agency at the network level by entering the data directly in a closed-form equation.

2.3 PAVEMENT CONDITION ASSESSMENT

One of the aims of a road agency is to maintain the entire network managed to a high level of service. Therefore, a successful PMS should include indicators that could reflect the current pavement condition of network sections’ and a set of tools able to predict pavement future condition supporting the decision-making process used to determine section’s maintenance needs and the suggested maintenance treatments at the network level.

Road agencies use pavement condition indicators based on the functional condition, the structural condition of a pavement section, or both. Researchers and highway agencies around the United States (Papagiannakis, Gharaibeh et al. 2009, Wolters, Zimmerman et al. 2011, Vavrik, Evans et al. 2013) have developed different pavement distress indices to measure the pavement’s structural integrity and pavement surface operation by aggregating several distress types to measure the overall condition of the pavement. For the extent of this research, the following sections have been focused on pavement condition assessment for flexible pavements based on pavement surface distress measurements.

2.3.1 ASTM D6433 STANDARDIZED PROCEDURE: PAVEMENT CONDITION INDEX

The main contribution to the state-of-the-art was the development of the Pavement Condition Index (PCI) rating procedure that measures the pavement integrity and surface operational condition based on a numerical scale, ranging from 100 (perfect condition) to 0 (failed pavement), developed by the U.S. Army Corps of Engineers (USACE), see Figure 2-16. According to this methodology, the pavement network is divided into branches, sections and sample units (SU). Pavement surveys are performed on SU. PCI method is an accurate, consistent, systematic and repeatable rating method based on a visual survey to assess pavement surface distresses by the identification of the type, severity, and density of each distress.

The American Society adopted the procedure for Testing and Materials (ASTM) and documented in ASTM D6433 (ASTM 2018) Standard Test Method for Roads and Parking Lots Parking Lots Pavement Condition Index Surveys, and ASTM D5340 (ASTM 2012) Standard Test Method for Airport Pavement Condition Index Survey. Since then, this methodology has been widely used throughout by roadagencies in the USA.

The ASTM D6433 defines 20 distress types for AC pavement (Table 2-7). Each combination of distress density, distress severity, and distress type has an associated deduct value as a weighting factor that indicates the influence of each combination on the pavement condition

Eq. 2-13. The ASTM D6433 Distress Identification manual provides guidelines to pavement inspectors to correctly identify each distress type and severity. Each deduct value is determined by using available master curves, Deduct Value (DV) curves, provided in the Distress Identification Catalogue documented in the ASTM D6433 Appendix X3 (ASTM 2018).

PCI calculation standardized procedure, described in ASTM D6433, involves the calculation of the deduct weighting value for each combination of distress type and severity by adding up the total quantity of each type distress, severity level from inspection data. An adjustment factor was them used to obtain the Corrected Deduct Value (CDV). PCI value for test section is calculated by subtracting the corrected deduct value to 100. The units for the quantities may be square meters, linear meters, or a number of occurrences, depending on the distress type, see Table 2-7.

$$\mathrm{PCI} = 100 - \sum_{i=1}^{p} \sum_{j=1}^{m_i} a\left[T_i, S_j, D_{ij}\right] \cdot F(t,d) \qquad \text{Eq. 2-13}$$

where: PCI = pavement condition index; a = deduct weighting value depending on distress type T_i, level of severity S_j, and density of distress D_{ij}; i = counter for distress types; j = counter for severity levels; p = total number of distress types of pavement type under consideration; m_i = number of severity levels on the i-th type of distress; and $F(t,d)$ = an adjustment factor for distresses that varies with total summed deduct value t and number of deducts d.

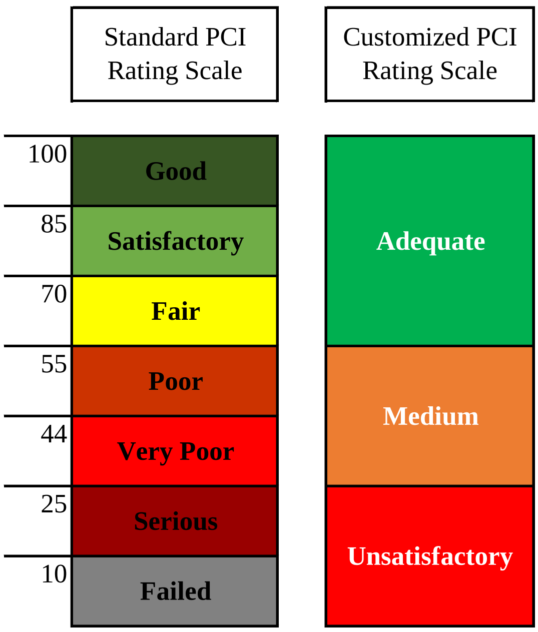


Figure 2-16. Standard and customized PCI rating scale (after Loprencipe and Pantuso (2017))

Table 2-7. American Society for Testing and Materials (ASTM) D6433 distress identification catalog asphalt concrete list of distresses.

Distress ID	Description	Unit of Measure	Group	Cause
1	Alligator cracking	Square meters	Cracking	Load
2	Bleeding	Square meters	Surface defects	Other
3	Block Cracking	Square meters	Cracking	Load
4	Bumps and sags	Linear meters	Visco-plastic deformations	Load, climatic, other
5	Corrugation	Square meters	Visco-plastic deformations	Climatic, other
6	Depression	Square meters	Visco-plastic deformations	Other
7	Edge cracking	Linear meters	Cracking	Climatic
8	Joint reflection	Linear meters	Cracking	Climatic
9	Lane/shoulder drop-off	Linear meters	Visco-plastic deformations	Other
10	Longitudinal and transverse cracking	Linear meters	Cracking	Climatic
11	Patching and utility cut patching	Square meters	Others	Other
12	Polished aggregate	Square meters	Surface defects	Traffic
13	Potholes	Number	Potholes	Traffic, load
14	Railroad crossing	Square meters	Others	Other
15	Rutting	Square meters	Visco-plastic deformations	Load
16	Shoving	Square meters	Visco-plastic deformations	Other
17	Slippage cracking	Square meters	Cracking	Traffic
18	Swell	Square meters	Visco-plastic deformations	Climatic
19	Raveling	Square meters	Surface defects	Other
20	Weathering (surface wear)	Square meters	Surface defects	Other

USACE subsequently developed the commercial software PAVER^{TM} (version 7.0, Colorado State University, Fort Collins, CO, USA) (PAVER 2014), which uses the ASTM PCI calculation for describing the pavement's condition and predicts future pavement conditions, helping managers to determine priorities and the optimal time to perform repair and maintenance activities. PAVER^{TM} has been widely used by the U.S. Department of Defense (DOD) for the management of U.S. military airfields worldwide.

However, it is necessary to use an adequate Distress Identification Catalogue reflecting the characteristics of pavement surveyed and a standardized procedure to make repeatable investigations for the entire network. To this end, some US highway agencies that apply PMS procedures have developed internal reference catalogs for pavement distress identification and use overall pavement condition indicators based on PCI evaluation procedure and ASTM D6433 distress identification catalog. Each distress rating manual is unique and adapted to state agency local conditions (Pierce, McGovern et al. 2013). Nevertheless, LTPP Distress

Identification Manual (Miller and Bellinger 2014) is another highlighted reference although was not explicitly designed as a pavement management tool; the *Distress Identification Manual* aimed to provide a standardized terminology and definitions of pavement condition distresses. Miller and Bellinger (2014) suggested: "*to modify survey procedures (but not the definitions) contained in the manual to meet road agency-specific needs, taking into account the desired level of detail, accuracy and timeliness of information, available resources, and predominant types of distress within the study area*."

Some authors (Gharaibeh, Zou et al. 2009, Papagiannakis, Gharaibeh et al. 2009, Gharaibeh, Freeman et al. 2011) as a part of a Texas DOT (TxDOT) project summarized the use of pavement scores by US states, including the rating methods used, the score scales, and descriptions. Particularly, they did a complete literature review comparing the level of agreement among six condition indexes from five DOTs in the United States, testing pavement data obtained from the Pavement Management Information System of the TxDOT.

Some pavement state agencies are using the pavement score indexes combined with another measurement of pavement functional condition, such as roughness regarding IRI. More recently, some researchers are working to the development of pavement performance models combining the pavement functional condition expressed by overall pavement condition indicators with a pavement structural capacity indicator, such as pavement deflection (Flora 2009), or structural capacity indicators within pavement deflection devices or CCD devices (Bryce, Flintsch et al. 2012, Bryce, Flintsch et al. 2016, Katicha, Ercisli et al. 2016, Katicha, Flintsch et al. 2017, Shrestha, Katicha et al. 2018).

2.3.2 VIRGINIA DOT: CRITICAL CONDITION INDEX (CCI)

The Virginia DOT summarizes pavement condition data using a CCI pavement rating, a numerical composite index ranging between 100 (perfect condition) and 0 (failed condition). This index, similar to PCI, reflects the road surfaces condition as a function of the type, level of severity and quantity of distress (McGhee 2002, Shahin 2005). The CCI is determined as the minimum of two rating indexes Eq. 2-14, the Load Related Distress (LDR) and Non-Load Related Distress (NDR) (McGhee 2004), see Table 2-8.

Table 2-8. VDOT CCI Flexible Pavement Distresses (after McGhee (2002))

LOAD RELATED DISTRESSES		**NON-LOAD RELATED DISTRESSES**	
Distress	**Description**	**Distress**	**Description**
Alligator Cracking	• One longitudinal crack in the wheel path, or • Interconnected cracks having a chicken wire or "alligator hide pattern"	**Bleeding**	• Excess asphalt cement, or • Surface appears shiny or glassy
Delaminations	Areas of pavement surface missing due to the loss of adhesion between the surface and underlying layers.	**Block Cracking**	A pattern dividing pavement into approximately rectangular pieces of less than or equal to 6 feet by 6 feet.
Patching	• Areas of pavement replaced, or • Areas of pavement covered with new material	**Linear Cracking**	Longitudinal cracks outside the wheel paths or transverse cracks.
Potholes	Bowl-shaped holes in the pavement surface, usually extending through more than one pavement layer.	**Reflection Cracking**	Relatively straight Transverse cracks directly over joints and cracks in underlying PCC pavement
Rutting	Longitudinal depressions of the pavement surface in the wheelpaths.		

LDR and NDR indexes are deduct-value indexes having a value of 100 when the pavement being evaluated has no obvious distress. The magnitude of deduct-value is related to the distress type as well as the severity and frequency of occurrence of that load-related distresses and non-load related distresses, respectively.

$$\mathrm{CCI} = \min(\mathrm{LDR}, \mathrm{NDR}) \qquad \text{Eq. 2-14}$$

VDOT deduct value master curves are mainly based on ASTM D6433 PCI procedure, deduct values were derived through consideration of the deducts VDOT personnel had become accustomed to using them(McGhee 2002). Additionally, it should be noted that VDOT deduct value's curves were provided in an analytical form simplifying the computerization of the CCI value calculation.

Recently, VDOT contracted out pavement distress data collection and automated the calculation of CCI thought the processing of digital images (Chowdhury 2012).

2.3.3 NATIONAL PARK SERVICES: PAVEMENT CONDITION RATING (PCR)

The Federal Highway Administration (FHWA), Road Inventory Program (RIP) for the National Park Service (NPS), collects roadway condition data on paved asphalt surfaces including roads, parkways, and parking areas in national parks nationwide. FHWA RIP referenced LTPP *Distress Identification Manual* (Miller and Bellinger 2014) for the distress type definitions considered (FHWA 2009).

FHWA RIP uses the Pavement Condition Rating (PCR) as pavement condition indicator Eq.2-15, which is a combination of pavement roughness in terms of IRI (Roughness Condition Indicator-RCI, Eq.2-16) and pavement surface distresses (Surface Condition Rating-SCR, Eq.2-17).

$$\mathrm{PCR} = (0.6 \cdot \mathrm{SCR}) + (0.4 \cdot \mathrm{RCI}) \qquad \text{Eq.2-15}$$

The calculation of the RCI was done using the correspondence between IRI values and RCI rating scale. To simplify the computerization of the PCR procedure, FHWA RIP provided a relationship between RCI and IRI values, starting from threshold values reported in Table 2-9.

$$\mathrm{RCI} = 32 \cdot 5 \cdot 2.718282^{\,-0.0041 \cdot \mathrm{avg\,IRI}} \qquad \text{Eq.2-16}$$

Table 2-9. Roughness Condition Indicator, RCI (after FHWA (2009))

IRI - RCI Range Description			
Rating Category	**IRI value range (inch/mi)**	**IRI value range (m/km)**	**RCI value range**
Excellent	<=127	<=2.004	95-100
Good	128-154	2.02-2.431	85-94
Fair	155-240	2.446-3.788	61-84
Poor	>240	>3.788	<=60

SCR is computed through five single surface distress indexes (alligator cracking, longitudinal cracking, transverse cracking, patching and rutting) based on a scale ranging from 0 (failed condition) to 100 (perfect condition). Single distress indexes are calculated for each distress type, see Table 2-10,where the values %LOW, %MED and %HI report the percentage of the observed pavement that contains alligator cracking within the respective severities.

$$\mathrm{SCR} = 100 - \begin{bmatrix} (100 - \mathrm{AC\_INDEX}) + (100 - \mathrm{LC\_INDEX}) + (100 - \mathrm{TC\_INDEX}) + (100 - \mathrm{PATCH\_INDEX}) \\ + (100 - \mathrm{RUT\_INDEX}) \end{bmatrix} \qquad \text{Eq.2-17}$$

Table 2-10. Single Distress Indicator Formulas, SCR (after FHWA (2009))

Single Distress Index Indicator	**Single Distress Index Indicator formula**
Alligator Cracking AC_INDEX	$\mathrm{AC\_INDEX} = 100 - 40 * [(\%\mathrm{LOW}/70) + (\%\mathrm{MED}/30) + (\%\mathrm{HI}/10)]$
Longitudinal Cracking LC_INDEX	$\mathrm{LC\_INDEX} = 100 - 40 * [(\%\mathrm{LOW}/350) + (\%\mathrm{MED}/200) + (\%\mathrm{HI}/75)]$
Transverse Cracking TC_INDEX	$\mathrm{TC\_INDEX} = 100 - [20 * (\%\mathrm{LOW}/15.1) + (\%\mathrm{MED}/7.5)] + [40 * (\%\mathrm{HI}/1.9)]$
Patching PATCH_INDEX	$\mathrm{PATCH\_INDEX} = 100 - 40 * (\%\mathrm{PATCHING}/80)$
Rutting RUT_INDEX	$\mathrm{RUT\_INDEX} = 100 - 40 * [(\%\mathrm{LOW}/160) + (\%\mathrm{MED}/80) + (\%\mathrm{HI}/40)]$

2.3.4 WISCONSIN DOT: PAVEMENT SURFACE EVALUATION AND RATING (PASER)

PASER methodology developed by Wisconsin DOT is intended to assist local managers in understanding and rating pavement surface condition. PASER methodology is based on windshield survey. Distress severity and quantity is estimated without direct measurements. It is implemented on the computerized pavement management system PASERWARE (Walker, Entine et al. 2002). PASER rating scale ranges from 10 (perfect condition) to 1 (failed condition). Pavement surface rating is obtained with an understanding of surface distress surveyed. PASER rating value reflects the pavement life cycle phase that is represented by representative photograph included in PASER manual. Associated with the surface rating, PASER methodology suggests an appropriate repair and maintenance treatment. Table 2-11 summarizes the PASER methodology rating scale and suggested/appropriate repair and maintenance treatment.

Table 2-11. PASER Rating Scale Categories and Methodologies (after Walker, Entine et al. (2002)

Surface rating	Visible distress	General condition/ treatment measures
10 Excellent	None.	New construction.
9 Excellent	None.	Recent overlay. Like new.
8 Very Good	No longitudinal cracks except reflection of paving joints. Occasional transverse cracks, widely spaced (40’ or greater). All cracks sealed or tight (open less than ¼”).	Recent sealcoat or new cold mix. Little or no maintenance required.
7 Good	Very slight or no raveling, surface shows some traffic wear. Longitudinal cracks (open ¼”) due to reflection or paving joints. Transverse cracks (open ¼”) spaced 10’ or more apart, little or slight crack raveling. No patching or very few patches in excellent condition.	First signs of aging. Maintain with routine crack filling.
6 Good	Slight raveling (loss of fines) and traffic wear. Longitudinal cracks (open ¼”– ½”). Transverse cracks (open ¼”– ½”), some spaced less than 10’. First sign of block cracking. Sight to moderate flushing or polishing. Occasional patching in good condition.	Shows signs of aging. Sound structural condition. Could extend life with sealcoat.
5 Fair	Moderate to severe raveling (loss of fine and coarse aggregate). Longitudinal and transverse cracks (open ½” or more) show first signs of slight raveling and secondary cracks. First signs of longitudinal cracks near pavement edge. Block cracking up to 50% of surface. Extensive to severe flushing or polishing. Some patching or edge wedging in good condition.	Surface aging. Sound structural condition. Needs sealcoat or thin non-structural overlay (less than 2”)
4 Fair	Severe surface raveling. Multiple longitudinal and transverse cracking with slight raveling. Longitudinal cracking in wheel path. Block cracking (over 50% of surface). Patching in fair condition. Slight rutting or distortions (½” deep or less).	Significant aging and first signs of need for strengthening. Would benefit from a structural overlay (2” or more).
3 Poor	Closely spaced longitudinal and transverse cracks often showing raveling and crack erosion. Severe block cracking. Some alligator cracking (less than 25% of surface). Patches in fair to poor condition. Moderate rutting or distortion (greater than ½” but less than 2" deep). Occasional potholes.	Needs patching and repair prior to major overlay. Milling and removal of deterioration extends the life of overlay.
2 Very Poor	Alligator cracking (over 25% of surface). Severe rutting or distortions (2” or more deep). Extensive patching in poor condition. Potholes.	Severe deterioration. Needs reconstruction with extensive base repair. Pulverization of old pavement is effective.
1 Failed	Severe distress with extensive loss of surface integrity.	Failed. Needs total reconstruction.

2.3.5 PAVEMENT CONDITION ASSESSMENT IN THE EUROPEAN SCENE

It is important to say that the above-cited methodologies for a distress identification catalogue, such as ASTM D6433 (ASTM 2018), are suited for roadways in the United States, and not for pavement distresses frequently seen on European roads.

So far European experiences are very limited, mainly being concentrated on airfield pavements. In Ireland (Mulry, Jordan et al. 2015), the United Kingdom (Coley 2011), and the Netherlands (Drenth 2001), some procedures and guidelines have been development to use the PCI methodology (Shahin 2005) and PAVER^{TM} (PAVER 2014) to assess pavement condition and define pavement maintenance policies.

However, the European background in distress identification and management of roadway pavements is limited. The French institute of science and technology for transport, spatial planning, development and networks (in French: *Institut français des sciences et technologies des transports, de l'aménagement et des réseaux* IFSTTAR) has developed guidelines (Bertrand, Boutonnet et al. 1998) for pavement distress identification in roadways. Additionally, only recently, in Ireland, a study of the Irish regional road network in 2012 (Mulry, Feighan et al. 2015) includes the assessment of the condition of pavement surfaces using a simplified index from 1 (failed pavement) to 10 (perfect conditions), based on above mentioned Pavement Surface Evaluation and Rating (PASER) rating scale, (Walker, Entine et al. 2002).

2.3.6 PAVEMENT MANAGEMENT SYSTEMS IN URBAN NETWORKS

Most of the maintenance management systems available were created to manage large networks major road and airport infrastructures. Nowadays, because of the economic crisis, even the small administrations are trying to find an effective tool to allocate their reduced budget on their road property.

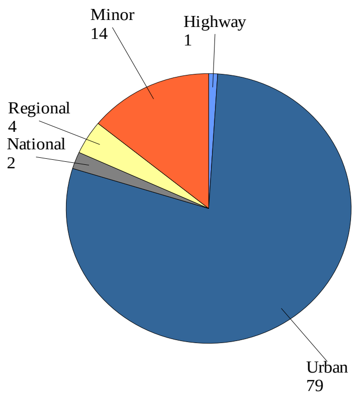


Figure 2-17. Distribution of road types on an Italian road network (Loprencipe, Pantuso et al. 2017)

Municipalities have to face the maintenance of a huge number of roads with a reduced budget. Moreover, urban roads are a large percentage of total roads in most of the countries all over the world. Particularly, in Italy, they amount to 79% (Figure 2-17).

However, the implementation of PMS practices in urban areas is a challenge for road pavement agencies because the different configuration of highway vs. urban networks. Some of the main differences between them are overviewed in the following table, see Table 2-12.

Table 2-12. Highway vs. Urban Network Configurations

Urban Areas	**Rural Highway**
Non homogeneous network: different speed for each branch	Homogeneous network: same speed for each branch.
Work zone and inspection requires usually traffic disruption	Work zone and inspection can be done in one lane of the road with no traffic disruption
M, R&R treatment create congestion and circulations problems	M, R&R treatment can be schedule and alternative routes can be proposed minimizing circulation problems.
Usually non-scheduled budget	Scheduled budget (by contract)
A complete inspection in urban areas is rarely performed and very expensive, because the use of automated data collection technologies is expensive and rarely performed.	It is easy to make a complete inspection of the network, by using automated distress data collection technologies.
Urban PMS is usually performed by internal staff, not specialised and using in-house data collection technologies.	Data collection and processing is usually contracted out and performed by private contractors with specialized staff.

Many PMSs include both the visual and automatic surveys in their procedures making the connections among the different indexes understandable. One of the most important features of the urban roads is the evenness that is related to the performance of the road such as skid resistance, noise and user comfort. Considering the total length of a road network, generally, the evenness is measured by high performance equipment (road profiling, profilograph, etc.) that can detect a longitudinal profile in a short period of time and the acquired data are analyzed with the well-known IRI.

These techniques have a wide use in highways and airports, but their application is uncommon in urban roads. The main reason for this scarcity of use is the need of an initial high investment and qualified staff to manage the acquired data. Therefore, the municipalities generally prefer to use a simple visual survey. The solution to this problem can be the use of experimental laws that link the index coming from the visual survey (e.g., PCI) to other indexes resulting from an automatic survey. Recently, pavement researchers have considered the IRI to reappraise urban roads management systems (Rens and Staley 2009, Arhin, Williams et al. 2015) relating this index to the pavement condition in terms of PCI which proposed to use previously PCI and IRI data of sections of roadways to establish a relationship between the PCI value and IRI. One of the major problems of pavement unevenness evaluation with IRI is the speed of the road: the IRI may be used to assess pavement in roads with speed limits above 80 km/h. In the case of urban roads, it is necessary to adapt the IRI thresholds, or use other indexes depending on vehicle speed, noise and user comfort (Cantisani and Loprencipe 2010, Cantisani, Fascinelli et al. 2013, Loprencipe and Zoccali 2017). Another reason for the limited use of the IRI for urban roads is its irrelevance to those surfaces with localized roughness, such as manholes, small patches, and pavement differences at crossings. Therefore, the application of PMS to urban areas are still very limited; as only recently, some researchers have been working to adapt the PCI rating system for its use in urban areas, to manage pavement maintenance of sidewalks (Corazza, Di Mascio et al. 2016) and roadway pavements (Cottrell, Bryan et al. 2009).

Recently, researchers (Chamorro, Tighe et al. 2009, Chamorro, Tighe et al. 2010, Osorio, Chamorro et al. 2014) have proposed the development of a comprehensive PCI called the Urban Pavement Condition Index (UPCI) for the assessment of Chilean urban pavements. They have developed new distress evaluation guidelines, including the principal distresses present in the urban network and not considered in ASTM D6433, such as catch basins and manhole covers, and including pavement roughness measure by means of IRI, see Table 2-13.

Table 2-13. Urban Pavement Distresses considered Chilean Urban Pavement Condition Index (after Osorio, Chamorro et al. (2014))

Pavement Distress	Number of Severity Levels	Units quantity measurement
Fatigue cracking	Three	m2
Edge cracking	Three	m2
Block cracking	Three	m
Wheelpath longitudinal cracking	Three	m
Nonwheelpath longitudinal cracking	Three	m
Reflection cracking	Three	m
Transverse cracking	Three	m
Patch deterioration	Three	no.,
Potholes	One	no.,
Rutting	One	mm
Shoving	One	no.,
Bleeding	One	no.,
Polished aggregate	One	no.,
Raveling	One	no.,
Curb deterioration	One	m
Water bleeding and pumping	One	no.,
Manholes and catch basins	Three	no.
Roughness (IRI)	One	m/km

In India (Shah, Jain et al. 2012, Shah, Jain et al. 2013), and in other developing countries (Adedimila, Olutaiwo et al. 2009, Bonin, Folino et al. 2017), some researchers have developed methods combining different distress index equations and PCI using a reduced number of distresses starting with the ASTM D6433 catalogue. To each distress, different weights were assigned based on expert opinion to take into account their influence on overall pavement conditions.

The absence of a road pavement inventory and maintenance planning is the major problem detected in small town road management in Italy. This situation leads to poor maintenance planning. The maintenance and repair works happen to be performed when the level of distress become critical in some network elements. This results in large inhomogeneity in the network. This situation can be avoided with a sustainable PMS flexible enough to adapt the method to the current situation and future demands.

2.4 PAVEMENT DETERIORATION MODELING AT THE NETWORK LEVEL

Road agencies in the United States have developed deterministic regression models for network-level pavement condition prediction since the 80’s (George, Rajagopal et al. 1989). The US Army Corps of Engineers (USACE) proposed the approach to use the data of sections grouped by their similar characteristics in pavement families (i.e. homogeneous sections) to develop pavement deterioration models (Nunez and Shahin 1986, Shahin, M et al. 1987, PAVER 2014). According to this approach, a pavement family should have the same characteristics as pavement type surface, belongs to the same maintenance system according to functional classification, similar traffic levels and similar repair and maintenance records. It is assumed that sections belonging to the same pavement family have the same pattern of deterioration over time and this pattern is considered to be representative of the overall performance of that pavement family.

The development of performance models for pavement families approach is currently used and implemented in agencies' PMS software by many US DOTs, such as TxDOT (Stampley, Miller et al. 1995), Indiana DOT (Gulen, Zhu et al. 2001), Louisiana DOT (Khattak, Baladi et al. 2009), Penn DOT (Wolters and Zimmerman 2010), Delaware DOT (Mills, Attoh-Okine et al. 2012), Colorado DOT (Saha, Ksaibati et al. 2017), and North Carolina DOT (Chen and Mastin 2016).

The use of family modeling techniques allows the agencies to incorporate simple default models capable of predicting the average behavior/pattern of a pavement family into the agency PMS. However, pavement deterioration and performance is highly variable due to many factors that introduce heterogeneity, bias, and uncertainty in the pavement condition. Furthermore, there often are important factors that affect pavement deteriorations that are not included in the model. These sources of uncertainty and high variability appear as “noise” that could affect expected pavement deterioration rates, and deterioration patterns that could be incoherent with engineering judgment (Pierce, McGovern et al. 2013). In fact, due to the high level of heterogeneity bias in pavement prediction (Prozzi and Madanat 2004, Chen and Mastin 2016), the pavement condition of specific sections will differ from the average model-predicted pavement condition.

Particularly, many deterioration models used in PMS have the limitation that predictions does not accurately reflect actual pavement conditions existing a gap between pavement network models and the specific pavement sections (Giummarra, Martin et al. 2007). Some authors have worked on the development of mathematical methods used in other disciplines, such as the Empirical Bayes approach (Zellner 1996) to combine the existing knowledge (prior) and information obtained from recent observations. Some researchers have been working to apply this approach to pavement deterioration modeling based on update pavement deterioration curves using pavement performance monitored in subsequent years (Han and Do 2015). Markov models are widely used in deterioration process of civil infrastructures using Transition Probability Matrices (TPM) between discrete condition states (CS), bayesian inference are used to periodically updating Markovian TPM as new inspection data become available. Tabatabaee and Ziyadi (2013) used Bayesian inference to accommodate uncertainties between expert-derived TPM and measurement error from inspections using Minnesota DOT MnROAD test facility. Hong and Prozzi (2006) studied the Bayesian Approach using the Markov Chain Monte Carlo simulation for Bridge deck deterioration process to update model parameters as data become available. Prozzi and Madanat (2004) developed pavement performance models using the Empirical Bayesian approach based on the Pavement Serviceability Index (PSI). Abaza (2016) proposed a technique based on the “back-calculation” of the discrete Markov model using two consecutive cycles of pavement distress assessment. Other researchers used the Bayesian approach to analyze pavement data average pavement indicators and identify homogenous sections (de León Izeppi, Flintsch et al. 2011, Cafiso and Di Graziano 2012).

More recently, Katicha *et al.* (2016) proposed a pavement model for the Virginia interstate network that includes the structural condition of the pavement as variable based. This model assumes that the deterioration process can be expressed as the NB regression and uses the LEB approach to calculate better estimation of the future pavement condition.

2.5 PAVEMENT PRESERVATION STRATEGIES EXPERIENCES

Pavement preservation has grown to become a standard practice in most highway agencies following different objectives related to the desire to improve overall pavement performance, to have a greater attention to customer satisfaction, and to the need of rehabilitation of pavement assets with a restricted budget (Peshkin 2011).

Consequently, pavement performance have become a critical issue to be monitored overtime to develop PMS cost-effective strategies, therefore, pavement states agencies along the USA are increasing their interest in developing studies on pavement performance (FHWA 2017). Recently, US federal government is requiring for the establishment of performance measurements in the National Highway System (NHS) for bridge and pavements. This requirement is founded on the Moving Ahead for Progress in the 21st Century Act (MAP-21), funding to govern USA federal surface transportation spending. As a consequence, some researchers are working on the performance measurements repeatability and the quality control and assurance (QA/QC) procedures to be implemented in the Interstate System (Rada, Nazarian et al. 2016, Visintine, Simpson et al. 2018) aiming to establish the guidelines for pavement condition sampling and performance measurements, using a combination of several metrics collected by State DOTs. These metrics provided a way to quantify pavement condition in terms of roughness (IRI), cracking, rutting, and faulting (intended as misalignment between concrete slabs for jointed concrete pavement surfaces).

Pavement M, R&R represent the first voice of most US DOT (Dong and Huang 2011), more than a third part of the total budget, followed nearly for highway construction (VDOT 2017). Evaluate the effectiveness and costs of maintenance strategies is a key issue for road agencies (de la Garza, Akyildiz et al. 2011) as a source of information to forecast the future pavement condition and select the right maintenance treatment to keep the pavements in good condition (Chen, Zhu et al. 2017). Therefore, road agencies are increasing their efforts in evaluate the M, R&R treatments' expected performance (Peshkin, Hoerner et al. 2004), that is, the maintenance effectiveness, appraised through the use of pavement performance indicators, such as, the roughness (IRI), cracking data, PCI metrics before and after the maintenance treatments as well as the time between maintenance projects on network pavement sections (Durango-Cohen and Madanat 2008, Dong, Huang et al. 2013, Dong, Huang et al. 2014, Chen, Zhu et al. 2017).

To the computation of the benefit associated with a maintenance treatment requires to know the performance previous to the application of M, R&R treatments (do nothing curve), the change in the performance condition, performance jump, and the performance after the maintenance treatment (post-treatment curve). Several researchers have treated this problem before evaluating the effectiveness of maintenance treatments calculating the benefit area as the area between the post-treatment curve and the pretreatment curve (Do nothing curve). Other researchers use performance analysis to calculate maintenance treatment service life, considering the end of service life when the performance model prediction reach the lower threshold (Peshkin, Hoerner et al. 2004). However, as highlighted by Dong and Huang (2011), this approach usually leads to high service life values (around 10-15 years) with pavement sections characterized by critical distress conditions requiring rehabilitation activities. Therefore, they suggested calculated the pavement service life as the time between two maintenance treatments on the section.

The performance indicators used among the existing research available are the IRI (Ouyang and Madanat 2004, Peshkin, Hoerner et al. 2004, Dong and Huang 2011, Chen, Zhu et al. 2017), Present Serviceability Index (PSI) (Dong, Huang et al. 2013, Dong, Huang et al. 2014), and, more recently, using the Critical Condition Index (CCI) (de León Izeppi, Morrison et al. 2015).

Maintenance, Repair, and Rehabilitation (M, R&R) works stored in the database is a very important source of information because it can help to identify the effectiveness of different treatments to maintain high service levels and would give information about the value generated in terms of benefits achieved for every euro invested in maintenance and repair treatments. However, there is still uncertainties and lack information about the functional performance of pavement preservation treatments, and their effects on extending pavement service life.

Continuous pavement performance, maintenance treatment effectiveness and expected life of the various treatments would help roadway managers to move toward a more proactive maintenance maintenance approach, avoiding ineffective “worst first” reactive approaches. Cost-effective approaches would optimize the roadway investment, enhancing the maintenance treatment selection.

2.6 PAVEMENT DATA MANAGEMENT ISSUES: NETWORK LEVEL CONSIDERATIONS

Pavement data collection techniques condition indicators used in the current state of practice involves a large quantity of data, and its correct management requires information from a large variety of sources. A cost-effective PMS should address the challenge of unifying the understanding between the pavement condition data and the use of data to support pavement management decisions. Therefore, road agencies need specific data quality and data cleansing criteria to support data processing and the decision-making strategies and spatial location procedures to integrate pavement data involved in the comprehensive PMS. Another issue pertaining with network-level pavement management is the sampling intervals and sectioning used by road agency. Bennett, De Solminihac et al. (2006) did a complete literature review of data collection issues for PMS applications.

2.6.1 LINEAR REFERENCING SYSTEMS AND SPATIAL POSITIONING SYSTEMS

Linear referencing system (LRS) is a method of spatial referencing, which consists of a set of tools and procedures to describe the location of features regarding the measurements as a distance or offset along a linear element (i.e., road), from a point of known location.

Road agencies use LRS to support their management systems as a useful tool to locate their data along with their road systems. Traditionally, PMS applications use LRS by road name and kilometer posts physically established along the road to refer events ‘locations.

An event is a linear, continuous, or point feature that occurs along a road feature (i.e., pavement distress data, roughness, FWD deflections, surface texture/friction measurements). In the road name and km-post referencing method, each roadway is given a unique name and number, and the distance along the route from a specific origin is used to locate points along the route.

Highway State Agencies in the USA use LRS because the FHWA through the HPMS program imposed as a federal requirement that the Highway State DOT must report annually on the condition and use of highways in a geospatial format that leverages linear referencing.

Besides, data collection technologies use LRS for recording data. The data are recorded between a start and an end point, using measures of distance or time along the lines, using calibrated odometers. The use of intermediate reference points (kilometers posts) improves the overall accuracy by limiting any accumulative error in the distance measurements, and the surveyed lengths are increased or decreased to match the accepted length of the section, and the data adjusted accordingly.

Linear referencing is also used to associate multiple sets of attributes to portions of linear features without requiring that underlying lines be segmented (split) each time that attribute values change. For example, most road centerline feature classes are segmented where three or more road segments intersect and where the road names change. This approach is very convenient when you want to combine multiple attributes of the roadway to understand where changes occur.

Some of the limitations of the LRS is the location of features when there is a modification of the alignment a road. For this reason, LRS should be continuously updated by road agency.

Nevertheless, PMS data collected can be related by its spatial locations using spatial referencing tools using their coordinates (x,y) or (latitude, longitude) obtained from Geographical Positioning Systems (GPS) of event features along pavement roads. The use of the spatial referencing systems in conjunction of LRS can be appropriate to integrate PMS data retrieved from different sources, providing consistent references for their management and for their representation that can used to report PMS analysis. The use of Geographical Information Systems (GIS) and web-based applications, such as Google Earth®, Bing Maps®, Here Maps® using WSG84 datum, would help road agencies to visualize data collected (Flintsch, Dymond et al. 2004). Nevertheless, the integration of PMS and their consistency should be analyzed carefully using specific matching and merging procedures based on data available and LRS implemented within the managed network.

2.6.2 SAMPLING INTERVALS AND SECTIONING, SURVEY FREQUENCY

One of the basic principles in network-level management and pavement deterioration modeling is that the analysis should be carried out on homogeneously/geographically located along road sections. Therefore, data collection and data interpretation are highly influenced by the agency sampling procedures, the survey frequency and the consistency between pavement condition inspections and M, R&R records on the managed network.

The sampling interval is the frequency along the road that data are collected. Appropriate pavement sampling interval has a significant influence on the cost of data collection and data processing as well as the quality of data.

The use of automated pavement data collection devices allows collecting an extensively high quantity of data in short times with high sampling, using integrated systems that consistently provide spatial information. Therefore, data collection and usability get increased.

Nevertheless, PMS data interpretation and analysis are traditionally based on the "homogeneous sections" concept, where sections have associated uniform/homogeneous attributes. Data quality and processing should take into account the importance of creating analysis sections and assure time history consistency between the pavement data retrieved from pavement inspections and the M, R&R records on the managed network.

2.7 SUMMARY OF LITERATURE REVIEW

Different tools are available in the current state of art and practice for the implementation of pavement management systems. However, they were focused for highway rural roadway conditions, and they are developed by road agencies which have implemented mature PMS. This dissertation proposed the development of a comprehensive set of tools to design and enforce pavement management applications at the network-level. Therefore, the objective of this dissertation was to include several methodologies in different pavement implementation stages with the aim of providing a set of tools and guidelines attempting to fill the gap in the current state-of-art. The limitation of current state-of-art have been encountered in different topics and opportunities for improvement developed:

- **Pavement Distress Identification Methodologies and flexible PMS implementations in Urban Areas**. Several pavement data collection methodologies have been studied, but they do not suit urban pavement networks. Pavement surface distress identification methodologies and distress definitions are adapted for rural roadways and not for urban systems. Some of the standard distresses have been not encountered in urban areas, and others with several occurrences in urban areas are not defined on standard catalogs, such as utility holes and tree roots. Nevertheless, the absence of pavement inventory and poor maintenance planning are one of the major problems in urban areas. The opportunity of the improvement is the development of distress identification guidelines and PMS tools. These tools should be simple, flexible and adaptable to local conditions to fulfill the challenge of European urban administrations who have to deal with limited funding and limited technical staff.
- **Pavement Condition Evaluation Indicators.** Several overall pavement condition indicators have been studied which are very useful for road agencies because they provide a scale (0, failed pavement-100, perfect pavement) to evaluate the functional pavement remaining service life. Those indexes have the advantage that can be analyzed and can be compared because most of them use the same scale of evaluation. However, pavement data collected is very influenced by data collection techniques available within pavement agency available procedures and by the characteristic of the network. The opportunity for improvement is the development of a methodology for a better evaluation of distress combination or aggregate index to best fit network-level characteristics. A further aim is to provide software to the road agency for the automatic and computer-aided calculation of overall pavement condition indicators including new distress definition for flexible pavements.
- **Network-level pavement management issues.** Pavement data is affected by high variability and noise inherent by the data processing and interpretation. The opportunity for improvement is the development of Linear Referencing Systems and spatial location methodologies to facilitate the data processing and management tools for the spatial matching of pavement data along the network will be provided to assure the consistency of time-history management sections and the merging with different attribute table and properties on the database.
- **Data performance modeling.** Modeling the pavement deterioration process is essential for a successful Pavement Management System (PMS). The pavement deterioration process is

profoundly influenced by the uncertainties related to the data acquisition and condition assessment. Therefore, road agencies need accurate pavement deterioration prediction models. The opportunity for improvement is the development of pavement deterioration model to obtain a better estimate of future pavement condition to account for the error in pavement deterioration data collection and interpretation addressing the critical need of predicting the future pavement condition to support network-level decision making.

- **Decision-Making process analysis and cost-effectiveness of M, R&R strategies.** One of the primary objectives of an agency is to maintain the entire network managed to a high level of service. Road agencies need to perform M, R&R on their managed network to achieve that goal. The opportunity for improvement is to evaluate pavement management strategies effectiveness by the analysis of M, R&R records within the PMS database. Time series pavement condition analysis would be useful to evaluate pavement performance comparing M, R&R treatment performed.

CHAPTER 3. PAVEMENT DISTRESS IDENTIFICATION GUIDELINES AND CONDITION ASSESSMENT IN URBAN AREAS

3. PAVEMENT DISTRESS IDENTIFICATION GUIDELINES AND CONDITION ASSESSMENT IN URBAN AREAS[1]

3.1 INTRODUCTION

A large majority of European citizens live in an urban environment where the links between transport and health are particularly pronounced. Therefore, urban pavement agencies are challenging to maintain the urban network managed at high level of service to fulfill user's expectations. To become successful, urban pavement agencies need easy-implementation PMSs to develop strategies to maintain, preserve and rehabilitate urban roads. As highlighted in Chapter 2: Literature review, this process involves the development of specified procedures for the assessment and identification of urban pavement distresses and the update of distress identification catalogues available on literature, such as ASTM D6433 (ASTM 2018), that are suited for roadways in the United States, and not for pavement distresses frequently seen on of European urban roads. Therefore, the research presented in this Chapter attempts to fill the existing gap in the state-of-the-art in urban pavement maintenance by introducing a simple, effective and affordable pavement distress identification and PCI calculation procedure.

3.2 OBJECTIVE AND FRAMEWORK

The objective of this research was the development of a methodology (see Figure 3-1) for the definition of customized pavement agency for urban areas. This research includes a procedure for automating ASTM D6433 PCI calculation through the digitalization DV master curves. This algorithm allows the construction of new distress DV to include new distress suited definitions for urban areas, such as manholes and tree roots, which are not collected until now in ASTM D6433 Distress Identification catalogue. As a result, a Distress Identification Manual for Urban Road Pavements was developed, see Appendix A. This research uses flexible pavement data of an Italian road network.

The main objectives of this methodology are to:

1. Perform a statistical analysis of distresses observed in an Italian road.

---

[1] Partial results of this research have been published as "Loprencipe, G., & Pantuso, A. (2017). A Specified Procedure for Distress Identification and Assessment for Urban Road Surfaces Based on PCI. *Coatings, 7*(5), 65. "

2. Determine how the ASTM D6433 distress definition could be upgraded to suit the pavement condition of urban roads looking for the identification, improvement and simplification of the pavement distress identification catalogue.
3. Find an analytical procedure to determine the ASTM D6433 deduct value curves for Asphalt Concrete (AC) roadway pavement distresses (20 distresses).
4. Achieve a simplified new distress identification catalogue for urban road AC pavements to assess the pavement condition at a PCI-based scale. Evaluate the reliability and the accuracy of the new methodology by comparing the PCI values calculated and those with the ASTM D6433 catalogue in an Italian urban road network (109 sample units).
5. Implement the ASTM D6433 PCI calculation procedure for urban roads through a Visual Basic for Application (VBA) language-based program.
6. Obtain new deduct value curves through the combination of ASTM D6433 distress deduct value curves to evaluate observed distress in Italian urban road networks that are not defined in ASTM D6433.

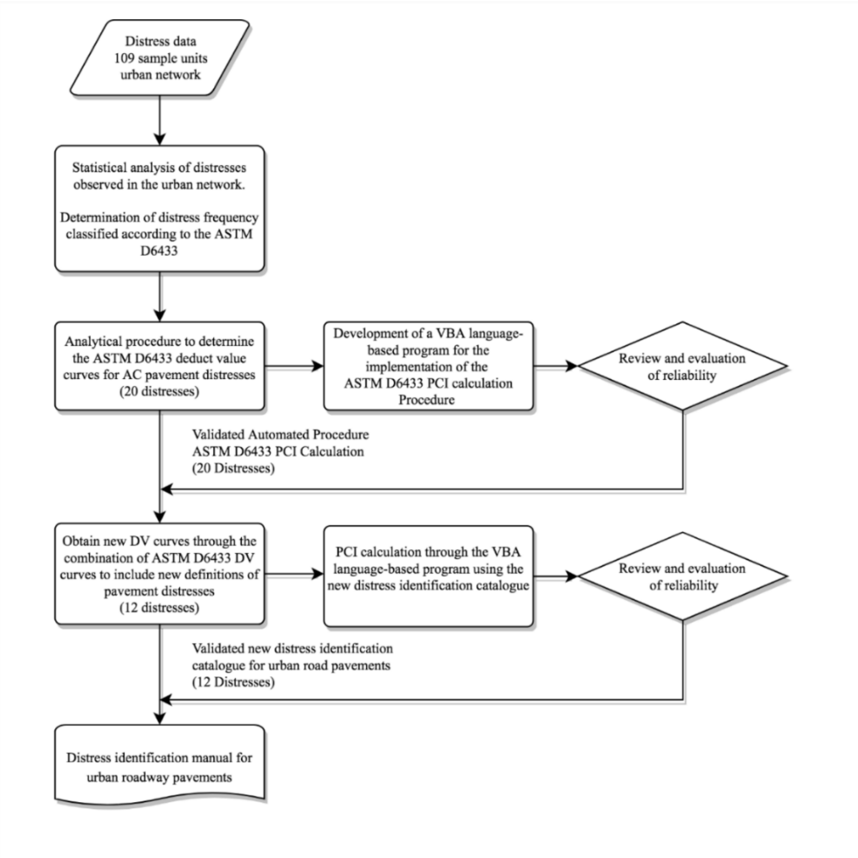


Figure 3-1. Urban Pavement Distress Identification and Assessment Methodology based on the PCI (after Loprencipe and Pantuso (2017))

3.3 THE PCI METHOD: VISUAL SURVEY ON VALENTANO ROAD NETWORK

As highlighted in Literature review (see § 2.3.1), the PCI methodology ASTM D6433 (2018), can be a valuable tool for small municipalities because it provides a systematic and rational basis for determining maintenance and repair needs and priorities. PCI can be used to establish the rate of pavement deterioration and early identification of major rehabilitation needs to reduce or defer costly, time-consuming rehabilitation and reconstruction projects.

In the same way the small and medium size towns, the pavement managers do not have an inventory of their road network. The pavement inventory can be created through GIS data and road data mapping.

The described PMS process has been tested on the road network in the town of Valentano, province of Viterbo, Italy. Valentano's road pavement network was ranked by pavement type, traffic data and construction data. Three different networks have been identified as shown in Figure 3-2.

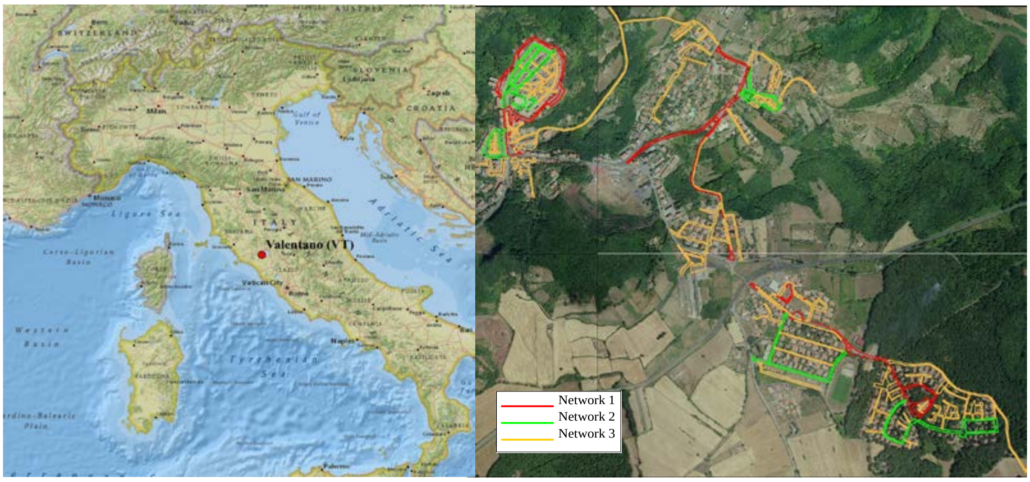


Figure 3-2. Valentano's Network characterization

- ***Network 1*** is the main network of the town. It is 5.47 km long and its function is to connect the provincial road network that connects Valentano with a neighboring town. The type of pavement in this network is flexible.
- ***Network 2*** is the internal traffic distribution network. Most of the pavement type of this network is paved by cobblestone. It is 3.52 km long. In some areas, the level of traffic is high.
- ***Network 3*** has the function to guarantee the access to the residential areas and there are some pedestrian areas as well. It is the longest network in town with a length of

12 km. The level of traffic is low and there is different type of pavement: flexible, rigid and cobblestone.

PCI survey has been performed by two inspectors with simple tools (a hand odometer wheel and a straightedge). They monitored the whole of network without high economical commitments, in a short period of time (a total of two weeks) and with minimal road interruptions. According to ASTM (2018), the network (total area of 33,190 m2) has been divided into branches corresponding to the streets. Each branch has been divided into sections. Each section has been divided into sample units. A sample unit is a defined portion of the pavement section with a standard size range of 225 ± 90 m^2; only a limited number of sample units have to be inspected for the pavement condition calculation of the section. This permits the reduction of time and personnel to carry out the inspection of the whole network. The type of analysis depends on the maintenance policy approach: at the *Network Level* or the *Project Level*.

The *Network Level* approach is defined by “top-down” logic: it aims to optimize the budget allocation for the network maintenance in medium and long terms.

The *Project Level* approach continues the decision-making process focusing on the improvement of the network condition by implementing M, R&R works in certain sections of the networks.

Both at network and project levels, the components of a PMS should provide all elements to planning any kind of action on road surfaces; that is, to be able to choose “how”, “where” and “when” to intervene on the pavements.

A total of 58 Sample Units (SU) at the network level and 127 SU at the project level was inspected (total area 33,190 m2). For the extent of this analysis, a total sample of 109 AC SU (25,711 m2) was considered, excluding the parking lot areas and those SU related to particular types of pavements (low traffic, pedestrian roads, etc.). This sampling is considered as representative of an Italian road network including SU with different construction dates and characteristics.

3.4 PAVEMENT DISTRESS DISTRIBUTION ANALYSIS

To perform a pavement distress urban distribution analysis, a statistical analysis has be performed to determine distress frequency defined according to the ASTM D6433 within the network. Distress density and associated deduct values of each distress type were summed for the entire sample. Distress types were grouped into distinct categories separating those that have a low distress density, those that cannot be grouped with other distresses and those with high density (Table 3-1).

Table 3-1. Distress Distribution Analysis on Case of Study Urban Network

Distress Category	ASTM D6433 Distress Description	Total Density Frequency	Density (%)	Total Deduct value (*DV*)	Deduct value (*DV*) (%)
Low frequency distresses	2. Bleeding; 12. Polished Aggregate; 14. Railroad crossing; 16. Shoving; 17. Slippage cracking; 18. Swell; 20. Weathering (surface wear)	0.49	1	168	1
Cracking	7. Edge cracking; 8. Joint reflection; 9. Lane/shoulder drop-off; 10. Longitudinal and transverse cracking	17.50	49	3805	27
Surface deformations	4. Bumps and sags; 5. Corrugation; 6. Depression	5.08	14	1917	14
Raveling	19. Raveling	2.80	8	2027	15
Alligator cracking	1. Alligator cracking	3.77	10	958	7
Block cracking	3. Block cracking	3.28	9	1467	11
Rutting	15. Rutting	1.18	3	102	1
Patching	11. Patching and utility cut patching	1.84	5	1492	11
Potholes	13. Potholes	0.32	1	1786	13
	Total	**36.26**	**100**	**13,722**	**100**

As observed, there are 7 distresses whose density quantities associated in the inspected urban road network has a very low frequency value (1%); other distresses can be grouped under two different categories by their similar characteristics (4 distresses into category *Cracking* and 3 distresses into category *Surface deformations*). Moreover, there are some distress types that cannot be considered under the same group due to their characteristics or due to frequencies or total deduct values that are very high (*Raveling*, *Alligator cracking*, *Block cracking*, *Rutting*, *Patching and Utility Cut Patching* and *Potholes*). This analysis highlights the influence of weight in the network pavement condition of certain defects that have reduced density values but a higher percentage of total deduct value as *Potholes*. This result agrees with the PCI methodology that assigns a greater weight with respect to other distresses. In another case, (i.e., *rutting*), the PCI methodology assigns a lower weight with respect to other distresses. However, this does not mean that this distress is less important with respect to other distresses. In fact, the distress *rutting* appears in the pavement at the end of the degradation process. The same happens for the distress *alligator cracking*: it appears in the pavement as the final stage of the degradation process. Therefore, the *rutting* and the *alligator cracking* distresses are more likely to be found in pavement with low PCI value.

Those findings have been confirmed in other statistical analyses performed on the same network. Therefore, distress types had been grouped according to the 4 categories defined above (*cracking*, *visco-elastic deformations*, *surface defects* and others) plus the 3 individual distresses (*rutting*, *potholes* and *alligator cracking*).

Considering the results of the statistical analysis of these 7 distresses, grouping the value of the SU into one of three PCI categories (adequate, medium or unsatisfactory, see Figure 2-16), it is possible to evaluate the global progressive degradation in all SU inspected. In this way, deduct values associated with distress type, severity levels and quantities have been summed and classified under the same PCI category, Figure 3-3 and Table 3-2.

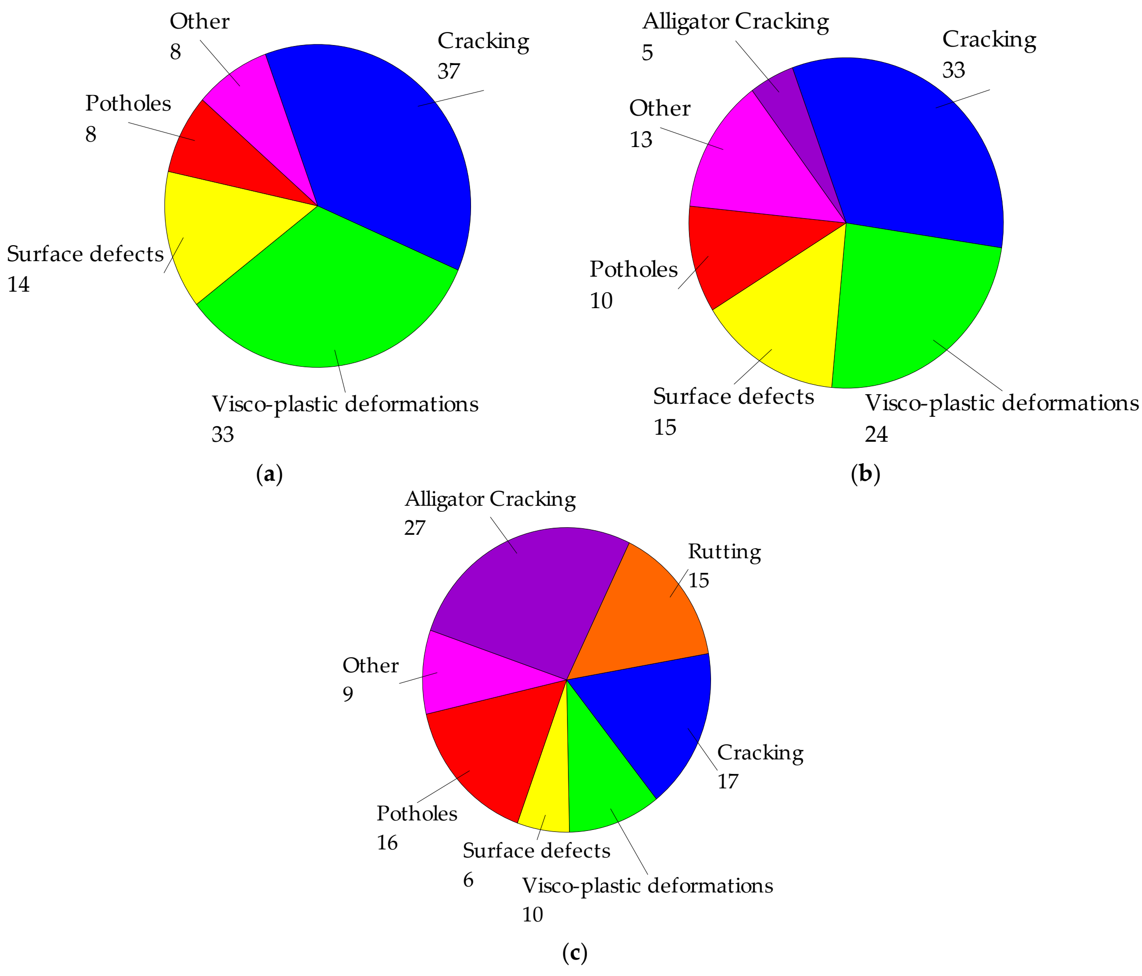


Figure 3-3. Road pavement observed distresses distribution by Pavement Condition Index (PCI) condition in urban areas. Percentage of observed distress in: (a) Adequate PCI sections; (b) Medium PCI sections; (c) Unsatisfactory PCI sections

Table 3-2. Road pavement distress distribution by PCI condition in Urban Areas

Distress Category	ASTM D6433 Distress Description	Percentage of Observed Distress in Adequate Conditions	Percentage of Observed Distress in Medium Condition	Percentage of Observed Distress in Unsatisfactory Conditions
Cracking	3. Block cracking, 7. Edge cracking, 8. Joint reflection, 9. Lane Shoulder Drop-off, 10 longitudinal and transverse cracking, 17. Slippage cracking	37%	33%	17%
Visco-plastic deformations	4. Bumps and sags, 5. Corrugation, 6. Depression, 16. Shoving, 18. Swell.	33%	24%	10%
Surface defects	2. Bleeding, 12. Polished Aggregate 19. Raveling, 20. Weathering	14%	15%	6%
Potholes	13. Potholes	8%	10%	16%
Other	11. Patching and utility cut patching, 14. Railroad crossing.	8%	13%	9%
Alligator Cracking	1. Alligator cracking)	0%	5%	27%
Rutting	15. Rutting	0%	0%	15%

When the PCI value is still high (adequate condition, Figure 3-3a) the most frequent distresses that occur in the sections are *cracking*, *visco-plastic deformations* and *surface defects*; accounting for 84% of total deduct value in SU. The distresses *rutting* and *alligator cracking* are still missing in the pavement, confirming our earlier assumption.

When the pavement degradation is medium (Medium Condition, Figure 3-3b) the categories *Visco-plastic deformations*, *Cracking*, *Surface defects* and *other* are the most frequent distresses that occur in the sections; accounting for 85% of the total deduct value in the SU. Finally, the typical distresses associated with the last stages of the degradation progress (*potholes* and *alligator cracking*) begin to appear in some pavement sections. When the PCI value is very low (unsatisfactory condition, Figure 3-3c) the most frequent distresses that occur in the sections are *rutting*, *potholes* and *alligator cracking*. This account for 58% of total deduct value in the SU, along with other distresses from different stages of the degradation progress.

3.5 Algorithm for the Computerization of PCI Calculation

3.5.1 VBA PCI Calculation Standardized Procedure

The PCI calculation standardized procedure Eq. 2-13 described in ASTM D6433 (see § 2.3.1) was implemented through a VBA language based program to calculate the PCI of each SU. Appendix B Software calculation PCI documentation shows a screenshot the main program window implemented in Microsoft Excel®, which enables distress combinations (distress, severity level and quantity) data entry for a given SU. The PCI of each SU is calculated as described in Eq. 2-13,by adding up the total quantity of each type distress, severity level from inspection data, and recording them in the "total severity". The units for the quantities are square meters, linear meters, or number of occurrences, depending on the distress type, see Table 2-7. Nevertheless, PCI calculation involves the determination of the Deduct value to weight each combination of type, severity and density of distress within each SU. Therefore, for the automatization of the calculation it was necessary to digitize each density/deduct value curve (master curves collected in ASTM D6433 on paper) to accurately determine the value of the deduct value in an Automated PCI calculation Procedure.

3.5.2 DEDUCT VALUE - POLYNOMIAL INTERPOLATING CURVE HERMITE PROCEDURE

For each distress type (20 in total for AC pavement) and level of severity (generally (L) low, (M) medium and (H) high), an interpolation curve can be determined using a Hermite Polynomial Interpolating Curve. For each density/deduct value curve, the coordinates of 10 points Qi were determined ($\log_{10}d_i = Ld_i$: logarithm of density, DV_i: deduct value) from ASTM D6433 Appendix X3 (ASTM 2018). These points were used to obtain a smooth continuous function in which each piecewise polynomial is a parametric cubic spline specified in Hermite procedure, see Figure 3-4.

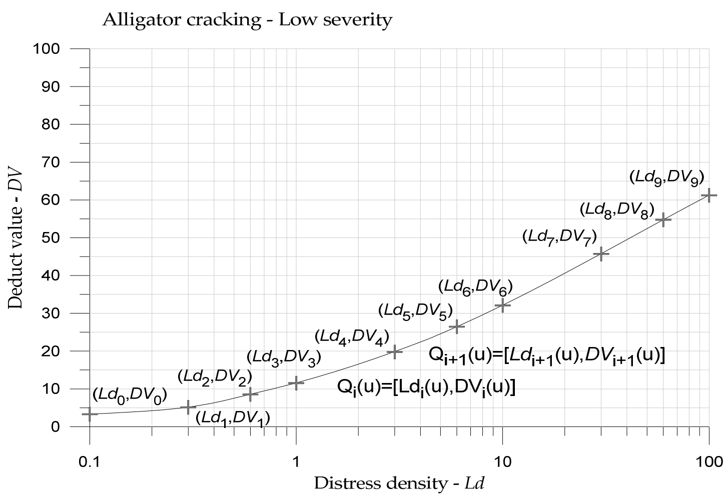


Figure 3-4. Logarithm Density- Deduct Value Polynomial Interpolating Curve Hermite Procedure

For each segment $Q_i(u)$ ($Ld_i(u)$,$DV_i(u)$), between each pair of known data points, the parametric cubic polynomial expressions can be written as follows (Eq. 3-1):

$$
\begin{aligned}
Ld_i(u) &= a_{Ld,i}u^3 + b_{Ld,i}u^2 + c_{Ld,i}u + d_{Ld,i} \\
DV_i(u) &= a_{DV,i}u^3 + b_{DV,i}u^2 + c_{DV,i}u + d_{DV,i}
\end{aligned}
\qquad \text{Eq. 3-1}
$$

with $u \in [0,1]$ and $i = 0, 1, 2, \dots 9$.

For 10 control points given in the plane (*Ld*, *DV*), interpolated by 9 cubic polynomial segments, 4 × 2 = 8 coefficients are needed to define each segment, therefore 8 × 9 = 72 coefficients should be determined (4 × 9 = 36 coefficients for each *Ld*(*u*) and *DV*(*u*) parametric function).

The available equations (for each *Ld* and *DV* variable) are the following:

1. 2 × 10 = 20 equations for each polynomial segment passing through each pair of adjacent control points;
2. 9 equations for first derivative continuity in contact points (C^1 continuity);
3. 9 equations for second derivative continuity in contact points (C^2 continuity).

The total equation count is 4 × 10 − 2 = 38 (76 in total) while the needed equations are 4 × 10 = 40 (80 in total).

To obtain the other 2 equations (4 in total) it was necessary to set tangent and/or curvature values at the first and at the last control points, through first and/or second parametric derivative values. The system of equations can be expressed through control point coordinates, first and second derivatives of the 9 internal and the 2 external control points.

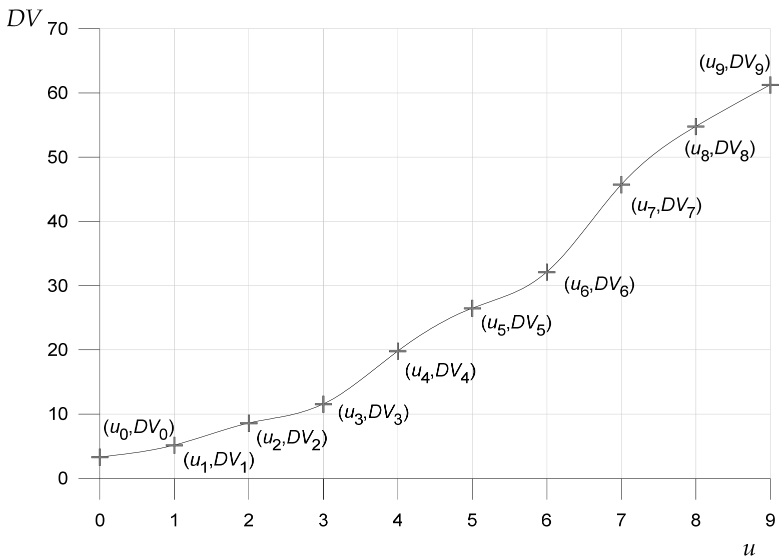


Figure 3-5. Parametric plane (*u, DV*) Polynomial Interpolating Curve Hermite procedure

In this way, for the *i*-th polynomial segment of each curve in the parametric plane (*u*, *Ld* or *u*, *DV*), see Figure 3-5, the equations, for example, consider the variable *DV* (the same for *Ld*), Eq. 3-2:

$$\begin{aligned} f_i(0) &= a_{DV,i} = DV_{i-1} \\ f_i(1) &= a_{DV,i} + b_{DV,i} + c_{DV,i} + d_{DV,i} = DV_i \end{aligned} \qquad \text{Eq. 3-2}$$

and the equations expressing the first derivative at the beginning (D_{i-1}) and at the end (D_i) of *i*-th polynomial segments are (Eq. 3-3):

$$f_i'(0) = c_{DV,i} = D_{i-1}$$
$$f_i'(1) = 3a_{DV,i} + 2b_{DV,i} + c_{DV,i} = D_i \qquad \text{Eq. 3-3}$$

In the Equations Eq. 3-2 and Eq. 3-3, DV_{i-1}, DV_i, D_{i-1} and D_i are respectively the control points coordinates and the first derivative value (at the beginning and at the end of *i*-th segment), From these equations, it is possible to obtain algebraic expressions of coefficients, Eq. 3-4:

$$d_{DV,i} = DV_{i-1}$$
$$c_{DV,i} = D_{i-1}$$
$$b_{DV,i} = 3(DV_i - DV_{i-1}) - 2D_{i-1} - D_i \qquad \text{Eq. 3-4}$$
$$a_{DV,i} = 2(DV_{i-1} - DV_i) + D_{i-1} + D_i$$

To obtain the C^2 continuity, it is necessary to impose the following second derivative conditions in the 9 internal control points,Eq. 3-5:

$$f_i''(1) = f_{i+1}''(0) \qquad \text{Eq. 3-5}$$

with i = 1, 2, …, 10; obtaining Eq. 3-6:

$$6a_{DV,i} + 2b_{DV,i} = 2b_{DV,i+1} \qquad \text{Eq. 3-6}$$

Then, substituting the algebraic expressions of coefficient Eq. 3-4 in Eq. 3-6 the expressions of second derivative of curve are obtained (Eq. 3-7):

$$6\left[2\left(DV_{i-1} - DV_i\right) + D_{i-1} + D_i\right] + 2\left[3\left(DV_i - DV_{i-1}\right) - 2D_{i-1} - D_i\right] = 2\left[3\left(DV_{i-1} - DV_i\right) - 2D_i - D_{i-1}\right] \qquad \text{Eq. 3-7}$$

and simplifying Eq. 3-8:

$$D_{i-1} + 4D_i + D_{i-1} = 3\left(DV_{i-1} - DV_{i-1}\right) \qquad \text{Eq. 3-8}$$

If the second derivative at the endpoints is equal to zero, the "*natural*" cubic spline is obtained. By imposing the second derivative condition at the first subdomain of the piecewise (i = 0), Eq. 3-9 is obtained:

$$2b_{DV,1} = 0 \qquad \text{Eq. 3-9}$$

Substitute the algebraic expression of coefficient Eq. 3-4 with Eq. 3-9, the following expression Eq. 3-10 is obtained and simplified Eq. 3-11:

$$2\left[3\left(DV_1 - DV_0\right) - 2D_0 - D_1\right] = 0 \qquad \text{Eq. 3-10}$$

$$2D_0 + D_1 = 3\left(DV_1 - DV_0\right) \qquad \text{Eq. 3-11}$$

Similarly, for the last subinterval of the piecewise ($i = 9$), Eq. 3-12:

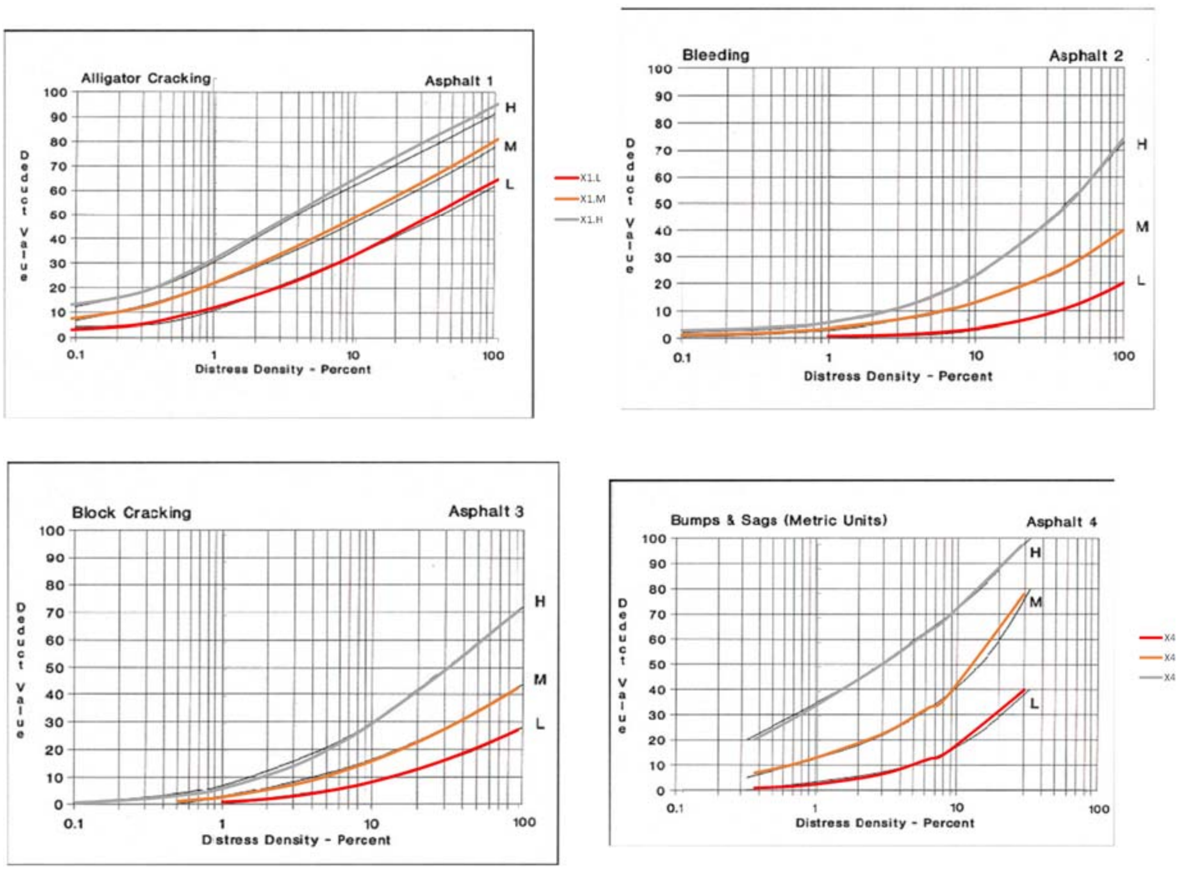
$$D_n + 2D_{n-1} = 3\left(DV_n - DV_{n-1}\right) \qquad \text{Eq. 3-12}$$

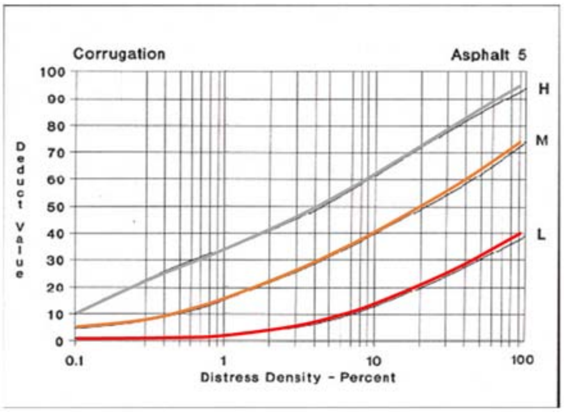
The Equations Eq. 3-8, Eq. 3-11 and Eq. 3-12 can be collected in matrix form (Eq. 3-13). The 10 unknowns are grouped in **D** = [D_0, D_1, …, D_9] that can be calculated by inverting the tridiagonal matrix **H** (and similarly for *Ld* variable):

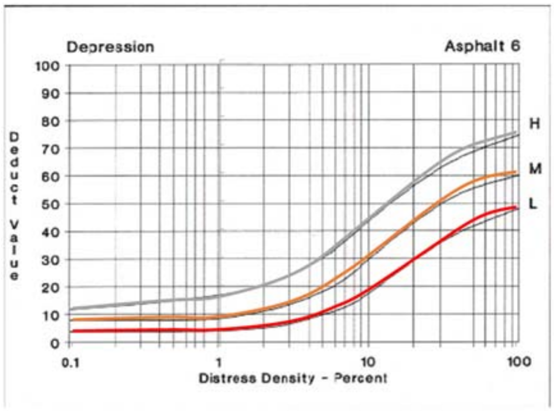
$$\begin{bmatrix} 2 & 1 & & & & & & \\ 1 & 4 & 1 & & & & & \\ & 1 & 4 & 1 & & & & \\ & & 1 & 4 & 1 & & & \\ & & & . & . & . & & \\ & & & & . & . & . & \\ & & & & & 1 & 4 & 1 \\ & & & & & & 1 & 2 \end{bmatrix} \cdot \begin{bmatrix} D_0 \\ D_1 \\ D_2 \\ D_3 \\ . \\ . \\ D_8 \\ D_9 \end{bmatrix} = \begin{bmatrix} 3(DV_1 - DV_0) \\ 3(DV_2 - DV_0) \\ 3(DV_3 - DV_1) \\ 3(DV_4 - DV_2) \\ \\ \\ 3(DV_9 - DV_7) \\ 3(DV_9 - DV_8) \end{bmatrix} = \mathbf{H} \times \mathbf{D} = \mathbf{DV} \Rightarrow \mathbf{D} = \mathbf{H}^{-1} \times \mathbf{DV} \qquad \text{Eq. 3-13}$$

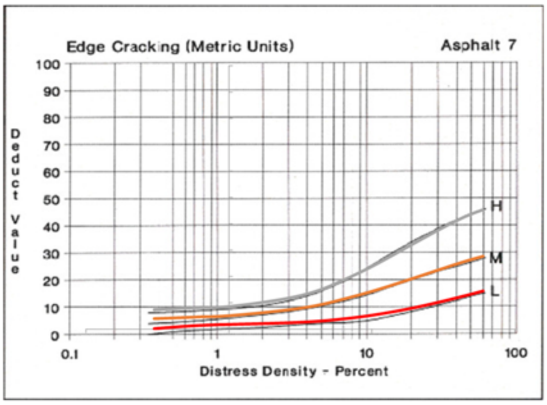
Knowing the coefficients of each segment of the curve (for all distress and for all levels of severity), it is possible to calculate, starting by a value of density, the corresponding value of deduct value and so on.

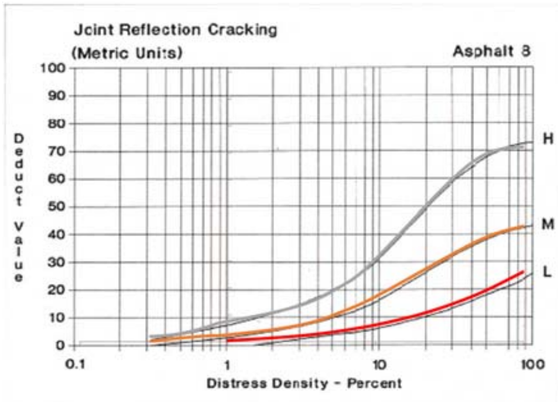
Figure 3-6 shows the results of the developed methodology, ASTM D6433 DV curves implemented on PAVER had been digitalized. The obtained analytical DV curves and ASTM D6433 DV curves have been represented together.

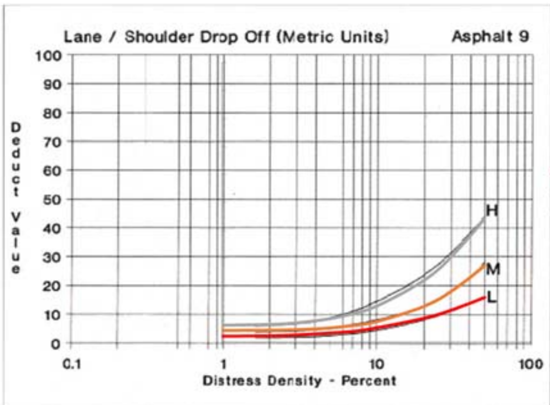


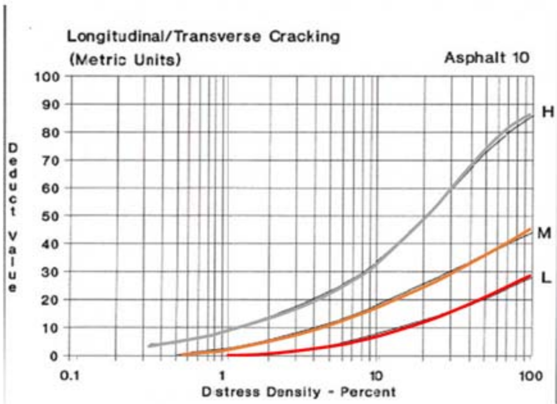

Corrugation
Asphalt 5
Deduct Value
Distress Density - Percent
H
M
L

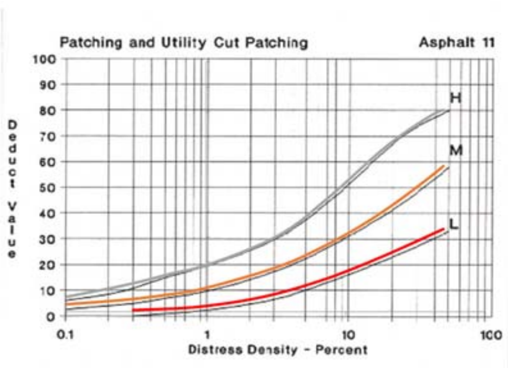

Depression
Asphalt 6
Deduct Value
Distress Density - Percent
H
M
L

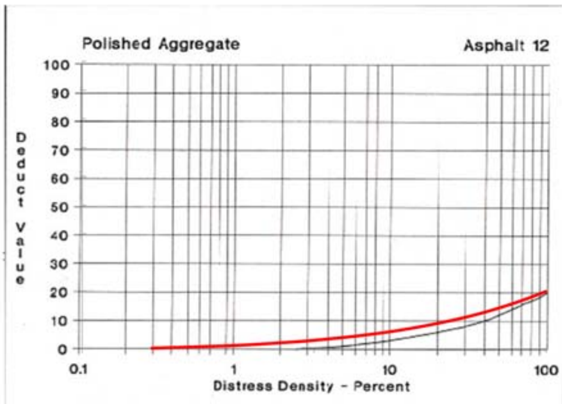

Edge Cracking (Metric Units)
Asphalt 7
Deduct Value
Distress Density - Percent
H
M
L


Joint Reflection Cracking
(Metric Units)
Asphalt 8
Deduct Value
Distress Density - Percent
H
M
L


Lane / Shoulder Drop Off (Metric Units)
Asphalt 9
Deduct Value
Distress Density - Percent
H
M
L


Longitudinal/Transverse Cracking
(Metric Units)
Asphalt 10
Deduct Value
Distress Density - Percent
H
M
L


Patching and Utility Cut Patching
Asphalt 11
Deduct Value
Distress Density - Percent
H
M
L


Polished Aggregate
Asphalt 12
Deduct Value
Distress Density - Percent

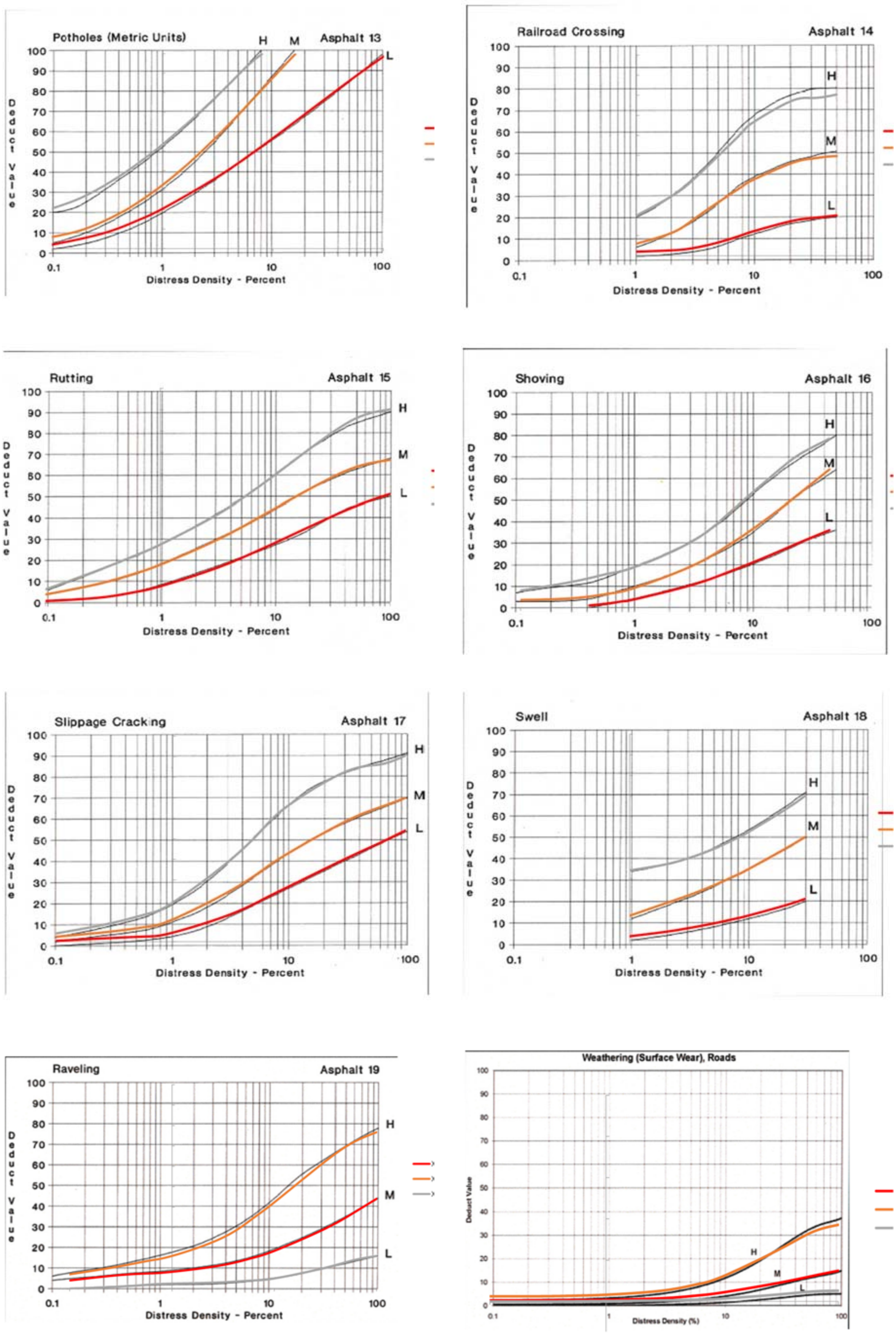


Figure 3-6. ASTM D6433 Digitalized DV Curves

The next step is the calculation of maximum Corrected Deduct Value (CDV) that will be subtracted from 100 obtaining the PCI of the sample unit. The PCI within each section is calculated by averaging the PCI of each sample unit. The whole algorithm was implemented writing a code in VBA (see Appendix B: Software calculation PCI documentation) and capable of calculating the SU PCI starting from its distress.

However, it is necessary to use an adequate Distress Identification Catalogue reflecting the characteristics of pavement surveyed and a standardized procedure to make repeatable investigations for the entire network. To this end, some US highway agencies that apply PMS procedures have developed internal reference catalogues for pavement distress identification.

Therefore, the procedure of digitalization of DV curves can be used to propose new DV curves based on the combination of existing ones for the assessment of new distress definitions based on agency needs'.

3.6 Definition of New Distress Identification Catalogue for Asphalt Concrete Pavements in Urban Areas

One of the objectives of this research was to achieve a simplified new distress identification catalogue for urban road AC pavements to assess the pavement condition in a PCI based scale. This section is intended to propose a simplified distresses catalogue comparing that one contained in ASTM D6433 considering the results of distress distribution analysis presented in this Chapter (Figure 3-3 and Table 3-2) on a case of study urban road network. Although the list of distresses in ASTM D6433 appears complete and exhaustive, the use of such a catalogue for urban pavements could be difficult to apply for the road network operators and maintenance agencies have little incentive to adopt a PMS. In addition, the use of ASTM D6433 catalogue has shown difficulties for some inspectors of urban pavements due to the widespread presence of pavements distresses that are not listed in ASTM D6433. Specifically, *Manholes* and *Tree roots*. These distresses, before this proposal of integration, were associated with other distresses respectively as *Patching* and *Swell*. However, it was often very difficult to instruct the inspectors on the choice of the type of distress to be attributed, so much so that often these distresses were completely neglected during inspections.

For this reason, it was considered appropriate to drastically reduce the number of distresses (from 20 to 10 distresses) present in the catalogue for the calculation of the PCI and add the two missing distresses. These adjustments should be made in order to avoid distorting the PCI numerical significance in its rating scale that appears to be universally accepted.

Table 3-3. New distress identification catalogue asphalt concrete pavement for urban areas

Distress ID	Distress Description	Unit of Measure	Group of Distress	Cause	Combination ASTM D6433 Distress
N_1	*Alligator cracking*	Square meters	Cracking	Load	1. Alligator Cracking
N_2	*Block cracking*	Square meters	Cracking	Climatic	3. Block cracking
N_3	*Linear and isolated cracking*	Linear meters	Cracking	Climatic/Poor construction	7. Edge cracking; 8. Joint reflection; 10. Longitudinal and transverse cracking, 9. Lane/Shoulder Drop-off
N_4	*Surface deformations*	Square meters	Visco-plastic deformations	Subsidence/Poor construction	4. Bumps and sags; 5. Corrugation, 6. Depression, 18. Swell
N_5	*Rutting*	Square meters	Visco-plastic deformations	Load	15. Rutting
N_6	*Failure surface grip*	Square meters	Surface Defects	Bituminous mixture low quality	2. Bleeding, 12. Polished Aggregate, 19. Raveling, 20. Weathering (Surface wear)
N_7	*Potholes*	Number	Potholes	Traffic, Load	13. Potholes
N_8	*Patching*	Square meters	Others	Other	11. Patching and Utility Cut
N_9	*Railroad crossing*	Square meters	Others	Other	14. Railroad crossing
N_10	*Slippage cracking*	Square meters	Visco-plastic deformations	Bituminous mixture low quality	16. Shoving, 17. Slippage cracking
N_11	*Manholes*	Square meters	Others	Other	6. Depression, 11. Patching and Utility Cut Patching
N_12	*Tree roots*	Linear meters	Others	Other	14. Railroad crossing, 11. Patching and Utility Cut Patching, 18. Swell

Consequently, to upgrade the ASTM D6433 catalogue for application on Italian urban roads, some distress types collected in the ASTM Standard Guide and whose quantities are measured in the same unit of measurement and belonging to the same group are combined with each other to facilitate the inspection activity as indicated in Table 3-3. Four new defects have been defined by combining others - N_3. *Linear and isolated cracking* (Figure A1), N_4. *Surface deformations* (Figure A2), N_6. *Failure surface grip* (Figure A3) and N_10. *Slippage cracking* (Figure A4); and two new distress types are defined—N_11. *Patching* (Figure A5) and N_12. *Tree Roots* (Figure A6); six distresses remain unchanged (N_1. *Alligator cracking*, N_2. *Block cracking*, N_5. *Rutting*, N_7. *Potholes*, N_8. *Patching* and N_9. *Railroad crossing*).

The proposed Distress Identification Catalogue is composed of 12 distress types, listed and classified in different groups. In the same way, the Distress Identification Catalogue defines for each distress type, 1) a description of the distress type, 2) a guide to determine the severity levels, 3) how to measure and count the distress type and severity, 4) representative photo (s) to help inspectors to identify the severity level of distress and, 5) the deduct value curve for each distress type.

The deduct value curve of new distresses is obtained as a combination of the ASTM D6433 deduct value curves of they are composed. The new deduct value curve was written as a continuous function using the Polynomial Interpolating Curve Hermite for the interpolating of ten pairs of values (density, *DV*) obtained by averaging the ten pairs of values (density, *DV*) of each deduct value curve that has to be combined. These inspection guidelines for the new distress definitions for urban road pavements are defined in Table 3-4. As an Appendix to this dissertation, it has been produced some guidelines for pavement surface distress identification in urban areas (Appendix A Distress Identification Manual for Urban Road Pavements).

The new distress type N_3. *Linear and Isolated Cracking* definition was obtained as the combination of ASTM D6433 7. *Edge cracking*; 8. *Joint reflection*; 9. *Lane/Shoulder drop-off* and 10. *Longitudinal and transverse cracking*, all of those distresses classified as cracking and measured in linear meters, allowing the inspector recording these distresses under the same code.

Likewise, the new distress N_4. *Surface deformations* is defined as the union of distresses classified under the same, group *Visco-plastic deformations*: ASTM D6433 distress 4. *Bumps and sags*, 5. *Corrugation*, 6. *Depression and* 18. *Swell*, measured in square meters, being much simpler to identify.

Distress N_6. *Failure surface grip*, classified under the *Surface defects* group is obtained as the combination of ASTM D6433 distress 2. *Bleeding, 12. Polished Aggregate, 19. Raveling* and *20. Weathering*, measured in square meters.

Similarly, distress N_10. *Slippage cracking* is defined as the combination of ASTM D6433 distresses 16. *Shoving*, 17. *Slippage cracking*, both distresses are measured in square meters, to consider the traffic tangential action. These distresses can be found in roadway points where there are repeated braking and accelerations linked to roadway areas overlays are poorly bonded with the underlying layer.

The main novelty of proposed distress identification catalogue for urban road pavements assimilates two innovative distress definitions in the catalogue that ASTM D6433 does not consider; N_11. *Manholes* and N_12. *Tree roots*.For the first distress, this choice is motivated to catalog the catch basins and manholes in pavement condition assessment due to their importance as a singular and very common distress in Italian Urban Roads (see Appendix A Distress Identification Manual for Urban Road Pavements). This innovative definition allows considerations of the catch basins, manholes. These features have a strong presence in urban roadways and induce other distresses in surrounding areas that very negatively affects drive quality and causes damage to the vehicles. These distress are not currently considered by the

ASTM D6433 PCI calculation procedure. The quantities of this distress are measured in square meters

Another distress that the Standard Practice ASTM D6433 does not include is N_12. *Tree roots* distress. The definition of this distress on the new distress identification catalogue for urban areas is important due to its importance and its wide presence in Italian Urban Roads (see Appendix A Distress Identification Manual for Urban Road Pavements). The appearance of this distress is linked to the presence of large trees in the roadside of urban streets which may create a raise in the pavement on the road side and /or a break in the pavement leading to accidents and vehicle damage. The density-deduct value curve associated with N_12. *Tree roots* is calculated as the combination of ASTM D6433 11. *Patching and Utility Patching* and 18. *Swell*. The quantities of this defect are measured in square meters.

Table 3-4. Severity levels definitions for urban pavement distresses identification catalogue (after Loprencipe and Pantuso (2017))

No	Distress Type	Severity Level	Description	Unit of Measure
N_3	*Linear and isolated cracking*	Low	Mild non-sealed cracks with a mean amplitude lower than 1 cm	Linear meters.
		Medium	Average cracks with a mean amplitude between 1 and 7.5 cmzWalker et al., 2002	
		High	Cracks with a mean amplitude >7.5 cm	
N_4	*Surface deformations*	Low	Depth of pavement depressions between 1 and 2.5 cm, with associated high ride quality.	Square meters.
		Medium	Depth of pavement depressions between 2.5 and 5 cm, with associated medium ride quality.	
		High	Depth of pavement depressions > 5 cm, with associated badly ride quality.	
N_6	*Failure surface grip*	Low	Sticky surface only a few days a year, aggregates and bitumen begin to be removed.	Square meters.
		Medium	Sticky surface few weeks a year or erosion level such as to have moderately wrinkled texture.	
		High	Sticky surface for at least a few weeks a year, or very rough texture.	
N_10	*Slippage cracking*	Low	Good ride quality and/or crack amplitude <1.0 cm.	Square meters.
		Medium	Average ride quality and/or crack amplitude between 1.0 and 4.0 cm with crack surrounding area moderately crushed.	
		High	Low ride quality and/or crack amplitude >4 cm; surrounding area highly cracked into removable pieces.	
N_11	*Manholes*	Low	Low influence on ride quality	Square meters.
		Medium	Medium influence on ride quality with an elevation of manhole <5 cm; The surrounding area is slightly cracked.	
		High	High influence on ride quality; Elevation over the roadway level >5 cm; the surrounding area is significantly cracked.	
N_12	*Tree Roots*	Low	The height above road surface is less than 10 cm with medium traffic influence.	Square meters.
		Medium	Height above road surface between 10 to 20 cm and low to medium traffic influence, or height <10 cm with high influence on traffic.	
		High	High influence on traffic with an height above road surface >10 cm.	

3.7 VALIDATION OF THE COMPUTERIZATION OF PCI CALCULATION

As mentioned above, the new definition of the catalogue distresses should not distort the PCI value. In addition, it is necessary to validate the digitization of *density/deduct value* curves to verify the correctness of the calculations.

Therefore, the results analysis was divided into three various different stages. In the first stage, a comparison between the sample units' PCI obtained with commercial software PAVER™ and the sample units' PCI calculated using the proposed procedure is conducted of the total sample of 109 sections of the Italian Road Network. The results of this regression are shown in the Figure 3-7a. The procedure adopted to interpolate deduct value curves for the ASTM D6433 distress identification catalogue was validating using this comparison.

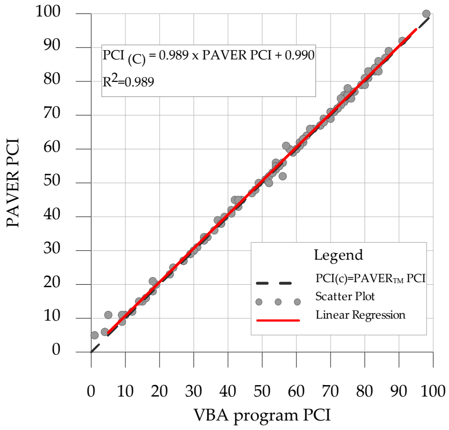
Next, new analytical density-deduct value curves were generated for the new distresses and a specified procedure for distress identification and assessment for urban road surfaces was conformed. In this second stage the two new distresses (N_11. *Manholes* and N_12. *Tree roots*) were not considered to evaluate whether the merging of the various distresses into the distress categories it can work. When these distresses were found in the sample unit, the calculation of the PCI was replaced by Patching and Swell. The objective of this second stage is to demonstrate that new deduct value curves can be obtained by the combination of the deduct value curves collected on ASTM D6433 and can be used to define new ones without a large variation in terms of PCI.

Finally, the last stage consisted of calculating the sample units' PCI using the new distress identification catalogue proposed by the reduction of the number of distresses from 20 distresses to 12 distresses, as presented above. The objective of this third stage is demonstrate that new distress identification catalogue for urban road pavements can be used without a large variation in terms of PCI.

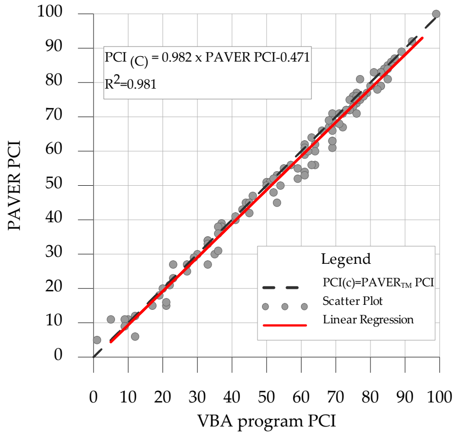
Next, the PCI was calculated with the VBA program following the definition of distresses by the merging of distress in the categories (in total 10 distresses); the regression with the PCI calculated with the PAVER™ is shown in the Figure 3-7b.

Finally, the PCI was calculated with the VBA program and using the definition of the 12 distresses (including the new ones for a total of 12 distresses); the regression with the PCI calculated with the PAVER™ is shown in the Figure 3-7c.

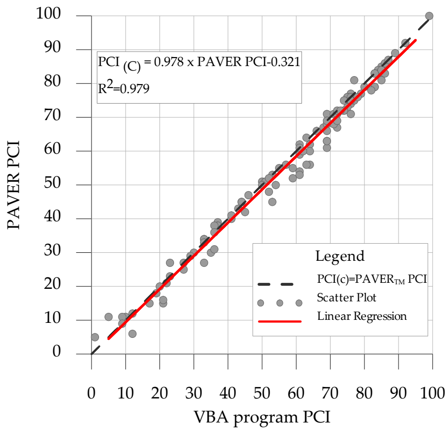
Figure 3-7 (a) ASTM D6433 (20 distresses); Figure 3-7 (b) New merged distress identification catalogue (10 distresses); Figure 3-7 (c) New distress identification catalogue including two new distress definitions (11. *Manholes* and 12. *Tree roots*).



(a)



(b)



(c)

Figure 3-7. Regressions on 109 SU between PAVER™ PCI and calculated PCI using VBA program

As observed from the Figure 3-7, there is a strong correlation with the PAVER™ software PCI value and *calculated* PCI through the automated procedure. In fact, the coefficient of determination, R^2, is very close to 1 in all cases, increasing from 0.979 (New distress identification Catalogue including N_11. *Manholes* and N_12. *Tree roots*) to 0.989 (ASTM D6433 PCI—20 distresses). The results of statistics regression are reported in Table 3-5 and Table 3-6.

Table 3-5. Statistical parameters of automated procedure PCI calculation (ANOVA F-test)

Sum of Squares (SS)			Mean Square (MS)	MS Regression/MS Residual (F)	Significance F
ASTM D6433 PCI calculation (20 distresses)	Regression	58578.94	58578.94	10171.10	6.70E-108
	Residual	616.25	5.76	–	–
	Total	59195.19	–	–	–
New distress identification catalogue (10 distresses)	Regression	58032.34	58032.34	5339.83	3.79E-93
	Residual	1162.86	10.87	–	–

	Total	59195.19	–	–	–
New distress identification catalogue (12 distresses)	Regression	57933.82	57933.82	4914.41	2.94E-91
	Residual	1261.38	11.79	–	–
	Total	59195.19	–	–	–

Table 3-6. Regression coefficient of the automated procedure for PCI calculation

Regression Model	Coefficient of Correlation	Coefficient of Determination	Adjusted R^2	Standard Error
ASTM D6433 PCI calculation (20 distresses)	0.995	0.989	0.989	2.40
New distress identification catalogue (10 distresses)	0.990	0.980	0.981	3.30
New distress identification catalogue (12 distresses)	0.989	0.979	0.979	3.43

Table 3-5 shows the results of the ANOVA-F test performed to test the overall (global) fit of the regression model, which is used to perform a statistical hypothesis test to compare an idealized null hypothesis (H_0) that proposes there is no relationship between the two data sets and the alternative hypothesis (H_1) states that there is a linear relationship between both data sets. The best regression results are found for the automated procedure for the PCI calculation (20 distresses) for which the lowest value of unexplained variance is 5.76. For these reasons, the values of the F statistic (MS Regression/MS Residual) for all the tested automated procedures are higher than 4914 with a Significance F less than 2.94×10^{-91}, that is, the observed data are inconsistent with the null hypothesis. So we can surely reject null hypothesis and state that there is a very good correlation between commercial software PAVER™ PCI and the SU PCI calculated using VBA language based on an automated procedure.

The comparison between the results of both analyses showed us that the standard errors calculated by the two regressions are very close. Certainly, the linear regression has a slope equal almost to 1 for all cases, near the theoretical line which indicates that the automated procedure calculation, and the PAVER™ PCI can be assumed to be the same, demonstrating the efficiency of the automated procedure calculation and the reliability of the constructed functions of deduct value curves.

3.8 Discussion

The proposed new distress identification catalogue can be used in the same scale of the standardized PCI with several advantages for the assessment of urban road surfaces. The advantages of the proposed method consist of the simplification of the pavement surveys (12 distresses instead of 20 distresses) and the identification of pavement distress very frequent in urban roads and others such tree roots and artificial elements (catch basins and manholes) not considered in the ASTM D6433 distress catalogue.

For example, these artificial elements, in the ASTM 6433 method, could be considered as 11. *Patching* and *Utility Cut Patching*, but this assumption is subjective to the inspector judgement and it has been found that often the inspectors completely neglected the inclusion of these distresses in the Pavement Condition Survey Data Sheet or included these as other not appropriate/unsuitable types of distress. This obviously caused a consequent reduction in the PCI value on the SU with an underestimation of the general conditions of the network.

Another example is the definition of the tree roots distress that is a very common distress in Italian Urban roads networks. The ASTM D6433 catalogue does not define this type of distress but it could be associated with the 18. *Swell* distress definition, but the deduct value curves associated with this distress are not defined for values of density <1 and severity level definitions are based only in the influence of the distress in ride quality. Moreover, the new distress identification catalogue includes in the definition of N_12. *Tree roots* distress the influence of the height of tree roots above road surface and the influence that this distress has on the traffic based on the position of the roots inside the roadway section; the associated deduct value curve is defined for all range of the density inside the sample units.

The possibility to define new types of distress allows the manager to define the most appropriate techniques to use for preventive and corrective maintenance in order to have, in the economic evaluation stage of the alternatives, the correct quantification of workings.

In fact, considering *Manholes*, often for this distress type there is no need to provide any maintenance intervention unless the damage is propagated outside of the manhole (see Figure 3-8). In this case, it would be appropriate to consider separately the two distresses: the "*artificial*" due to the surface discontinuity of the pavement and the "*natural*" result on the pavement in consequence of previous distress.

To complement the explanation expressed above, a comparative analysis between both distress identification catalogue methodologies is included. Two SU of the studied road network distress identification sheets was compared, see Table 3-7 and Table 3-8.

Figure 3-8. Example of manhole distress identification guidelines, Manholes

On the one hand, Table 3-7 shows a SU with manholes which are considered according to the ASTM catalogue as 11. *Patching and Utility Cut Patching* medium severity. The new distress identification manual allows the inspector to consider the influence of those artificial elements in ride quality when these create a vertical discontinuity in the pavement so, the inspector should consider the presence of manholes in SU as a N_11 *Manholes* High Severity (See Appendix A). This is reflected in a higher calculated *DV* value, this leads to a lower value of overall SU PCI value which causes the rating of the SU to fail (poor condition).

Table 3-7. Comparative analysis between ASTM D6433 and New Distress Identification Manual for Urban Pavements: N_11 Manholes

No	Distress Type	Severity Level	Distress Quantity	Distress Density	*DV*
10	Longitudinal and transverse cracking	L	6.40	2.10	0.98
6	Depression	L	1.30	0.43	4.46
13	Potholes	H	1.00	0.33	33.81
4	Bumps and sags	L	1.39	0.45	1.36
11	Patching and Utility Cut Patching	M	5.00	1.64	12.78
	–		**PCI**	**Fair**	**58**
N_3	Linear and isolated cracking	L	6.40	2.10	1.70
N_4	Surface deformations	L	1.30	0.43	2.70
N_7	Potholes	H	1.00	0.33	33.81
N_4	Surface deformations	L	1.39	0.45	2.71
N_11	Manholes	H	5.00	1.64	23.15
–	–	–	**PCI**	**Poor**	**54**

Table 3-8. Comparative analysis between ASTM D6433 and New Distress Identification Manual for Urban Pavements: N_12 Tree roots

No	Distress Type	Severity Level	Distress Quantity	Distress Density	*DV*
4	Bumps and sags	M	7.90	3.31	23.58
3	Block cracking	M	2.21	0.92	2.22
10	Longitudinal and transverse cracking	L	2.00	0.84	2.00
10	Longitudinal and transverse cracking	M	14.40	6.03	13.00
18	Swell	M	3.40	1.42	15.04
7	Edge cracking	M	2.10	0.88	4.89
3	Block cracking	L	17.09	7.15	6.21
7	Edge cracking	L	17.40	7.28	4.29
11	Patching and Utility Cut Patching	M	3.12	1.31	11.25
18	Swell	L	5.00	2.09	4.32
–	–	–	**PCI**	**Fair**	**59**
N_4	Surface deformations	M	8.61	3.60	22.31
N_2	Block cracking	M	2.21	0.92	2.22
N_3	Linear and isolated cracking	L	19.40	8.12	5.27
N_3	Linear and isolated cracking	M	16.50	6.90	12.88
N_2	Block cracking	L	17.09	7.15	6.21
N_8	Patching	M	3.12	1.31	11.25
N_12	Tree roots	H	5.00	2.09	30.62
–	–	–	**PCI**	**Poor**	**52**

The N_4 Surface deformations quantity is not the sum of 4. Bumps and sags and 18 Swell because the 4. Bumps and sags distress is measured in linear meters instead of square meters.

On the other hand, Table 3-8 gives an example of a SU with tree roots. As stated above, the tree roots have to be considered as 18. *Swell* according the ASTM D6433 catalogue. The position of the root within the section or its traffic influence is not considered in distress identification. However, since it is located in the pathway of SU analyzed, the inspector should identify the distress as a N_12. *Tree roots*, High level severity according the new distress identification manual for urban roads catalogue. As a consequence, the associated *DV* for this type of distress in those sections is significantly higher, changing the PCI rating scale category from Fair to Poor (see Figure 2). Besides, the use of new distress identification manual for urban road pavements facilitates the inspection and makes it faster by grouping various distresses by their similar characteristics, as can be noted at Table 3-8.

Therefore, the use of the new distress identification manual for urban road pavement allows us to consider the traffic influence of these common distresses in urban areas, which is reflected in a correction of overall SU PCI, declining the PCI rating category of sections when any of these elements arise.

Another significant advantage of this methodology is the reduction of the number of distresses of the catalogue without causing a substantial change in the PCI value. In general, it can be noted that this reduction is only applicable in the case in which the contribution of these removed distresses slightly varies the PCI value. In the present study, this has been verified through the regressions reported before, but in the future, the applicability of the proposed catalogue should be verified by an extensive campaign of inspections in urban road that can confirm the results presented here.

3.9 CONCLUSIONS

A methodology based on the definition of new deduct value curves to complete the ASTM D6433 Distress Identification Catalogue to assess the urban road pavement surfaces is proposed. It has been implemented a procedure for the determination of new distress deduct value curves to include new distress definitions—manholes and tree roots—which are defects that are very present in urban areas and have not been collected until now in the distress identification catalogue (see Appendix A Distress Identification Manual for Urban Road Pavements). The algorithm used for the digitalization of existing distress deduct value curves and the automated procedure for the calculation of the PCI should be a useful tool for pavement engineers and managers by allowing the implementation and the automatization of the pavement agency customized index for surfaces' overall condition assessment eliminating time-consuming procedures. A sample of 109 AC urban pavement surfaces was considered. By the statistical analysis of detected distresses included in this sample the most recurrent defects, those never encountered and those not defined concerning the list collected in ASTM D6433 have been determined by statistical analysis. The comparison of the PCI value using the new distress identification catalog and the values obtained using the commercial software PAVER^{TM} was performed. The results highlighted the suitability of the proposed automated distress procedure for the calculation of the PCI.

The method implemented using the new distress identification catalog, therefore, determines PCI values close to those can be obtained by applying the ASTM D6433. Conversely, the method could be very severe for the SU which presents many distresses like manhole covers and for tree roots. In fact, these distresses are not considered in ASTM D6433, and the inspectors often do not consider them or substitute them with another distress. Moreover, the accuracy in distress identification and pavement condition assessment is always linked to a mandatory operator training for the correct distress identification in urban roads since the assessment is always subject to subjective judgment. Finally, from the analysis of the results it is possible to state that specific urban road distress can be acquired through the definition of new deduct value curve, and the results provide strong correlations with the inspected survey data using the ASTM D6433 distress catalogue and provide to small and medium agencies of a valid tool for the assessment of their urban road network with reduced costs implementation. The automated procedure had been used successfully in other applications presented in this dissertation.

CHAPTER 4. PAVEMENT MANAGEMENT SYSTEM CONSIDERING THE VEHICLE OPERATING COSTS: A CASE OF STUDY IN URBAN AREAS

4. PAVEMENT MANAGEMENT SYSTEM CONSIDERING THE VEHICLE OPERATING COSTS: A CASE OF STUDY IN URBAN AREAS[2]

4.1 INTRODUCTION

Urban roads constitute most of the existing roads, and they are directly managed by small administrations. Normally, these small administrations do not have sufficient funds or sufficient qualified personnel to carry out this task.

The main objective of this chapter is the creation of the basis for the implementation of a sustainable, flexible, and affordable PMS for urban areas in order to be easily adapted to various situations and it does not require a large amount of time and money for its implementation.

This methodology includes the creation of the road network inventory, the visual surveys of the pavement and the evaluation of its condition by the Pavement Condition Index (PCI) through visual survey as proposed in ASTM D6433 (2018).

The method intends to give a valid tool to road managers to compare alternative maintenance strategies and perform the priority analysis on the network. For this, this PMS methodology to include the indirect operating costs induced by the urban vehicles (tire wear costs, repair and maintenance vehicle costs, and fuel consumption), recognized in international literature as Vehicle Operating Cost (VOC) in the network analysis.

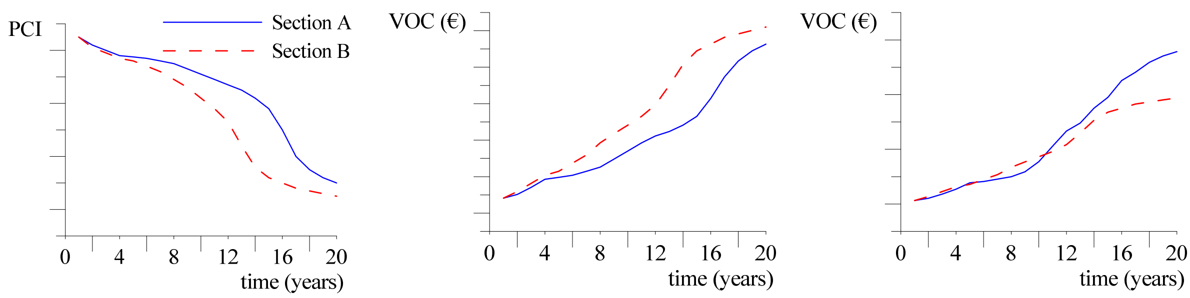
The VOC were considered and calculated to evaluate and prioritize the M, R&R work strategies. Therefore, this method assembles the network pavement condition data with the traffic surveys with the purpose of combining analysis to evaluate the influence of the indirect costs in M, R&R decision making process.

According to the VOC models used, the traffic is directly proportional to the indirect user costs, therefore it can be related to the global pavement distress. Consequently, the user costs play an important role in the decision-making process to assess the cost-effectiveness of M, R&R treatments on the urban network.

For example, if the traffic level is the same for two pavement sections A and B (Figure 4-1), the Repair and maintenance work will be performed in the section with higher rate of deterioration (Section B). However, if the level of traffic in A was much higher than the level of traffic in B, it would be better to perform maintenance works in the section with higher level

2 Partial results of this research have been published as “Loprencipe, G., Pantuso, A., & Mascio, P. D. (2017). Sustainable Pavement Management System in Urban Areas Considering the Vehicle Operating Costs. *Sustainability*, 9(3), 453”

of traffic (Section A) to mitigate the increase in indirect costs, expressed by VOC. Therefore, the Annual Average Daily Traffic (AADT) and its associated VOC influences the choice of the most suitable M, R&R work strategy.



(**a**) (**b**) (**c**)

Figure 4-1. Comparative of traffic influence on VOC
(**a**) pavement condition performance of sections A and B; (**b**) VOC section A and section B, where traffic at section A is equal to traffic at section B; and (**c**) VOC section A and section B, where traffic at section A is higher than traffic at section B.

The World Bank's Highway Development Model (HDM) research has developed some Vehicle Operating Costs (VOC) models. It begins with the study by De Weille (De Weille 1966) and continued by the development of the Highway Cost Model by Becker (1972). In the 1980s, Zaniewski et al. (1982) adapted the VOC model to US conditions. Most of the VOC models were derived from other previous models that have been updated and improved over time. The most recent VOC found in the literature review is the HDM-4 (Bennett and Greenwood 2001). In the National Cooperative Highway Research Program (NCHRP) Report 720 (Chatti and Zaabar 2012), the calibration of the field tests conducted for the calibration HDM-4 VOC model were described. The report studied the effect of roughness on the main components of the VOC such as fuel consumption, tire wear, repair and maintenance costs. The equations developed by (Chatti and Zaabar 2012) has been chosen to be used to estimate the VOC for the road in the study.

4.2 METHODOLOGY FOR PMS IMPLEMENTATION IN URBAN AREAS

The described PMS process (Figure 4-2) has been developed by using visual surveys to assess the distress of pavement. Condition evaluation has been performed by PCI as described in ASTM D6433 American Standard Practice. In the following, the steps of the proposed methodology are listed:

1. The method shows the need to create a network pavement inventory, which, in most cases, is non-existent. A Geographical Information System (GIS) is a useful tool to create the **network inventory**. The network's elements are divided into sub-units

(branches and sections) with homogeneous properties (type of pavement, date of construction, materials, etc.).

2. First, the **pavement distress catalogue** adopted in the ASTM 6433 could be used, in order to make it comparable with international implementations, future implementations can adopt a list of distresses and different levels of severity specifically defined for urban areas and already tested in this dissertation (see § 3.6 and Appendix A Distress Identification Manual for Urban Road Pavements).
3. The inventory is updated (**network GIS insert data**) and completed through pavement visual surveys and traffic counts. This step is vital for determining the current state of the network and to include the effect of traffic in the decision-making process (VOC).
4. The **pavement condition evaluation** is assessed by the PCI value.
5. The **Vehicle Operating Costs (VOC)** is estimated for all the network sections. A written regression between PCI and IRI is used as well as a relation between IRI and VOC.
6. The **pavement deterioration models** are adopted to predict the pavement condition during pavement life.
7. The **maintenance and repair treatment unit costs** are defined to estimate the total costs of every considered strategy (LCCA) together in comparison to user costs (VOC).
8. A **comparison among the different strategies** is proposed to establish priority in the network sections and to perform the correct choice of M, R&R strategy.

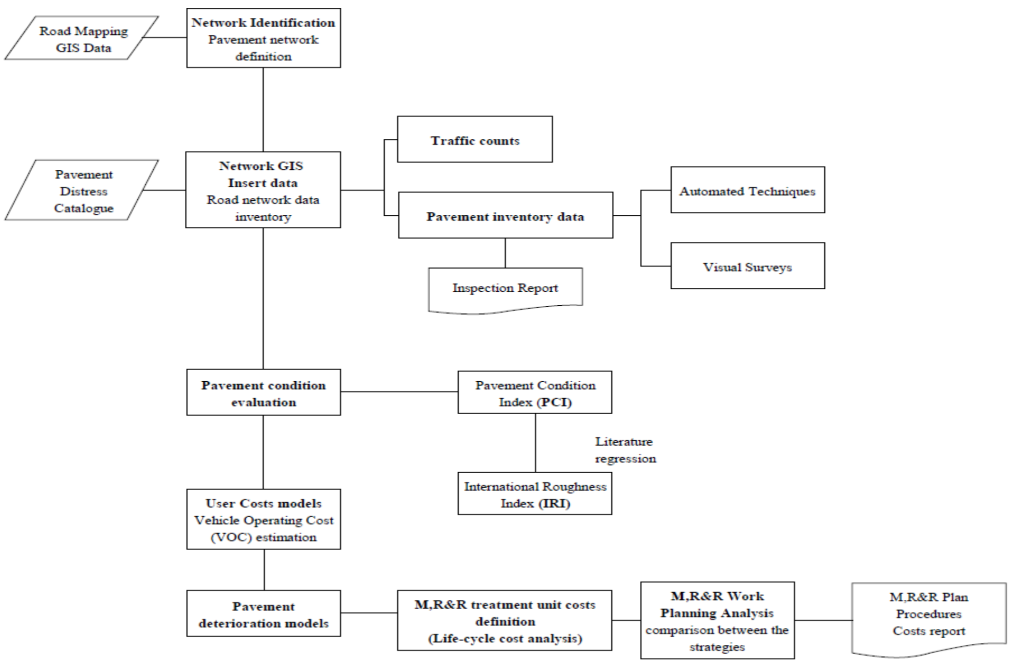


Figure 4-2. Framework of the proposed PMS

4.3 RESULTS OF A PILOT PMS IMPLEMENTED IN URBAN ROAD NETWORK

The described PMS process has been tested on the road network in the town of Valentano, province of Viterbo, see § 3.3 and § 3.4 for more details. The average PCI of the network is 54 (poor condition, as seen in Figure 4-3). The network is very deteriorated: 34% of the pavement area is under the PCI critical threshold (PCI < 55).

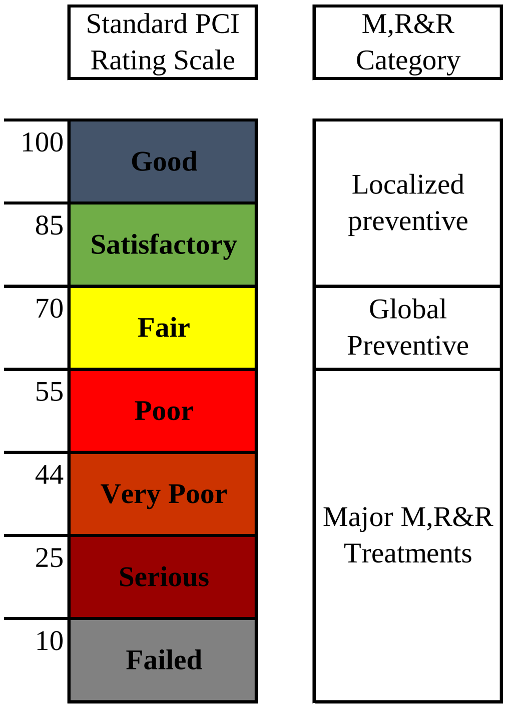


Figure 4-3. PCI Rating Scale for Urban PMS Applications

After that, it has estimated the budget to improve the condition of all sections of the network to bring them above poor condition (PCI > 55). The necessary budget for that purpose is 1,195,720 €; this budget is oversize for a small municipality.

However, the municipality of Valentano has a limited budget of about 100,000 € every five years. Therefore, different alternative strategic repair and maintenance programs have been studied.

Then, priorities and improvement conditions of pavements have been established in order to meet the needs of the network by optimizing their results. This process is especially important in limited funding scenarios as the pavement management of municipal networks.

4.3.1 REPAIR AND MAINTENANCE PLANNING

The strategic aspect of M, R&R requires a planned approach to determine the maintenance policy by setting rules and guidelines the different strategies as well as to examine the impact of alternative funding scenarios.

In this section, the M, R&R alternative strategies are described, and the maintenance policies are set. For this purpose, five repair and maintenance strategies were designed and their effects on the network condition were studied. All of these were based on the current limited budget available by the municipality (100,000 € for a five-year period) and on the network traffic.

For each strategy, the type of M, R&R work, its unit costs, and its effects on traffic and prolonging the life of the pavement were defined, so one strategy can be compared to another.

These strategies combine minor and major treatments on certain sections of the network, defined as critical. Then, the repair and maintenance costs were considered together as to the user costs through the estimation of the VOC.

Table 4-1. Type of M, R & R minor and major treatments

Treatment description (code)		Unit cost
Minor M, R & R treatment	No Localized M&R (NONE)	-
	Crack-Sealing (CR-SEAL)	2.31 (€/m)
	Slurry-Seal (SR-SL)	4.06 (€/m^2)
	Manhole-Reconstruction (RC_MH)	82.99 (€/piece)
	Grinding-Localized (GR-LOC)	4.38 (€/m^2)
	Patching-Leveling (PAT-LV)	11.06 (€/m^2)
	Patching-Deep (PAT-DP)	22.12 (€/m^2)
Major M, R &R treatment	Overlay-Thin-AC (OL-AC)	10.62 (€/m^2)
	Surface-Reconstruction (SR-RC)	21.85(€/m^2)
	Complete-Reconstruction (CPL-RC)	37.77 (€/m^2)

In the following, some possible M, R&R strategies are presented:

- *Strategy 1. Do Nothing.* Aims to show how the network pavement condition evolves by time if no treatment is performed. It is used here to compare the differences with the other strategies.
- *Strategy 2. Run to failure maintenance.* Run to failure maintenance (sometimes known as reactive maintenance) means doing zero prevention or planned maintenance; only repair treatments in the most critical distress areas during the time. *Strategy 2* simulates the current maintenance policy of the municipality which consists in making minor treatments (Table 4-1) when the failure of the pavement has occurred. These repair treatments are performed when some quantities of distress are detected above certain values without any planning. The repair and maintenance works are distributed in the five years of the analysis period, and its influence and cost-effectiveness were analysed (Table 4-2). The results of this analysis suggest that

a reactive maintenance policy would not be cost-effective if the overall pavement network condition is below the PCI critical threshold.

- *Strategy 3. Corrective Maintenance (single investment in the first year). Strategy 3* proposes making major treatments in the first year of the period of analysis with a single investment of 100,000 € to improve the overall PCI of the network into satisfactory condition, from PCI 54 to 74. This strategy includes some major treatments in the 15 most critical sections of the network that are in very poor and serious condition with the highest level of traffic. All the available funds are invested for reconstruction and rehabilitation treatments to bring each of these sections into a satisfactory condition in the first year of the period of study. There is no available budget for minor repairs and maintenance treatments (preventive maintenance) to maintain the overall pavement condition the next four years. Therefore, the average PCI of the network falls to 40 at the end of the five-year period.
- *Strategy 4. Corrective Maintenance (investments equally distributed in the period of study).* Following the same approach of the previous strategy, *Strategy 4* proposes making major treatments in sections with very poor and serious condition but, in this case, the investment is equally distributed in the five years. The repair and maintenance treatments are performed in the most deteriorated sections similarly with *Strategy 3*, so the sections are the same for both strategies. In this strategy, 20,000 €/year for major treatment is allocated and the overall PCI deterioration rate has a slight decrease from 59 PCI value after the first year to 40 value at the fifth year. The critical state of the network influenced this type of policy (*corrective maintenance*) to involve immediate action to avoid imminent failures and preserve the condition and service level of the critical pavement sections. The funds were made equally distributed in the period of study because it is difficult for the municipality to address the entire budget in the first year.
- *Strategy 5. Corrective maintenance (treatments on low traffic sections). Strategy 5* proposes making major treatments in some sections of the network concentrating the M, R&R works in the first, second and third year of the period of study. These sections are different from those chosen for *Strategy 3* and *Strategy 4.* The objective of *Strategy 5* is evaluating the effect of the performance of the M, R&R treatments in those sections with different levels of traffic (traffic level lower than those of the selected sections of the other strategies) in order to compare the results with the aforementioned. The PCI trend (Figure 4-4) over time of *Strategy 5* is similar to that

of *Strategy 4*, so the treatments carried out in these sections are able to maintain the overall pavement condition of the network with an overall PCI value of 40 at the fifth year.

Strategy 3 concentrates the M, R&R works in the first year of the period of study and succeeds in improving pavement condition of the worst sections of the network and reducing the percentage of pavement area in critical condition. Nevertheless, the absence of preventive maintenance treatments in the following years leads to a rapid deterioration of pavement area below the critical PCI threshold.

Strategy 4 maintenance policy could maintain the pavement condition of the network sections with higher traffic levels and in the worst pavement condition.

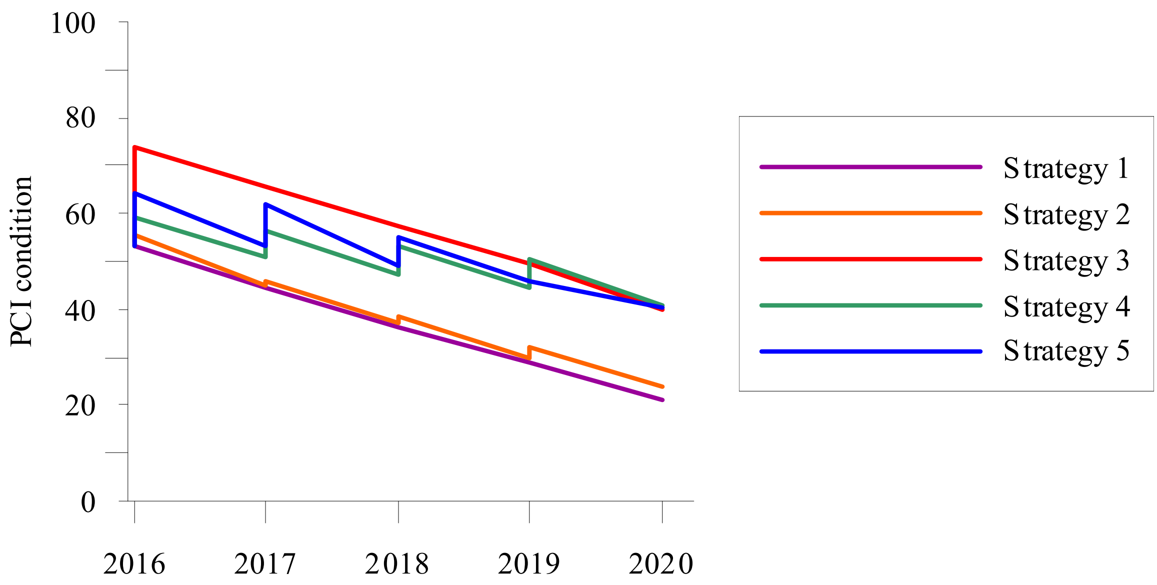


Figure 4-4. PCI condition calculated for each strategy

Table 4-2. M, R&R strategies at the Project Level for Strategies 3, 4 and 5.

Strategy	Branch Id.	Section No.	Year	Section Area	PCI before	M, R&R treatments costs (€)
Strategy 3	B1	S1	2016	175	24	1,858.50
	B3	S2	2016	431	29	4,577.22
	B4	S2	2016	810	35	8,602.20
	B4	S6	2016	233	6	2,474.46
	B4	S7	2016	241	69	2,559.42
	B5	S2	2016	2,127	8	22,588.74
	B5	S3	2016	512	19	5,437.44
	B5	S5	2016	410	28	4,354.20
	B5	S7	2016	438	15	4,651.56
	B6	S8	2016	1,021	14	10,843.02
	B9	S4	2016	647	31	6,871.14
	B10	S4	2016	428	65	4,545.36
	B10	S5	2016	438	27	4,651.56
	B10	S6	2016	233	58	2,474.46
	B11	S2	2016	1,097	62	11,650.14
Strategy 4	B5	S2	2016	2,127	8	22,588.74
	B4	S6	2017	233	6	2,474.46
	B5	S3	2017	512	19	5,437.44
	B6	S8	2017	1,021	14	10,843.02
	B1	S1	2018	175	24	1,858.50
	B3	S2	2018	431	29	4,577.22
	B4	S2	2018	810	35	8,602.20
	B9	S4	2018	647	31	6,871.14
	B5	S5	2019	410	28	4,354.20
	B5	S7	2019	438	15	4,651.56
	B10	S4	2019	428	65	4,545.36
	B10	S5	2019	438	27	4,651.56
	B10	S6	2019	233	58	2,474.46
	B4	S7	2020	241	69	2,559.42
	B11	S2	2020	1,097	62	11,650.14
Strategy 5	B2	S1	2016	952	71	9,919.84
	B4	S2	2016	809	35	8,429.78
	B11	S2	2016	733	77	7,637.86
	B4	S6	2016	236	6	2,430.80
	B1	S1	2017	175	18	1,823.50
	B11	S1	2017	733	72	12,993.74
	B4	S1	2017	2,009	38	20,933.78
	B4	S4	2017	1,894	62	19,735.48
	B3	S1	2017	1,247	71	7,637.86
	B8	S1	2018	321	61	3,344.82
	B7	S1	2018	448	61	4,668.16

Figure 4-5 shows a graphic chart with the percentage of pavement area rated by the PCI scale (below and above critical PCI) of each strategy for every year of the period of study. It is important to highlight that the *Strategy 3* and *Strategy 4*— Figure 4-5 (c) and (d)—are those that keep the network in a better condition with the highest percentage of pavement area above the critical PCI.

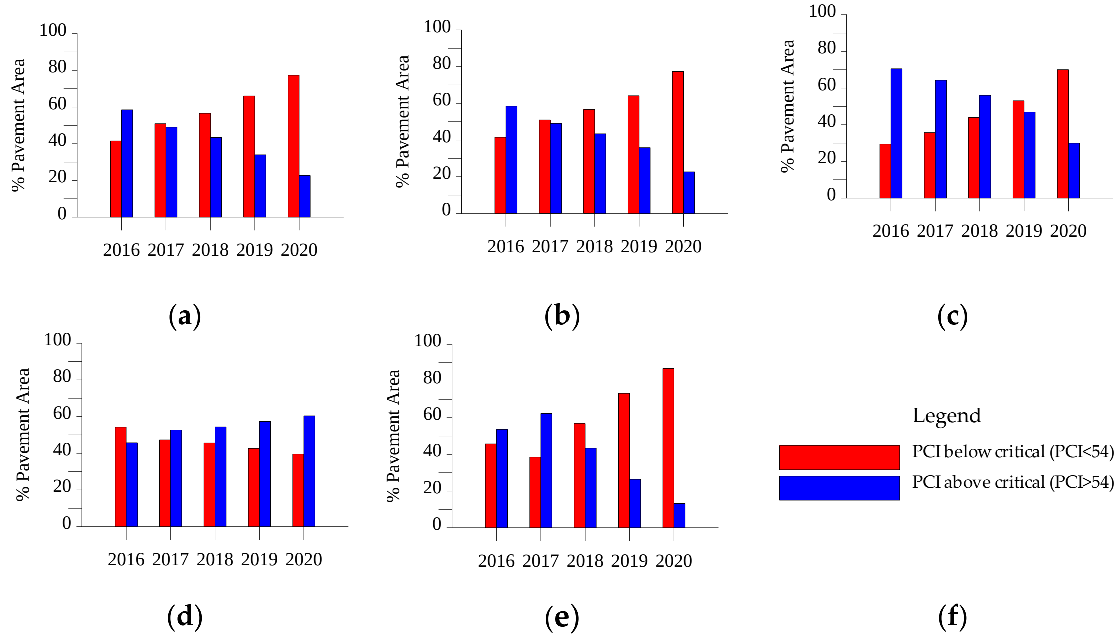


Figure 4-5. PCI section rating for each strategy at the considered period of study

(**a**) *Strategy 1*; (**b**) *Strategy 2*; (**c**) *Strategy 3*; (**d**) *Strategy 4*; and (**e**) *Strategy 5*; and (**f**) the legend.

4.3.2 Life-Cycle Cost Analysis—Vehicle Operating Costs Estimation

An innovative approach is used to verify the traffic influence in the M, R&R works planning; a Life-Cycle Cost tool was proposed to evaluate the benefits of the strategies. As advanced in the description of the proposed PMS, an inventory was created that includes the traffic count for each section. Therefore, the Vehicle Operating Costs may be calculated and all the strategies compared between them.

The VOC are the costs associated with owning, operating, and maintaining a vehicle, including the fuel consumption, tire wear, repair and maintenance costs, as the main components of VOC. The proposed method calculates the VOC focusing on its three main components. They depend on the pavement condition and traffic of each section (type and number of vehicles).

In the proposed VOC model, the effects of pavement condition in VOC are expressed in terms of roughness using the IRI (Chatti and Zaabar 2012). However, the PMS presented in this article assesses the pavement condition in terms of PCI. Therefore, the calculation of VOC requests the estimation of roughness measure through the PCI. For this, a regression between PCI and IRI (Eq. 4-1), which was developed by Arhin et al. (2015) with a study of pavement condition in the District of Columbia (USA), was used. Arhin et al. proposed different regression models changing the functional classification of road (freeways, arterials, collectors and locals) and the pavement type (asphalt, composite and concrete), and they obtained R^2 values between 0.56 (for freeways road) and 0.82 (for asphalt pavement). In our study, the regression model between PCI and IRI obtained for asphalt pavement was used:

$$PCI = -0.224 \times (IRI) + 120.02 \qquad \text{Eq. 4-1}$$

with R^2 = 0.82. The results of the ANOVA tests showed statistically significant F-statistic: the *p*-value for this regression model was determined to be less than 0.05, indicating that the regression model is adequate.

Therefore, IRI can be calculated as a function of the PCI by means of Equation. (2):

$$\mathrm{IRI} = \frac{\mathrm{PCI} - 120.02}{(-0.224) \times 63.4} \qquad \text{Eq. 4-2}$$

where IRI = International Roughness Index, in m/km units, considering 1 m/km = 63.4 in/mi; and PCI = Pavement Condition Index.

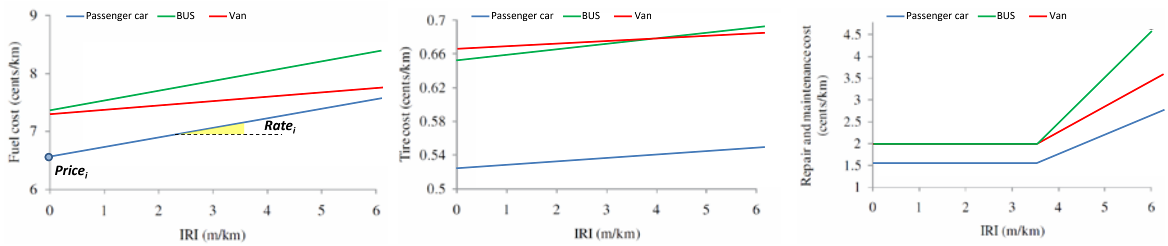
Consequently, as expressed in Eq. 4-3, the annual VOC in the network was calculated as the sum of the VOC in all sections. The VOC in a section was calculated multiplying the number of days in a year (365), the AADT (see Table 4-3) of the IRI, the length of the section and the unit VOC. The unit VOC is the summation of the products of the consumption rates (named *Rate* and depending on IRI) and the theoretical VOC when the IRI = 0 (named *Price*) of the main components of VOC.

The *Rate* and *Price* of each component (such as fuel consumption, tire wear costs and repair and maintenance costs) was calculated using the diagrams from Chatti and Zaabar (2012) of the HDM-4 VOC calibrated to US conditions. Each of these costs are calculated separately and summed up to obtain the annual VOC.

$$VOC = \sum_{j=1}^{N} \left[365 \times AADT_j \times Length_j \times IRI_j \times \sum_{i=1}^{9} \left(Rate_i \times Price_i \right) \right] \qquad \text{Eq. 4-3}$$

where VOC = annual Vehicle Operating Costs of the entire network; N = total number of sections; *AADT* = Annual Average Daily Traffic in section *j*; *Rate* = consumption rate of each component of VOC (tire wear costs, fuel consumption, repair and maintenance costs) (% m/km); *Price* = theoretical VOC of the main components of VOC (cents €/km) when the IRI = 0; *Length* = length of section *j* (km); *IRI* = IRI value of the section *j*; *i* = index of summation of the three VOC components per three different vehicles (nine in total); and *j* = index of summation of the total number of sections of the network.

Figure 4-6 shows some examples of the diagram used to determine the VOC for each strategy in the study period.



Effect of roughness on fuel costs *Effect of roughness on tire costs* *Effect of roughness on repair and maintenance costs*

Figure 4-6. Diagrams to evaluate the effect of roughness on VOC

In the present study, the effect of pavement surface texture (MPD) on VOC was not considered.

Table 4-3. Traffic distribution in network's sections

Branch Id.	Section No.	Passenger Cars	Van	Bus
B1	S1	800	10	0
B1	S2	800	10	0
B2	S1	800	20	0
B3	S1	1,000	30	0
B3	S2	1,000	30	0
B4	S1	800	20	20
B4	S2	800	20	20
B4	S4	1,300	30	20
B4	S6	1,300	30	20
B4	S7	1,300	30	20
B5	S1	3,000	70	50
B5	S2	3,000	70	50
B5	S3	2,000	40	1
B5	S4	2,000	40	1
B5	S5	2,000	40	1
B5	S6	2,000	40	1
B5	S7	2,000	40	1
B6	S1	2,000	40	0
B6	S7	2,000	40	0
B6	S8	2,000	40	0
B6	S9	2,000	40	0
B7	S1	800	10	0
B8	S1	150	10	0
B9	S1	3,700	100	0
B9	S3	3,700	100	0
B9	S4	3,700	100	80
B10	S4	1,000	30	50
B10	S5	1,000	30	50
B10	S6	1,000	30	50
B10	S3	1,000	30	0

B11	S1	500	10	0
B11	S2	500	10	0

The results of the calculation of the VOC for each strategy in the study period are shown in Table 4-4 and Figure 4-7**.**

Table 4-4. Vehicle Operating Costs calculated for each strategy.

Strategy	VOC Total [€]	VOC Reduction [€]	Medium PCI before	Medium PCI after	No. Sections below Critical PCI after
Strategy 1	2,114,987	0	53	21	41
Strategy 2	2,091,395	23,593	53	23	41
Strategy 3	1,673,448	441,539	53	40	37
Strategy 4	1,746,079	368,909	53	40	32
Strategy 5	1,975,060	139,927	53	40	46

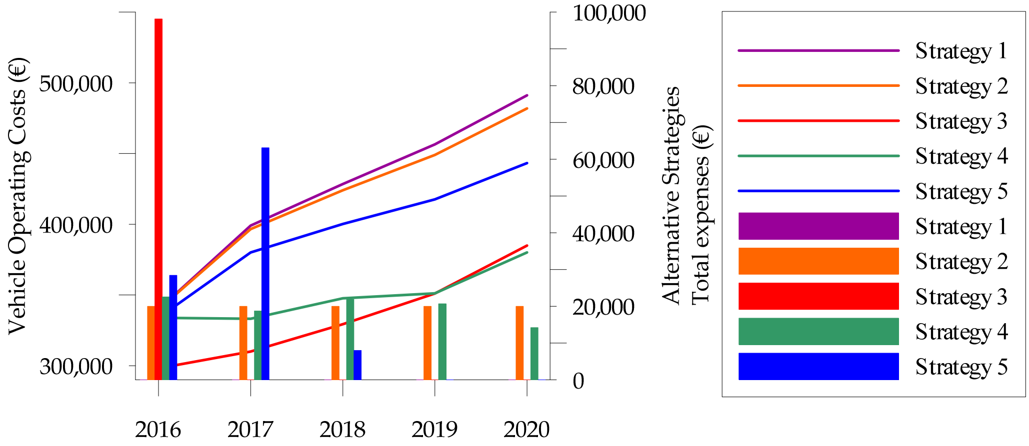


Figure 4-7. Vehicle Operating Costs calculated for each strategy.

4.4 Discussion

Although *Strategy 3* and *Strategy 4* provide similar results, *Strategy 4* is preferred because it plans the budget allocation gradually. Indeed, it is scarcely plausible that the municipality can have the total resources available for five years all together during the first year. The results of this study highlight that *Strategy 4* best fits the needs of the pavement network and achieves the optimization of M, R&R treatments. Actually, it obtains a high reduction of indirect costs and lower deterioration of the network, keeping more sections above the PCI critical threshold. Other findings of this study are the following:

- The analysis carried out is influenced by the agency's initial target budget to maintain pavements because the proposed method is based on the distribution of the available resources; a change in the initial economic conditions will change the management analysis.

- The lack of planned M, R&R treatments and minor treatments without established priorities to allocate funds (*Strategy 2*) is equivalent to doing nothing (*Strategy 1*).
- This case study shows that the budget allocated by the administration is not enough to keep the global network pavement above the critical PCI's threshold, and it would be necessary to study strategies involved in only a few sections and prioritize actions to achieve the maximum improvement of the global network within the constraints of actual budget (€100,000 every five years).
- If the initial status of the network is very poor or serious, it will be very difficult to achieve an acceptable level of PCI of the global network. It would be advisable to invest in the most critical sections of the network (corrective maintenance) to raise the average PCI to a fair level and then invest in preventive maintenance to maintain the achieved network status (see *Strategy 3*).
- Although the PCI trend is similar to *Strategy 4* and *Strategy 5* as shown in Figure 4-4, *Strategy 5* does not reduce the VOC in the same way that *Strategy 4* does (Figure 4-7) because *Strategy 4* maintenance policy could maintain the overall conditions in the network sections with higher traffic levels and in worse conditions and, therefore, significantly reduce the indirect costs estimated by the VOC.
- Evidence has been shown that VOC is a valuable tool to include the influence of the level of traffic in the proposed PMS. Indeed, if the repair and maintenance works has been carried out in the sections with higher level of traffic, the indirect costs would be reduced considerably.

4.5 CONCLUSIONS

The findings of this case of study for the implementation of a PMS in urban small and mid-sized municipalities suggests that the proposed PMS establishes a new approach to the management of the urban network looking at the development of a rational and justified allocation of the resources with a reduced cost of implementation. The proposed method can be readily used in practice because it is a very flexible method that can easily adapt to the needs of a municipality road network.

The results have shown that it is essential to make an initial estimation of the available budget at the network level and, immediately afterwards, determine what kind of repair and maintenance policy must be adopted (preventive or corrective maintenance). As mentioned before, it is strongly influenced by the initial state of the pavement network because the proposed method is based on the distribution of available resources.

The traffic distributions in urban networks influences the maintenance planning prioritization and may be used to allocate the limited available budget of the municipalities by making comparisons of cost-effectiveness between different M, R&R work strategies.

CHAPTER 5. PAVEMENT MANAGEMENT DATA STRATEGIES: GUIDELINES TO ENHANCE DATA PROCESSING AND INTERPRETATION

5. PAVEMENT MANAGEMENT DATA STRATEGIES: GUIDELINES TO ENHANCE DATA PROCESSING AND INTERPRETATION

5.1 Pavement Data Management Methodologies Studied During the Ph.D. Program

Specific methodologies and processing tools were developed during the all the Ph.D. program, to suit the needs of pavement survey data available. Pavement data used were retrieved through collaboration with highway state agencies worldwide. In particular, two PMSs have been studied, utterly different from each other, because are in different geographical areas and the different level of the maturation of the systems. On the one hand, this research uses pavement data of Kostanay region in Kazakhstan pavement data for a pilot PMS to support the development of a pavement management program for the highway National network of Kazakhstan. High-speed survey vehicle collecting pavement roughness, rutting, texture and cracks and FWD deflection tests were performed to the pilot network, ten Kazakhstan Republican Roads for a total of 1500 km for the first inspection. Pavement inventory data is stored in the Road Assessment Management System (RAMS) including pavement traffic data and pavement thickness information (Bonin, Folino et al. 2017), however pavement inventory data does not include M, R&R records because it is a newly implemented system and even these data are not available. Raw survey vehicles provided data for the development of the evaluation of survey data and analysis for maintenance planning with the incorporation of functional and structural evaluation measurements at the network level in developing countries (See *§ Chapter 6 Analysis of Pavement Condition Survey Data for Effective Implementation of a Network Level Pavement Management Program For a Developing Country*).

On the other hand, this study used pavement data available in the VDOT web-based comprehensive PMS database (Figure 5-1), a sophisticated PMS system is implemented (it is have been used since 2005), of pavement sections in the Virginia Highway State roads for the State pavement systems: interstate, primary and secondary. VDOT contracted out pavement vendor for monitoring their highway systems through automated distress data collection and automated calculation of CCI thought the processing of digital images (Chowdhury 2012); yearly for Interstate and Primary systems and five-yearly for the Secondary system. VDOT uses a comprehensive PMS to support decisions regarding the preservation and renewal of the networks.

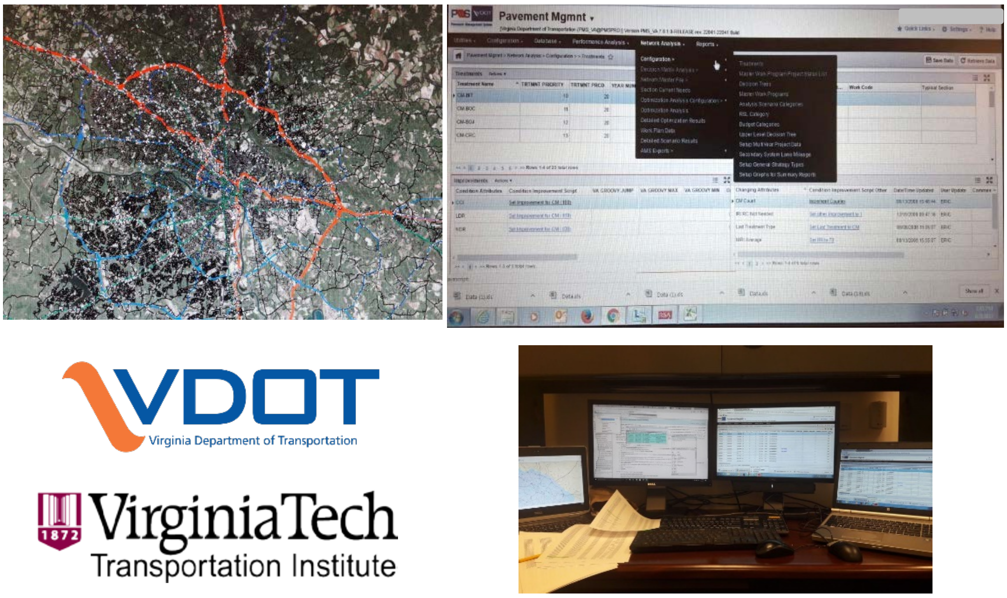


Figure 5-1. VDOT web-based comprehensive PMS database

The data included pavement condition, such as cracks, roughness and rutting data summarized through the CCI indicator, ranging from 0 (failed pavement) and 100 (perfect pavement). Data included 616,360 observations of different pavement ages of pavement sections inspected each year from 2007 to 2017 (11 years of data) for a total length of 4,651 miles (7,485 km) of 357 Interstate, Primary and Secondary routes reported every 0.1 miles.

Pavement inventory data including complete maintenance history (type, year and mileage of start –end of each maintenance story in maintenance section). A total of 42,786 maintenance sections were available and 299,502 M, R&R records were analyzed (See *§ Chapter 7. Development of Pavement deterioration curves of flexible pavements using the Linear Empirical Bayesian Approach at the Network Level*).

5.2 Spatial Integration Tools

Since 2000s VDOT have implemented a Linear Referencing System (LRS) for all their highway systems; in an effort of the agency to integrate asset management data. The objective of LRS is to define locations in space and within the pavement network (spacial referencing), defining the connectivity of assets to the network and of part of the network and to supporting planning scenario to allow the comparison between different temporal versions of the spatial and attribute data (FHWA 2005). LRS allows to locate pavement survey data along with pavement inventory data by use LRS road names and mile/kilometer posts physically distributed along the road to refer survey data events' (i.e., cracks data, roughness data, rutting data and deflection data).

However, pavement survey data collected involves a high quantity of data, and they can be related by its spatial location using the GPS coordinates of the survey events used in conjunction of LRS to integrate data of different sources. For the extension of this research, spatial integration tools were developed to fulfill these needs providing consistent references and assuring the reliability of used pavement data.

Specifically, the primary challenge of managing pavement survey data is the geolocation of inspected feature in the network and their integration in PMS database to enhance the decision-making process (Zhou, Wang et al. 2009, Rydholm and Luhr 2015).

During the development of this research, data integration problems within the PMS components were identified, mainly due to the vast amount of data acquired from different sources (i.e., automated distress data collection, rutting and roughness data, traffic data, …) and necessary to be related with the existing data in the database (i.e., past year pavement survey, inventory data, M, R&R records). Consequently, spatial integration tools were developed to be able to carry out this research successfully. In the following sections, an overview of recommendations and guidelines for the implementation of spatial and geographical procedures are presented.

5.2.1 Implementation of LRS System from Pavement Survey Data

Generally, pavement vendors performed the data collection and pre-processing of pavement data on general access exchange formats such as text (‘*.txt’ files), comma-separated value (‘*.csv’ files) or Microsoft® Excel® files that included collected data and images/video from the surveyed sections.

Pavement data surveyed can be considered as “events” of the pavement network, they can be a linear event: the surveyed property can be associated to a linear feature or segment of pavement route (i.e., CCI values, summarized pavement roughness regarding IRI, summarized pavement rutting) or a point event: the surveyed property can be associate to a point of pavement route (i.e., deflection data).

Nevertheless, the geolocation of survey event to the network elements required the use of a LRS to the entire network, defining each route by a code or route identifier (i.e., IS00095NB – IS for Interstate, 00095 is the number of the route, and NB is to indicate the direction “North Bound”). In an LRS, the elements or features are localized by a linear distance or offset from a known point of the route (i.e., the beginning of the route). Usually, to simplify the works of maintenance districts and to define to the general public known elements in the road, the distances from the known point are physically located in the road using a visible element, "Mile Post" or "Kilometer Post."

Specific database elements are located using this reference system and maintained by the agency to get it updated. Therefore, the survey events can be referenced to the known distance of the LRS using GPS coordinates of the events and the linear or offset from the milepost measured by the survey vehicle.

5.2.2 ENHANCING DATA INTEGRATION USING SPATIAL GPS MATCHING PROCEDURES

In the context of monitoring high size pavement networks, the roads are surveyed by high-speed vehicles equipped with an odometer, which measure the distance traveled by the vehicle acquiring GPS coordinates of each point. Consequently, GPS latitude longitude of known points associated with traveled distances (GPS stations) are used to manage the data of magnitudes surveyed; Table 5-1 shows a sample of the data retrieved from the survey vehicle.

GPS matching procedures were developed to simplify the data integration process and provide consistent and reliable data for their use in the next steps of analysis. In this section, the LRS implementation through GPS data is presented.

Table 5-1. Sample data GPS stations from survey vehicle

RouteID	Direction	km_GPS_station	Lat_GPS	Long_GPS	Alt_GPS
A23	Straight	0,001917	52,4610781	61,72854535	228,59
A23	Straight	0,106965	52,46194553	61,72797045	230,58
A23	Straight	0,209251	52,46279890	61,72743613	232,09
A23	Straight	0,309883	52,46362198	61,72684650	233,39

LRS implementation through GPS data procedure reconstructs the road profile of the surveyed road resampling the profile with a predetermined spatial base according to the purpose of the analysis (i.e., 100 meters, 0.1 miles, 200 meters, 1 km, 1 mile). The algorithm assigns GPS geographical coordinates to the begin and the end of the spatial base-length segments. An algorithmic description of this method is provided below (Figure 5-2).

Algorithm LRS implementation through GPS data

for i=1 to N **do**
Find km_GPS_station between the lower end and the higher end of the spatial analysis base (Begin_km and End_km); r index of input table where there is a match
if r>1 **then** % if more than one match
Find the nearest GPS point.
elseif r=1 **then**
Set GPS coordinates to lower end and higher end of the spatial analysis base
else
Missing value
end if
end for

Figure 5-2. Algorithm GPS procedure LRS implementation through GPS data

This procedure is essential at the beginning of the analysis and processing of data because it provides a geographic basis to integrate the other analysis results successfully. The results of the application of the method are depicted in Table 5-2 and Figure 5-3.

Figure 5-3. Results of the application of GPS matching procedure for the implementation of LRS

Table 5-2. Example output LRS implementation table

RouteID	**Direction**	**Begin_km**	**End_km**	**Begin_Lat_GPS**	**Begin_Long_GPS**	**Begin_Alt_GPS**	**End_Lat_GPS**	**End_Long_GPS**	**End_Alt_GPS**
A23	Straight	0.00	0.10	52.4612	61.7285	228.360	52.4612	61.7285	228.360
A23	Straight	0.10	0.20	52.4612	61.7285	228.360	52.4612	61.7285	228.360
A23	Straight	0.20	0.30	52.4612	61.7285	228.360	52.4612	61.7285	228.360
A23	Straight	0.30	0.40	52.4612	61.7285	228.360	52.4612	61.7285	228.360
A23	Straight	0.40	0.50	52.4612	61.7285	228.360	52.4612	61.7285	228.360
A23	Straight	0.50	0.60	52.4612	61.7285	228.360	52.4621	61.7278	230.520

5.2.3 PAVEMENT DATABASE INTEGRATION – MERGING DATA FROM DIFFERENT SOURCES

In this section, routine to integrate pavement data associating multiple sets of attributes to portions of linear features within LRS is presented. This approach has the advantage to combine different attributes stored in independent tables into the same linear segments.

For example, this procedure can be used to relate pavement condition data and pavement inventory data, see Table 5-3. The algorithm is based on the matching of the upper end and the lower end of LRS segments between tables containing attributes that are referenced in the same LRS. The algorithm is described below, Figure 5-4.

Table 5-3. Pavement condition and pavement inventory datasets

Pavement condition data – CCI Inspections	Pavement construction data M, R & R treatments
Route ID	Route ID
Begin Mile Post	Begin Mile Post
End Mile Post	End Mile Post
Year inspection	Year construction
CCI and Distresses	M,R& T Treatments
GPS data (Latitude, Longitude)	

Pavement Age = Years after Major Maintenance Treatment

Algorithm Data Integration between tables within a PMS

```
for i=1 to N do
Find the linear portions of table 1 (Begin_km1, End_km1) that are completely contained in linear portions of table 2 (Begin_km2,End_km2) having as a result the row of table 2 where the linear portions of table 1 are contained ind(i).
ind1 = find(Begin_km2<=Begin_km1(i) & End_km2>Begin_km1(i));
ind2 = find(Begin_km2<=End_km1(i) & End_km2>End_km1(i));
    if ~isempty(ind1) then
        ind(i,1) = ind1(1);
    end if
    if ~isempty(ind2) then
        ind(i,2) = ind2(1);
    end if
end for
```

Figure 5-4. Algorithm Data Integration between tables within a PMS

Nevertheless, pavement survey data integration can be affected by uncertainties related to the approximation de the extremes of LRS analysis segments used. To relate this data with other tables can be challenging because some of the mileposts - kilometer posts of the LRS can be duplicated or, on the contrary, be left exposed and not effectively coincide with the selected analysis base segments (see columns “BeginMile” and “EndMile” of Table 5-4). Therefore, an algorithm has been developed that allows these segments to be approximated by taking into account the length of the base of the analysis segments used. The result is the adjustment of all the upper and lower ends of each of the rows of the tables (see "Round_BeginMile" and "Round_EndMile" of Table 5-4). The algorithm is described below.

Table 5-4. Example of Application of LRS data integration; get nearest LRS milepost

VDOTSystemID	Route	BeginMile	EndMile	Round_BeginMile	Round_EndMile
Primary	SR00007EB	19.08	19.18	19.10	19.20
Primary	SR00007EB	19.18	19.28	19.20	19.30
Primary	SR00007EB	19.28	19.38	19.30	19.40
Primary	SR00007EB	19.38	19.48	19.40	19.50
Primary	SR00007EB	19.48	19.58	19.50	19.60
Primary	SR00007EB	19.58	19.68	19.60	19.70
Primary	SR00007EB	19.68	19.78	19.70	19.80

Algorithm Data Integration – Get round milepost, kilometer post to find the nearest analysis delimitation milepost, kilometer posts.

---

for i=1 to N **do**
Compare Begin Mile against End Mile dataset, and then evaluate the distance between them to assign it to the spatial base LRS choosen. And consequently, round toward positive (ceil) or round toward zero (fix) the pair of extremes that delimitate analysis segments.
if abs(round(E_MPraw(i),1)/1-round(B_MPraw(i),1)/1)==0 **then**
    if abs(fix(E_MPraw(i)*10)/10-E_MPraw(i))<= 0.05 && abs(fix(B_MPraw(i)*10)/10-B_MPraw(i)) <=0.05 **then**
        if abs(fix(B_MPraw(i)*10)/10 - fix(E_MPraw(i)*10)/10)==0 **then**
          roundB_MP1(i) = fix(B_MPraw(i)*10)/10;
          roundE_MP1(i) = ceil(E_MPraw(i)*10)/10;
        else then
        roundB_MP1(i) = fix(B_MPraw(i)*10)/10;
        roundE_MP1(i) = fix(E_MPraw(i)*10)/10;
        end if
    elseif abs(ceil(E_MPraw(i)*10)/10-E_MPraw(i))< 0.05 && abs(fix(B_MPraw(i)*10)/10-B_MPraw(i)) <0.05 **then**
        roundB_MP1(i) = fix(B_MPraw(i)*10)/10;
        roundE_MP1(i) = ceil(E_MPraw(i)*10)/10;
    elseif abs(ceil(B_MPraw(i)*10)/10-B_MPraw(i))<0.05 && abs(fix(E_MPraw(i)*10)/10-E_MPraw(i)) <0.05 **then**
        roundB_MP1(i) = ceil(B_MPraw(i)*10)/10;
        roundE_MP1(i) = ceil(E_MPraw(i)*10)/10;
    elseif abs(ceil(B_MPraw(i)*10)/10-B_MPraw(i))<0.05 && abs(ceil(E_MPraw(i)*10)/10-E_MPraw(i)) <0.05 **then**
        roundB_MP1(i) = ceil(B_MPraw(i)*10)/10;
        roundE_MP1(i) = ceil(E_MPraw(i)*10)/10;
    elseif abs(ceil(B_MPraw(i)*10)/10-B_MPraw(i))<0.05 && abs(fix(E_MPraw(i)*10)/10-E_MPraw(i)) <0.05 **then**
        roundB_MP1(i) = ceil(B_MPraw(i)*10)/10;
        roundE_MP1(i) = ceil(E_MPraw(i)*10)/10;
    elseif abs(fix(B_MPraw(i)*10)/10-B_MPraw(i))>0.05 && abs(fix(E_MPraw(i)*10)/10-E_MPraw(i)) <0.05 **then**
        roundB_MP1(i) = ceil(B_MPraw(i)*10)/10;
        roundE_MP1(i) = ceil(E_MPraw(i)*10)/10;
    elseif abs(ceil(E_MPraw(i)*10)/10-E_MPraw(i))< 0.05 && abs(ceil(B_MPraw(i)*10)/10-B_MPraw(i)) <0.05 **then**
        roundB_MP1(i) = ceil(B_MPraw(i)*10)/10;
        roundE_MP1(i) = ceil(E_MPraw(i)*10)/10;

```
        else then
        end if
    else then
        roundE_MP1(i)=round(E_MPraw(i),1)/1;
        roundB_MP1(i)=round(B_MPraw(i),1)/1;
    end if
end for
```

Figure 5-5. LRS data integration algorithm; get the nearest milepost

5.3 PAVEMENT HISTORICAL DATA MANAGEMENT

Pavement data retrieved from the PMS can be characterized by uncertainties in data collection methods and interpretation (i.e., subjective judgement of paver rater in manual or semi-automated methods, uncertainties on automated cracks detections in automated data collection methods, interpretation of distress data results, use of pavement condition indicators) that can introduce bias in the data. As overviewed, this study used pavement condition data stored in VDOT comprehensive PMS that included pavement condition inspected yearly from 2007 to 2016.

In general, the usual pavement performance decrease over time, until the pavement is treated (is receiving maintenance treatment). However, the analysis of pavement data reveals that there are some not expected jumps on pavement performance (a positive variation on pavement condition) as a result of pavement sources of uncertainty above mentioned, which make difficult the pavement performance analysis. Therefore, a significant definite jump of pavement performance over time can be caused by significant maintenance treatment applied to the pavement that restores its initial condition; that is, a maintenance treatment that should reset pavement age.

Data filtering and cleansing criteria would be used to enhance pavement performance analysis. From the analysis of pavement data over time of different inspections, the following situations can be highlighted as depicted in Figure 5-6.

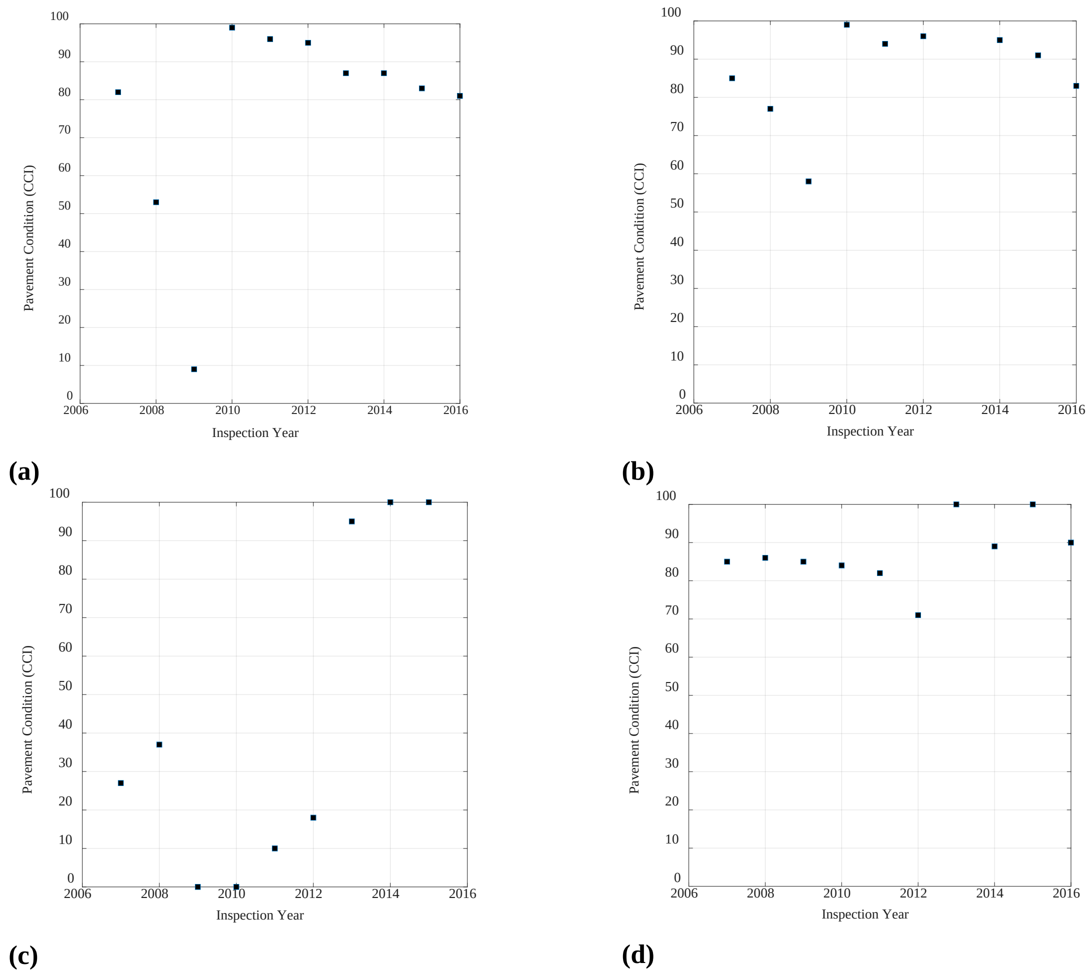


Figure 5-6. Data Processing Analysis: Typical Pavement Performance Situations

The analysis of pavement condition data from different sections inspected yearly in the network from 2006 to 2016 shows a variety of situations of observations in the network. As it can be observed, the evolution of the condition of the pavement in time shows the expected deterioration of the pavement in time, CCI index is decreasing over time following a deterioration trend. However, it can also be observed that some sections have been treated in different stages of the useful life of the section under examination. For example, Figure 5-6 (a) shows that in 2009, the section was quickly deteriorating and pavement is close to the end of its useful life. Therefore, the pavement section was treated with a conservation work to re-establish its useful life, so two life cycles of the pavement must be defined, one that had been developing until 2009 and another that begins in 2010 (after the conservation work). Pavement deterioration cycle is starting again the condition of 100; that means that the M, R&R had "reset" its pavement life. To adequately model the deterioration of this section, pavement section age should start counting from zero after 2010.

In Figure 5-6 (b) pavement is deteriorating over time, and the significant jump on pavement condition in 2009 suggests that an M, R&R had been carried out in the section, and similarly, of the case (a), the pavement is starting to deteriorate from a value of 100 of CCI. As it can be noted, there is not a pavement observation for that specific section in 2013. This is one of the sources of uncertainties in pavement data.

However, Figure 5-6 (c) situation is different from (a) and (b), different observations are rather unusual on pavement deterioration process, in 2007 CCI is almost 28 points and contrary to how expected CCI is higher in 2008 near to 40. That pavement condition improvement is unusual because the pavement condition is decreasing again, even to a failed condition of the section (CCI to 0), and successfully increasing in 2011 and 2012. After 2012, a definite jump of pavement condition is detected (from 20 points to 99), pavement was restored. Analyzing this situation, it can be considered that there are observations in this section that should be filtered out from the analysis dataset and should not be taken into consideration since they could lead to a biased situation that effectively does not represent the deterioration process.

Figure 5-6 (d) shows another case, different from the previous ones since the road section analyzed had a better condition in early stages of it (high values of CCI) and pavement deteriorates slowly following a very soft tendency that can be identified qualitatively. The analysis of the data shows that when conservation managers detected a slight worsening more pronounced in 2012, applying a strategy of pavement preservation. Additionally, it is detected that in 2015 the condition of the pavement worsens and improves again in successive observations. If that observations were included in the pavement deterioration process analysis dataset, it would introduce errors in the modeling of the deterioration process. The unusual improvement of pavement condition section could have been produced because a small M, R&R preventive maintenance activity.

5.4 DATA VISUALIZATION TOOLS

Network level pavement management utilizes a high quantity of information for their practical use. Data comes from a large variety of sources containing pavement inventory data, results from pavement survey, budgeting and investment business information, etc. However, the high amount of data stored in PMS database can difficult the extraction of the information the manager need that can be hidden in the database. Therefore, to aid the decision-making process and to improve the quality of the information retrieved, useful data visualization tools and data mining applications need to be implemented in the PMS, including decision trees and association rules to associate attribute data stored in the PMS with the geographical position of network road segments with the aim of performing priority analysis and cost-effective strategies.

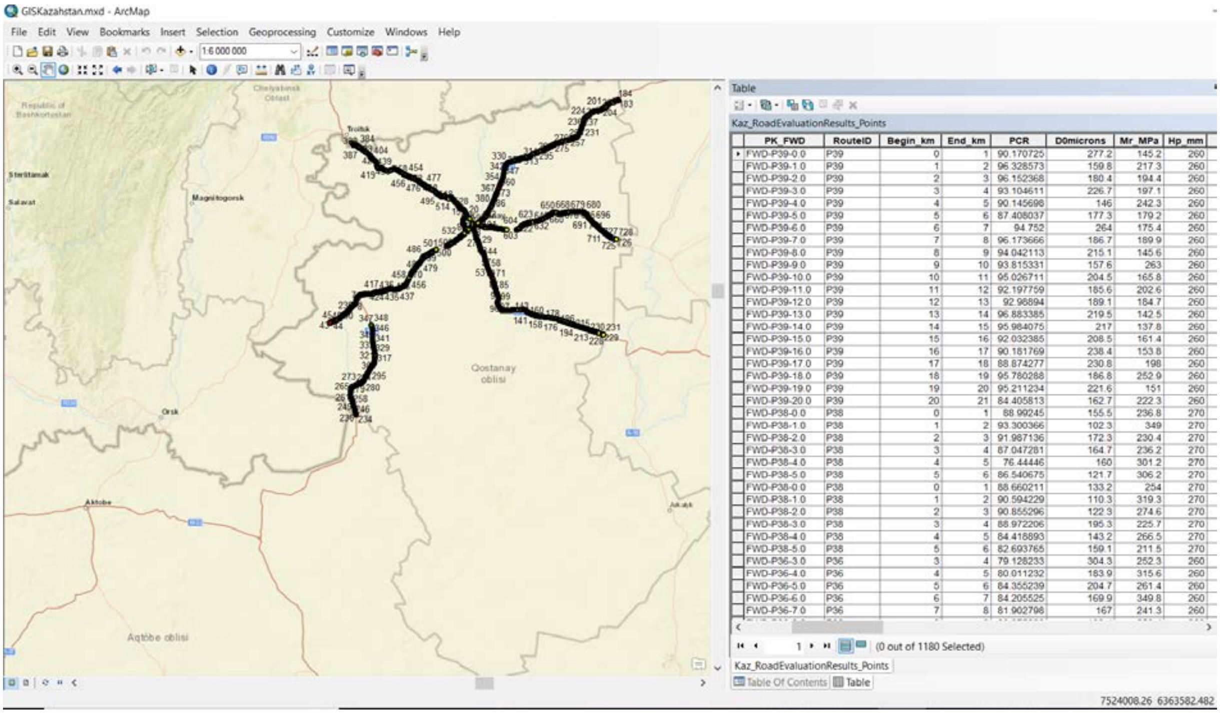
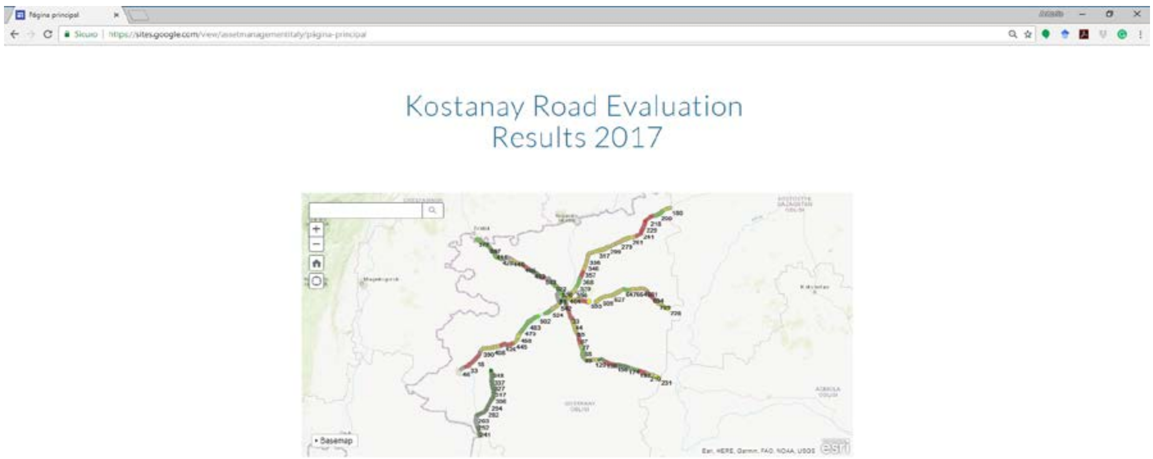


Figure 5-7. Use of Geographical Information Systems to support pavement decision-making at the network level

Data spatial integration tools and database merging procedures presented in this chapter demonstrated to be effective in the integration of data from different sources by their spatial location, and the use of LRS have turned to be effective to provide a geographical background to the data. GIS models can be developed by the road agency for their internal use to support business process and to provide information to the general public through online portals. GIS models can be useful to visualize, handling pavement data and to perform geographical analysis (i.e., land use, network size and coverage, …) using piecharts, maps, and using GIS interface would be also useful to the operator to communicate key attributes about their groups.



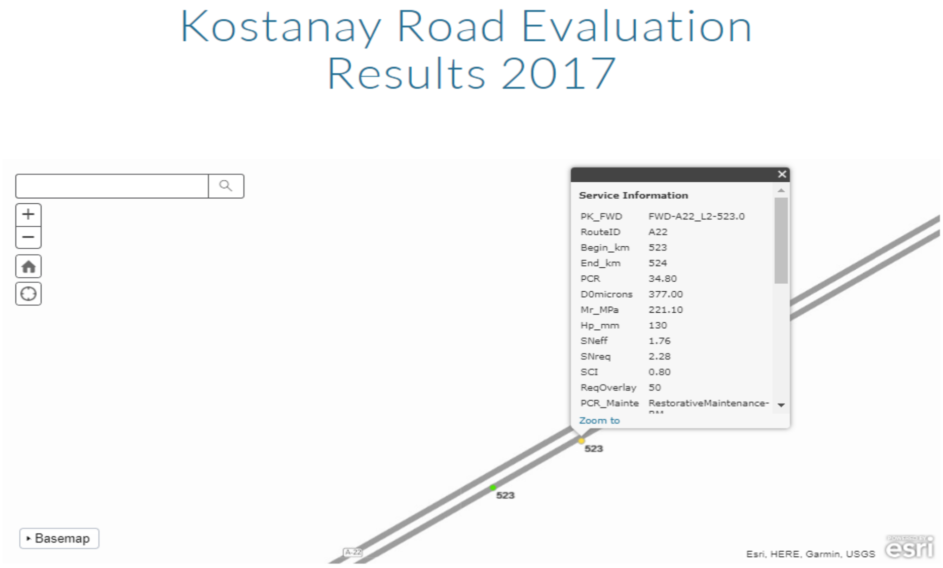


Figure 5-8. Web-based API interface uploaded online to present the results of the study

GIS models can be produced and visualized using proprietary software (ArcGIS®, Global Mapper or AutoCAD MAP 3D) or using freeware or open source (gvGIS, qGIS) using attribute tables stored in exchange files such as shapefile (.shp) or relational database using Database Management Systems (DBMS), see Figure 5-7.

In the recent years, there has been easily accessible web-based mapping software such as, Google Maps® and Bing Maps ® and other freeware tool as OpenStreetMap, that give public access to huge amount of information and geographic data allowing the transportation users including geolocalized information using their tablets or mobile phones, providing feedback and suggestions about their perception of transportation systems, providing a valuable source of information that can be used by road agency as a criteria for priority analysis. To provide information for the users, it can be the application programming interface (API) such as ArcGIS® that can be used to create web-based applications for presenting pavement data, see Figure 5-8.

5.5 Discussion and Conclusions

As presented in this chapter, pavement recorded condition can have a very high variance because of uncertainties in the data collection methods interpretation of distresses, and the inherent variability of individual sections. Therefore, the implementation of cost-effective PMSs necessarily involves collecting and analysing M, R&R treatments recorded in the network. Compare the M, R&R works with the pavement inspections and their extensive analysis is essential to model pavement deterioration. Data processing and data quality procedures were developed on the base of situations depicted in Figure 5-6, and the effectiveness of M, R&R treatments were analysed, in order to reset pavement age when the maintenance records reveal a significant maintenance treatment that restore pavement condition.

However, it must be considered that as introduced at the beginning of the chapter, the PMS implemented in Virginia works exhaustively since 2005, with data from structured inspections in the PMS database since 2006. Therefore, how pavement deteriorate over time can be analysed for those years in which the roadway has been restored to its initial condition

For this reason, modelling deterioration is still a big challenge. The analysis that can be accomplished is limited. PMS stored data is reliable for the 2006-2016 period, in which the history of deterioration of the pavement is well known (material used, pavement thickness, contractor code, the extension of treatment), see Table 5-3.Besides, pavement inspections carried out in the network are known and together with M, R&R records, pavement section time-series can be reconstructed. Nevertheless, the effectiveness of maintenance strategies in the whole system (analysing the CCI before and after) can be investigated.

The next Chapters of the dissertation presents studies developing PMS strategies and database conformations/ analysis procedures that were used for introducing knowledge in data processing and analysis from survey data:- to support the decision-making process and priority analysis at the network level in development countries (Chapter 6); - for the development of deterioration models for flexible pavements in the State of Virginia (Chapter 7) and; - to analyze the effects of pavement preservation practices currently in use for road agencies to design cost-effective pavement management maintenance practices at the network level (Chapter 8).

CHAPTER 6. ANALYSIS OF PAVEMENT CONDITION SURVEY DATA FOR EFFECTIVE IMPLEMENTATION OF A NETWORK LEVEL PAVEMENT MANAGEMENT PROGRAM FOR A DEVELOPING COUNTRY

6. ANALYSIS OF PAVEMENT CONDITION SURVEY DATA FOR EFFECTIVE IMPLEMENTATION OF A NETWORK LEVEL PAVEMENT MANAGEMENT PROGRAM FOR A DEVELOPING COUNTRY[3]

6.1 INTRODUCTION

As introduced in § *5.1 Pavement Data Management Methodologies Studied During the Ph.D. Program*, pavement survey data used during the Ph.D. were retrieved as a result of international collaborations with highway state agencies worldwide. This Chapter uses pavement data from Kostanay Region in Kazakhstan.

Transportation highway systems are essential for the political stability and economic growth in developing countries (Mačiulis, Vasiliauskas et al. 2009). However, backlogs and high costs of maintenance characterized the highway systems in those countries. Therefore, road agencies need appropriate procedures and methods for survey data processing to develop business planning of future investment at the strategic level based on pavement condition assessment.

Development banks are promoting the implementation of Performance-Based Maintenance Contracting (PBMC) for their funded projects, since the payments for these projects are based on performance measures. However, network-level pavement management strategies (PMS) in these countries are limited, and there is a lack of experience of road agencies personnel.

The research presented in this Chapter is focused on the development of a methodology for the analysis and processing of survey data Kazakhstan to implement a national pavement management program for the national network able to identify maintenance needs at the network level to define mid-term M, R&R strategies. The developed methodology successfully integrates functional survey data (cracking, roughness, rutting, and texture) and structural performance measurements (deflection data using Falling Weight Deflectometer (FWD), into a decision-making process to ensure cost-effective maintenance strategies. Besides, this research also deals with the use of spatial data processing techniques to ensure consistency and enable its management through the implementation of Linear Referencing Systems (LRS) to

[3] Partial results of the presented work have been accepted for presentation at the 98th Transportation Research Board Annual Meeting 2019 as "Pantuso, A., Loprencipe, G., Bonin, G. & Teltayev, B. (2019). Analysis of Pavement Condition Survey Data for Effective Implementation of Performance-Based Maintenance Contract. "

support data integration and the creation of pavement management database (see § 5.2.1 for further information). Matching spatial procedures were accomplished using feature coordinates in adopted LRS and Geographical Positioning Systems (GPS) locations recorded of events features (latitude, longitude) of surveyed data within the managed network (see *Chapter 5: Pavement Management Data Strategies: Guidelines to Enhance Data Processing and Interpretation § 5.2.1 Implementation of LRS System from Pavement Survey Data,* for more details). Therefore, the implementation of LRS provides a base reference of the road profile and allow to resample the profile with a predetermined spatial base according to the purpose of the analysis (100 m base length for rutting, IRI-roughness, macrorough and cracking data, and 1 km base length for deflection data).

6.2 PAVEMENT SURVEY DATA AVAILABLE

This research used data collected in the Republic Road Network of Kazakhstan for Asphalt Concrete (AC) pavements in the region of Kostanay Oblast, 1514 kilometres of highway roads (see *§ 5.1*).

The pavement surface distress data were collected and processed using continuous digital imaging, and automated cracking detection techniques of pavement surface distress for straight and reverse traveling directions (see Figure 6-1 a). Distress type, severity, and extension of surveyed distresses (alligator cracking, longitudinal and transverse cracking, block cracking, raveling, and patching) were processed with LCMS, (Pavemetrics 2018) on a continuous basis. Simultaneously, longitudinal and transverse profiles measurements such as roughness measured according to Class I profilometer (ASTM E 950-94, (ASTM 2018)) transverse profile to measure transverse evenness using the measurements of sensors of a profilometric bar mounted on surveying integrated vehicle traveling at highway speed, and surface texture measurement was surveyed for straight and reverse directions. Profiling data were provided each 100 meters continuously from the pavement survey data available. For ease of interpretation, surface distress data summarized each 100 m.

On the other hand, pavement deflection data were collected using a Dynatest FWD device (see § 2.2.5). Pavement structural thickness and material data were available from obtained from road inventory data available (passport data) (see Figure 6-1, b)).

(a) Profiling and cracks survey vehicle
Source: SPT Srl– Studi e Pianificazione del Territorio

(b) Dynatest© FWD Testing

Figure 6-1. Pavement Survey Equipment

6.3 PAVEMENT DATA ANALYSIS AND PROCESSING

6.3.1 ANALYSIS OF PAVEMENT FUNCTIONAL EVALUATION: PAVEMENT CONDITION RATING

Pavement survey data were analyzed using a dual approach which consists of integrating pavement functional condition and structural evaluation to ensure cost-effective mid-term maintenance treatment strategy.

The PCR (FHWA 2009), overall global functional condition indicator has been proposed to combine the surface distress effectively into a composite index (Eq. 6-1) – cracks collected, rutting data, and roughness data – ranging from 0 (failed pavement) to 100 (perfect pavement), see § *2.3.3 National Park Services: Pavement Condition Rating (PCR).*

$$\mathrm{PCR} = 0.6 \cdot \mathrm{PCI} + 0.4 \cdot \mathrm{RCI} \quad \text{Eq. 6-1}$$

$$\mathrm{RCI} = 253.67 \cdot e^{-0.459 \cdot \mathrm{IRI}} \quad \text{Eq. 6-2}$$

where PCI is the standardized ASTM D6433 PCI (ASTM 2018) and RCI is the Roughness Condition Index, and RCI (Eq. 6-2) is a roughness indicator ranging from 0 to 100 with the aim to adopt a similar scale to the PCI calculated using roughness data expressed in the International Roughness Index (IRI) (ASTM 2015). Table 6-1 describes the correspondence between IRI and RCI rating scale, adapted from Table 2-9. To allow the computerization of RCI calculation, (Eq. 6-2) an exponential regression equation was defined to calculate the RCI starting from IRI, imposing an upper limit 100 and lower limit 0, respectively.

The use of PCR as the combination of the two indexes depends on the fact that the calculation of the PCI was done based on available distress type and severity recorded from automatic identification of distresses using continuous imaging. As a consequence, some distress types associated with ride quality were not measured. However, the use of RCI makes possible to consider their effects regarding ride quality adequately. Rutting data and cracking distress data (distress severity, type, and quantity) were summarized every 100 m assumed as a sampling interval, and standardized ASTM D6433 PCI (ASTM 2018) was calculated using VBA procedure presented in §3.5.1.

Table 6-1. Correspondence between IRI and RCI rating scale

Suggested Maintenance Treatment Category	Rating Category	IRI Value Range (m/km)	RCI Value Range
Do Nothing(DN)	Excellent	< 2.50	85 - 100
Preventive Maintenance (PM)	Good	2.50 - 3.00	60 - 85
Corrective Maintenance (CM)	Fair	3.00 - 4.00	40 - 60
Restorative Maintenance (RM)	Poor	4.00 - 5.00	26 - 40
Rehabilitation (RB)	Very Poor	> 5.00	< 26

6.3.2 ANALYSIS OF TRAFFIC DATA

Traffic data effects on pavement performance can be estimated using the ESAL concept (AASHTO 1993) to establish a damaged relationship for comparing the effect of axles of different loads. The damage effects of the full spectrum of traffic of axle loading (different axle loads and axle configurations predicted over the design period) are estimated using an equivalent number of single axle load applications summed over that period (Watanatada 1987), see §2.2.5.

This research involves the study of the Kazakhstan's traffic spectrum and the calculation of ESAL (see Eq. 2-6 and Eq. 2-7) per vehicle category have been calculated based on vehicle counts for each traffic category group (see Table 6-2).

Table 6-2. ESAL per vehicle category group Kazakhstan full traffic spectrum

Vehicle Category	GCM	Axles	Axle 1	Axle 2	Axle 3	Axle 4	Axle 5	Axle 6	$ESAL_F$
Light loads and minibuses	<3.5 t	2	S *	S *	-	-	-	-	**0.0013**
Bus medium, 20-40 seats	8 t	2	S 2.4	D 5.6	-	-	-	-	**0.239**
Bus heavy, more than 40 seats	15 t	2	S 4.5	D 10.5	-	-	-	-	**2.958**
Single trucks, less than 2 t	2 t	2	S 0.6	S 1.4	-	-	-	-	**0.002**
Single trucks, from 2 to 5 t	5 t	2	S 1.5	S 3.5	-	-	-	-	**0.037**
Single trucks, from 5 to 10 t	10 t	2	S 3.0	D 7.0	-	-	-	-	**0.584**
Single trucks, from 5 to 10 t	10 t	3	S 2.6	TN 3.7	TN 3.7	-	-	-	**0.139**
Single trucks, from 10 to 20 t	20 t	4	S 4.2	S 5.0	TN 5.4	TN 5.4	-	-	**1.017**

Road-trains with trailer, 11=11	25 t	4	S 5.25	S 6.25	TN 6.75	TN 6.75	-	-	**2.482**
Road-trains with trailer, 11=12	25 t	5	S 4.0	S 5.25	S 5.25	TN 5.25	TN 5.25	-	**1.403**
Road-trains with trailer, 12=11	25 t	5	S 4.0	S 5.25	D 5.25	TN 5.25	TN 5.25	-	**1.174**
Road-trains with trailer, 12=12	25 t	6	S 3.0	D 4.75	D 4.75	S 3.0	D 4.75	D 4.75	**0.712**
Truck tractors with semitrailer, 111	14 t	3	S 4.2	S 4.2	D 5.6	-	-	-	**0.550**
Truck tractors with semitrailer, 112	20 t	4	S 4.2	D 5.0	TN 5.4	TN 5.4	-	-	**0.828**
Truck tractors with semitrailer, 113	30 t	5	S 4.2	D 6.6	TR 6.4	TR 6.4	TR 6.4	-	**2.077**
Truck tractors with semitrailer, 121	38 t	4	S 7.22	D 10.26	D 10.26	TN 10.26	-	-	**10.752**
Truck tractors with a semitrailer, 122	38 t	5	S 6.84	TN 7.79	TN 7.79	TN 7.79	TN 7.79	-	**5.687**
Truck tractors with a semitrailer, 123	38 t	6	S 4.94	TN 6.94	TN 6.94	TR 6.46	TR 6.46	TR 6.46	**3.203**
Tractors, Light loads with trailer	3 t	3	S 0.7	S 1.3	S 1.0	-	-	-	**0.002**
Tractors, Heavy with trailer	10 t	4	S 1.0	S 2.0	TN 3.5	TN 3.5	-	-	**0.101**

Note: GCM = Gross Combination Mass; S = Single-Wheel Single Axle; D = Dual-Wheel Single Axle; TN = Tandem-Wheel Tandem Axle; TR = Dual-Wheel Triple Axle
*** Axle loads are different for each vehicle category; $ESAL_F$ of this class is the average of all $ESAL_F$ vehicles considered.**

6.3.3 ANALYSIS OF DEFLECTION DATA: REQUIRED OVERLAY PARAMETER

Pavement deflection data were merged with inventory data (pavement thickness and materials) to analyze deflection bowl or deflection basin obtained from FWD test. Pavement thickness and materials, and traffic counts in inventory data were used to calculate the effective SN of existing pavement, (SN_{eff}), Eq. 2-4 (AASHTO 1993) using Rohde (1994) method (see § 2.2.5), and the required SN_{req} that indicates the total thickness required for a given pavement thickness as a function of the accumulated traffic values over a design period (20 years) expressed in ESALs, $ESAL_{20}$ (Eq. 2-8). The design period has been considered of 20 years to be consistent with the definition of AASHTO Guide and PSI serviceability rating.

The subgrade support (resilient modulus, M_R) was estimated with the outer FWD deflection (considered as almost entirely controlled by the subgrade) using Boussinesq equation (Eq. 2-9) for single-point load, P, applied on a semi-infinite elastic space, according to the formulation derived from (Papagiannakis and Masad 2017), considering the Poisson ratio as 0.35 and using the distance of the outer geophone expressed in mm (1800).

The elastic modulus at the asphalt temperature measured during testing, E_p^{Tm}, is obtained as an estimate of the SN_{eff} (Eq. 2-4):

$$E_p^{Tm} = \left(\frac{SN_{eff}^{Tm}}{0.0045 \cdot H_p} \right)^3 \qquad \text{Eq. 6-3}$$

The back-calculated asphalt moduli can be adjusted for temperature to the reference temperature (º68 F – 21 º C) using the relationship proposed by Lukanen, Stubstad et al. (2000)

for the LTPP, (Eq. 6-4). Therefore, the corrected asphalt concrete elastic modulus $E_{\mathrm{p}}^{\mathrm{Tr}}$ is obtained through (Eq. 6-5). The use of this equations is a necessary step to make a comparison between other back calculation methods that can be used from deflection measures (see *Chapter 2: Literature review § 2.2.5 Pavement structure evaluation*)

$$\mathrm{ATAF} = 10^{-0.021\cdot(T_r - T_m)} \qquad \text{Eq. 6-4}$$

$$E_{\mathrm{p}}^{\mathrm{Tr}} = \mathrm{ATAF}\cdot \mathrm{E}_{\mathrm{p}}^{\mathrm{Tm}} \qquad \text{Eq. 6-5}$$

where: ATAF = Asphalt Temperature Adjustment Factors; T_m = Measured temperature (ºC); T_r = Reference temperature 20 ºC (º68 F);

The objective of this research is to use FWD deflection measurements into the M, R&R decision making process at the network level. With this aim, it has been defined the required overlay parameter (d) (Eq. 6-6) that give information about the maintenance treatment thickness that is needed to ensure the pavement structural life over the design period and as a consequence design rehabilitation treatment that would improve pavement structural capacity:

$$d = \frac{SN_{\mathrm{req}} - SN_{\mathrm{eff}}}{0.44\cdot(1-C)} \qquad \text{Eq. 6-6}$$

where d = the required overlay in inches, SN_{req} and SN_{eff} are the required and effective SN, respectively; C is a factor based on the condition of the pavement (Huang 2004); the C factor represents the percent of the contributing structure that remains in the removed layer of asphalt; for the extent of this study, the value C = 0.35 would be used. This relationship based on a relationship proposed by Bryce, Flintsch et al. (2012) assumes that the asphalt has an equivalent structural coefficient of 0.44 per in., and the thickness of the design rehabilitation treatment that is associated with the contribution of pavement overlay thickness to increase the current structural capacity to support the predicted accumulated traffic over the design period.

Therefore, the rehabilitation design treatment to restore the pavement structural capacity can be calculated, as a function of *d*. The remaining service life (*RL*) can be calculated as the time until the cumulated number of ESALs would reach the $\mathrm{ESAL_{eff}}$ calculated from (Eq. 6-7):

$$\log\left(ESAL_{eff}\right) = \left(Z_R\cdot S_0\right) + 9.36\cdot\log\left(SN_{\mathrm{eff}}+1\right) - 0.2 + \frac{\log\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.4+\frac{1094}{\left(SN_{\mathrm{eff}}+1\right)^{5.19}}} + 2.32\cdot\log\left(M_R\right) - 8.07 \qquad \text{Eq. 6-7}$$

6.4 Methodology for Determining Homogeneous Sections and a Decision-Making Process Based on Survey Data

6.4.1 Definition of Pavement Homogeneous Section Criteria

As a result, pavement surface distress data, roughness, and rutting data expressed through PCR (Eq. 6-1), and pavement deflection data summarized by the required overlay parameter (*d*), are used simultaneously to determine the section maintenance needs' and to define the suggested maintenance repair treatments. This dual approach is essential because PCR provides a general idea about the current pavement condition. Nonetheless, pavement surface distresses would not always be able to reflect structural deficiencies in the examinated pavement sections, which are revealed only when certain distress types visually appear in the pavements. Consequently, cost-effective decision-making maintenance strategies require to perform pavement structural evaluation that would be incorporated into the decision-making process through the parameter *d*. Practically, *d* is used to design M, R&R treatments and, in conjunction with the PCR, roadway managers would get into the final suggested maintenance treatment. However, once pavement indicators have been defined, this research defines criteria to divide network routes into homogeneous sections. For this purpose, FWD deflection data can be analyzed and processed to get the center deflection data normalized to the 40 kN-standard load (Eq. 6-8). The accumulated differences of the center deflections were used to divide the length of the test run into homogeneous sections.

$$AD_0 = \sum \delta_i - \mu \cdot i \qquad \text{Eq. 6-8}$$

where AD0 = Accumulated difference at the i_{th} station; $\sum\delta_i$ = sum of deflections from the first station to the i_{th} station inclusively; i = station number from δ_1 to δ_i ; μ = mean deflection tested on the road. According to this approach, a section will be considered homogeneous when the cumulative differences are in the same upward or downward trend (Figure 6-2).

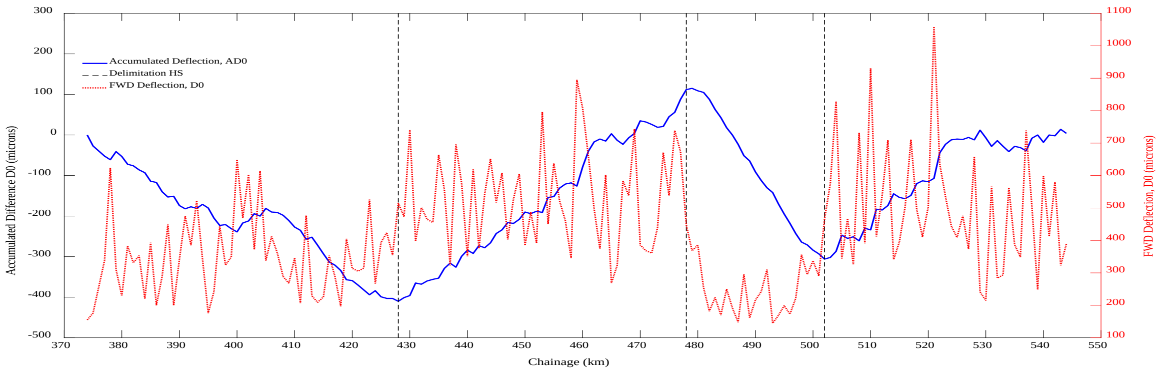
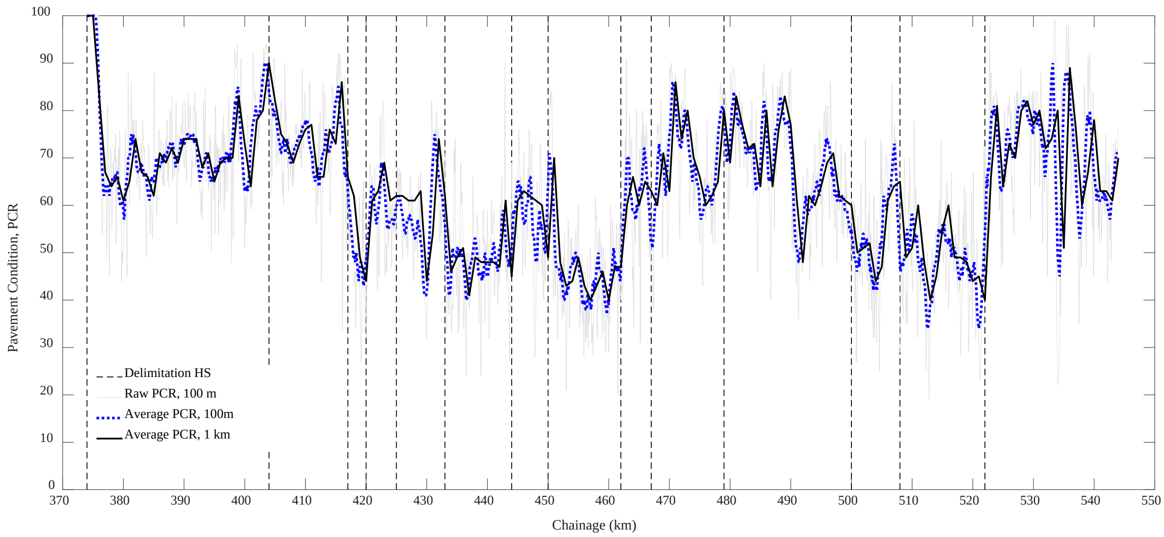
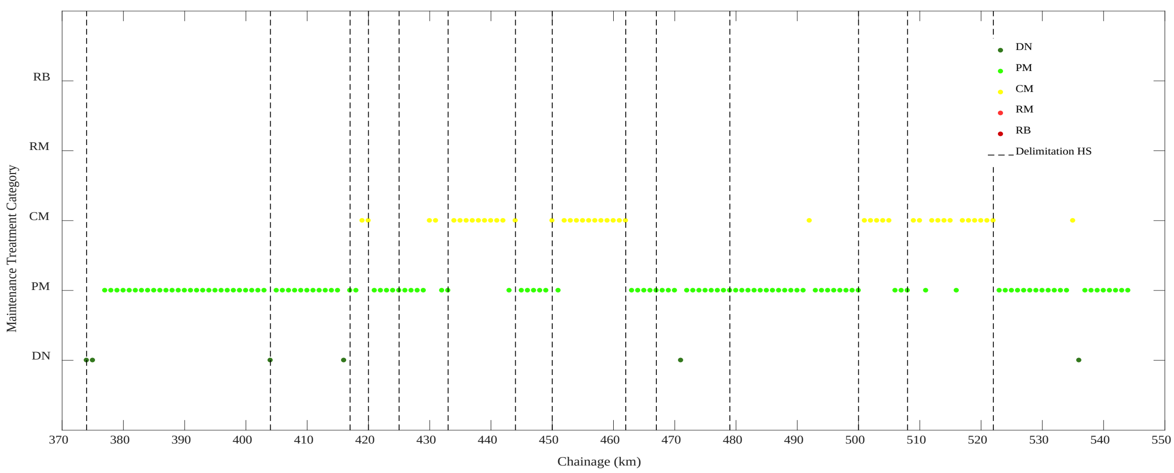


Figure 6-2. Accumulated Difference Centered Deflection along road chainage.

In the same way, PCR values classified by maintenance categories, can be used to delimitate homogeneous sections. Due to the high variability of the PCR values in consecutive 100 m length segments, a smoothing process was applied to the PCR signal, Figure 6-3 (a). Processed PCR values compared with PCR rating scale, define the most proper maintenance activities for each analyzed section, Figure 6-3 (b). The final sectioning of the length of the road are obtained with the combination of the delimitation of both methodologies.



(a) PCR indicator smoothed



(b) PCR maintenance categories

Figure 6-3. Pavement Condition Rating obtained from Survey along road chainage.

6.4.2 DEFINITION OF THE DECISION-MAKING PROCESS

The application of the decision-making process has been made based on the use of *PCR* values to trigger the suggested maintenance category. Table 6-3 summarizes the pavement maintenance activities considered by the extent of this study, which is aligned with the Kazakhstan road agency maintenance standards implemented.

Specifically, deflection measurements are used to define *d* and the remaining service life (*RL*) that reflects the homogeneous sections maintenance needs regarding the structural capacity (see Table 6-4).

In fact, the use of PCR rating as the only decision treatment, may lead to a suboptimal decision. For example, PCR rating suggests PM maintenance treatments in almost all the length of the road but, looking to the structural indicators (RL and d) of the pavement, some sections have severe structural deficiencies that require CM, RM maintenance treatments to guarantee that the pavement lasts over the design period. The thickness of the maintenance treatment are designed using the parameter d. In addition, RL parameter provides useful information to road manager about the current pavement structural condition.

Table 6-3. Pavement Maintenance Activities under Different Categories

Cat.	PCR Rating Scale Values	*d* (cm) Rating Scale Values	Pavement Treatment Category	Pavement Condition Indicators	Pavement Maintenance Category Treatment Description
RB	<26	>20	Rehabilitation (RB)	Deep-seated failure of the pavement showed by extensive cracking, shoving and rutting of the surface.	Reconstruction. Mill, Break, compaction of the subgrade, and Seat and AC Overlay
RM	26-40	13-20	Restorative Maintenance (RM)	Failure of the pavement top layers - not deep-seated but may include some local surface failures and a few isolated deep-seated failures.	Full depth recycling of 20 cm with the addition of new material (50% of crushed stone, 7% of bitumen); with AC overlay
CM	40-60	8-13	Corrective Maintenance (CM)	Pavement is sound but needs to be re-profiled and strengthening of the surfacing layers. The surface may be cracked and crazed in limited locations.	Mill and AC Overlay (5 cm)
PM	60-85	4-7	Preventive Maintenance (PM)	The shape of pavement is OK but may be suffering from minor cracking and crazing on the surface and with occasional patch repairs.	Preventive Maintenance Surface Treatment (Chip Seal, Slurry Seal, Micro-surfacing)
DN	85-100	<=3	Do Nothing (DN)	Pavement surface is in excellent conditions and only minor distresses are detected	Do Nothing

Table 6-4. Maintenance Decisions for the homogeneous sections along road chainage

Chainage HS	Thickness (cm)	PCR	SN_{eff}	SN_{req}	d(cm)	RL (years)	PCR Trigger	Structural Trigger	Final Decision
374-404	24.00	73	2.81	3.30	5	19	PM	PM	PM
404-417	23.00	73	2.90	3.20	3	16	PM	PM	PM
417-420	17.00	54	2.51	2.79	3	13	CM	DN	CM
420-425	17.00	63	2.36	3.01	6	5	PM	PM	PM
425-433	20.00	58	2.30	3.20	8	3	CM	CM	CM
433-444	20.00	49	2.29	3.22	8	3	CM	CM	CM
444-450	20.00	61	2.29	3.36	10	2	PM	CM	CM
450-462	20.00	48	2.11	3.09	9	3	CM	CM	CM
462-467	20.00	62	2.18	3.20	9	4	PM	CM	CM
467-479	20.00	70	2.16	4.45	20	1	PM	RM	RM
479-500	32.00	68	3.98	3.98	3	30	PM	DN	PM
500-508	15.00	54	2.17	4.18	18	3	CM	RM	RM
508-522	13.00	50	1.92	3.20	11	1	CM	CM	CM
522-544	13.13	69	2.05	2.80	7	5	PM	PM	PM

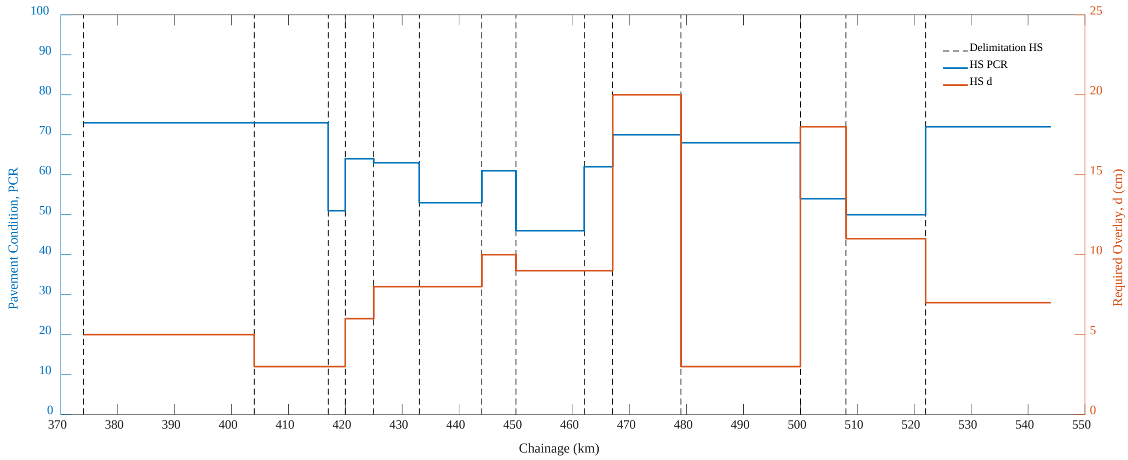


Figure 6-4. Pavement Condition Rating and required overlay for Homogeneous Sections along Road Chainage.

Figure 6-4 summarizes the averaged PCR and the parameter (*d*) for homogenous pavement sections along the road. It also includes the suggested maintenance treatment category., where the final decision is obtained from the most conservative maintenance treatment. As it can be observed, the use of structural indicators is useful in deciding which pavement sections requires full depth maintenance (RM) or the rehabilitation (RB) treatments. Besides, it should be noted that criteria to define homogeneous sections according to central deflection (D_0) is useful in determining the areas in terms of structural capacity.

6.5 CONCLUSIONS

This Chapter presents a methodology for the implementation of a network level pavement management program in Kazakhstan. This research was tested for priority roads in Kostanay (one region in Kazakhstan). Based on the analysis of the results achieved, some remarks can be highlighted as follows:

- Pavement survey at the network level involves high quantity of data. The use of the data towards cost-effective PMS strategies, needs specific procedures and rules to integrate them into a structured database. LRS and GPS merging data procedures are essential for the further analyses.
- The use of the PCR, to give an overall score of pavement functional condition, would be valid for deciding pavement needs at the network level, and it can be used as a first screening tool for identifying the most suitable pavement maintenance treatment in pavement sections. However, deflection measurements at the network level allow to enhance pavement decision-making because they can help to discriminate, among the pavement sections resulting in good conditions according to *PCR*, those in need of structural maintenance treatments.
- The definition of rules to design maintenance and rehabilitation strategies for the homogeneous sections would support the decision-making process at the network level. The proposed approach through pavement condition indicators provides information to predict the maintenance and rehabilitation strategies of road segments. The indicators and they can estimate the expected benefits of maintenance treatments to be performed on the network.
- The proposed approach leads to the implementation of a PMS for the national network of Kazakhstan, but it is still a young experience that would grow up with next pavement inspections on the network. At the moment, it gives a glance at the current state of the network that needs to be calibrated to the local conditions of the country (construction practices, procurement contracting).

Even though the methodology developed in this study is based on the data available from the surveys performed in Kostanay for a single year, the availability of additional data may allow forecasting future pavement conditions. It is anyway possible to extend this method to the other regions of Kazakhstan and, further that, it is possible to perform an economic evaluation involving stakeholders at the strategic level. Nevertheless, the proposed methodology can be used for survey data analysis in other countries and networks, since it is based on pavement data collected using survey methodologies and pavement condition indicators widely recognized and accepted worldwide, such as the PCI, IRI.

CHAPTER 7. DEVELOPMENT OF PAVEMENT DETERIORATION CURVES OF FLEXIBLE PAVEMENTS USING THE LINEAR EMPIRICAL BAYESIAN APPROACH AT THE NETWORK LEVEL

7. DEVELOPMENT OF PAVEMENT DETERIORATION CURVES OF FLEXIBLE PAVEMENTS USING THE LINEAR EMPIRICAL BAYESIAN APPROACH AT THE NETWORK LEVEL[4]

7.1 INTRODUCTION

Modeling the pavement deterioration process is essential for a successful Pavement Management System (PMS). The pavement deterioration process is highly influenced by the uncertainties related to the data acquisition and condition assessment. Road agencies collect a large amount of pavement condition data through inspections performed along their network. Because of uncertainties in the data collection methods interpretation of distresses, and the inherent variability of individual sections, the pavement recorded condition can have a very high variance. Therefore, deterioration models can provide a good representation of the overall condition of the network, but are bad at representing the performance of individual pavement sections.

This research was focused on developing a novel modeling approach for predicting the pavement Critical Condition Index (CCI) used by Virginia Department of Transportation (VDOT). The model builds on the Negative Binomial (NB) regression that has been used to model pavement deterioration as a function of the pavement *age* (Katicha, Flintsch et al. 2016). Network-level pavement condition models were developed for *interstate*, *primary* and *secondary* pavement road families using Virginia DOT PMS (see § 5.1) and were compared with traditional non-linear regression models.

The current network-level pavement performance models used by VDOT give an adequate network-level prediction of the average pavement condition for each pavement family as a function of *age*, but do not consider the deviation of specific pavement sections from the average family performance. Thus, the pavement deterioration process is predicted using two types of models: specific-site project-level models based on the construction and maintenance historical records, and default network-level models when the data of specific sections is not available. The default models are nonlinear regression models for interstate and primary roads

[4] Partial results of the presented work have been submitted at ASCE Journal of Transportation Engineering: Part B as "Pantuso, A., Flintsch, G.W., Katicha, S.W. & Loprencipe, G. (2018). Development of Network Level Pavement Deterioration Curves using the Linear Empirical Bayes Approach."

developed by Stantec Inc. (2007) to estimate the *CCI* as a function of the pavement *age*, where the pavement *age* is defined as the number of years since construction or last significant maintenance treatment.

A Linear Empirical Bayesian (LEB) approach is proposed to combine the pavement condition estimated by network-level pavement deterioration models for *families of pavements,* with the most recent observed/measured condition recorded from the inspection of the network. The LEB estimator is used to effectively account for the variance of observed condition data in the deterioration model, for which the estimate of the measurement error is the same than the observed measurement error.

This approach results in a better estimate of the pavement condition compared to the average family deterioration model or the measured condition. Compared to the measured condition, the LEB approach is estimated to reduce the mean square error of predicting the future condition (next year's condition) by as much as 33 %, 36%, and 41 % for the interstate, primary, and secondary network families, respectively. A more accurate estimate of the future condition addresses the critical need of predicting the future pavement condition to support network-level decision making.

7.2 OVERVIEW DATA ANALYSIS AND METHODOLOGY USED

This study used pavement data available in the VDOT PMS for flexible pavement sections interstate, primary and secondary networks in three maintenance districts (Richmond, Hampton Roads and Northern Virginia) (see § 5.1). The data included pavement condition data and repair and maintenance records from 2007 to 2016. The pavement condition rating table (pavement condition in terms of CCI) and the structure data table (including the year and type of last maintenance treatment performed on each 0.1 mile-section) were merged using geographical location. The location is based on VDOT's Linear Referencing System (LRS), which uses milepost position along the routes (see § 2.6.1 and §5.2.1). The matching procedure took into account that the beginning and ending point of the condition data and repair and maintenance treatments and even, the inspection sections may not always match. The matching produced a list of pair of values (*age*, *CCI*) for a total amount of 61,501 pavement sections (0.1-mile road segments) and 343,589 observations of different pavement *age*, number of years between *CCI* inspection year and year of last significant treatment record, which reset pavement *age* (each pavement section had between 5 and 6 years of data).

7.2.1 ANALYSIS OF CCI DATA: DATA CLEANSING AND FILTERING PROCESS CONSIDERATIONS

The use of suitable filtering and cleansing procedures for the pre-processing of data analysis is a key aspect to develop a pavement deterioration model that can accurately explain the observed data. The following considerations have been found relevant in this study:

1. **Pavement condition data at old ages is often inherently biased**: one of the main objectives of highway agencies is to provide road users with a high level of service, thus, when the *CCI* falls below a minimum value, the section typically receives a corrective maintenance (CM), restorative maintenance (RM), or reconstruction (RC) treatment, which reset its life. Thus, there are not many sections with low *CCI* condition. This can lead to a biased deterioration model, because condition data for old pavements may not represent the average pavement condition of all typical sections, but rather the behavior of the best performing pavements in the family. To account for this, the model should be fitted to a range of pavement ages for which this effect is not too pronounced.
2. **Some treatments are not recorded**: Analysis of the *CCI* data revealed that there are many instances of sections with significant differences in condition between two consecutive years. More importantly, there are instances where the *CCI* difference between two consecutive years is highly positive, which is unusual for the process of deterioration of pavements. Some of these are thought to be due to maintenance treatment that are not recorded in the database.

7.2.2 DATA CLEANSING AND FILTERING PROCESS

Figure 7-1, Figure 7-2, and Figure 7-3 present boxplots of pavement condition *CCI* as a function of pavement *age* for the three systems studied. Pavement *CCI* data grouped by pavement *age* is depicted using a box plot where the median value of *CCI* condition is represented by a solid line, and the box outlines represents the data between the 25^{th} and 75^{th} percentiles of *CCI*. The figures show that the distributions of pavement condition have a high variability within each pavement family. This is due to the grouping of various pavement sections in the family, in addition to the inherent pavement data condition variability (related with the collection techniques, use of a composite index, localization of the distress within the sections). The general trends are reasonable as the average *CCI* decrease as the *age* increase, but the average *CCI* reaches an almost constant level for the *age* of 14, 11 and 8 for the Interstate, Primary and Secondary pavement families, respectively. After these ages, the average *CCI* for each family is inconsistent with the engineering expectation because there is a positive trend of *CCI* average for some years. This could be due to missing treatments in the database and/or the bias previously discussed because bad pavements are treated at younger ages. Considering old sections with very good condition for the development of the pavement model can lead to a biased model that would not be representative of the average performance of all pavement sections. For this reason, the regression models were fitted to sections with ages less than 14, 11 and 8 years for the interstate, primary and secondary network pavement families, respectively.

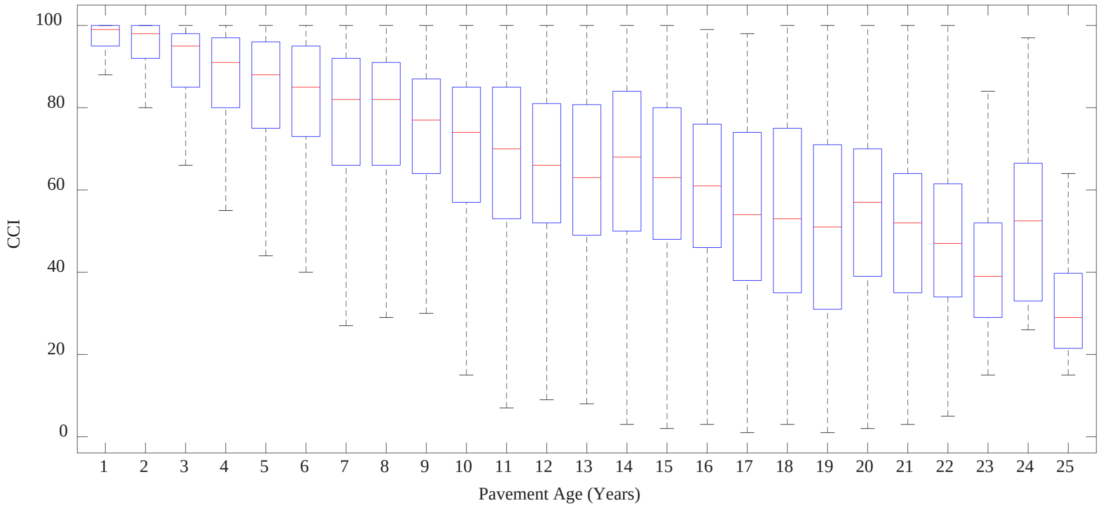


Figure 7-1. Pavement CCI as function of pavement age of Interstate Network

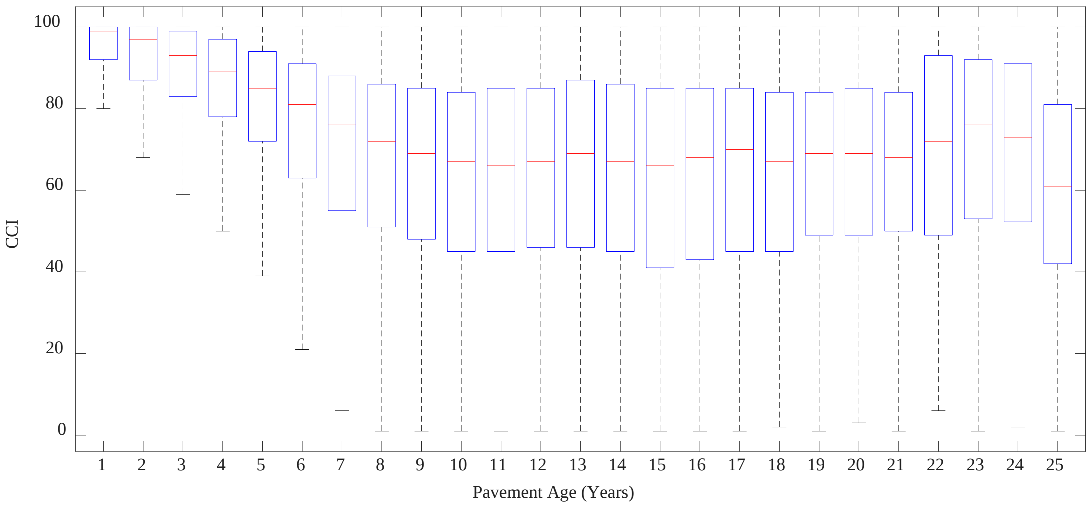


Figure 7-2. Pavement CCI as function of pavement age of Primary Network

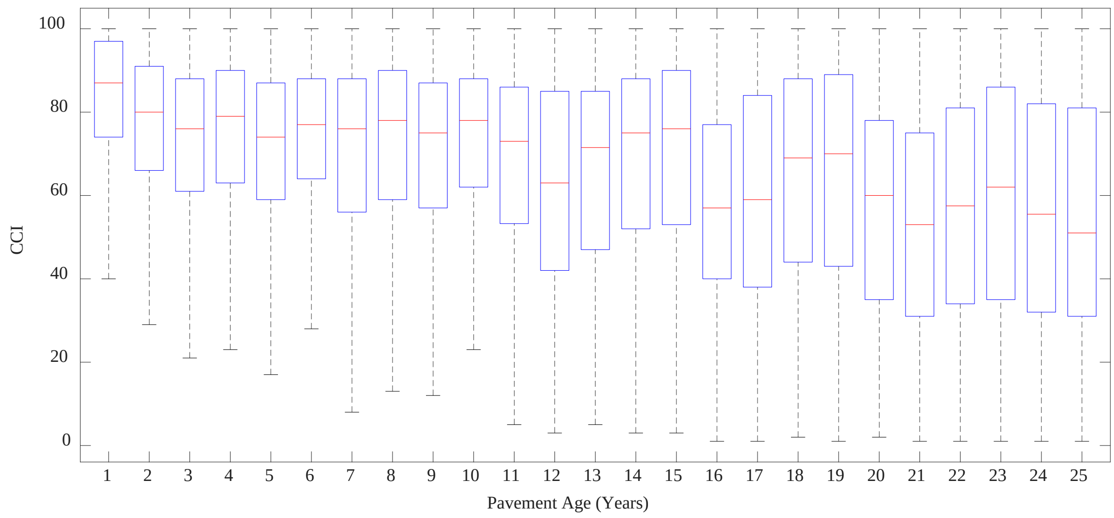


Figure 7-3. Pavement CCI as function of pavement age of Secondary Network

A detailed study of the differences of pavement condition between two consecutive years showed that for interstate and secondary roads, a difference of +/- 10 points between two consecutive years of *CCI* can be considered outliers according to the average behavior of the pavement, and therefore they have been excluded from the set used to fit the models (the model was fitted for 29,971 observations in 3,601 sections, 91 % of the data). However, the same filtering criteria could not be applied for primary network because it would eliminate too many segments. Thus, only the sections with *age* greater than 11 years were filtered out (leaving 170,810 observations in 22,316 sections, or 74 % of the data).

For the secondary network, there seems to be even more uncertainty about repair and maintenance treatments recorded in the database. This can significantly influence the general behavior of pavement models because introduce bias in the model in this shows in Figure 7-3. The average of pavement condition for the roads of secondary network after the *age* of 4 years is approximately constant and increases in several ages. Specifically, 16,773 measurements (21 % of the dataset) have *CCI* values greater than 80 points for pavement ages greater than 4 years. Since this good performance of pavement sections is unusual, this set of measurement have been excluded from the dataset used to fit the model for secondary roads' pavement family (the model was fitted for 38,354 observations in 26,167 sections, or 48 % of the data). Table 7-1 summarizes the sample sizes considered to fit the deterioration models for each family.

Table 7-1. Sample Size of Pavement Families included in the study

Pavement family model	No. Sections	No. Observations	No. Sections model	No. Observations model
Interstate Network	3,601	32,685	3,601	29,971
Primary Network	22,316	231,197	18,272	170,810

Secondary Network	35,584	79,707	26,167	38,354
Total	61,501	343,589	48,040	239,135

7.2.3 The Empirical distribution of Pavement Condition: Negative Binomial (Poisson-Gamma model)

The measured pavement deterioration (*DI*) can be defined according to Eq. 7-1, as the complementary to the measured *CCI*.

$$DI = 100 - CCI, \qquad \text{Eq. 7-1}$$

The distribution of *DI* for each pavement family were fitted with different theoretical statistical distributions, a Normal distribution, a Poisson distribution and a Negative Binomial distribution. Figure 7-4, Figure 7-5, and Figure 7-6 illustrate the distribution fittings and the empirical distribution for the interstate, primary and secondary system, respectively.

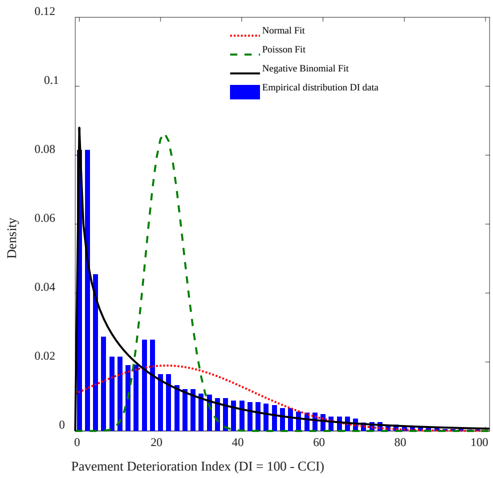


Figure 7-4. Distribution of Pavement Deterioration (DI) of the Interstate Network

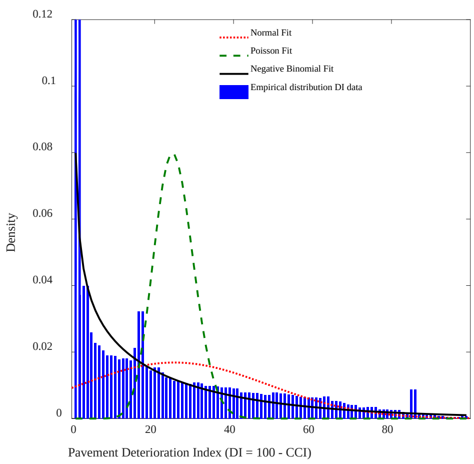


Figure 7-5. Distribution of Pavement Deterioration (DI) of the Primary Network

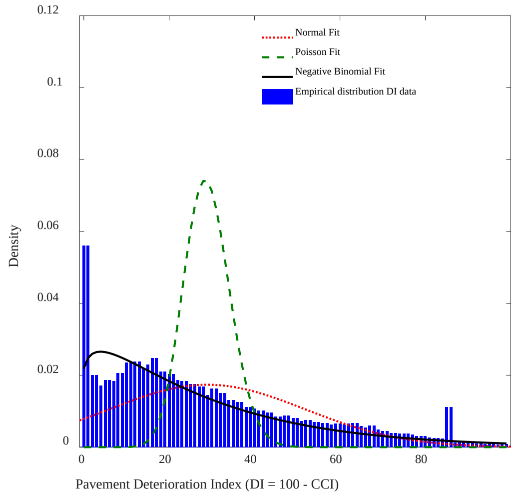


Figure 7-6. Distribution of Pavement Deterioration (DI) of the Secondary Network

This analysis showed that the Poisson distribution (with equal mean and variance by its definition) provides a poor representation of pavement data; however, the Poisson distribution combined with the Gamma distribution which arises to the Negative Binomial distribution provides a good representation of the pavement data of each pavement family outperforming the Normal Distribution. The NB distribution (Poisson-Gamma model) is a discrete distribution that is more flexible than the Poisson distribution (for which the mean is equal to the variance), because it allows the variance to be greater than the mean. Statistical analysis of the empirical distribution of the pavement deterioration data showed that the NB distribution provides a better representation of observed data if compared with a normal distribution (Ercisli 2015). The empirical distribution of the pavement condition over time have been studied for each pavement family. Figure 7-7, Figure 7-8, and Figure 7-9 illustrate the observed pavement condition (*CCI*) with descriptive histograms and the empirical distribution cummulative distribution function (CDF) and NB distribution fitting for each pavement family.

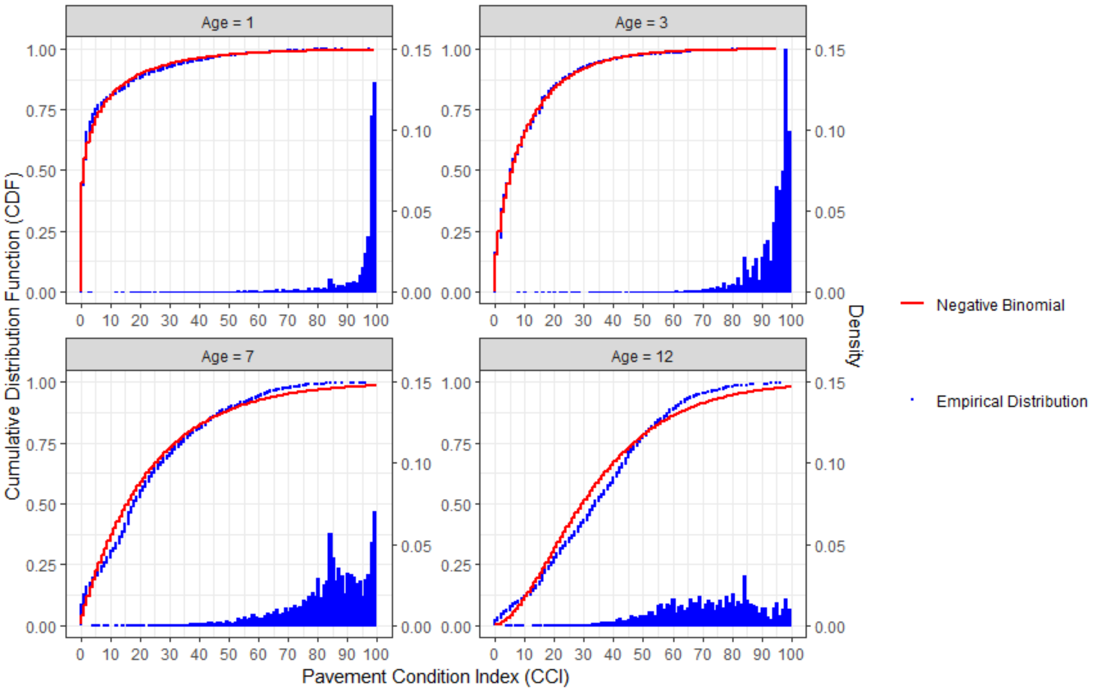


Figure 7-7. Empirical Distribution of pavement *CCI* and Negative Binomial Distribution fit along Pavement *age* for the Interstate System

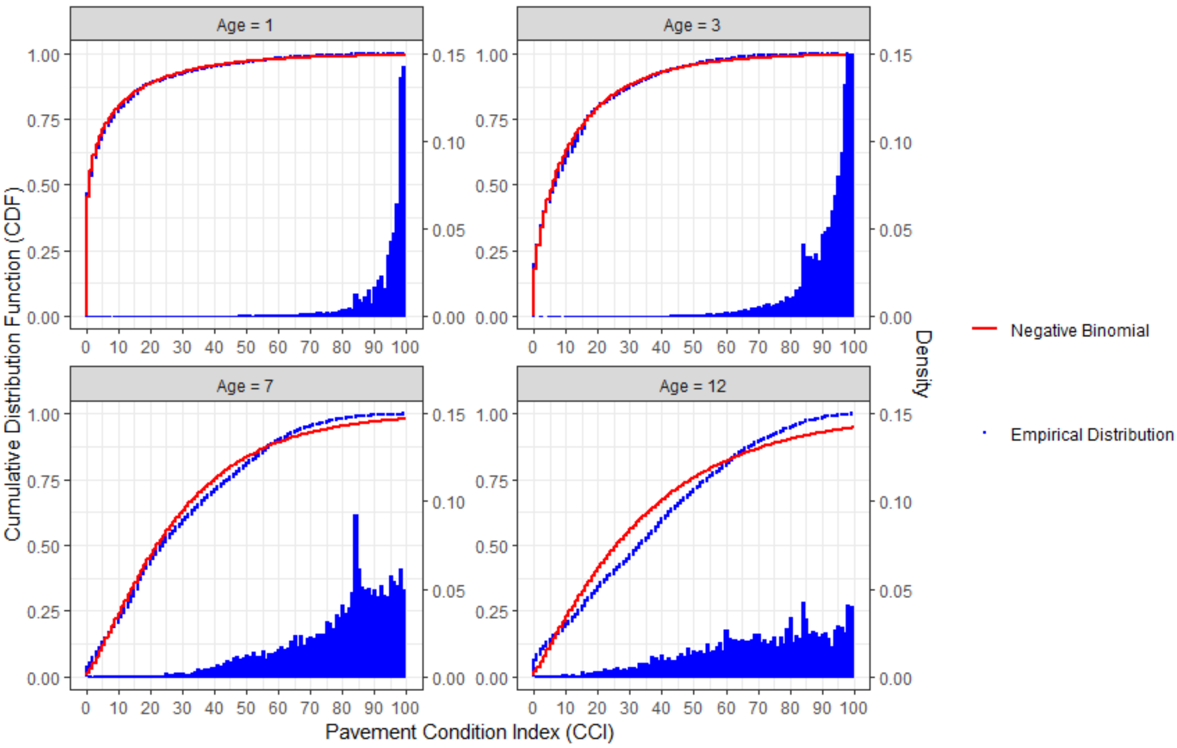


Figure 7-8. Empirical Distribution of pavement *CCI* and Negative Binomial Distribution fit along Pavement *age* for the Primary System

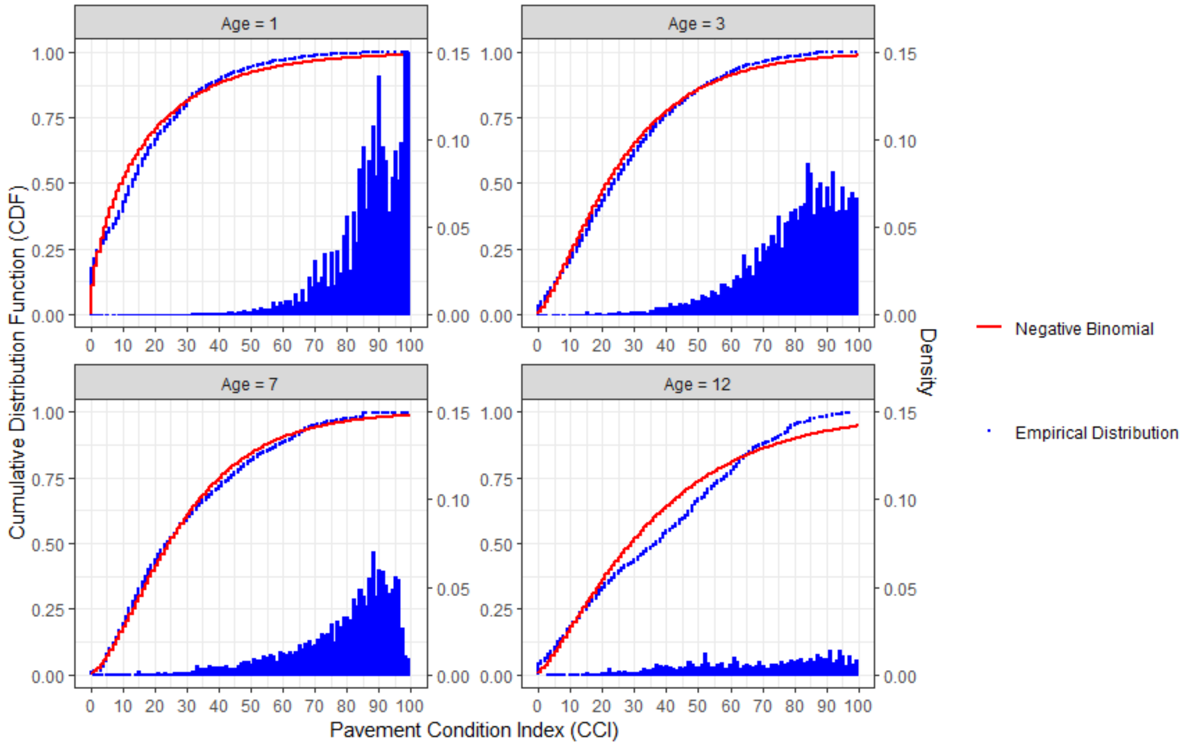


Figure 7-9. Empirical Distribution of pavement *CCI* and Negative Binomial Distribution fit along Pavement *age* for the Secondary System

Table 7-2 summarized with descriptive statistics of *CCI* for different pavement ages (*age*=1, *age* =3, *age*=7, and *age* =12), and the NB distribution fitting parameters have been provided. The statistic χ^2 -test was used for testing the goodness of fit for the NB distribution of the empirical distribution of *DI* (complementary of *CCI*).

Table 7-2. Descriptive Statistics and χ^2 – test of Negative Binomial Distribution Fitting

Family		NB size	NB μ	χ2 test	DF	p-value
Interstate	*age*=1	0.234	6.862	395.561	82	1.37E-42
	age=3	0.661	10.454	396.362	76	7.46E-45
	age=7	1.014	22.903	526.278	88	1.21E-63
	age=12	2.231	34.809	526.278	88	1.21E-63
Primary	*age*=1	0.234	6.862	395.561	82	1.37E-42
	age=3	0.661	10.454	396.362	76	7.46E-45
	age=7	1.014	22.903	526.278	88	1.21E-63
	age=12	2.231	34.809	373.335	89	1.72E-36
Secondary	*age*=1	0.234	6.862	395.561	82	1.37E-42
	age=3	0.661	10.454	396.362	76	7.46E-45
	age=7	1.014	22.903	526.278	88	1.21E-63
	age=12	2.231	34.809	373.335	89	1.72E-36

7.2.4 NEGATIVE BINOMIAL MODEL DEVELOPMENT AND VIRGINIA DOT PAVEMENT DETERIORATION MODELS

Because the NB model provides a good representation of the data, NB regression, which is a type of Generalized Linear Model (GLM), was used to determine the deterioration model for each type of pavement family.

The deterioration model relates the pavement deterioration as a function of pavement *age* as shown in Eq. 7-2. The NB regression is performed to determine the parameters β_0, β_1 and β_2 and an overdispersion parameter ϕ, which is related to the model variance as shown in Eq. 7-3.

$$DI_{NB\_model} = \exp(\beta_0 + \beta_1 \cdot age + \beta_2 \cdot \log(age)) = age^{\beta_2} \cdot \exp(\beta_0 + \beta_1 \cdot age), \quad \text{Eq. 7-2}$$

$$Var(DI) = DI_{NB\_model}(1 + \phi\, DI_{NB\_model}) \quad \text{Eq. 7-3}$$

Current VDOT default network-level pavement deterioration models are used whenever there are no section-specific data available to predict the pavement condition in terms of *CCI* for a given pavement *age*. Thus, as a baseline, these three VDOT default models - Power model Eq. 7-4, Sigmoidal model Eq. 7-5, VDOT exponential model Eq. 7-6, developed by Stantec Inc. (2007) - were fitted to the filtered and cleaned data for the interstate, primary and secondary networks:

$$CCI_{P\_model} = a \cdot age^{b} + c \quad \text{Eq. 7-4}$$

$$CCI_{S\_model} = 100 - d \cdot \exp\left(\frac{-e}{age}\right) \quad \text{Eq. 7-5}$$

$$CCI_{VDOT\_model} = 100 - \exp\left(f + g \cdot h^{\left(\frac{1}{age}\right)}\right) \quad \text{Eq. 7-6}$$

where the regression parameters (a, b, c, d, e, f, g, and h) are obtained using Ordinary Least-Squares fitting. The VDOT default deterioration models fitted for the pavement condition data from PMS were compared with the proposed NB deterioration model.

7.2.5 EMPIRICAL BAYESIAN APPROACH

The NB distribution arises as a compound mixture of Poisson distributions where the mixing distribution of the Poisson rate is the Gamma distribution (Poisson – Gamma model) and can be expressed parametrically in Eq. 7-7 (Hilbe 2011).

$$f_{\mathrm{NB}}(x;r,p)=\int_0^\infty f_{\mathrm{P}}(x;\lambda)\cdot f_{\mathrm{G}}(\lambda;r,p)\cdot d\lambda=\int_0^\infty\left(\frac{\lambda^x}{x!}\cdot\exp(-\lambda)\right)\cdot\left(\frac{\lambda^{r-1}}{\left(\frac{p}{1-p}\right)^r\cdot\Gamma(r)}\cdot\exp\left(-\lambda\cdot\frac{(1-p)}{p}\right)\right)\cdot d\lambda \qquad \text{Eq. 7-7}$$

The Poisson – Gamma model gives rises to a Bayesian model with the Gamma distribution prior. (conjugate prior of the Poisson distribution). The Gamma distribution prior is parametrized by two parameters which are related to the mean and overdispersion, according to Eq. 7-8, Eq. 7-9, and Eq. 7-10.

$$DI_{NB\_Model}=\frac{p\cdot r}{1-p} \qquad \text{Eq. 7-8}$$

$$Var(DI)=\frac{p\cdot r}{(1-p)^2} \qquad \text{Eq. 7-9}$$

$$\phi=\frac{Var(DI)-DI_{NB\_Model}}{DI_{NB\_Model}}=\frac{\frac{p\cdot r}{(1-p)^2}-\frac{p\cdot r}{1-p}}{\frac{p^2\cdot r^2}{(1-p)^2}}=\frac{\frac{p\cdot r-p\cdot r\cdot(1-p)}{(1-p)^2}}{\frac{p^2\cdot r^2}{(1-p)^2}}=\frac{1}{r} \qquad \text{Eq. 7-10}$$

In the empirical Bayes approach, these are DI_{NB_model} and ϕ, determined from the data using the NB regression. Once the Gamma distribution parameters are evaluated, the posterior distribution is obtained as expressed in Eq. 7-11.

$$f_{\mathrm{Gposterior}}\left(\lambda;\frac{1}{\phi}+DI,\frac{\phi\cdot DI_{NB\_Model}}{\phi\cdot DI_{NB\_Model}+1}\right)=\frac{\lambda^{\frac{1}{\phi}+DI-1}}{\left(\frac{1}{\phi\cdot DI_{NB\_Model}}\right)^{\frac{1}{\phi}+DI}\cdot\Gamma\left(\frac{1}{\phi}+DI\right)}\cdot\exp\left(-\lambda\cdot\left(\frac{\phi\cdot DI_{NB\_Model}+1}{\phi\cdot DI_{NB\_Model}}\right)\right) \qquad \text{Eq. 7-11}$$

Consequently, a point estimate (the posterior mean) of the pavement deterioration, DI_{EB},can be calculated as a weighted average of the pavement condition predicted from the prior distribution ($DI_{\mathrm{NB\_Model}}$) and the observed mean pavement deterioration (*DI*), obtained as follows in Eq. 7-12:

$$DI_{EB}=\left(\frac{1}{\phi}+DI\right)\cdot\left(\frac{\phi\cdot DI_{NB\_Model}}{\phi\cdot DI_{NB\_Model}+1}\right)=\frac{DI_{NB\_Model}+\phi\cdot DI_{NB\_Model}\cdot DI}{\phi\cdot DI_{NB\_Model}+1} \qquad \text{Eq. 7-12}$$

The posterior pavement deterioration estimated (DI_{EB}) is calculation rewriting Eq. 7-12 in Eq. 7-13, as follows:

$$DI_{EB}=\frac{1}{\phi\cdot DI_{NB\_model}+1}\cdot DI_{NB\_model}+\left(1-\frac{1}{\phi\cdot DI_{NB\_model}+1}\right)\cdot DI \qquad \text{Eq. 7-13}$$

In practical terms, Bayes' formula combines the average behavior of all pavement sections (expressed by the prior, DI_{NB_model}) with recent observations of specific sections (observed *DI*) to obtain a better estimate of the pavement condition DI_{EB} (complementary of CCI_{EB} which is the posterior).

For example, from the latest pavement inspections, 10 pavement observations of different pavement ages (data retrieved from PMS after applying the matching procedure explained before) were picked up randomly, for each of these observations the EB was applied to get a better estimate of the pavement condition for that specific sections (see Figure 7-10).. Questionable observed data unusual for the process of deterioration of pavements (highly deteriorated pavements at early ages or lowly deteriorated pavement at late ages) were corrected to obtain a better estimate of the deterioration prediction, closer to the mean performance of pavement sections (represented by the proposed deterioration model).

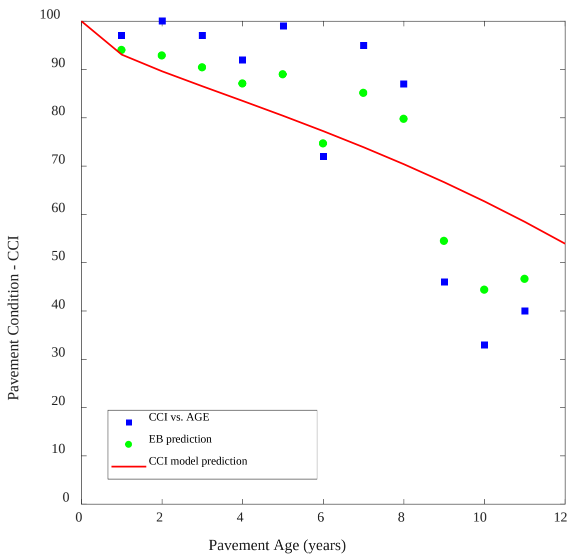


Figure 7-10. Example of application of the Empirical Bayesian Approach

According to Efron and Morris (1973), the improvement of the LEB approach is such that the mean square error is reduced by the factor shown in Eq. 7-14:

$$\frac{\sigma_s{}^2}{\mathrm{Var}(DI)} = \frac{\sigma_s^2}{\sigma_s{}^2 + \sigma_{\mathrm{error}}^2} \qquad \text{Eq. 7-14}$$

where the σ^2_s is the variance of the data and the Var(*DI*), is the total variance of the observed data.

According to the assumption of the Poisson-Gamma model, Var(*DI*) can be decomposed by the variance of the pavement sections' performance, σ^2_s estimated by the Gamma distribution, and the variance of the error estimated from the pavement data, estimated by the Poisson distribution, $\sigma^2_{Poisson}$. However, the variance of the estimated error σ^2_{error} in the measured pavement deterioration is higher than what is predicted by the Poisson distribution (see Figure 7-11), thus, the error of the measured/observed deterioration is underestimated, leading to a suboptimal use of EB estimator.

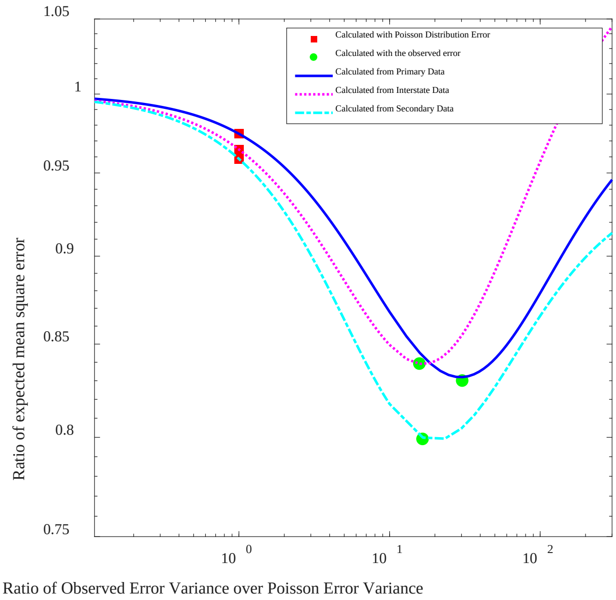


Figure 7-11. Error in estimating the Pavement Condition with the EB Method as a Function of Estimated Error Variance

Figure 7-11 illustrates the error in estimating the pavement condition with the EB method as a function of estimated error variance. When the measurement error is assumed practically zero, EB estimate is equal to the observed pavement condition and the error is equal to 1. As the estimate of the measurement error gets closer to the observed measurement error variance, the mean square error of estimating the pavement condition using the EB method decreases and, on the other hand, when the estimate of the measurement error variance is greater of the observed measurement error of the variance, the error of EB starts to increase. The best estimate is obtained when the measurement error variance is correctly estimated.

Therefore, an adjustment has been made in the model to minimize the mean square error between the estimated pavement deterioration and the true deterioration, $\sigma^2_{error} = \alpha \cdot \sigma^2_{Poisson}$ and the EB approach becomes a Linear Empirical Bayes (LEB). The adjustment can be done by modifying Eq. 7-13 as shown in Eq. 7-15, where $\phi_c = \phi/\alpha$.

$$DI_{EB} = \left(\frac{1}{\phi_c \cdot DI_{NB\_model} + 1}\right) \cdot DI_{NB\_model} + \left(1 - \frac{1}{\phi_c \cdot DI_{NB\_model} + 1}\right) \cdot DI, \quad \text{Eq. 7-15}$$

Alternatively, by substituting Eq. 7-10 and Eq. 7-14 in Eq. 7-13, the Linear EB estimator is calculated using Eq. 7-16, which can be used for any distribution and without the knowledge of the appropriate form.

$$DI_{EB} = \left(1 - \frac{\sigma_s^2}{\sigma_s^2 + \sigma_{error}^2}\right) \cdot DI_{model} + \left(\frac{\sigma_s^2}{\sigma_s^2 + \sigma_{error}^2}\right) \cdot DI, \quad \text{Eq. 7-16}$$

where $\sigma^2_{error}=\alpha\ \sigma^2_{Poisson}$;$\sigma^2_{Poisson,}$ is the variance predicted by the Poisson distribution (the mean value of the data that fit the model), and σ^2_{error} is the variance estimated from the data using the difference sequence method Katicha *et al.* (2016a,2016b).

7.3 DETERIORATION MODEL DEVELOPMENT

For each pavement family, the parameters of the NB pavement deterioration model Eq. 7-3 were estimated using the NB Regression on the filtered data measurements. In addition, the filtered data were fitted with the VDOT default deterioration models). Table 7-3 summarized the estimate parameters obtained using the various models and the coefficient of determination.

Table 7-3. Goodness of fit pavement deterioration models for the defined pavement families

Pavement family model	Estimated Parameters	R^2
Interstate Network		
Power model;	a=-5.150; b=0.756; c=100	0.292
Sigmoidal model;	d=56.030; e=5.956	0.275
VDOT model;	f=-16.070; g=17.710; h=0.961	0.291
Negative Binomial	β_0=1.658; β_1=0.018; β_2=0.654; ϕ=1.125; α = 15.86	0.289
Primary Network		
Power model;	a=-0.09; b=2.27; c=95.29	0.227
Sigmoidal model;	d=46.360; e=7.856	0.184
VDOT model;	f=1.384; g=0.115; h=0.313	0.226
Negative Binomial;	β_0=1.353; β_1=0.228; β_2=-0.250; ϕ=1.322; α = 30.50	0.227
Secondary Network		
Power model;	a=-21.9; b=0.412; c=105.000	0.190
Sigmoidal model;	d=46.230; e=1.132	0.184
VDOT model;	f=-16.040; g=18.880; h=0.975	0.190
Negative Binomial;	β_0=2.837; β_1=0.001; β_2=0.487; ϕ=0.709; α = 16.64	0.190

The four models are represented graphically for the Interstate family (Figure 7-12), Primary family (Figure 7-13), and Secondary family (Figure 7-14). In general, all the models adequately fit the average value of the pavement deterioration. However, due to the high variability of the data, the fitted models will not fit well individual pavement sections.

The parameter ϕ presented for the Negative binomial model in Table 7-3 is the over dispersion parameter as defined previously. The R^2 was calculated manually for the NB regression using the residual sum of squares (SSE) of the model and the total variance of the data used to fit the models.

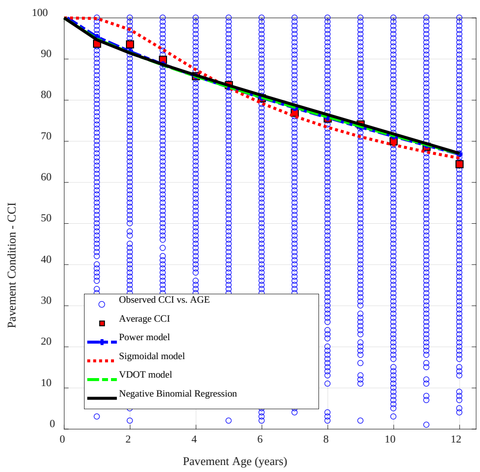


Figure 7-12. Comparison between *CCI* deterioration curves using Virginia DOT default models and Negative Binomial Regression model for the Interstate System

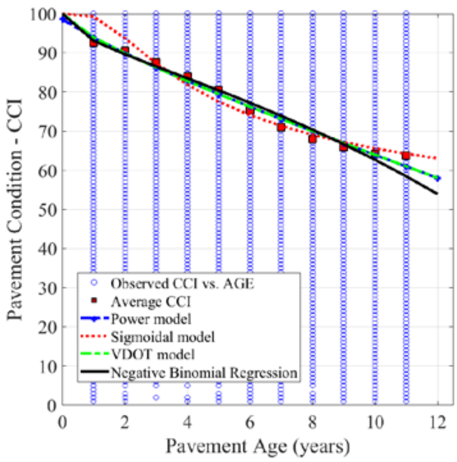


Figure 7-13. Comparison between *CCI* deterioration curves using Virginia DOT default models and Negative Binomial Regression model for the Primary System

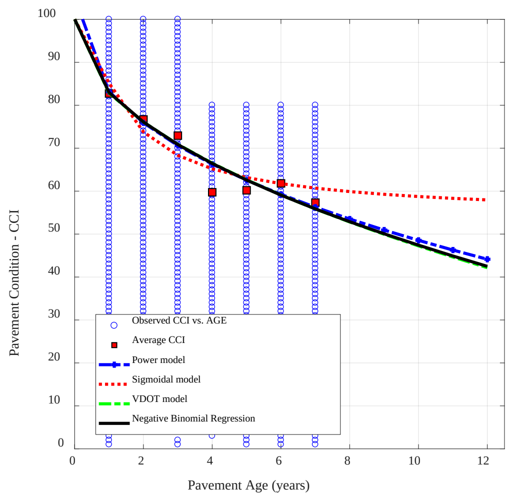


Figure 7-14. Comparison between *CCI* deterioration curves using Virginia DOT default models and Negative Binomial Regression model for the Secondary System

7.4 VALIDATION OF MODELLING PROCEDURE

As mentioned previously, the EB approach can provide a better estimation of pavement condition compared with the estimation of the *CCI* model prediction or the *CCI* value recorded in the database. The best way to validate the LEB approach would be to estimate the “true” value that is being calculated by the model and verify that the chosen procedure gives a better estimation of the true value (next year condition) compared with the obtained from estimation of pavement condition (actual condition) without use any modeling procedure. Therefore, a leave-one-out cross-validation procedure has been implemented and 5 different methods have been compared using the mean square prediction error (MSE) as the evaluation criterion. The procedure followed including the following steps:

1. Process the data to get the measurements (observations) *O* on each section *S* and determine the year of inspection of each measurement.
2. Remove from pavement section S_i all the measurement obtained after year Y_{i}.
3. Fit the model to the remaining data in the data set.
4. Evaluate the different methods to determine estimation of the remove measurement and calculate the mean square error.

Five different approaches were compared in terms of their ability to predict the pavement deterioration (illustrated in Figure 7-15):

1. **Method 1**. Predict the future pavement condition with the most recent observation on the section S_i in the PMS. This choice implies not modeling the pavement condition and assumes that the pavement does not deteriorate from last recorded inspection (Eq. 7-17):

$$CCI_{i+1}^{M1} = CCI_i \quad \text{Eq. 7-17}$$

2. **Method 2**. Predict the pavement condition using the fitted family model using the negative binomial model, Eq. 7-2:

$$CCI_{i+1}^{M2} = CCI_{\mathrm{NB\_Model\_i+1}} \quad \text{Eq. 7-18}$$

3. **Method 3**. Predict the pavement condition using the LEB approach using Eqs. Eq. 7-15 and Eq. 7-16:

$$CCI_{i+1}^{M3} = CCI_{\mathrm{EB}\_i} \quad \text{Eq. 7-19}$$

4. **Method 4**. Predict the condition from the most recent observation of section S_i and adding the deterioration of the section S_i obtained as follows Eq. 7-20:

$$CCI_{i+1}^{M4} = CCI_i + \left(CCI_{\mathrm{NB\_model}\_i+1} - CCI_{\mathrm{NB\_model}\_i}\right) \quad \text{Eq. 7-20}$$

5. **Method 5**. Predict the future condition using the Linear Empirical Bayes approach adding the deterioration of the section S_i obtained from the model Eq. 7-21:

$$CCI_{i+1}^{M5} = CCI_{\mathrm{EB}\_i} + \left(CCI_{\mathrm{NB\_model}\_i+1} - CCI_{\mathrm{NB\_model}\_i}\right) \quad \text{Eq. 7-21}$$

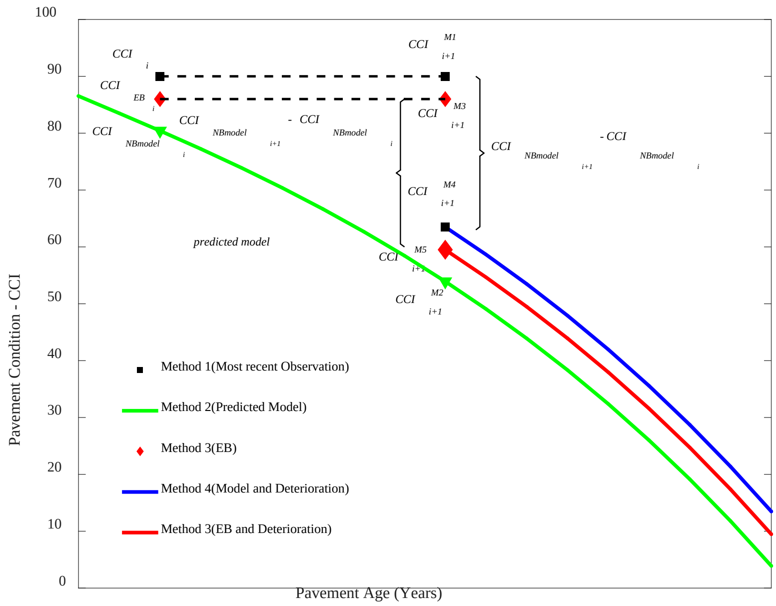


Figure 7-15. Illustration of the Approaches to Calculate the Estimate of Future Pavement Condition

Each pavement condition measurement recorded in the database is excluded from the model and the value is estimated by fitting the model for the remaining measurement in the database, and the obtained estimate of the "true" observation is compared with the observed measurement that it is recorded in the database.

Table 7-4. Mean Square Error of prediction of the Estimate of Pavement Condition using different approaches

VDOT System Family	MSE Prediction	MSE Ratio with respect to Method 1	Improvement prediction ratio with respect to Method 1
Interstate Network Family			
Method 1	292.17	1.000	-
Method 2	325.68	1.115	-12 %
Method 3	247.69	0.873	13 %
Method 4	228.60	0.782	22 %
Method 5	188.83	0.666	33 %
Primary Network Family			
Method 1	94.99	1.000	-
Method 2	113.69	1.197	-20 %
Method 3	87.89	0.925	8 %
Method 4	67.05	0.706	29 %
Method 5	60.91	0.641	36 %
Secondary Network Family			
Method 1	499.79	1.000	-
Method 2	474.17	0.949	5 %
Method 3	459.33	0.919	8 %
Method 4	461.32	0.923	8 %
Method 5	295.94	0.592	41 %

The results for the five different approaches (illustrated in Figure 7-15) are summarized in Table 7-4, including the mean square error prediction of each method. Method 1 represents the results of not considering any modeling technique, that is, consider that the future pavement condition is the same of most recent observation recorded in the database, and thus is used as the baseline to be compared the other methods. Method 2 represents the results of predicting pavement condition using fitting NB regression model, the cross-validation process showed that for the tested datasets (affected by high variance such as Interstate family and primary family) is ineffective to predict future pavement condition prediction error, -12 % for the Interstate family, and -20 % for the Primary family, ratio of prediction error compared with the Method 1. However, Method 2 is more effective for the Secondary family (+ 5 % ratio of prediction error compared with the Method 1) affected by less variance of specific pavement section deterioration prediction compared with the other pavement families.

The EB approach used in Method 5, which predicts the future pavement condition as the combination of the EB of the individual pavement section and the expected deterioration predicted by the model, has the best predicting ability. It gives the lowest prediction error with an improvement of 33% for the Interstate family, 36% for the Primary family, and 41 % for the Secondary family compared with not considering any pavement deterioration technique (Method 1).

Color maps were used to visually illustrate the effectiveness of the proposed modelling technique for the Interstate (Figure 7-16), Primary (Figure 7-17), and Secondary (Figure 7-18) pavement families. Pavement condition data in terms of VDOT *CCI* were represented as a function of pavement *age*. Density were estimated average density across the observed data points using color scale; darker colors represent a greater density of observations and, reversely, brighter colors represent lower density of observations. The same scale of colors is used in figures Figure 7-16, Figure 7-17, and Figure 7-18 a) and b). The visual inspection indicates that the application of the Linear EB estimator is effective because more reasonable deterioration trends were observed in the color map after applying the modeling technique: the LEB estimator corrects unreasonable pavement condition predictions for specific pavement sections (lowly deteriorated pavement sections at late ages and/or highly deteriorated sections at early ages) getting closer to the mean deterioration of pavement family, a significant reduction of the variance of pavement condition data is obtained.

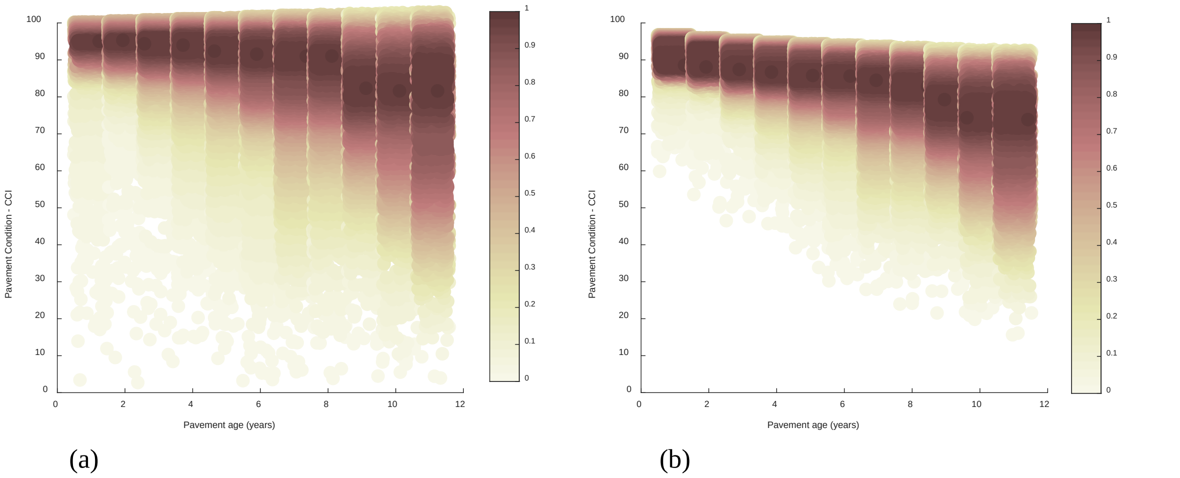


Figure 7-16. Color Map Plot of the Interstate Network Pavement Family; a) Raw observed pavement condition data; b) Pavement condition modeled using the LEB approach

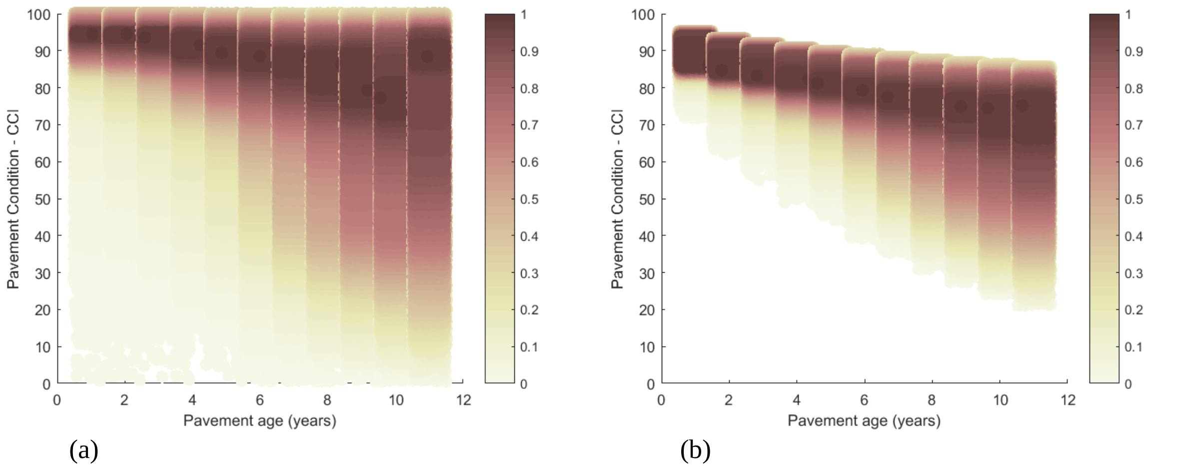


Figure 7-17. Color Map Plot of the Primary Network Pavement Family; a) Raw observed pavement condition data; b) Pavement condition modeled using the LEB approach

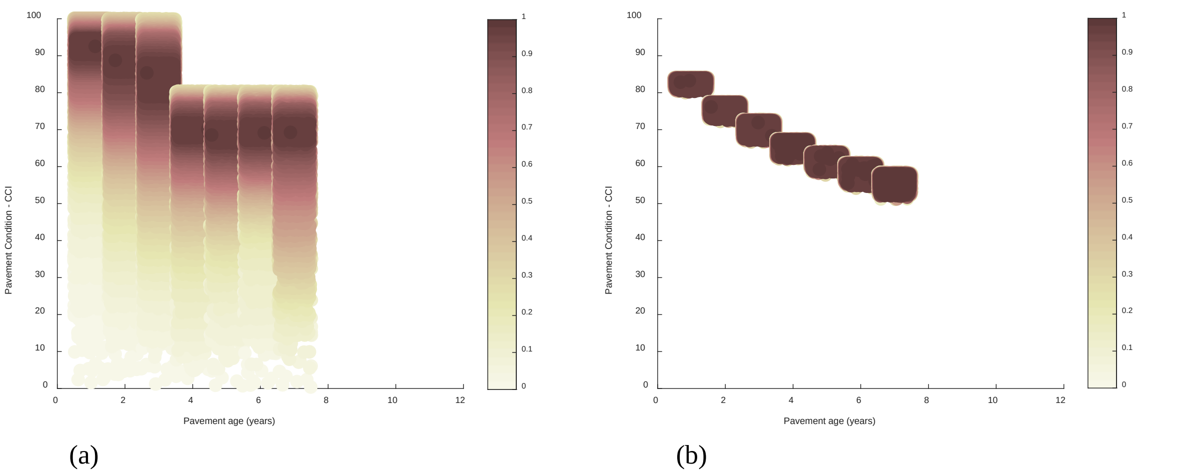


Figure 7-18. Color Map Plot of the Secondary Network Pavement Family; a) Raw observed pavement condition data; b) Pavement condition modeled using the LEB approach

7.5 DISCUSSION AND CONCLUSIONS

The results presented shows that the proposed deterioration modeling approach can improve the prediction of future pavement condition in terms of *CCI* for the Virginia DOT for the pavement families studied (interstate, primary and secondary).

The empirical distribution of the data is well represented by the NB distribution (providing a better distribution of the normal distribution), a form of Poisson-Gamma model allowing to use the Negative Binomial regression to build a regression model that fits VDOT data well, the resulting model provides a similar coefficient of determination with respect to the current default models but has the advantage of allowing to estimate the pavement condition using the LEB approach as a weighted combination of the average condition provides by the model and the observed pavement condition. The definition of the NB distribution as a Poisson–Gamma model allows the use of the EB approach that improves the overall performance of the model by calculating the best estimate of the pavement condition based on the average condition predicted by the model and the last measurement recorded in the network for each specific section. This approach can be useful for pavement network-level predictions by combining the prediction of the fitted model with the observations of the pavement condition. The main advantage of this method is that it allows better predictions of the next year's pavement condition compared with the prediction of average family pavement deterioration regression model, improving the consistency between the network- and project-level deterioration curves.

However, there are still inherent errors in the PMS collected data (missing construction record data, uncertainties in the data collection process) that is necessary to deal with. The cross-validation process proves that estimate the pavement condition from the new deterioration regression model would cause significant errors in the prediction of the deterioration in pavement sections (Method 2). Nevertheless, the model is still needed to account for the mean deterioration of pavement sections and to obtain the LEB estimator. The proposed LEB modelling approach can account for the variance of pavement condition data and allows to the maximize the effectiveness of EB estimator obtaining a deterioration model that can get the estimate of the measurement error as the observed measurement error. The use of the LEB approach proved to be an effective method to estimate the future pavement condition (next year) by subtracting from the LEB estimator the modeled pavement deterioration by the model (Method 5). Method 5 improve the MSE prediction of the future (next year) pavement condition between the observed and predicted future condition by 33 % for the interstate family, 36 % for the primary family, and 41 % for the secondary family.

CHAPTER 8. DESIGN PAVEMENT MANAGEMENT STRATEGIES: PERFORMANCE ASSESSMENT FOR NETWORK LEVEL PAVEMENT PRESERVATION

8. DESIGN PAVEMENT MANAGEMENT STRATEGIES: PERFORMANCE ASSESSMENT FOR NETWORK LEVEL PAVEMENT PRESERVATION

8.1 INTRODUCTION

Road agencies should budget a high amount of public money to reduce the number of accidents and achieve a high level of service of the highway system. Managing and preserving those investments is increasingly one of the main objectives of road agencies, even more in the actual panorama of limiting funding. The implementation of PMS will allow the continuous monitoring of the network and the prediction of future performance, helping in establish cost-effective pavement preservation strategies to fulfill user's expectation and increase the life of pavement assets.

This research reviews pavement preservation policies along time-series to evaluate the effectiveness of maintenance treatment performed at the network level. Historical pavement condition and maintenance records were retrieved from VDOT PMS database for flexible pavement sections for primary system in three maintenance districts: Richmond, Northern Virginia, and Hampton Roads analyzing CCI indicator (see § 5.1).

The overall objective of this chapter is to increase the knowledge about the current practices of pavement maintenance and repair strategies (Table 8-1) and would have a potential impact on decision making at the network level. This main objective implies the development of other pursuit activities, such as, a) evaluate the field performance of maintenance treatment strategies; b) the development of performance models for flexible pavement sections receiving different maintenance treatments at different ages; c) study the effects of maintenance treatments in extending pavement life; d) evaluate the maintenance treatments categories cost-effectiveness to compare them among maintenance treatment categories..

Table 8-1. Virginia Pavement Maintenance Central Office Cost Estimate (Wu, Flintsch et al. 2008) and CCI triggers for Maintenance Treatments (de León Izeppi, Morrison et al. 2015)

Treatment Code	Treatment type	CCI Trigger for Primary System	Pavement Condition	Cost ($ per Lane-mile) for Primary System
DN	Do Nothing	90-100	Excellent	-
PM	Preventive Maintenance	60-89	Good	6,977
CM	Corrective Maintenance	40-59	Fair	59,686
RM	Restorative Maintenance	25-39	Poor	153,229
RC	Rehabilitation/Reconstruction	0-24	Very Poor	480,494

8.2 METHODOLOGY

Pavement condition data used in this research were retrieved from inspections and pavement historical maintenance treatments recorded in VDOT PMS database (see § *5.1*) Spatial analysis and referencing tools where used to match survey data records with inventory data stored in VDOT database (see § 5.1 and § 5.2.3) using VDOT Linear Referencing System (LRS). The dataset resulting from matching procedures has been used to reconstruct pavement performance timeseries.

To fulfill the objectives of this research, sequential activities has been carried out. First, post-treatment performance deterioration curves for the last maintenance cycle of M, R&R treatments in use by VDOT (Preventive Maintenance, Corrective Maintenance, Restorative Maintenance, and Reconstruction) have been developed using inspected sections where at least CCI inspection records available for 6 consecutive years by applying NB regression model (Eq. 8-1), see *§7.2.4 Negative Binomial model Development and Virginia DOT Pavement deterioration models*, .

$$CCI_{NB\_model} = 100 - age^{\beta_2} \cdot exp(\beta_0 + \beta_1 \cdot age), \qquad \text{Eq. 8-1}$$

Secondly, the effects of maintenance projects on pavement condition have been evaluated by determining the immediate improvement caused by a maintenance treatment, it can be defined as Performance Jump (PJ) (Haider and Dwaikat 2011). PJ can be obtained by considering the difference between the nearest inspection CCI before and after the maintenance treatment recorded in the database (e.g., if 2014 is the year of competition of maintenance treatment, it is considered the difference between the CCI before, 2013 inspection, and the CCI after, 2015 inspection).

In the light of previous studies summarized in the literature review (see *§ 2.5 Pavement Preservation Strategies Experiences*), there are two consolidated approaches to calculate the lifespan of the maintenance treatment.

On the one hand, Peshkin, Hoerner et al. (2004) considered the end of service life when pavement section the performance model prediction reaches a threshold defined by the road agency. This approach leads to high theoretical lifespans and it is not representative of road agency maintenance policy, because road agencies concentrate their efforts in maintaining their networks ata high level of service. As a consequence, they perform maintenance treatments before pavements reach their end of life.

On the other hand, Dong and Huang (2011) proposed that the service life can be calculated as the time between two consecutive maintenance records on the same section, reflecting the road agency's real maintenance policy. In this research, VDOT maintenance policies had been carefully studied in the following sections in terms of maintenance treatments' application timing, improvement of pavement CCI, that is, PJ recorded in the database, and pavement service life of maintenance treatments to increase the knowledge about VDOT road agencies maintenance policies in use.

Cost-effectiveness analysis were carried out and discussed later in this chapter by making a comparison between maintenance treatment strategies performed. Maintenance treatment costs are estimated using VDOT Central Office's cost estimate (Table 8-1) and maintenance strategies effectiveness are estimated as benefit area (B) calculated as the bounded by the pretreatment performance curve (Do Nothing curve obtained through NB regression for the Primary System in § 7.2.4), the post-treatment performance curve and VDOT critical threshold (CCI=40), considered as the lower cut off value, as depicted in Figure 8-1. This area was calculated performing numerical integration via the trapezoidal method. This method approximates the definite integration over a given interval by breaking the area under the curve into trapezoids that can be easily calculated. Accordingly, the benefit area (B) and Do-nothing area (A) can be calculated using Eq. 8-2, and Eq. 8-3.

$$BenefitArea(B) = \left(\int_a^c CCI_{M,R\&RPostTreatment}(CCI)dage - \int_a^b CCI_{DoNothing}(CCI)dage\right) - (c-a)\times d \quad \text{Eq. 8-2}$$

$$DoNothing\ Area(A) = \left(\int_0^b CCI_{DoNothing}(CCI)dage\right) - (b\times d) \quad \text{Eq. 8-3}$$

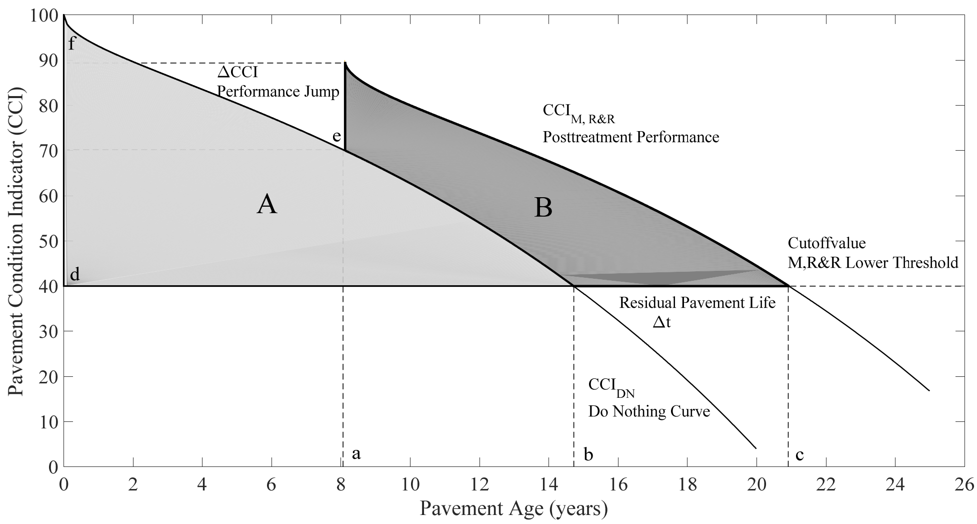


Figure 8-1. Pavement Maintenance Treatment Effectiveness Calculation

8.3 RESULTS

Pavement maintenance representative projects evaluated in this study to build post-treatment performance models for maintenance treatments VDOT categories (Table 8-1) were selected from VDOT database and summarized in Table 8-2.

Table 8-2. Pavement sections used in this study

Type of Treatment	Project Sections used in this study	Number of Observations
PM (Chip Seal, Microsurfacing, Slurry Seal, and Thin Hot Mix (HMA) Asphalt Concrete Overlay (THMACO)	60	328
CM (Milling and HMA Layer Inlay <=2 inches)	148	785
RM (Milling and HMA Layer Inlay <= 4 inches)	29	130
RC (Milling, Break, and HMA Overlay >6 inches)	69	370

8.3.1 PAVEMENT PERFORMANCE MODELS

Post-treatment performance models have been obtained using NB regression for the above-mentioned pavement maintenance categories (age, CCI) subset (Table 8-2). Figure 8-2, Figure 8-3, Figure 8-4, and Figure 8-5 show post-treatment performance models for the primary system for PM, CM, RM, and RC maintenance treatment categories, respectively. This models have been used to forecast pavement condition and to compare among them pavement sections receiving different maintenance treatment categories. The parameters of the performance models are detailed in Table 8-3.

Table 8-3. Post-treatment performance models for VDOT representative preservation maintenance strategies

Type of Treatment	Post-treatment performance models	R^2
PM	$\beta0=1.8756$; $\beta1=-0.1959$; $\beta2=-0.0021$; $\phi=0.3325$	0.4580
CM	$\beta0=2.2119$; $\beta1=-0.0710$; $\beta2=0.9459$; $\phi=0.6397$	0.2749
RM	$\beta0=2.6858$; $\beta1=-0.1512$; $\beta2=1.0669$; $\phi=0.7078$	0.1458
RC	$\beta0=2.1196$; $\beta1=-0.1764$; $\beta2=1.4815$; $\phi=0.4138$	0.4575

In general, performance models are good at representing the average deterioration (depicted with red filled dots at above mentioned figures) but there are observations that move away from average deterioration pavement condition data because data is affected by high variance, as already highlighted in § 7.2.1.

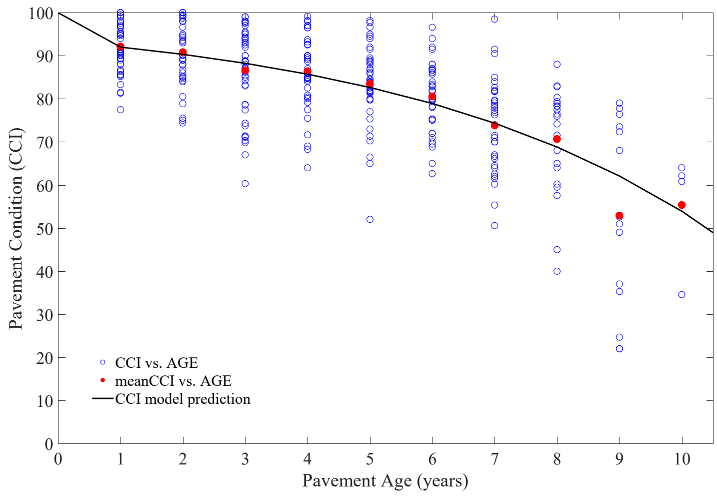


Figure 8-2. Preventive Maintenance Post-Treatment Performance Model for Primary System.

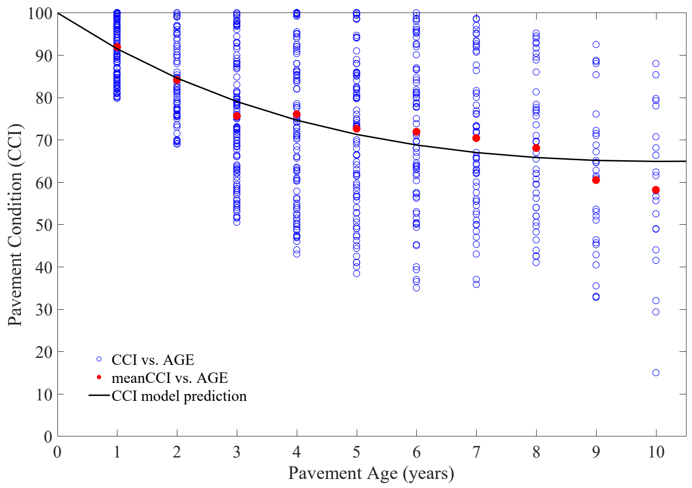


Figure 8-3. Corrective Maintenance Post-Treatment Performance Model for Primary System.

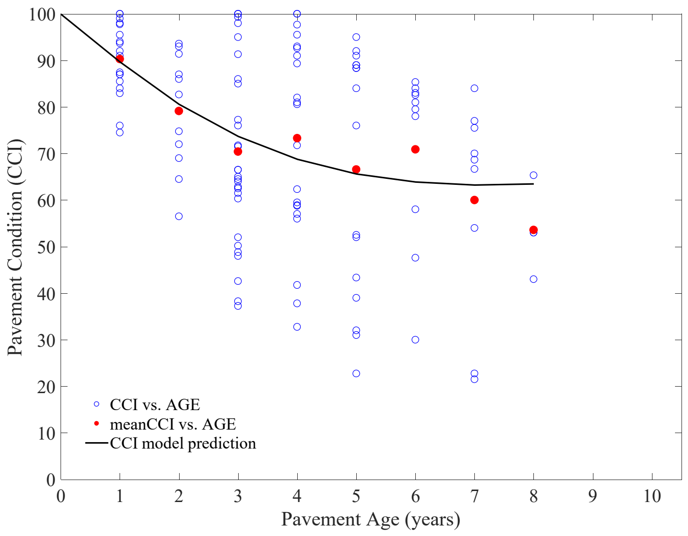


Figure 8-4. Restorative Maintenance Post-Treatment Performance Model for Primary System.

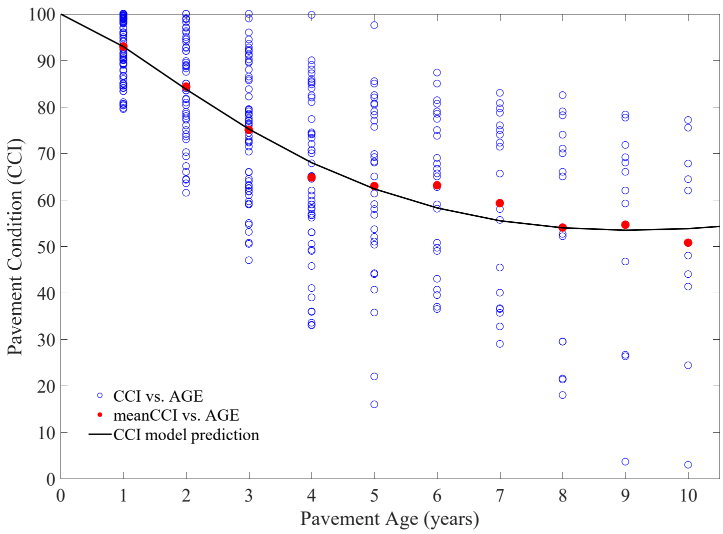


Figure 8-5. Reconstruction Maintenance Post-Treatment Performance Model for Primary System.

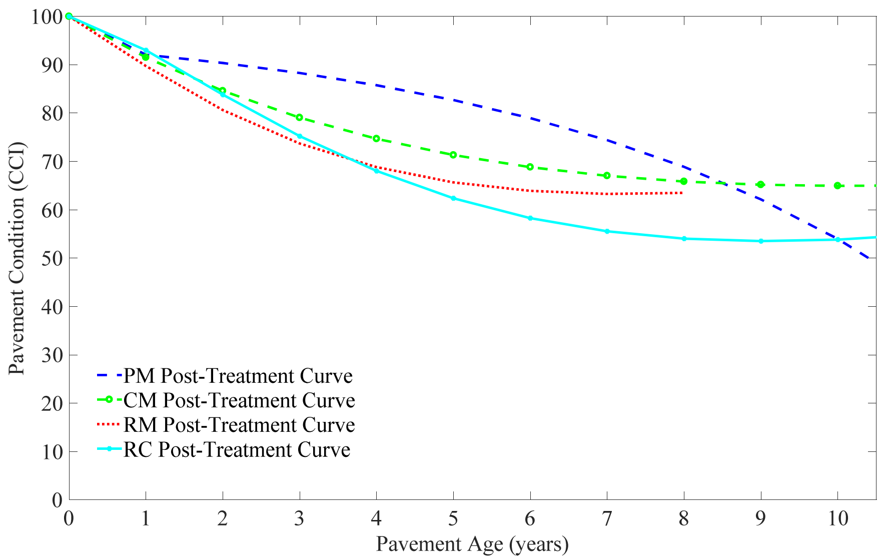


Figure 8-6. Post-Treatment Performance Curves Comparison

Figure 8-6 shows the comparison of pavement post-treatment performance curves of VDOT maintenance policies in use. It can be observed that pavement sections that have received preventive maintenance treatments (blue dashed line) perform better than other receiving other maintenance treatments. This is mainly due to pavement sections receive PM treatment at younger ages (early stage of deterioration). However, pavement sections receiving major treatments (RM and RC) curves shows a faster deterioration if compare with PM curves, this occur because RC treatments are applied lately in almost all the cases (65 of 69 sections) when pavement have already failed and pavement sections have not received preservation pavement treatments (PM, CM) before RC.

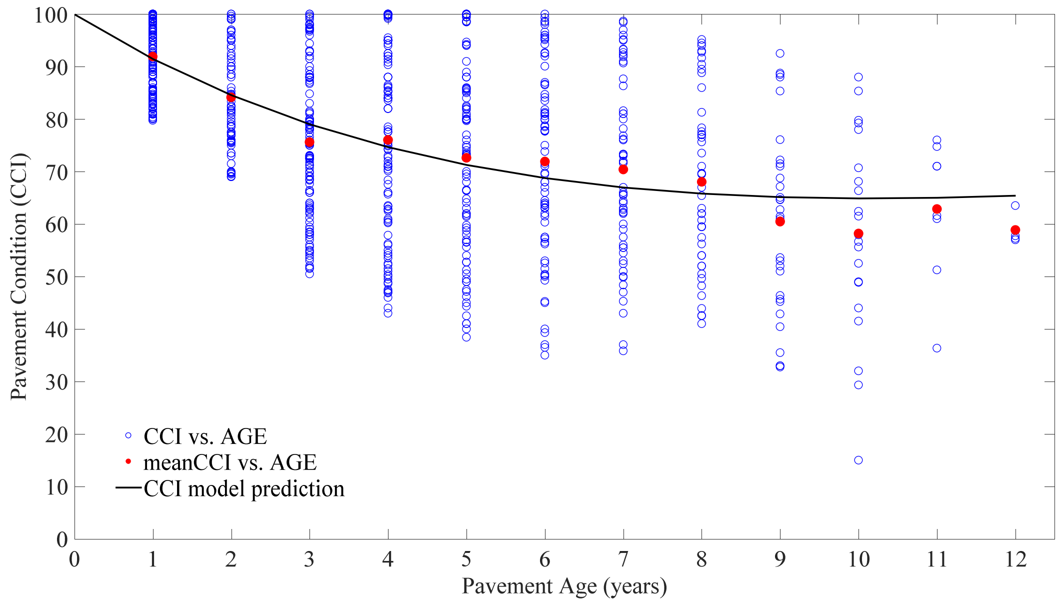
Moreover, CCI performance curves do not follow expected pavement condition decreasing over time for pavement sections with age more than 6. After this age, pavement performance models follow almost a constant trend or even a positive trend for CM (Figure 8-3), RM (Figure 8-4), and RC (Figure 8-5). This is against the usual deterioration process (pavement deteriorates faster at older ages).

This happens because the performance models are built for the last maintenance cycle stored in PMS database with at least more than 6 inspection records for consecutive year after the application of maintenance treatment. The maintenance projects used time-baseline is the year before (2006) the first inspection year (2007), that is, there are a total number of 11 CCI observations recorded in the PMS and pavement age can achieve a maximum value of 12 years.

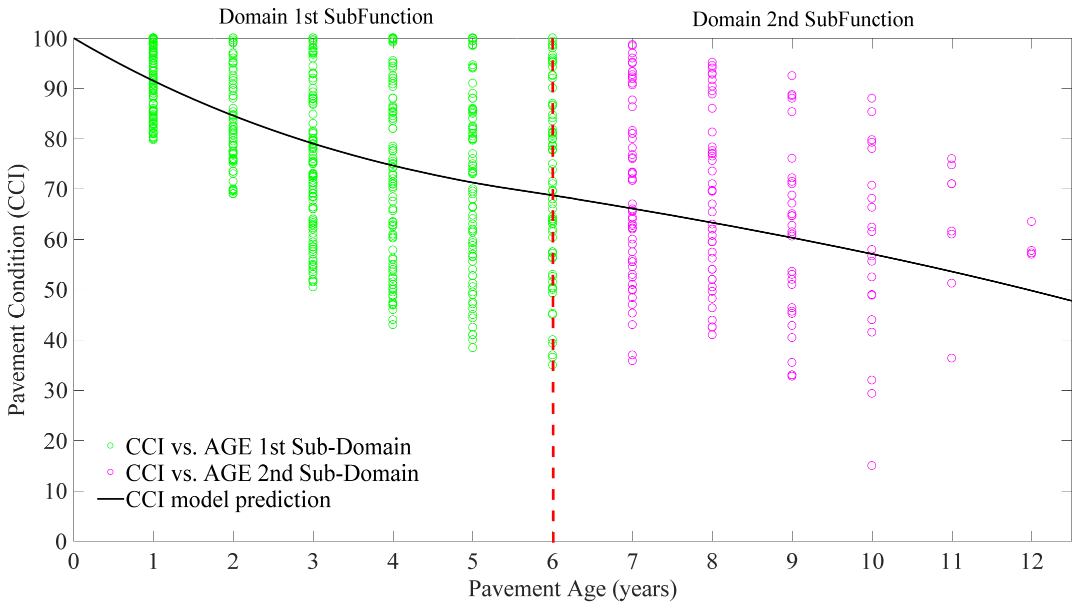
The number of sections with 6 consecutive CCI observations is higher than pavement sections with more than 6, that is because all projects used for the population of the dataset (age, CCI) have at least 6 years of pavement data. Instead, there are less sections that have 11 CCI consecutive inspections. For this, performance models have been enhanced to improve the models for pavement ages between 6 and 12 and these can be used in cost-effectiveness analysis, see Figure 8-7.

Post-treatment enhanced performance models have been constructed as a piecewise functions (Eq. 8-4), with two sub-functions in associated subdomains: age<=6 years and age>6 years. Both sub-functions build on NB regression, using different subset for each piecewise sub-functions: the first piecewise sub-function (subdomain age<=6 years) uses (age,CCI) subset for CCI observations with age less of 6 years, the second piecewise sub-function (subdomain age >6 years) uses the rest of the observations of the dataset, see Figure 8-7(b). Additionally, it is imposed that the second sub-function for the abscissa age=6, has to have as coordinate the same value that the first sub-function. This condition is verified by shifting of a value that is the difference of the ordinates between the two sub-functions.

$$CCI_{NB,Model} = \begin{cases} 100 - age^{\beta_{2,1}} \cdot exp\left(\beta_{0,1} + \beta_{1,1} \cdot age\right), age \leq 6 \\ 100 - (age-6)^{\beta_{2,2}} \cdot exp\left(\beta_{0,2} + \beta_{1,2} \cdot (age-6)\right) - f, age \in]6,12] \end{cases} \quad \text{Eq. 8-4}$$



(a) Post-Treatment Performance Model



(b) Enhanced Post-Treatment Performance Model

Figure 8-7. Enhanced Post-Treatment Piecewise Performance models

As a result, enhanced post-treatment performance models are calculated for CM (Figure 8-8), RM (Figure 8-9), and RC (Figure 8-10) maintenance categories. Performance model parameters are summarized in Table 8-4. Following this approach to enhance maintenance treatment, it can be observed that maintenance performance models showing a sharper CCI decreasing after age 6, as expected in pavement deterioration process.

Table 8-4. Enhanced Piecewise Post-Treatment Performance Models Parameters

Maintenance Treatment Category	Enhanced Post-Treatments Piecewise Performance Model Parameters
CM	β0,1= 2.2382; β1,1=-0.0987; β2,1=1.0014; ϕ1=0.6393; f=3.4257;β0,2=3.2417; β1,2=-0.0824; β2,2=0.0138; ϕ2=0.3899
RM	β0,1= 2.4889; β1,1=-0.1591; β2,1=1.1450; ϕ1=0.6855; f=6.9896;β0,2=3.4595; β1,2=-0.0887; β2,2=0.5866; ϕ2=0.2296
RC	β0,1= 2.1064; β1,1=-0.1582; β2,1=1.4367; ϕ1=0.4160; f=7.5609;β0,2=3.4872; β1,2=0.1209; β2,2=-0.1428; ϕ2=0.2077

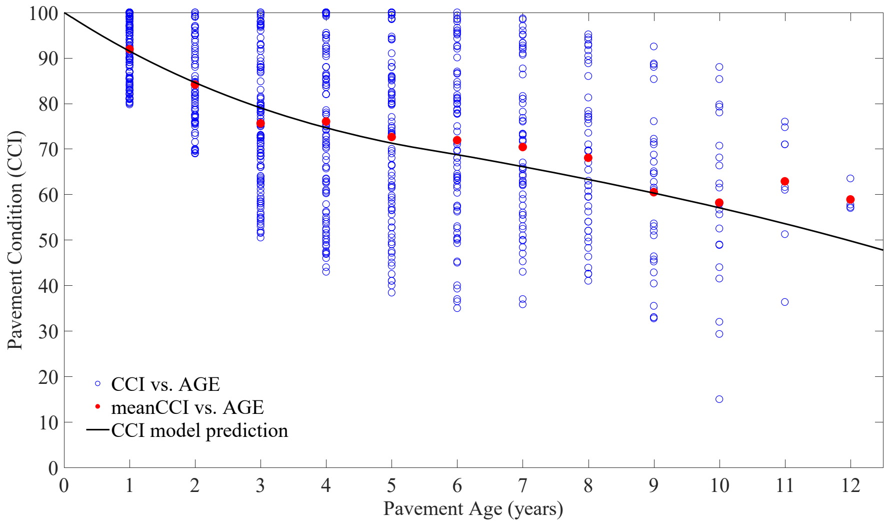


Figure 8-8. Enhanced Post-Treatment Corrective Maintenance Performance Model

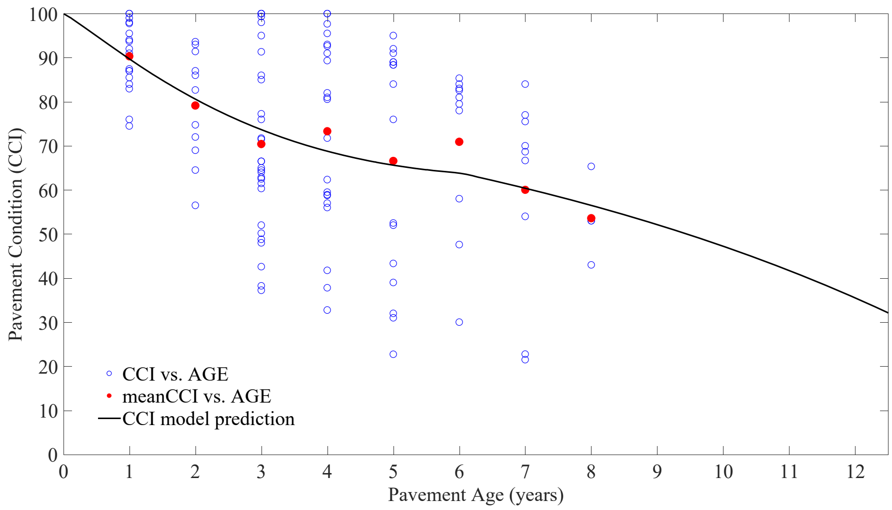


Figure 8-9. Enhanced Post-Treatment Restorative Maintenance Performance Models

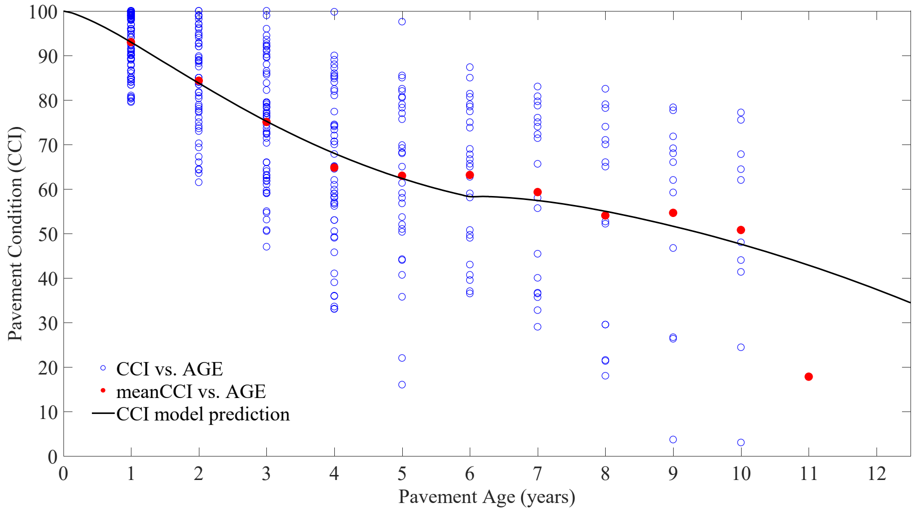
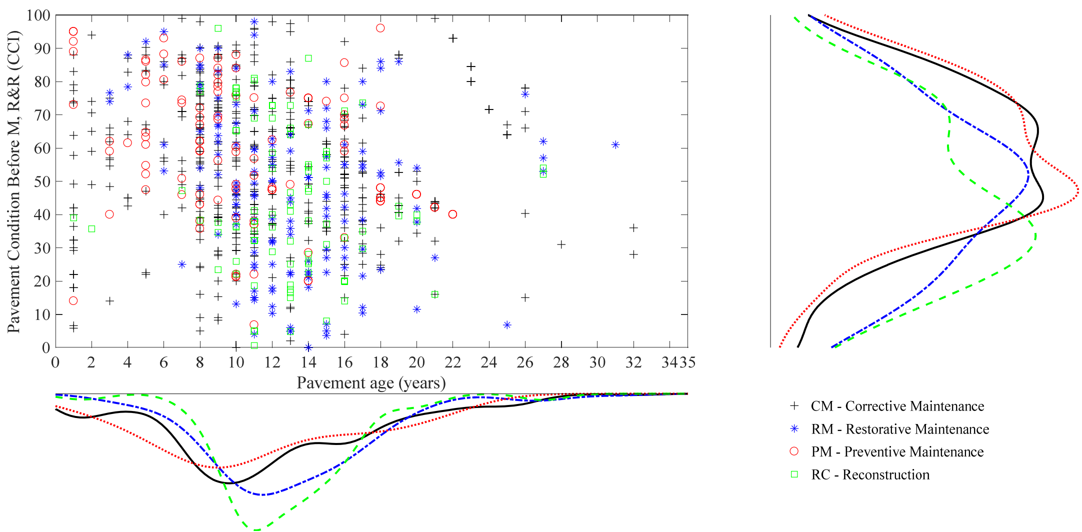


Figure 8-10. Enhanced Post-Treatment Reconstruction Performance Model

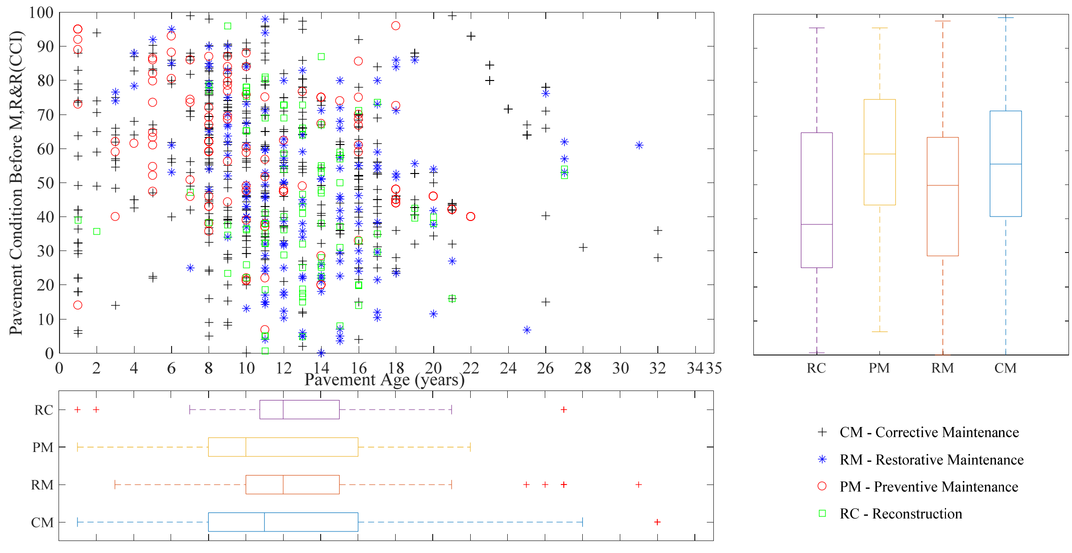
8.3.2 VDOT Pavement Maintenance Policies Effectiveness

Pavement maintenance policies applied in a road network by a road agency and its effectiveness are strictly linked to the phase of deterioration when maintenance treatment is applied.

Figure 8-11 shows a scatter plot of (age, CCI) data retrieved from PMS historical data for the years before the application of maintenance treatments. Marginal histograms (a) and boxplots (b) have been generated sorting maintenance projects by the categories defined in Table 8-2. The figure illustrates that PM maintenance treatment applications are concentrated at the first stages of deteriorations, CM, RM can be localized at the medium phase of deterioration, and RC are localized in the last phase of deterioration at late ages for pavement sections with CCI values less than 50 points.



(a)



(b)

Figure 8-11. Pavement Preservation Policies (age, CCI) before M, R&R treatment (a) Marginal Histogram and (b) Boxplot

8.3.2.1 PERFORMANCE JUMP

Figure 8-12 depicts the boxplots of empirical distribution of CCI before and after the application of the maintenance treatments sorted by VDOT categories. It can be observed that maintenance treatments succeed in restoring the pavement condition (CCI after values close to 100, perfect pavement).

This figure captures pavement agency maintenance policies showing that the application of one or another maintenance treatment varies according pavement section CCI (CCI before). Lower values of CCI_{before} suggest a higher maintenance treatment category (RM, RC). Usually, road agencies defined certain windows of intervention (pavement maintenance thresholds) to suggest pavement engineers which pavement maintenance treatments categories may apply configuring road agency maintenance policies. VDOT has defined thresholds in terms of CCI (Table 8-1) to suggest the pavement maintenance category to use. Figure 8-12 presents graphically VDOT pavement preservation policies trigger thresholds' in use are respected for almost all pavement projects evaluated.

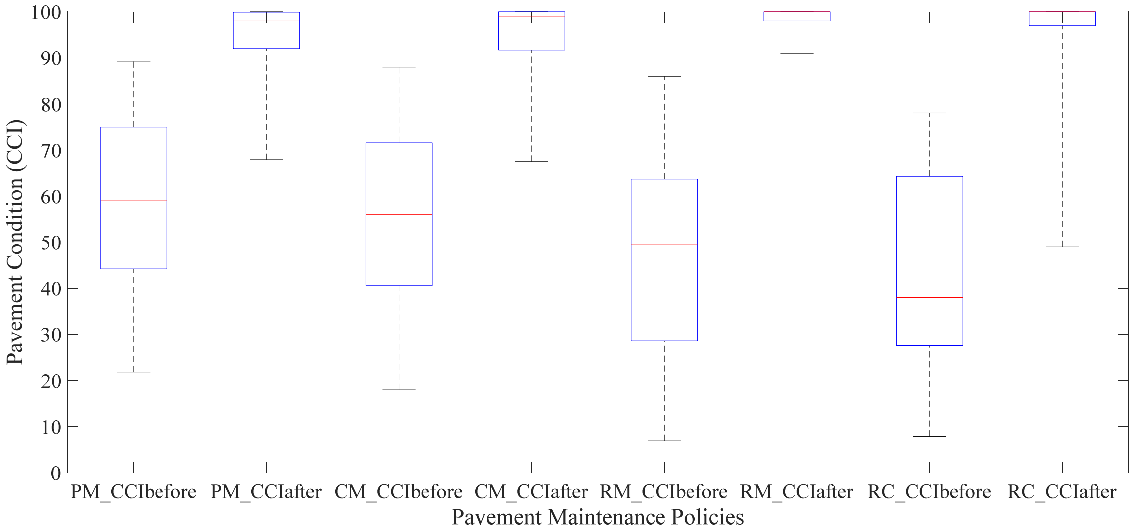


Figure 8-12. Pavement Condition CCI Empirical Distributions Before and After the Application of Maintenance Treatments for VDOT Primary System

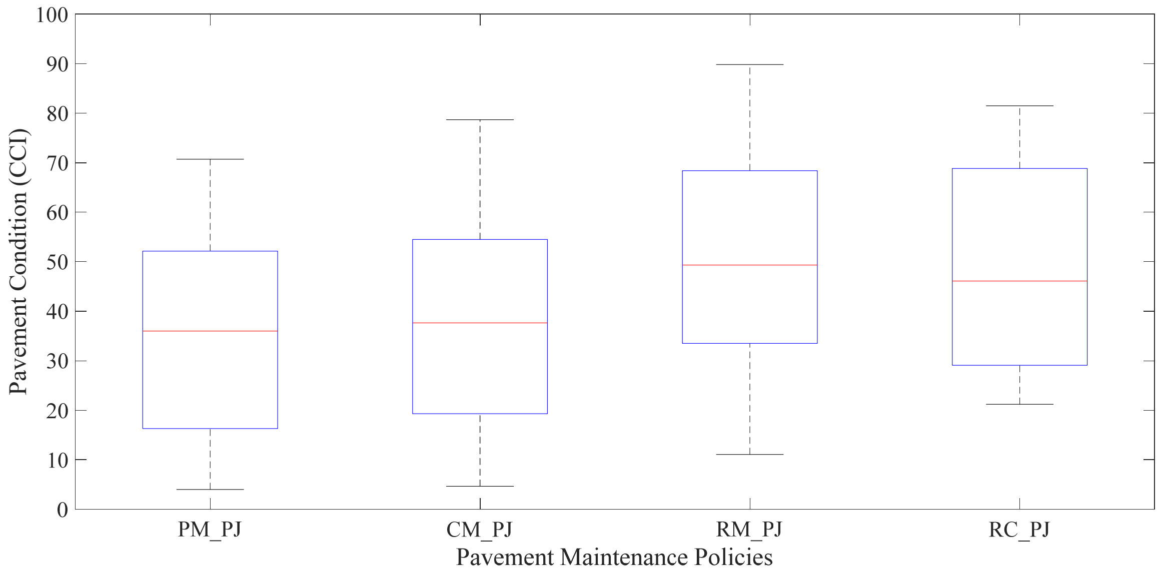


Figure 8-13. Boxplot of Performance Jump by VDOT on the Primary System

Figure 8-13 shows the boxplot of PJ obtained as a function of maintenance treatment categories. Table 8-5 shows the average, 25th percentile, and 75th percentile of CCI $_{after}$, CCI $_{before}$ CCI $_{PJ}$ for the above-mentioned maintenance treatment categories.

Table 8-5. Average, 25th Percentile, and 75th Percentile of CCI $_{after}$, CCI $_{before}$, CCI $_{PJ}$

Item	Percentile50 (average)	Percentile25	Percentile75
Preventive Maintenance CCI_{after}	59.00	44.00	75.00
Preventive Maintenance CCI_{before}	98.00	92.00	99.90
Preventive Maintenance CCI_{PJ}	36.85	19.35	55.50
Corrective Maintenance CCI_{after}	56.00	40.60	71.60
Corrective Maintenance CCI_{before}	98.90	91.70	100.00

Corrective Maintenance CCI_{PJ}	39.00	22.55	57.90
Restorative Maintenance CCI_{after}	49.75	29.05	63.85
Restorative Maintenance CCI_{before}	100.00	98.00	100.00
Restorative Maintenance CCI_{PJ}	48.90	33.50	67.90
Reconstruction CCI_{after}	38.30	25.58	65.20
Reconstruction CCI_{before}	100.00	97.68	100.00
Reconstruction CCI_{PJ}	46.10	29.10	68.85

8.3.2.2 MAINTENANCE TREATMENT SERVICE LIFE

Another parameter of interest for the road agency is how many years the pavement would last after the maintenance treatment application, that is, the service life duration of the single maintenance intervention. Table 8-6 and Figure 8-14 display the service life for evaluated projects in VDOT network. As explained in the methodology section, pavement service life is calculated as the years between consecutive maintenance sections recorded in each single section.

Table 8-6. Average, 25th Percentile, and 75th Percentile of Service Life After Maintenance Treatments

Item	Percentile50 (average)	Percentile25	Percentile75
Service Life Preventive Maintenance	8	5	12
Service Life Corrective Maintenance	9	6	13
Service Life Restorative Maintenance	9	6	14
Service Life Reconstruction	10	4	15

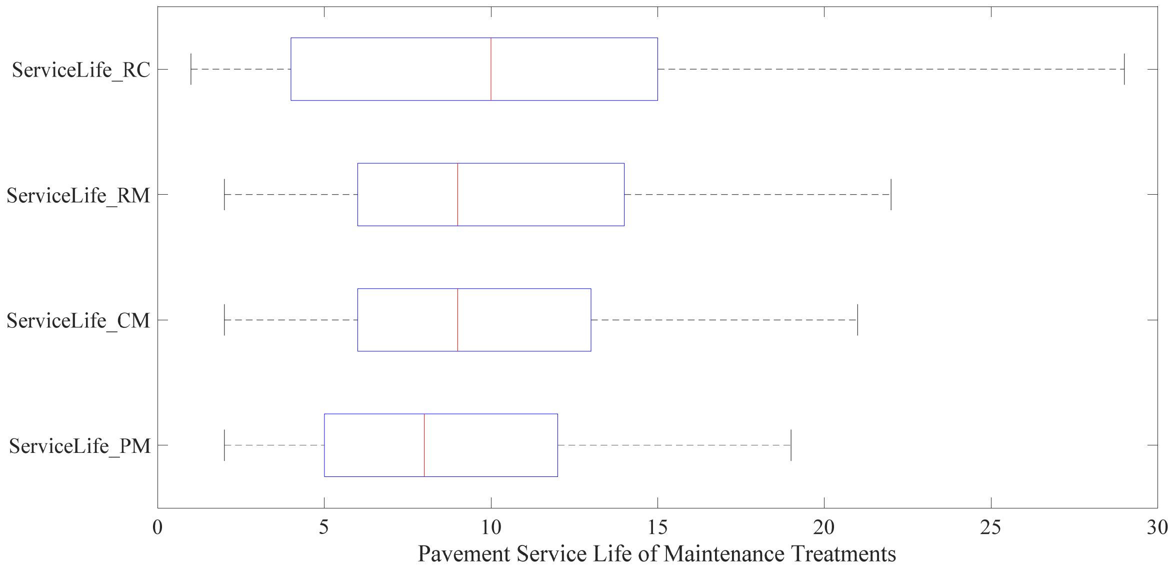


Figure 8-14. Pavement Treatment Service Life for VDOT Primary System

8.3.2.3 COST-BENEFIT ANALYSIS

Maintenance projects were chosen to illustrate the influence of acting on different times of pavement lifecycle with different types of maintenance treatments categories. Figure 8-15 (PM), Figure 8-16 (CM), Figure 8-17 (RM), and Figure 8-18 (RC) shows the benefit area (B) along the do nothing area (A). The computation of these areas are used as an indicator of relative benefits obtained by the application of one maintenance treatment against other considering the costs of the single maintenance treatments.

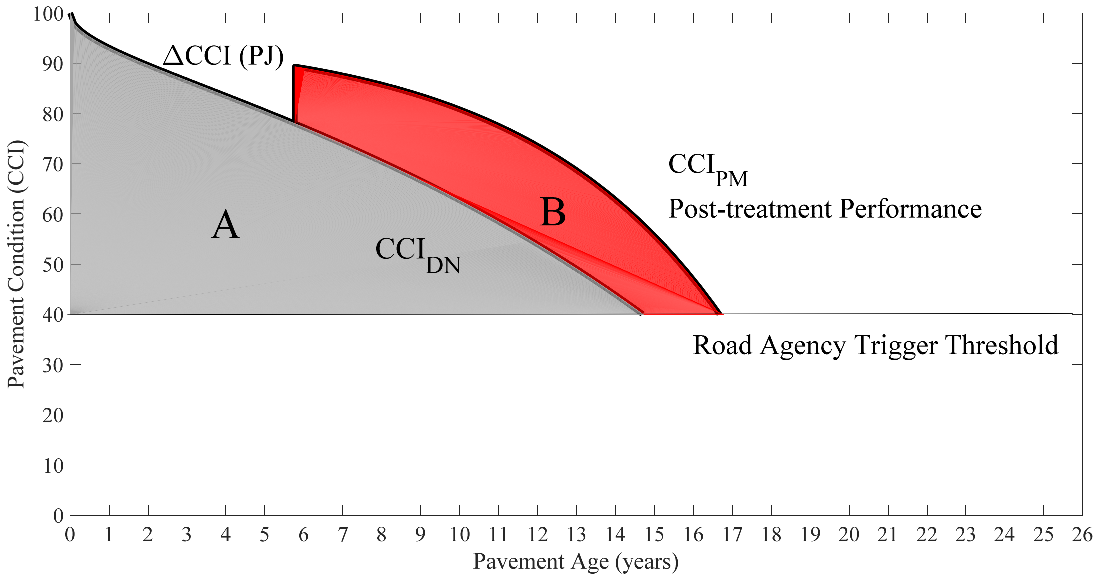


Figure 8-15. Computation of Benefit Area and Do-Nothing Area of Preventive Maintenance Project

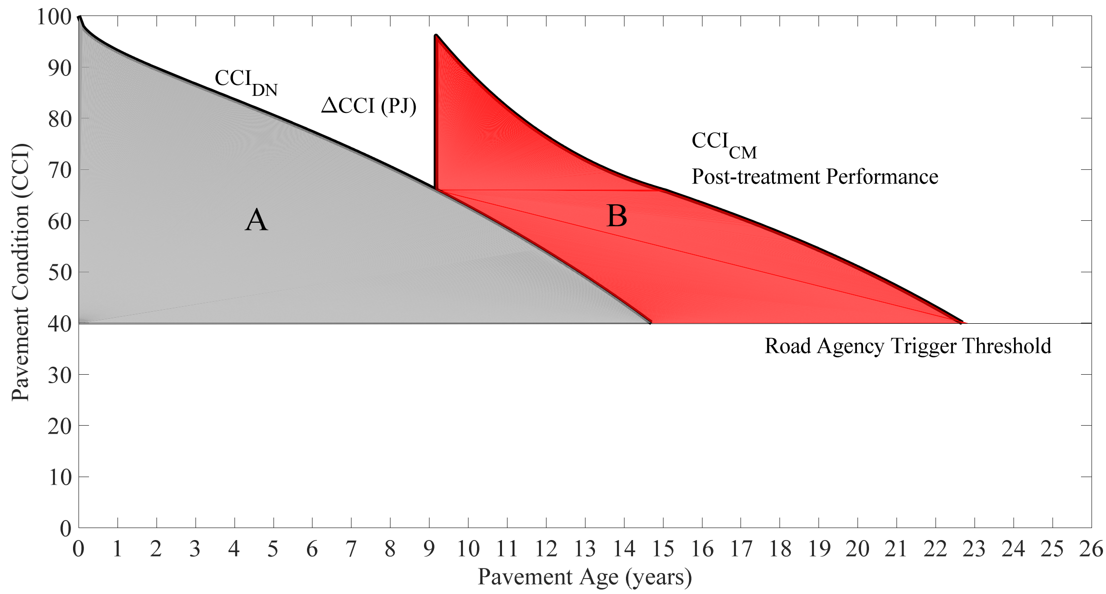


Figure 8-16. Computation of Benefit Area and Do-Nothing Area of Corrective Maintenance Project

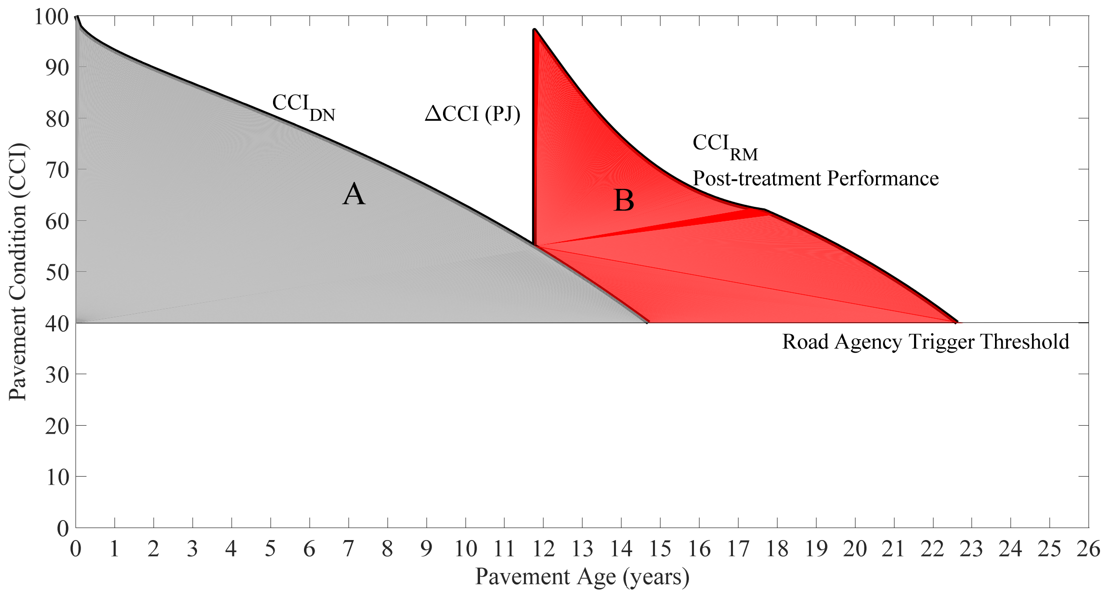


Figure 8-17. Computation of Benefit Area and Do-Nothing Area of Restorative Maintenance Project

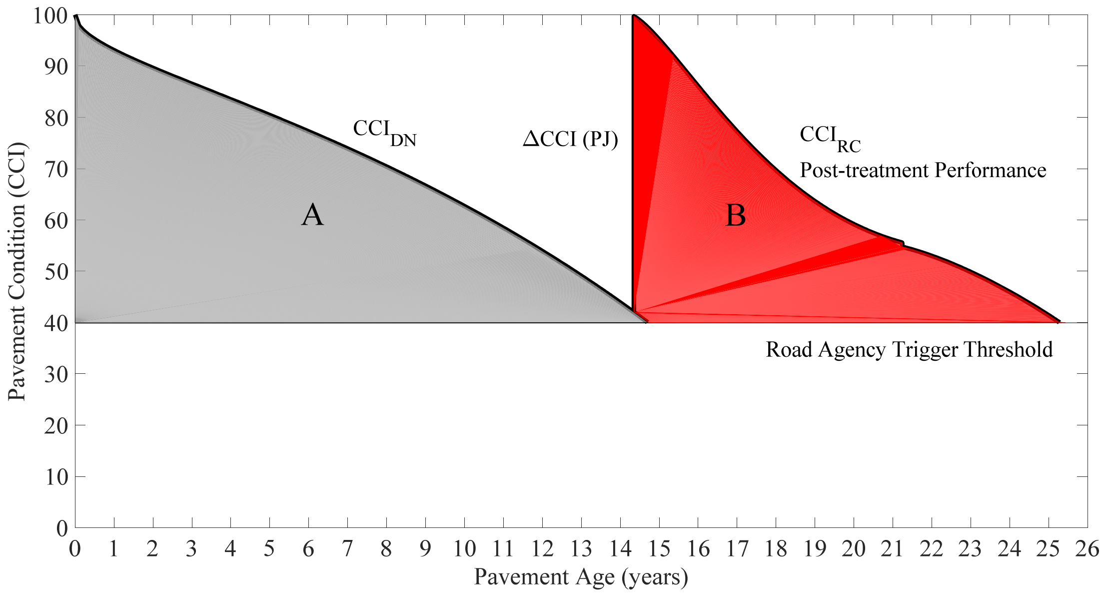


Figure 8-18. Computation of Benefit Area and Do-Nothing Area of Reconstruction Project

The computation of the Benefit Area (B), Eq. 8-2, Do Nothing Area (A), Eq. 8-3, and cost estimate of maintenance treatments categories are outlined in Table 8-7 and Figure 8-19.

Table 8-7. Relative Benefit Cost-Effectiveness of VDOT Pavement Preservation Policies

Pavement Section	CCI_{before}	Do Nothing Area (A)	Benefit Area (B)	Relative Benefit (B/A) (%)	Cost ($/ Lane-Mile)	Benefit Cost Ratio (B/C) x1000
PM	78	448.06	172.20	38.43	6,977	25.41
CM	66	448.06	252.36	56.32	59,686	4.23
RM	55	448.06	238.71	53.28	153,229	1.56
RC	42	448.06	265.83	59.33	480,494	0.55

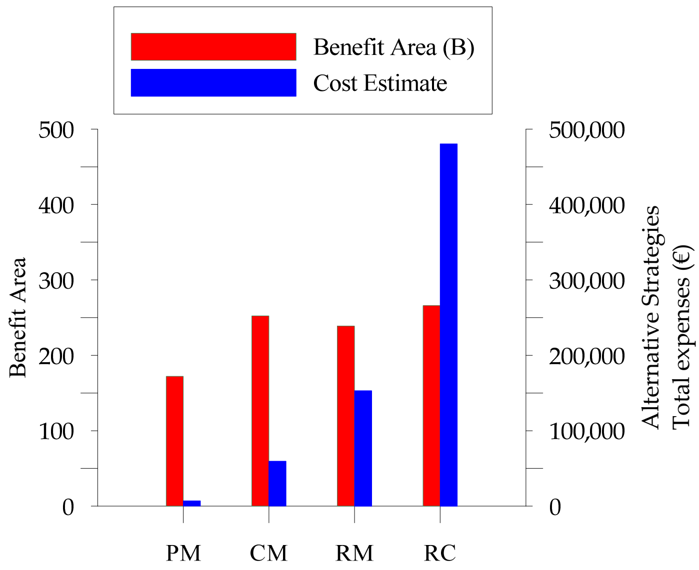


Figure 8-19. Cost-Effectiveness of Maintenance Treatments Category

8.4 DISCUSSION OF RESULTS

The results show that PM treatments have a reduced cost implementation but a relative high benefit area (B), if compared with the other studied maintenance categories. Benefit is 35 % lower than the higher benefit area calculated (RC) computation, but it has a significant reduction in the cost estimate maintenance cost (PM treatments cost 69 times less than RC treatments). Therefore, PM preservation activity has the highest cost-benefit ratio (Figure 8-19). However, it should be noted that PM are ineffective when pavements are in poor conditions.

Moreover, it can be observed that CM, RM, and RC are effective in extending pavement service life in pavements in fair to poor conditions but these interventions have the highest associated costs. that are corresponding with the lifespan maintenance treatments VDOT maintenance policies above studied. This analysis shows the effects of deferred maintenance treatment application.

8.5 RECOMMENDATIONS FOR FUTURE RESEARCH

In this research, efforts were focused on analyzing the effectiveness of individual pavement preservation treatments performed by VDOT on the Commonwealth of Virginia pavements. Future developments of this research would include in the analysis pavement sections with two or more subsequent maintenance treatments with the aims of analyzing the cumulative effect of preservation treatments to postpone reconstruction and rehabilitation activities.

Moreover, VDOT collect roughness data summarized using the International Roughness Index (IRI) along the overall condition indicator CCI. Future developments of this research can be the data processing and analysis in terms of IRI as it is recognized internationally as an indicator of serviceability and it can be related also with ride quality and user's costs (fuel consumption, maintenance costs, vehicle delay costs). Other recommendations for future research would include in the cost-effectiveness analysis the user's costs benefits such as the VOC (see § 4.3.2 for more details).

CHAPTER 9. CONCLUSIONS, RECOMMENDATIONS AND FUTURE RESEARCH

9. CONCLUSIONS, RECOMMENDATIONS AND FUTURE RESEARCH

9.1 SUMMARY OF DISSERTATION

The main objective of this dissertation was to provide guidelines for the implementation of cost-effective PMS strategies for road agencies based on their local conditions. During this research, different methodologies were developed to provide a set of guidelines that constituted a reliable tool, as a kind of umbrella, for supporting pavement management solutions at the network level. The work started with a review of existing network data collection technologies, the study of consolidated indexes of pavement condition assessment based on pavement surface distresses evaluation; functional pavement evaluation and structural capacity analysis; the analysis of PMS models for pavement networks of different sizes and with different availability of pavement data, the development of pavement deterioration models at the network level, and cost-effectiveness analysis of pavement preservation strategies.

Collected data network level gave valuable information for the evaluation of current pavement condition and allow pavement managers to make prediction about future condition. This information turned out to be essential for planning M, R&R on the network to maintain high levels of service, fulfilling one of the main objective transportation agencies. However, it was detected that there are still uncertainties on pavement data collection methods, geospatial location of survey data and interpretation of distresses. Pavement condition recorded have a high variance and appropriate processing data techniques were needed to build pavement deterioration models. Therefore, road agencies should consider to increase their efforts on monitoring pavement networks and data processing tools, to promote cost-effective PMS strategies, to structure the data in a comprehensive PMS database ensuring reliable decisions based on the data and consider rules and procedures to analyze data systematically.

9.2 FINDINGS

Based on the analysis of PMS models and strategies worldwide of local agencies, it was found that the biggest loophole of the PMS is the inexistence of a single method of analyze pavement survey data, because data processing is linked to the way data is provided (if available) and the existing rules and conservation policies of each road agency. However, data analysis and processing rules assembled in this dissertation give some of the require information to fulfill this shortcoming.

This dissertation showed the advantage of collect and analyzed pavement data at the network level proactively to analyze current pavement condition and model pavement deterioration to achieve better prediction of pavement condition and design cost-effective maintenance strategies to extend road service life. In particular, this research led to the following findings:

- Road pavement agencies uses customized-agency indexes and distress identification protocols for pavement surface's overall condition assessment. The method developed in Chapter 3 for the computerization of pavement index calculation eliminating time-consuming procedures would be helpful and can be applied for the definition of new distress identification rules. This is especially important in urban areas where there is still a gap of pavement distress definitions and assessment: manholes and tree roots distress identification and assessment were done successfully.
- During the dissertation, the application of PMS in small-mid-sized municipalities (urban areas) have been studied (Chapter 4) to give an analytical tool to help pavement managers in the decision-making process. This application settled up a novel approach for the management of urban network looking at the development of a rational and justified allocation of resources with a reduced cost of implementation. The proposed method can be readily used in practice because it is a flexible method that can be adapted to the needs of each municipality. The results have shown that is essential to make a first estimation of the available budget at the network level and, immediately afterwards, determine what kind of repair and maintenance policy has to be adopted. However, the present condition of the pavement network strongly influenced the budgeting analysis. Therefore, the use of overall index for the urban networks to summarize the pavement condition such as the PCI had been effective to rate pavement section at the network level.
- Pavement survey data involves high quantity of data and are affected by high variability and noise inherent by the data processing and interpretation. LRS and spatial location methodologies have shown to ease data processing and their management giving tools for the effective spatial matching along the network assuring the consistency of data coming from diverse sources.
- The experience of implementing a pilot PMS in Kazakhstan (0) has proved to be effective to give a consistent methodology for the implementation of pavement management program at the network level by analyzing survey data to establish a priority analysis for the entire network in a developing country. The implementation of procedures aligned with international standards would be helpful for this would make easier the comparison of obtained results with other more mature systems and would simplify the training of local staff.

- Pavement deterioration modeling is essential for a successful PMS. Current network level pavement performance models used by VDOT give an adequate of the average pavement deterioration as a function of pavement age but are bad to represent the performance of individual pavement sections. Negative Binomial distribution represented the empirical distribution of deterioration data (providing a better distribution of the normal distribution) that allows to use the NB regression for fitting the pavement deterioration data, the resulting model provides a similar coefficient of determination with respect to the current default models. NB regression model have the advantage of using the LEB approach as a weighted combination of the average condition and the last measurement recorded in the network for each specific section obtaining a better prediction of next year's pavement condition if compare with the one predicted by the model, improving the consistency between network and project level deterioration curves.
- Historical pavement condition time series analysis and maintenance treatment historical records gave valuable information about pavement preservation strategies. VDOT currently in use pavement maintenance policies have been studied statistically to determine the pavement condition improvement and pavement service life extension caused by maintenance treatments, as well as, the lifecycle phase when maintenance treatments are applied. Additionally, cost-effectiveness comparison between preservations strategies evidenced that PM have a reduced cost of implementation but have the highest cost-effective ratio if compared with CM, RM, and RC treatment applications. These results show that a proactive decision-making approach supported by PMS leads to effective decisions with limited costs in extending pavement life.

9.3 CONCLUSIONS

This dissertation proposed a set of guidelines for data analysis and processing of pavement survey data to evaluate maintenance and rehabilitation strategies for the implementation of pavement management systems.

The main scientific contributions of this research were represented by the evaluation of road agencies pavement condition assessment and the evaluation of deterioration models and maintenance effectiveness at the network level. This dissertation analyzed real data from different road agencies of various levels of development (pilot PMS and mature PMS) to analyze pavement condition and predict pavement deterioration based on commonly used parameters such as PCI, IRI, and pavement deflections from FWD. Furthermore, this research contributed to develop procedures to integrate pavement condition data in a PMS database.

From an engineering point of view, this dissertation provided a comprehensive set of guidelines to support pavement data analysis and the decision-making process.

Although the stand-alone tools assembled in this dissertation represent a good outcome, they have to be enhanced to get them valid to local conditions using real data and experiences. Of course, it could be necessary to include new variables and analysis as a consequence of the different local conditions. However, road agencies can benefit from these experiences already tools that in many cases they do not involve expenses because they already collect pavement survey data and these tools would enhance the implementation and results of cost-effective PMS strategies. Nevertheless, these tools can be used by those agencies that do not collect data even if they require expenses and the benefits are not immediate. In few years, they will give economic results managing the in an effective way to enhance the pavement decision-making process.

9.4 RECOMMENDATIONS FOR FUTURE RESEARCH

Although the proposed technologies represent a step forward in the incorporation of decision-making analysis at the network level, other variables and the proposed tools have to be adapted to road agency local conditions. For example, they are closely linked to the data collection methodologies implemented in the road agency and related to the training of the operators of manager and practitioners. Likewise, the time that the system is implemented and the availabity of historical data influence the quality of the analysis.

Moreover, the implementation of cost-effective management systems requires the development of processes and procedures to evaluate the quality of the data surveyed. The development of quality management checks at the various stages of the data collection and interpretation states are required. the implementation of control and assurance plans should fit road agency specific needs.

In the European scene, the use of pavement management systems is beginning to require the attention of practitioners, managers and politicians. But road agencies do not collect and manage pavement data in a comprehensive PMS database (only few airport authorities and highway concessionaires have implemented PMS with less or more success). This represents an important challenge for the public administrations and private contractors to ptimize the expenses in maintenance and rehabilitation treatments at the network level based on reliable data.

References

336, C. (1998). Guidelines for Evaluation of Flexible Pavements at Project Level Using, Falling Weight Deflectometer, European Commission Brussels.
AASHTO (1993). AASHTO Guide for Design of Pavement Structures, 1993, AASHTO.
AASHTO (2012). AASHTO Pavement Management Guide, 2nd Edition 2012.
Abaza, K. A. (2016). "Back-calculation of transition probabilities for Markovian-based pavement performance prediction models." International Journal of Pavement Engineering **17**(3): 253-264.
Adedimila, A. S., A. Olutaiwo and O. Kehinde (2009). "A simplified PCI-based Condition Rating System for a Developing Country." J. Eng. Appl. Sci **5**: 27-40.
Amadore, A., G. Bosurgi and O. Pellegrino (2014). "Classification of measures from deflection tests by means of fuzzy clustering techniques." Construction and Building Materials **53**: 173-181.
Arhin, S. A., L. N. Williams, A. Ribbiso and M. F. Anderson (2015). "Predicting pavement condition index using international roughness index in a dense urban area." Journal of Civil Engineering Research **5**(1): 10-17.
ASTM (2009). ASTM E1911-09. Standard Test Method for Measuring Paved Surface Frictional Properties Using the Dynamic Friction Tester (Withdrawn 2018). West Conshohocken, PA, ASTM International.
ASTM (2012). ASTM D5340-12, Standard Test Method for Airport Pavement Condition Index Surveys. West Conshohocken, PA, ASTM International.
ASTM (2012). ASTM E867 - 06, Standard Terminology Relating to Vehicle-Pavement Systems. West Conshohocken, PA, ASTM International.
ASTM (2013). ASTM E303 - 93, Standard Test Method for Measuring Surface Frictional Properties Using the British Pendulum Tester. West Conshohocken, PA, ASTM International.
ASTM (2013). ASTM E2133-03. Standard Test Method for Using a Rolling Inclinometer to Measure Longitudinal and Transverse Profiles of a Traveled Surface.
ASTM (2015). ASTM D6951 / D6951M - 09, Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications. West Conshohocken, PA, ASTM International.
ASTM (2015). ASTM E274, Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire. West Conshohocken, PA, ASTM International.
ASTM (2015). ASTM E965 - 15, Standard Test Method for Measuring Pavement Macrotexture Depth Using a Volumetric Technique. West Conshohocken, PA, ASTM International.
ASTM (2015). ASTM E1703-10, Standard Test Method for Measuring Rut-Depth of Pavement Surfaces Using a Straightedge. West Conshohocken, PA, ASTM International.
ASTM (2015). ASTM E1845 - 15, Standard Practice for Calculating Pavement Macrotexture Mean Profile Depth. West Conshohocken, PA, ASTM International.
ASTM (2015). ASTM E1926 - 08, Standard Practice for Computing International Roughness Index of Roads from Longitudinal Profile Measurements. West Conshohocken, PA, ASTM International.
ASTM (2018). ASTM D6433-18, Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys. West Conshohocken, PA, ASTM International.
ASTM (2018). ASTM E950 / E950M - 09, Standard Test Method for Measuring the Longitudinal Profile of Traveled Surfaces with an Accelerometer-Established Inertial Profiling Reference. West Conshohocken, PA, ASTM International.
Becker, M. A. (1972). Selection of optimal investment strategies for low volume roads, Massachusetts Institute of Technology.
Bennett, C. R., H. De Solminihac, A. Chamorro and G. W. Flintsch (2006). "Data collection technologies for road management."

Bennett, C. R. and I. D. Greenwood (2001). Volume 7: Modeling Road User and Environmental Effects in HDM-4. La Défense Cédex World Road Association (PIARC), Paris/The World Bank: Washington, DC, USA.
Bennett, C. R. and W. D. Paterson (2003). Volume 5: HDM-4 Calibration Reference Manual. La Défense Cédex World Road Association (PIARC), Paris/The World Bank: Washington, DC, USA.
Bertrand, L., M. Boutonnet, J. Cazeneuve, J. Chabrol, M. . Dauzats, J. Griselin, A. Coquereau, J. Poilane, B. Robert and P. Lepert (1998). Catalogue des degradations de surface des chaussees-Version 1998. 58, boulevar Lefebvre - 75732, Paris, France, Laboratoire Central des Ponts et Chaussees.
Bonin, G., N. Folino, G. Loprencipe, G. Oliverio Rossi, S. Polizzotti and B. Teltayev (2017). Development of a Road Asset Management System in Kazakhstan. Proceedings of the TIS 2017 International Congress on Transport Infrastructure and Systems, Rome, Italy.
Bryce, J. M., G. W. Flintsch, S. W. Katicha and B. K. Diefenderfer (2012). "Developing a network-level structural capacity index for asphalt pavements." Journal of Transportation Engineering **139**(2): 123-129.
Bryce, J. M., G. W. Flintsch, S. W. Katicha, B. K. Diefenderfer and A. Sarant (2016). Development of Pavement Structural Capacity Requirements for Innovative Pavement Decision-Making and Contracting: Phase II, Virginia Transportation Research Council.
Cafiso, S. and A. Di Graziano (2012). "Definition of Homogenous Sections in Road Pavement Measurements." Siiv-5th International Congress - Sustainability of Road Infrastructures 2012 **53**: 1069-1078.
Cairney, P. (1997). Skid resistance and crashes: a review of the literature.
Cantisani, G., A. D'Andrea, P. Di Mascio and G. Loprencipe (2016). Reliance of Pavement Texture Characteristics on Mix-Design and Compaction Process, Dordrecht, Springer Netherlands.
Cantisani, G., G. Fascinelli and G. Loprencipe (2013). Urban road noise: The contribution of pavement discontinuities. ICSDEC 2012: Developing the Frontier of Sustainable Design, Engineering, and Construction**:** 327-334.
Cantisani, G. and G. Loprencipe (2010). "Road roughness and whole body vibration: Evaluation tools and comfort limits." Journal of Transportation Engineering **136**(9): 818-826.
Chamorro, A., S. Tighe, N. Li and T. Kazmierowski (2009). "Development of Distress Guidelines and Condition Rating to Improve Network Management in Ontario, Canada." Transportation Research Record: Journal of the Transportation Research Board **2093**: 128-135.
Chamorro, A., S. Tighe, N. Li and T. Kazmierowski (2010). "Validation and Implementation of Ontario, Canada, Network-Level Distress Guidelines and Condition Rating." Transportation Research Record: Journal of the Transportation Research Board **2153**: 49-57.
Chatti, K. and I. Zaabar (2012). Estimating the effects of pavement condition on vehicle operating costs, Transportation Research Board.
Chen, D. and N. Mastin (2016). "Sigmoidal Models for Predicting Pavement Performance Conditions." Journal of Performance of Constructed Facilities **30**(4): 04015078.
Chen, X., H. Zhu, Q. Dong and B. Huang (2017). "Optimal thresholds for pavement preventive maintenance treatments using LTPP data." Journal of Transportation Engineering, Part A: Systems **143**(6): 04017018.
Choi, J. W., R. Wu, J. M. Pestana and J. Harvey (2009). "New layer-moduli back-calculation method based on the Constrained Extended Kalman Filter." Journal of Transportation Engineering **136**(1): 20-30.
Chowdhury, T. (2012). A Guide to Evaluating Pavement Distress Through the Use of Video Images, Virginia Department of Transportation, Maintenance Division.
Coley, C. (2011). Inspections of Airfield Pavements - Practitioner Guide 06/11, UK Ministry of Defense.

Corazza, M. V., P. Di Mascio and L. Moretti (2016). "Managing sidewalk pavement maintenance: A case study to increase pedestrian safety." Journal of Traffic and Transportation Engineering (English Edition) **3**(3): 203-214.

Cottrell, W. D., S. Bryan, B. R. Chilukuri, V. Kalyani, A. Stevanovic and J. Wu (2009). "Transportation Infrastructure Maintenance Management: Case Study of a Small Urban City." Journal of Infrastructure Systems **15**(2): 120-132.

de la Garza, J. M., S. Akyildiz, D. R. Bish and D. A. Krueger (2011). "Network-level optimization of pavement maintenance renewal strategies." Advanced Engineering Informatics **25**(4): 699-712.

de León Izeppi, E., G. W. Flintsch, A. R. Archilla and W. Sequeira (2011). Continuous friction measurement equipment (CFME) data processing and analysis software. Transportation Research Board Annual Meeting, Washington DC, USA.

de León Izeppi, E., S. W. Katicha, G. W. Flintsch and K. K. McGhee (2015). "Pioneering the use of continuous pavement friction measurements to develop new Safety Performance Functions, improve the accuracy of crash count predictions, and evaluate possible treatments for the roads in Virginia."

de León Izeppi, E., A. Morrison, G. W. Flintsch and K. K. McGhee (2015). Best Practices and Performance Assessment for Preventive Maintenance Treatments for Virginia Pavements.

De Weille, J. (1966). Quantification of road user savings, The World Bank.

Diefenderfer, B. K., T. Chowdhury and R. Shekharan (2009). Implementation of a Network-Level Falling Weight Deflectometer Survey of Virginia's Interstate System. Bearing Capacity of Roads, Railways and Airfields. 8th International Conference (BCR2A'09) University of Illinois, Urbana-Champaign.

Dong, Q. and B. Huang (2011). "Evaluation of effectiveness and cost-effectiveness of asphalt pavement rehabilitations utilizing LTPP data." Journal of Transportation Engineering **138**(6): 681-689.

Dong, Q., B. Huang and S. H. Richards (2014). "Calibration and application of treatment performance models in a pavement management system in Tennessee." Journal of Transportation Engineering **141**(2): 1-9.

Dong, Q., B. Huang, S. H. Richards and X. Yan (2013). "Cost-effectiveness analyses of maintenance treatments for low-and moderate-traffic asphalt pavements in Tennessee." Journal of Transportation Engineering **139**(8): 797-803.

Drenth, K. P. (2001). Management of Dutch Airfield Pavements at Network Level. Advancing Airfield Pavements, 1801 Alexander Bell Drive Reston, VA USA 20191-4400, American Society of Civil Engineers ({ASCE}).

Drusin, S. (2014). Evaluating the Performance of New and In-service Pavements in Italy using High-Speed Non-Destructive Testing. Pavement Evaluation Conference 2014.

Durango-Cohen, P. L. and S. M. Madanat (2008). "Optimization of inspection and maintenance decisions for infrastructure facilities under performance model uncertainty: A quasi-Bayes approach." Transportation Research Part A: Policy and Practice **42**(8): 1074 - 1085.

Dynatest. (2018). Retrieved 05/05/2018, from https://www.dynatest.com/.

Efron, B. and C. Morris (1973). "Stein's Estimation Rule and its Competitors—An Empirical Bayes Approach." Journal of the American Statistical Association **68**(341): 117-130.

Ercisli, S. (2015). Development of Enhanced Pavement Deterioration Curves. Masters, Virginia Polytechnic Institute and State University.

Fernandes, A. and J. Neves (2014). "Threshold values of pavement surface properties for maintenance purposes based on accidents modelling." International Journal of Pavement Engineering **15**(10): 917-924.

FHWA (2005). Transportation Asset Management Case Studies; Data Integration: The Virginia Experience. Washington, D.C., USA, FHWA.

FHWA (2009). Pavement Distress Identification Manual for the National Park Service Road Inventory Program, United States. Federal Highway Administration. Office of Infrastructure Research and Development.

FHWA (2017). National performance management measures; assessing pavement condition for the National Highway Performance Program and bridge condition for the National Highway Performance Program,82 FR 5886. Federal Register. **2017-00550:** 5886-5970.

Flintsch, G., E. de Leon, K. McGhee and I. AI-Qadi (2003). "Pavement surface macrotexture measurement and applications." Transportation Research Record: Journal of the Transportation Research Board(1860): 168-177.

Flintsch, G. W., R. Dymond and J. Collura (2004). Pavement management applications using geographic information systems, National Academies Press.

Flintsch, G. W., S. W. Katicha, J. M. Bryce, B. Ferne, S. Nell and B. K. Diefenderfer (2013). Assessment of continuous pavement deflection measuring technologies.

Flintsch, G. W. and K. K. McGhee (2009). Quality Management of Pavement Condition Data Collection, National Academies Press.

Flintsch, G. W., K. K. McGhee, E. de León Izeppi and S. Najafi (2012). "The little book of tire pavement friction." Pavement Surface Properties Consortium.

Flora, W. F. (2009). Development of a structural index for pavement management: An exploratory analysis, Purdue University.

George, K. P., A. S. Rajagopal and L. K. Lim (1989). "Models for predicting pavement deterioration." Transportation Research Record(1215): 1-7.

Gharaibeh, N. G., T. Freeman, A. Wimsatt and Y. Zou (2011). Evaluation and Development of Pavement Scores, Performance Models and Needs Estimates: Phase I Activities, TxDOT Project 0-6386 Technical Report. Texas Transportation Institute, the Texas A&M University System, June.

Gharaibeh, N. G., Y. Zou and S. Saliminejad (2009). "Assessing the agreement among pavement condition indexes." Journal of Transportation Engineering **136**(8): 765-772.

Giummarra, G., T. Martin, Z. Hoque and R. Roper (2007). "Establishing deterioration models for local roads in Australia." Transportation Research Record(1989): 270-276.

Gulen, S., K. Zhu, J. Weaver, J. Shan and W. Flora (2001). "Development of improved pavement performance prediction models for the indiana pavement management system." Joint Transportation Research Program: 69.

Haider, S. and M. Dwaikat (2011). "Estimating optimum timing for preventive maintenance treatment to mitigate pavement roughness." Transportation Research Record: Journal of the Transportation Research Board(2235): 43-53.

Hall, J., K. L. Smith, L. Titus-Glover, J. C. Wambold, T. J. Yager and Z. Rado (2009). "Guide for pavement friction." Final Report for NCHRP Project: 01-43.

Han, D. and M. Do (2015). "Life Cycle Cost Analysis on pavement inspection intervals considering maintenance work delay." Ksce Journal of Civil Engineering **19**(6): 1716-1726.

Han, D. and K. Kobayashi (2013). "Criteria for the development and improvement of PMS models." KSCE Journal of Civil Engineering **17**(6): 1302-1316.

Hilbe, J. M. (2011). Negative binomial regression: modeling. Negative Binomial Regression, Cambridge University Press**:** 221–283.

Hong, F. and J. A. Prozzi (2006). "Estimation of Pavement Performance Deterioration Using Bayesian Approach." Journal of Infrastructure Systems **12**(2): 77-86.

Hossain, M., T. Chowdhury, S. Chitrapu and A. J. Gisi (2000). "Network-level pavement deflection testing and structural evaluation." Journal of Testing and Evaluation **28**(3): 199-206.

Huang, Y. H. (2004). "Pavement analysis and design."

Inc., S. C. and H. W. L. Inc. (2007). Development of performance prediction models for Virginia Department of Transportation pavement management system. Richmond,VA, Virginia Dept. of Transportation.

Jones, D. and J. T. Harvey (2005). "Relationship between DCP, Stiffness, Shear Strength, and R-value."
Katicha, S. W., S. Ercisli, G. W. Flintsch, J. M. Bryce and B. K. Diefenderfer (2016). Development of Enhanced Pavement Deterioration Curves. Virginia Transportation Research Council Charlottesville, VA United States, Virginia.
Katicha, S. W., G. W. Flintsch and J. M. Bryce (2016). "Modeling Pavement Condition and Deterioration: Linear Empirical Bayes Approach." (17-04619).
Katicha, S. W., G. W. Flintsch, J. M. Bryce and B. K. Ferne (2014). "Wavelet Denoising of TSD Deflection Slope Measurements for Improved Pavement Structural Evaluation." Computer-Aided Civil and Infrastructure Engineering **29**(6): 399-415.
Katicha, S. W., G. W. Flintsch and B. Ferne (2012). Estimation of pavement TSD slope measurements repeatability from a single measurement series. Proc. 91st Transp. Res. Board Annu. Meeting.
Katicha, S. W., G. W. Flintsch, S. Shrestha and S. Thyagarajan (2017). "Demonstration of Network Level Structural Evaluation with Traffic Speed Deflectometer." Final report. Prepared for the Federal Highway Administration, USDOT.
Khattak, M. J., G. Y. Baladi and X. Sun (2009). Development of index based pavement performance models for pavement management system (PMS) of LADOTD.
Livneh, M. (1989). "Validation of correlations between a number of penetration tests and in situ California bearing ratio tests." Transportation Research Record **1219**: 56-67.
Loprencipe, G. and A. Pantuso (2017). "A Specified Procedure for Distress Identification and Assessment for Urban Road Surfaces Based on PCI." Coatings **7**(5): 65.
Loprencipe, G., A. Pantuso and P. Di Mascio (2017). "Sustainable Pavement Management System in Urban Areas Considering the Vehicle Operating Costs." Sustainability **9**(3): 453.
Loprencipe, G. and P. Zoccali (2017). "Comparison of methods for evaluating airport pavement roughness." International Journal of Pavement Engineering: 1-10.
Loprencipe, G. and P. Zoccali (2017). "Use of generated artificial road profiles in road roughness evaluation." Journal of Modern Transportation **25**(1): 24-33.
Lukanen, E. O., R. Stubstad and R. Briggs (2000). Temperature Predictions and Adjustment Factors for Asphalt Pavements. Washington, D.C., FHWA.
Mačiulis, A., A. V. Vasiliauskas and G. Jakubauskas (2009). "The impact of transport on the competitiveness of national economy." Transport **24**(2): 93-99.
McGhee, K. H. (2002). Development And Implementation Of Pavement Condition Indices For The Virginia Department Of Transportation, Phase I Flexible Pavements. 1401 E. Broad St. Richmond, VA 23219, USA, Virginia Department of Transportation (VDOT).
McGhee, K. H. (2004). Automated Pavement Distress Collection Techniques, Transportation Research Board.
Miller, J. S. and W. Y. Bellinger (2014). Distress identification manual for the long-term pavement performance program, United States. Federal Highway Administration. Office of Infrastructure Research and Development.
Mills, L. N. O., N. O. Attoh-Okine and S. McNeil (2012). "Developing Pavement Performance Models for Delaware." Transportation Research Record **2304**(2304): 97-103.
Moretti, L. (2014). "Technical and economic sustainability of concrete pavements." Modern Applied Science **8**(3): 1.
Moretti, L., G. Cantisani and P. Di Mascio (2016). "Management of road tunnels: Construction, maintenance and lighting costs." Tunnelling and Underground Space Technology **51**: 84-89.
Moretti, L., G. Cantisani, P. Di Mascio and S. Caro (2017). "Technical and economic evaluation of lighting and pavement in Italian road tunnels." Tunnelling and Underground Space Technology **65**: 42-52.
Moretti, L., P. Di Mascio and A. D'Andrea (2013). "Environmental impact assessment of road asphalt pavements." Modern Applied Science **7**(11): 1.

Moretti, L., V. Mandrone, A. D'Andrea and S. Caro (2017). "Comparative "from cradle to gate" life cycle assessments of Hot Mix Asphalt (HMA) materials." Sustainability **9**(3): 400.

Mulry, B., K. Feighan, J. McCarthy and others (2015). Development and Implementation of a Simplified System for Assessing the Condition of Irish Regional and Local Roads. 9th International Conference on Managing Pavement Assets, Alexandria, VA, USA, Virginia Tech Transportation Institute (VTTI).

Mulry, B., M. Jordan, D. O'Brien and others (2015). Automated Pavement Condition Assessment Using Laser Crack Measurement System (LCMS) on Airfield Pavements in Ireland. 9th International Conference on Managing Pavement Assets, Alexandria, VA, USA, Virginia Tech Transportation Institute (VTTI).

Najafi, S., G. W. Flintsch and K. K. McGhee (2013). "Assessment of operational characteristics of continuous friction measuring equipment (CFME)." International Journal of Pavement Engineering **14**(8): 706-714.

Najafi, S., G. W. Flintsch and A. Medina (2017). "Linking roadway crashes and tire–pavement friction: a case study." International Journal of Pavement Engineering **18**(2): 119-127.

Nunez, M. M. and M. Y. Shahin (1986). "Pavement condition data analysis and modeling." Transportation Research Record(1070): 125-132.

ODoherty, J. (2007). At the Crossroads. Preserving Our Highway Investment, National Center for Pavement Preservation Michigan State University, 2857 Jolly Road Okemos, MI USA 48864.

Osorio, A., ,, A. Chamorro, S. Tighe and C. Videla (2014). "Calibration and Validation of Condition Indicator for Managing Urban Pavement Networks." Transportation Research Record: Journal of the Transportation Research Board **2455**: 28-36.

Ouyang, Y. and S. Madanat (2004). "Optimal scheduling of rehabilitation activities for multiple pavement facilities: exact and approximate solutions." Transportation Research Part A: Policy and Practice **38**(5): 347-365.

Paige-Green, P. and L. Du Plessis (2009). Use and interpretation of the dynamic cone penetrometer (DCP) test, CSIR.

Papagiannakis, A., N. Gharaibeh, J. Weissmann and A. Wimsatt (2009). "Pavement scores synthesis." College Station, TX: Texas Transportation Institute.

Papagiannakis, A. T. and E. A. Masad (2017). Pavement design and materials, John Wiley & Sons.

Paterson, W. D. and T. Scullion (1990). Information systems for road management: draft guidelines on system design and data issues.

Pavemetrics. (2018). Retrieved 05/05/2018, from http://www.pavemetrics.com/applications/road-inspection/laser-crack-measurement-system/.

PAVER (2014). User Manual - PAVERTM Version 7.0. Champaign, Illinois, US Army Corps of Engineers Engineer Research and Development Center (ERDC) Construction Engineering Research Laboratory (CERL).

Perera, R., S. Kohn and G. R. Rada (2008). LTPP manual for profile measurements and processing.

Peshkin, D. G. (2011). Guidelines for the preservation of high-traffic-volume roadways, Transportation Research Board.

Peshkin, D. G., T. E. Hoerner and K. A. Zimmerman (2004). Optimal Timing of Pavement Preventive Maintenance Treatment Applications, Transportation Research Board.

Pierce, L. M., G. McGovern and K. A. Zimmerman (2013). Practical guide for quality management of pavement condition data collection. 1200 New Jersey Avenue, SE Washington, DC USA 20590, Federal Highway Administration.

Pierce, L. M., G. McGovern and K. A. Zimmerman (2013). "Practical guide for quality management of pavement condition data collection."

Prozzi, J. A. and S. M. Madanat (2004). "Development of Pavement Performance Models by Combining Experimental and Field Data." Journal of Infrastructure Systems **10**(1): 9-22.

Puzzo, L., G. Loprencipe, C. Tozzo and A. D'Andrea (2017). "Three-dimensional survey method of pavement texture using photographic equipment." Measurement **111**: 146-157.
Rada, G. R., S. Nazarian, B. A. Visintine, R. Siddharthan and S. Thyagarajan (2016). Pavement structural evaluation at the network level.
Rens, K. L. and B. Staley (2009). "Rating City and County of Denver Urban Pavement Constructability Using a Profiler." Practice Periodical on Structural Design and Construction **15**(1): 44-49.
Roadware, F. (2018). Retrieved 05/05/2018, from http://www.roadware.com/products/software/.
Rohde, G. and T. Scullion (1990). "MODULUS 4.0: Expansion and validation of the MODULUS backcalculation system."
Rohde, G. T. (1994). "Determining pavement structural number from FWD testing." Transportation Research Record(1448).
Rydholm, T. C. and D. R. Luhr (2015). Visualizing pavement management data at the project level. 9th international conference on managing pavement assets.
Saha, P., K. Ksaibati and R. Atadero (2017). "Developing Pavement Distress Deterioration Models for Pavement Management System Using Markovian Probabilistic Process." Advances in Civil Engineering **2017**.
Sayers, M. W. and S. M. Karamihas (1998). "The little book of profiling." University of Michigan.
Shah, Y. U., S. S. Jain and M. Parida (2012). "Evaluation of prioritization methods for effective pavement maintenance of urban roads." International Journal of Pavement Engineering **15**(3): 238-250.
Shah, Y. U., S. S. Jain, D. Tiwari and M. K. Jain (2013). "Development of Overall Pavement Condition Index for Urban Road Network." Procedia - Social and Behavioral Sciences **104**: 332-341.
Shahin, M. Y. (2005). Pavement Management for Airports, Roads, and Parking Lots. New York, NY, USA, Springer Publishing Company.
Shahin, M. Y., N. M. M, M. R. Broten, C. S. H and A. Sameh (1987). "New techniques for modeling pavement deterioration." Transportation Research Record(1123): 40-46.
Shrestha, S., S. W. Katicha, G. W. Flintsch and S. Thyagarajan (2018). "Application of Traffic Speed Deflectometer for Network-Level Pavement Management." Transportation Research Record: 0361198118758675.
Sollazzo, G., K. C. Wang, G. Bosurgi and J. Li (2016). "Hybrid procedure for automated detection of cracking with 3D pavement data." Journal of Computing in Civil Engineering **30**(6): 04016032.
Stampley, B. E., B. Miller, R. Smith and T. Scullion (1995). "Pavement management information system concepts, equations, and analysis models."
Standardization, I. O. f. (1997). Mechanical vibration and shock-Evaluation of human exposure to whole-body vibration-Part 1: General requirements, The Organization.
Tabatabaee, N. and M. Ziyadi (2013). "Bayesian Approach to Updating Markov-Based Models for Predicting Pavement Performance." Transportation Research Record(2366): 34-42.
Tavakoli, A., M. S. Lapin and J. L. Figueroa (1992). "PMSC: Pavement Management System for Small Communities." Journal of Transportation Engineering **118**(2): 270-280.
Ullidtz, P. (1987). Pavement Analysis. Developments in Civil Engineering, 19.
Ullidtz, P. and R. N. Stubstad (1985). "Analytical-empirical pavement evaluation using the falling weight deflectometer." Transportation Research Record **1022**: 36-44.
USDOT (2017). National Performance Management Measures; Assessing Pavement Condition for the National Highway Performance Program and Bridge Condition for the National Highway Performance Program. U. D. o. Transportation. **Vol. 82, No. 11**.

Vavrik, W. R., L. D. Evans, J. A. Stefanski and S. Sargand (2013). "PCR Evaluation–Considering Transition from Manual to Semi-Automated Pavement Distress Collection and Analysis."

VDOT (2017). Fiscal Year 2018, VDOT Annual Budget**:** 32.

Visintine, B. A., A. L. Simpson and G. R. Rada (2018). Validation of Pavement Performance Measures Using LTPP Data.

Von Quintus, H. L., C. Rao and L. Irwin (2015). Long-Term Pavement Performance Program Determination of In-Place Elastic Layer Modulus: Backcalculation Methodology and Procedures.

Walker, D., L. Entine and S. Kummer (2002). Pavement Surface Evaluation and Rating: Paser Manual. 432 North Lake Street, Madison, WI 53706, Wisconsin Transportation Information Center.

Watanatada, T. (1987). The Highway Design and Maintenance Standards Model: User's manual for the HDM-III model, Johns Hopkins University Press.

Wolters, A., K. A. Zimmerman, K. Schattler and A. Rietgraf (2011). Implementing pavement management systems for local agencies—State-of-the-art/state-of-the-practice synthesis.

Wolters, A. S. and K. A. Zimmerman (2010). Research of Current Practices in Pavement Performance Modeling, Pennsylvania Department of Transportation.

Wu, Z., G. W. Flintsch and T. Chowdhury (2008). "Hybrid Multiobjective Optimization Model for Regional Pavement-Preservation Resource Allocation." Transportation Research Record **2084**(1): 28-37.

Zaniewski, J. P., B. Butler, G. Cunningham and R. Machemehl (1982). Vehicle operating costs, fuel consumption, and pavement type condition factors, United States. Federal Highway Administration.

Zellner, A. (1996). An Introduction to Bayesian Inference in Econometrics, John Wiley \& Sons Inc.

Zhang, Z., G. Claros, L. Manuel and I. Damnjanovic (2003). Evaluation of the pavement structural condition at network level using Falling Weight Deflectometer (FWD) data. 82nd Transportation Research Board meeting, Washington, DC, USA.

Zhang, Z., L. Manuel, I. Damnjanovic and Z. Li (2003). "Development of a new methodology for characterizing pavement structural condition for network-level applications." Texas Dept. of Transportation, Austin, TX.

Zhou, G., L. Wang, D. Wang and S. Reichle (2009). "Integration of GIS and data mining technology to enhance the pavement management decision making." Journal of Transportation Engineering **136**(4): 332-341.

Zimmerman, K. A. (2017). Pavement Management Systems: Putting Data to Work, Transportation Research Board.

APPENDIX A. DISTRESS IDENTIFICATION MANUAL FOR URBAN ROAD PAVEMENTS

This Appendix includes a list of the new distresses defined in the proposed Distress Identification Catalogue for urban road pavements. Besides, it contains guidelines for distress identification and severity level assessment and provides recommendations to conduct the pavement survey. Deduct Value Curves, pavement inspection sheets and representative distress characteristics photos have been provided.

Preface

This manual has been prepared as a result of research with the following objective, the improvement of the ASTM D6433 Distress Identification Catalogue to be adapted to urban road surfaces. ASTM Definitions *1. Alligator Cracking, 2. Block Cracking, 5. Rutting, 7. Potholes, 8. Patching and 9. Railway Crossing,* has been not modified in the distress identification manual for urban road pavements, and original ASTM definitions have been reported unchanged in this manual as *1. Alligator Cracking, 3. Block Cracking, 15 Rutting, 13. Potholes, 14. Railway crossing, 11. Patching and Utility Cut,* respectively.

References:

ASTM. (2018). ASTM D6433-18, Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys. In. West Conshohocken, PA: ASTM International.

Loprencipe, G., & Pantuso, A. (2017). A Specified Procedure for Distress Identification and Assessment for Urban Road Surfaces Based on PCI. *Coatings*, *7*(5), 65.

Distress Identification Guidelines

1. Alligator Cracking

Description:
Alligator or fatigue cracking is a series of interconnecting cracks caused by fatigue failure of the asphalt concrete surface under repeated traffic loading. Cracking begins at the bottom of the asphalt surface or stabilized base, where tensile stress and strain are highest under a wheel load. The cracks propagate to the surface initially as a series of parallel longitudinal cracks. After repeated traffic loading, the cracks connect, forming many sided, sharp-angled pieces that develop a pattern resembling chicken wire or the skin of an alligator. The pieces are generally less than 0.5 m (1.5 ft) on the longest side. Alligator cracking occurs only in areas subjected to repeated traffic loading, such as wheel paths. Pattern-type cracking that occurs over an entire area not subjected to loading is called “block cracking,” which is not a load associated distress.

Severity levels:

- **L**: Fine, longitudinal hairline cracks running parallel to each other with no, or only a few interconnecting cracks. The cracks are not spalled (Figure A- 1a).
- **M**: Further development of light alligator cracks into a pattern or network of cracks that may be lightly spalled (Figure A- 1b).
- **H**: Network or pattern cracking has progressed so that the pieces are well defined and spalled at the edges. Some of the pieces may rock under (Figure A- 1c).

Alligator cracking is measured in Square Meters.

(**a**) (**b**) (**c**)

Figure A- 1. Alligator Cracking (ASTM D6433 definition). (a) Low Severity; (b) Medium Severity; and (c) High Severity.

2. Block Cracking

Description:
Block cracks are interconnected cracks that divide the pavement into approximately rectangular pieces. The blocks may range in size from approximately 0.3 by 0.3 m (1 by 1 ft) to 3 by 3 m (10 by 10 ft). Block cracking is caused mainly by shrinkage of the asphalt concrete and daily temperature cycling, which results in daily stress/strain cy- cling. It is not load-associated. Block cracking usually indicates that the asphalt has hardened significantly. Block cracking normally occurs over a large portion of the pavement area, but sometimes will occur only in non-traffic areas. This type of distress differs from alligator cracking in that alligator cracks levels of severity cannot be divided easily; the entire area should be rated at the highest severity present. If alligator cracking and rutting occur in the same area, each is recorded separately as its respective severity level.
Severity levels:

- **L**: Blocks are defined by low-severity cracks (Figure A- 2a).
- **M**: Blocks are defined by medium-severity cracks (Figure A- 2b).
- **H**: Blocks are defined by medium-severity cracks (Figure A- 2c).

Block cracking distress is measured in Square meters.

(**a**) (**b**) (**c**)

Figure A- 2. Block cracking (ASTM D6433 Definition). (a) Low Severity; (b) Medium Severity; and (c) High Severity.

3. Linear and Isolated Cracking

Description:
Linear and isolated cracks are parallel to the pavement or perpendicular of laydown direction. They may be caused by a poorly constructed paving lane joint, shrinkage of the AC surface due to low temperature or hardening of the asphalt, or daily temperature cycling, or both. The distress is accelerated by traffic loading. Transverse cracks extend across the pavement at approximately right angles to the pavement centerline or direction of laydown. These types of cracks are not usually load associated.

Severity levels:

- **L**: mild cracking (if not sealed lower amplitude of 1 cm, if sealed of any amplitude), possibly due to the opening of a joint, without crumbling (Figure A- 3a).
- **M**: Average cracking (they can be unsealed amplitude between 1 and 7.5 cm, or not sealed amplitude <7.5 cm surrounded by cracking read, or still can be sealed of any width surrounded by light cracking), possibly due to the opening of a joint with little crumbling (Figure A- 3b).
- **H**: any slot, sealed or not, surrounded by medium or high cracking; slot unsealed >7.5 cm; cracking of any amplitude with about 10 cm of severely damaged surrounding paving. We are in this level even if there is a considerable crumbling; if there are any joints are completely open.

The linear and isolated cracking is measured in linear meters (Figure A- 3c).

(**a**) (**b**) (**c**)

Figure A- 3. Linear and isolated cracking. (a) Low Severity; (b) Medium Severity; and (c) High Severity.

4. Surface Deformations

Description:

Swellings, depressions and spaced crests of the road surface that generally are developed parallel or orthogonal direction of traffic laydown direction. This type of distress can be caused by a surface or unstable pavement surface combined by tangential actions of the traffic, in other cases, it can be the results of a failure of the substrate. More generally, the main cause of this distress in urban areas should be the wrong construction of pavement.

The severity level of this distress is determined by the criterion of driving quality:

- **L**: High ride quality, depth of pavement depressions between 1 and 2.5 cm (Figure A- 4a).
- **M**: Medium ride quality, depth of pavement depressions between 2.5 and 5 cm (Figure A- 4b).
- **H**: badly ride quality, the depth of pavement depressions is more than 5 cm (Figure A- 4c).

The surface deformations are measured in square meters.

(a) (b) (c)

Figure A- 4. Surface deformations. (a) Low Severity; (b) Medium Severity; and (c) High Severity.

5. Rutting

Description:
A rut is a surface depression in the wheel paths. Pavement uplift may occur along the sides of the rut, but, in many instances, ruts are noticeable only after a rainfall when the paths are filled with water. Rutting stems from a permanent deformation in any of the pavement layers or subgrades, usually caused by consolidated or lateral movement of the materials due to traffic load.
Severity levels:

- **L**: Mean Rut Depth; 6 to 13 mm (1⁄4 to 1⁄2 in.) (Figure A- 5Figure A- 3a).
- **M**: Mean Rut Depth; > 13 to 25 mm (>1⁄2 to 1 in.) (Figure A- 5b).
- **H**: Mean Rut Depth; >25 mm (>1 in.) (Figure A- 5c).

The rutting distress is measured in square meters of surface area; and its severity is determined by the mean depth of the rut.

(a) (b) (c)

Figure A- 5. Rutting (ASTM D6433 Definition). (a) Low Severity; (b) Medium Severity; and (c) High Severity.

6. Failed Surface Grip

Description:
This type of distress can cause alteration in the status of surface conditions with direct consequences on the supply of skid resistances. This distress includes the loose of coarse aggregate of pavement surface, in general, these losses are caused by repeated traffic cycles and the formation of a film of bituminous material on the pavement surface that creates a glossy, reflective and shiny surface. The causes of this distress are excess of bitumen, low percentage of voids, bad construction or insufficient connection between the upper layer and the one below that a poor-quality or poor compacted mixture is present.
Note. The ASTM D6433 distress; 20 *Weathering* represent the surface deterioration by effect of worn out of asphalt concrete surface, the deduct value of this distress are very low and they can be included on the new distress definition failed surface grip (206).

Severity level:

- **L.** Sticky surface only a few days a year, significant sanding aggregates, aggregates and bitumen begin to be removed (Figure A- 6a).
- **M.** Sticky surface few weeks a year or erosion level such as to have a moderately wrinkled texture (Figure A- 6b).
- **H.** Sticky surface for at least a few weeks a year, or very rough texture (Figure A- 6c).

The failed surface grip distress is measured in square meters.

(a) **(b)** **(c)**

Figure A- 6. Failed surface grip. (a) Low Severity; (b) Medium Severity; and (c) High Severity.

7. Potholes

Description:
Potholes are small—usually less than 750 mm (30 in.) in diameter—bowl-shaped depressions in the pavement surface. They generally have sharp edges and vertical sides near the top of the hole. When holes are created by high-severity alligator cracking, they should be identified as potholes, not as weathering.
Severity levels:
If the pothole is more than 750 mm (30 in.) in diameter, the area should be determined in square feet and divided by 0.5 m2 (5.5 ft2) find the equivalent number of holes. If the depth is 25 mm (1 in.) or less, the holes are considered medium-severity. If the depth is more than 25 mm (1 in.), they are considered high-severity (Figure A- 7).

Potholes are measured as the number that are in surveyed section.

(**a**) (**b**) (**c**)

Figure A- 7. Potholes (ASTM D6433 Definition). (a) Low Severity; (b) Medium Severity; and (c) High Severity.

8. Patching

Description:
A patch is an area of pavement that has been replaced with new material to repair the existing pavement. A patch is considered a defect no matter how well it is performing (a patched area or adjacent area usually does not perform as well as an original pavement section). Generally, some roughness is associated with this distress.
Severity levels:

- **L**: Patch is in good condition and satisfactory. Ride quality is rated as low severity or better (Figure A- 8a).
- **M**: Patch is moderately deteriorated, or ride quality is rated as medium severity, or both (Figure A- 8b).
- **H**: Patch is badly deteriorated, or ride quality is rated as high severity, or both; needs replacement soon (Figure A- 8c).

Patching is rated in ft^2 of surface area; however, if a single patch has areas of differing severity, these areas should be measured and recorded separately. For example, a 2.5 m^2 (27.0 ft^2) patch may have 1 m^2 (11 ft^2) of medium severity and 1.5 m^2 (16 ft^2) of low severity. These areas would be recorded separately. Any distress found in a patched area will not be recorded; however, its effect on the patch will be considered when determining the patch's severity level. No other distresses, for example, are recorded within a patch. Even if the patch material is shoving or cracking, the area is rated only as a patch. If a large amount of pavement has been replaced, it should not be recorded as a patch but considered as new pavement, for example, replacement of a complete intersection.

(**a**) (**b**) (**c**)

Figure A- 8. Patching and Utility Cut Patching (ASTM D6433 Definition). (a) Low Severity; (b) Medium Severity; and (c) High Severity.

9. Railway Crossing

Description:
Railroad crossing defects are depressions or bumps around, or between tracks, or both.
Severity levels:

- **L**: Railroad crossing causes low-severity ride quality (Figure A- 9a).
- **M**: Railroad crossing causes medium-severity ride quality (Figure A- 9b).
- **H**: Railroad crossing causes high-severity ride quality (Figure A- 9c).

The area of the crossing is measured in square meters (square feet) of surface area. If the crossing does not affect ride quality, it should not be counted. Any large bump created by the tracks should be counted as part of the crossing.

(**a**) (**b**) (**c**)

Figure A- 9. Railway Crossing(ASTM D6433 Definition). (a) Low Severity; (b) Medium Severity; and (c) High Severity.

10. Slippage Cracking (Sliding Deformations)

Description:
This type of distress is characterized by a permanent sliding in the longitudinal direction of a localized area of road surface, located in areas in which the mixture used is too fluid and unstable.
Severity levels:

- **L**. Good ride quality and / or crack amplitude <1.0 cm (Figure A- 10a).
- **M.** Average ride quality and/or amplitude of 1.0 to 4.0 cm crack; the surrounding area is moderately crushed (Figure A- 10b).
- **H.** Low ride quality and/or crack amplitude >4 cm; the surrounding area is highly cracked into removable pieces (Figure A- 10c).

This type of distress is measured in square meters.

(a) **(b)** **(c)**

Figure A- 10. Slippage cracking. (a) Low Severity; (b) Medium Severity; and (c) High Severity.

11. Manholes

Description:
This distress definition includes all artificial elements that are on the urban pavement, designed to collect rain water, or allow access to the area below the roadway. These distress definition includes manholes and catch basins.
Severity levels:

- **L.** Low influence on ride quality (Figure A- 11a).
- **M.** Medium influence have medium influence on ride quality with an elevation of the element less than 5 centimeters over the road level and the surrounding area is slightly cracked (Figure A- 11b).

- **H.** High influence in ride quality, the element creates a vertical discontinuity of the roadway, height over the roadway level ≥ 5 cm and the surrounding area is significantly cracked.

This type of distress is measured in square meters (Figure A- 11c).

(a) **(b)** **(c)**

Figure A- 11. Manholes. (a) Low Severity; (b) Medium Severity; and (c) High Severity.

12. Tree Roots

Description:
This distress is defined as a localized lifting of the road surface due to the present of large tree roots at the road side: this type of distress is very common in urban areas. They can quickly degenerate into cracks, which widen as a result of infiltration of rainwater and the growth of the roots below the road level. The density-deduct value curve associated with 12. *Tree roots* is calculated as the combination of ASTM D6433 11. *Patching and Utility Patching* and 18. *Swell.*
Severity levels:
The severity level of this distress should be considered by Figure A- 12, representative photos of new distress definitions are given in Figure A- 13.

Height (cm) above road surface	*Low Traffic influence*	*Medium Traffic Influence*	*High Traffic Influence*
0–10	Low Severity	Low Severity	Medium Severity
10–20	Low Severity	Medium Severity	High Severity
≥20 cm	Medium Severity	Medium Severity	High Severity

Figure A- 12. Tree roots severity levels.

The severity level is defined with the table which relates the height difference between the tree root and the road level and the influence of roots have in the traffic. The definition of severity level is also based on the position of roots roadway section, to take into account if the roots are present where the vehicles pass transversally or if they are present on the roadside.
This type of distress is measured in square meters; other type of distress in surrounding area has not to be consider as tree roots.

(a) **(b)** **(c)**

Figure A- 13. Tree roots. (a) Low Severity; (b) Medium Severity; and (c) High Severity.

Pavement Inspection Sheet

DISTRESS IDENTIFICATION MANUAL FOR URBAN ROAD PAVEMENTS
SURVEY DATA SHEET FOR SAMPLE UNIT

Notes:

Branch Section Sample unit

Surveyed by Date Sample area

1. Alligator Cracking	6. Failed Surface Grip	11. Manholes
2. Block Cracking	7. Potholes	12. Tree Roots
3. Linear and Isolated Cracking	8. Patching	
4. Surface Deformations	9. Railway Crossing	
5. Rutting	10. Slippage Cracking (Sliding Deformation)	

DISTRESS SEVERITY	QUANTITY (linear meters, square meters, or number)								TOTAL	DENSITY %	DEDUCT VALUE

SAMPLE UNIT SKETCH

Deduct Value Master Curves

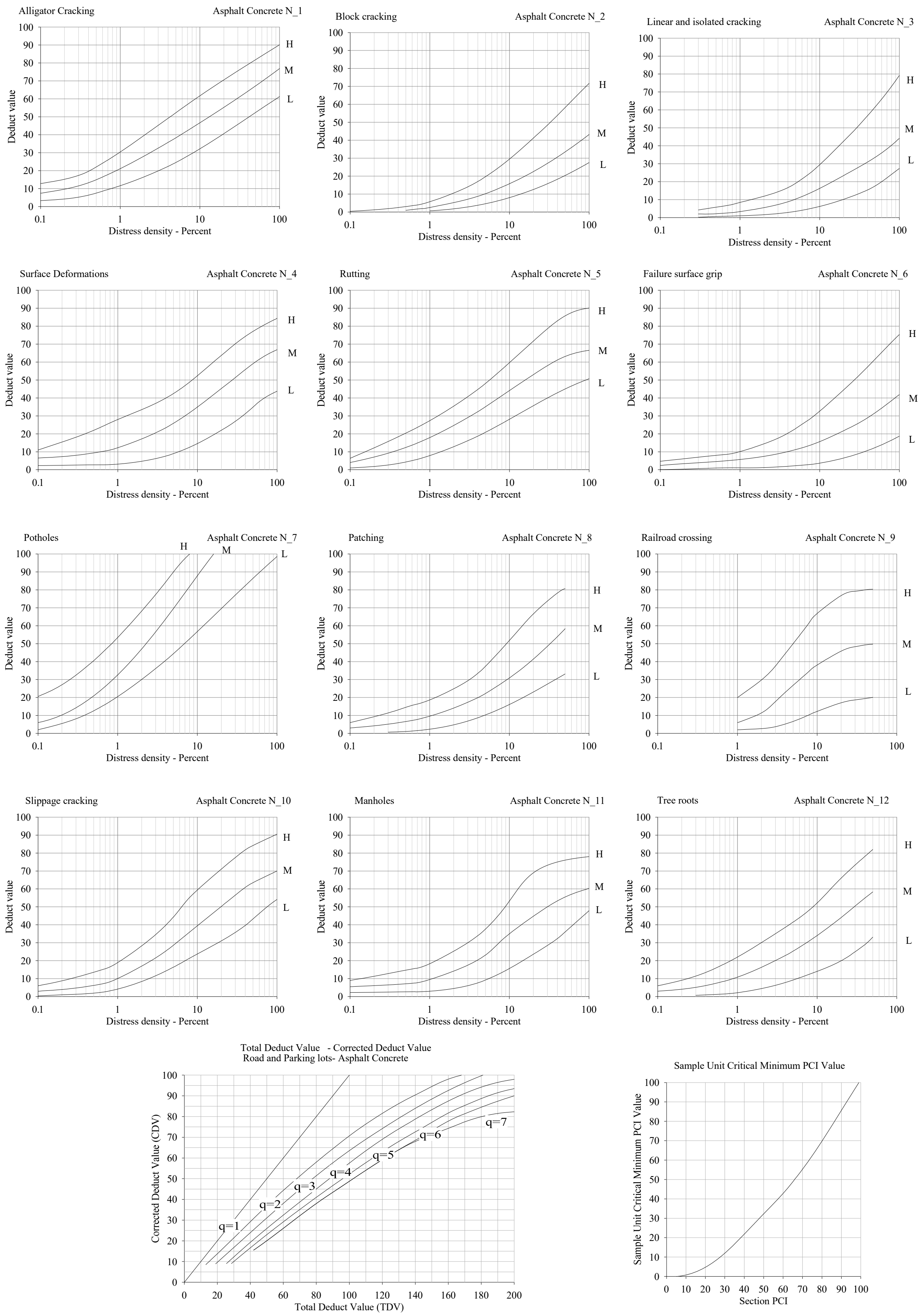


Figure A- 14. Deduct Value Curves and Correct Deduct Value Diagrams

APPENDIX B. SOFTWARE CALCULATION PCI DOCUMENTATION

This Appendix describes the use of the software tool developed in Visual Basic for Application (VBA) and implemented in Microsoft ® Excel. The software was developed for the calculation of Pavement Condition Index (PCI) according to the ASTM D6433 for flexible pavements, and for the calculation of PCI for Urban Areas using distress identification manual developed by Loprencipe and Pantuso, 2017.
Figure B- 1 depicts the program interface that consists of a data entry module on the left side where distress type, severity and quantity needs to be provided. Distress type are included by their designation code – 1-20 for flexible pavements using ASTM D6433 methodology or 201-212 for flexible pavement using Loprencipe & Pantuso, 2017 methodology – distress severity is entered as L- Low, M – Medium and H – High.

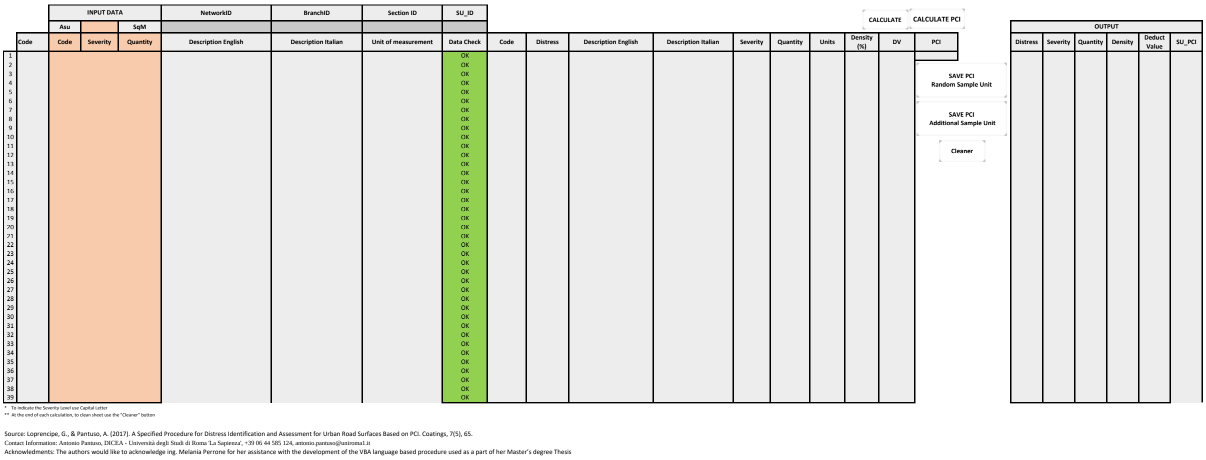


Figure B- 1. PCI calculation program interface implemented in Microsoft® Excel

Once the data has been entered, the user must pulse the button in order to run the code that calculated the Deduct Value (DV) associated for each distress type, severity and quantity introduced. On the central part of the interface, unique distress type and severity have been grouped and distress quantities have been summed up, the DV results are summed up in the DV column. For the calculation of the PCI, the user must click the button ("CALCULATE PCI"). The output of the calculation is summarized in the right size of software interface (see Figure B- 2).

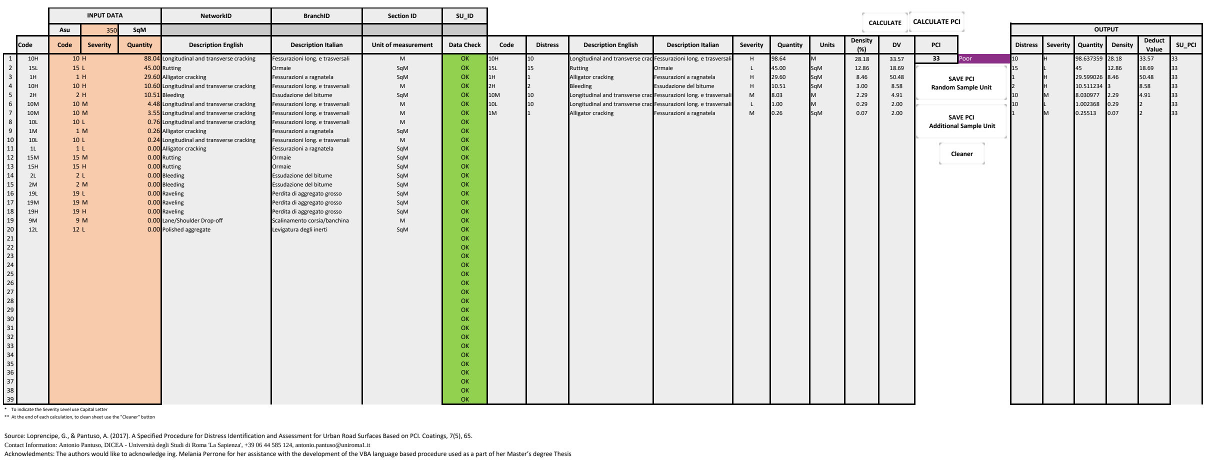


Figure B- 2. Output of the calculation of PCI using the VBA software

References:

ASTM. (2018). ASTM D6433-18, Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys. In. West Conshohocken, PA: ASTM International.

Loprencipe, G., & Pantuso, A. (2017). A Specified Procedure for Distress Identification and Assessment for Urban Road Surfaces Based on PCI. *Coatings*, *7*(5), 65.

APPENDIX C. CODE USED IN CHAPTER 6

This appendix documents the Excel VBA Functions used in Chapter 6: Analysis of Pavement Condition Survey Data for Effective Implementation of a Network Level Pavement Management Program For a Developing Country to process FWD deflection data.

These functions are summarized in the following list:

1. Function ***Dx***. This function is used to interpolate the deflection sensors measurements using the closest three deflection sensors.

```
Function Dx(Rx As Single, ind As Single, D1st As Range, R1st As Range) As Variant
    Dim Da As Single
    Dim Db As Single
    Dim Dc As Single
    Dim Ra As Single
    Dim Rb As Single
    Dim Rc As Single


If ind = 1 Then
        Ra = Cells(R1st.Row, R1st.Column)
        Rb = Cells(R1st.Row, R1st.Column + 1)
        Rc = Cells(R1st.Row, R1st.Column + 2)
        Da = Cells(D1st.Row, D1st.Column)
        Db = Cells(D1st.Row, D1st.Column + 1)
        Dc = Cells(D1st.Row, D1st.Column + 2)

    ElseIf ind = 9 Then
        Ra = Cells(R1st.Row, R1st.Column + ind - 2)
        Rb = Cells(R1st.Row, R1st.Column + ind - 1)
        Rc = Cells(R1st.Row, R1st.Column + ind)
        Da = Cells(D1st.Row, D1st.Column + ind - 2)
        Db = Cells(D1st.Row, D1st.Column + ind - 1)
        Dc = Cells(D1st.Row, D1st.Column + ind)

    Else
        Ra = Cells(R1st.Row, R1st.Column + ind - 1)
        Rb = Cells(R1st.Row, R1st.Column + ind)
        Rc = Cells(R1st.Row, R1st.Column + ind + 1)
        Da = Cells(D1st.Row, D1st.Column + ind - 1)
        Db = Cells(D1st.Row, D1st.Column + ind)
        Dc = Cells(D1st.Row, D1st.Column + ind + 1)
    End If

    Dx = ((Rx - Rb) * (Rx - Rc)) / ((Ra - Rb) * (Ra - Rc)) * Da + (((Rx - Ra) * (Rx - Rc)) /
((Rb - Ra) * (Rb - Rc))) * Db + (((Rx - Ra) * (Rx - Rb)) / ((Rc - Ra) * (Rc - Rb))) * Dc

    End Function
```

2. Function ***ESALt***. This function is used to determine the cumulated ESAL number for a section and after certain number of years "t" as a function of the daily ESAL

number, traffic lane distribution coefficient (trfLaneDistr), and growth rate (rategrowth) assumed.

```
Function ESALt(ESALdaily As Single, t As Single, trfLaneDistr As Single, rategrowth As Single) As Variant

ESALt = (ESALdaily * 365 * rategrowth * trfLaneDistr) + t * (ESALdaily * 365 * rategrowth * trfLaneDistr)

End Function
```

3. Function ***RL***. This function is used to determine residual remaining life of a pavement section based on AASHTO 93 calculated ESAL end of life (ESALendlife), daily ESAL of the section (ESALdaily), traffic lane distribution coefficient, and growth rate (rategrowth) assumed.

```
Function RL(ESALendlife As Single, ESALdaily As Single, trfLaneDistr As Single, rategrowth As Single) As Variant

For i = 1 To 120
RL = i
If ESALendlife < (ESALdaily * 365 * rategrowth * trfLaneDistr) + i * (ESALdaily * 365 * rategrowth * trfLaneDistr) Then Exit For

Next i

End Function
```

4. Function ***ATAFDefault.*** This function is used to correct measured asphalt temperature to the standard temperature of FWD test.

```
Function ATAFDefault(sTempMeasured As Single, sTempRef As Single) As Single
Dim sSlope As Single
sSlope = -0.021
ATAFDefault = 10 ^ (sSlope * (sTempRef - sTempMeasured))
End Function
```

5. Function ***customaverageHS.*** This function is used to calculate analysis parameters within a specific range that delimitate homogeneous sections (upper limit, lower limit).

```
Function customaverageHS(col As Range, upper As Single, lower As Single) As Variant
Dim rng As Range
Set rng = Range(Cells(upper, col.Column), Cells(lower, col.Column))
Dim cell As Range, total As Double, count As Integer
For Each cell In rng
    total = total + cell.Value
```

```
    count = count + 1
    Next cell
customaverageHS = total / count
End Function
```

6. Function ***locationHS***. This function is used to localize the position of a position (km) within a set of homogeneous sections locations range (rng).

```
Function locationHS(rng As Range, km As Integer) As Variant
Dim cell As Range
locationHS = 1
For Each cell In rng
If cell.Value <= km Then
locationHS = locationHS + 1
ElseIf cell.Value >= km Then Exit For
End If
Next cell
End Function
```

APPENDIX D. CODE USED IN CHAPTER 7

This appendix documents the regression modelling codes used in Chapter 7: Development of Pavement deterioration curves of flexible pavements using the Linear Empirical Bayesian Approach at the Network Level for fitting of pavement condition vs age dataset (age,CCI) using Generalized Linear Models (GLM) to model the deterioration of pavement homogeneous families using currently-in-use functions by Virginia Department of Transportation and comparing the results with the Negative Binomial regression model developed in this research and presented in Chapter 7. As an output, the programming code produces the parameters of each regression type and provides goodness-of-fit indicators (in a '.txt' file) and the production of a graphic comparing models between them.

The models evaluated are enumerated in the following bulleted list:

1. Power model, Eq. D- 1

$$CCI_{\mathrm{P\_model}} = a \cdot age^{b} + c \qquad \text{Eq. D- 1}$$

2. Sigmoidal Model, Eq. D- 2

$$CCI_{\mathrm{S\_model}} = 100 - d \cdot \exp\left(\frac{-e}{age}\right) \qquad \text{Eq. D- 2}$$

3. VDOT Model, Eq. D- 3

$$CCI_{\mathrm{VDOT\_model}} = 100 - \exp\left(f + g \cdot h^{\left(\frac{1}{age}\right)}\right) \qquad \text{Eq. D- 3}$$

4. Linear Model, Eq. D- 4

$$CCI_{\mathrm{L\_model}} = j \cdot age + k \qquad \text{Eq. D- 4}$$

5. Negative Binomial, Eq. D- 5

$$CCI_{\mathrm{NB\_model}} = 100 - age^{\beta_2} \cdot \exp(\beta_0 + \beta_1 \cdot age), \qquad \text{Eq. D- 5}$$

The code was written into a MATLAB® function that makes simpler their use for making the analysis for pavement homogeneous families into a sequential mode.

Contents

- Initialization.

- Fit: 'Power'.
- Fit: 'Linear'.
- Fit: 'Sigmoidal'.
- Fit: 'VDOT'.
- Negative Binomial regression
- Plot all the fit models together to compare them

```
function outfit= fit_glmVDOTFamily(age,CCI,directory,HomogeneousFamilyId)
% This function fits the dataset of each pavement family and gives as
% outputs the equation model, the fit parameters, the bound parameters and
% the goodness of fit of all the fits and gives as a result a .txt file

% See also Matlab Functions nbreg() -https://it.mathworks.com/matlabcentral/fileexchange/40642-negative-binomial-regression- and fit() - https://it.mathworks.com/help/curvefit/fit.html.
```

Initialization.

```
% Initialize arrays to store fits and goodness-of-fit.
fitresult = cell( 4, 1 );
gof = struct( 'sse', cell( 5, 1 ), ...
    'rsquare', [], 'dfe', [], 'adjrsquare', [], 'rmse', [] );
```

Fit: 'Power'.

```
[xData, yData] = prepareCurveData( age, CCI );

% Set up fittype and options.
ft = fittype( 'power2' );
opts = fitoptions( 'Method', 'NonlinearLeastSquares' );
opts.Display = 'Off';
opts.StartPoint = [95.0526022183619 -0.0469148465751339 0.112906483354135];



% Fit model to data.
[fitresult{1}, gof(1)] = fit( xData, yData, ft, opts );
 syms x
hpower=fitresult{1}.a*x^fitresult{1}.b+fitresult{1}.c;
fpower=matlabFunction(hpower);

% Plot fit with data.
figure( 'Name', 'Power ' )
plot( fitresult{1}, xData, yData,'predobs' );
legend('cci vs. age', 'Power','Lower bounds (Power)', 'Upper bounds (Power)', 'Location', 'SouthWest' );
% Label axes
title('Power ')
xlabel AGE
ylabel CCI
xlim auto
ylim([0 100]);
set(gca,'FontSize',8.5);
grid on
```

Fit: 'Linear'.

```
[xData, yData] = prepareCurveData( age, CCI );

% Set up fittype and options.
ft = fittype( 'poly1' );
```

```
% Fit model to data.
[fitresult{2}, gof(2)] = fit( xData, yData, ft);
syms x
hlin=fitresult{2}.p2+fitresult{2}.p1*x;
flin=matlabFunction(hlin);
% Create a figure for the plots.
figure( 'Name', 'Linear '  )

% Plot fit with data.
h = plot( fitresult{2}, xData, yData, 'predobs' );
legend( h, 'cci vs. age', 'Linear', 'Lower bounds (Linear)', 'Upper bounds (Linear)', 'Location', 'SouthWest' );
% Label axes
title(['Linear '])
xlabel AGE
ylabel CCI
xlim auto
ylim ([0 100]);
grid on
```

Fit: 'Sigmoidal'.

```
[xData, yData] = prepareCurveData( age, CCI );
% Set up fittype and options.
ft = fittype( '100-a*exp(-b/x)', 'independent', 'x', 'dependent', 'y' );
opts = fitoptions( 'Method', 'NonlinearLeastSquares' );
opts.Display = 'Off';
opts.MaxFunEvals = 200000;
opts.MaxIter = 200000;
opts.StartPoint = [.5 .8];


% Fit model to data.
[fitresult(336), gof(3)] = fit( xData, yData, ft, opts );
syms x
hsigmoidal=100-fitresult(336).a*exp((-(fitresult(336).b)/x)^1);
fsigmoidal=matlabFunction(hsigmoidal);
% Plot fit with data.
figure( 'Name', 'Sigmoidal '  )
h = plot( fitresult(336), xData, yData, 'predobs' );
legend( h, 'cci vs. age', 'Sigmoidal','Lower bounds (Sigmoidal)', 'Upper bounds (Sigmoidal)', 'Location', 'SouthWest' );
% Label axes
title('Sigmoidal ')
xlabel AGE
ylabel CCI
xlim auto
ylim ([0 100]);
grid on
```

Fit: 'VDOT'.

```
[xData, yData] = prepareCurveData( age, CCI );
% Set up fittype and options.
ft = fittype( '100-exp(a+b*c^(log(1/x)))', 'independent', 'x', 'dependent', 'y' );
opts = fitoptions( 'Method', 'NonlinearLeastSquares' );
opts.Display = 'Off';
opts.MaxFunEvals = 200000;
```

```
opts.MaxIter = 200000;
opts.StartPoint = [-10 11 1];
opts.Upper = [20 20 20];

% Fit model to data.
[fitresult{4}, gof(4)] = fit( xData, yData, ft, opts );
syms x
hvdot=100-exp(fitresult{4}.a+fitresult{4}.b*fitresult{4}.c^log(1/x));
fvdot=matlabFunction(hvdot);

% Plot fit with data.
figure( 'Name', 'VDOT model '  )
h = plot( fitresult{4}, xData, yData, 'predobs' );
legend( h, 'cci vs. age', 'VDOT model','Lower bounds (VDOT model)', 'Upper bounds (VDOT model)', 'Location', 'SouthWest' );
% Label axes
title(['VDOT model '])
xlabel AGE
ylabel CCI
xlim auto
ylim ([0 100]);
grid on
```

Negative Binomial regression

Eq. CCI=100-DI=100-exp(b0+b1*log(age)) define the vector intersect as a vector of length the size of the data to fit (DI) and the use of nbreg function on matlab to do that

```
% define intercept as a vector of ZEROS
intercept=zeros(size(age,1),1);
for i=1:length(intercept)
   intercept(i)=1;
end
% create the matrix X with the coefficient to use the negative binomial
% regression according to the definition of the model that want to be
% approximate
X=[intercept age log(age)];
Y=nbreg(X,100-CCI); % application of function nbreg gives as a result a struct with the overdispersion parameter (alpha), the parameters of the model under the variable b
DI=100-CCI;

syms x
CCInbreg=100-exp(Y.b(1,1)+Y.b(2,1)*x+Y.b(3,1)*log(x));
fnbreg=matlabFunction(CCInbreg);
ytot=0;
for i=1:length(age)
   ytot=ytot+DI(i);
end
ymed=(1/(length(age)))*ytot;

SSres=0;
for i=1:length(age)
   SSres=SSres+((DI(i)-(100-fnbreg(age(i))))^2);
end

SStot=0;
for i=1:length(age)
   SStot=SStot+((DI(i)-ymed)^2);
end
```

```
R2=1-(SSres/SStot);
rmsenb=sqrt(SStot/length(age));

gof(5).sse=SSres;
gof(5).rsquare=R2;
gof(5).rmse=rmsenb;
gof(5).dfe='Not defined';
gof(5).adjrsquare='Not defined';
figure( 'Name', 'Negative Binomial Regression '  )
h = plot(age,fnbreg(age),'LineWidth',2),hold on,scatter(age,CCI,10)
legend(h,'cci vs. age','Negative Binomial Regression')
% Label axes
title(['Negative Binomial Regression '])
xlabel AGE
ylabel CCI
xlim auto
ylim ([0 100]);
grid on
```

Plot all the fit models together to compare them

```
clf
figure('Name', 'Regression Model Comparison '); % plot(0:16,fpol(0:16),'m--','LineWidth',2),hold on,
% scatter(age,CCI,10),hold on
plot(0:16,fpower(0:16),'b--+','LineWidth',2),hold on,plot(0:16,flin(0:16),'c:','LineWidth',2),hold on,plot(0:16,fsigmoidal(0:16),'-.','LineWidth',2),hold on,plot(0:16,fvdot(0:16),'-','LineWidth',2),hold on,plot(0:16,fnbreg(0:16),'m--','LineWidth',2)
% legend( 'cci vs. age', 'Power model','Linear model','Sigmoidal model','VDOT model','Negative Binomial', 'Location', 'SouthWest' );
legend('Power model','Linear model','Sigmoidal model','VDOT model','Negative Binomial', 'Location', 'SouthWest' );
hold on
boxplot(CCI,age,'Widths',.3,'OutlierSize',0.001,'symbol','')
% Label axes
title(['Regression model comparison ',HomogeneousFamilyId])
xlabel AGE
ylabel CCI
xlim auto
ylim([0 100]);
grid on

set(gcf, 'PaperUnits', 'centimeters');
set(gcf, 'PaperOrientation','landscape','PaperPosition', [0 0 29 18]); %x_width=29cm y_width=18cm
saveas(gca,[directory HomogeneousFamilyId ' age vs CCI Regression Model Comparison'],'pdf');
saveas(gca,[directory HomogeneousFamilyId ' age vs CCI Regression Model Comparison'],'fig');

T=struct2table(gof);
diary on
diary([ directory 'outputfit_' HomogeneousFamilyId '.txt']);
display('Model 1. Power');
display(fitresult{1});
display('Model 2. Linear model');
display(fitresult{2});
display('Model 3. Sigmoidal model');
display(fitresult(336));
display('Model 4. VDOT model');
display(fitresult{4});
display('Model 5. Negative Binomial model');
display('General model:')
```

```
display('y(x)=100-(x^b2)*exp(b0+b1*x)')
display('Coefficients')
display([' b0 = ',num2str(Y.b(1,1))])
display([' b1 = ',num2str(Y.b(2,1))])
display([' b2 = ',num2str(Y.b(3,1))])
display([' alpha = ',num2str(Y.alpha)])
display('Goodness of fit of the selected models');
display(T);
diary off

outfit.fitresult=fitresult;
outfit.gof=gof;

save([directory 'outfit.mat'],'age','CCI','outfit','fpower','flin','fsigmoidal','fvdot','fnbreg');
```

Published with MATLAB® R2017a

outputfit_PrimaryFamily.txt - Blocco note

File Modifica Formato Visualizza ?

```
Model 1. Power
     General model Power2:
     y(x) = a*x^b+c
     Coefficients (with 95% confidence bounds):
       a =      -4.763  (-5.156, -4.369)
       b =      0.8618  (0.8319, 0.8916)
       c =       98.57  (97.98, 99.16)
Model 2. Linear model
     Linear model Poly1:
     y(x) = p1*x + p2
     Coefficients (with 95% confidence bounds):
       p1 =      -3.277  (-3.307, -3.248)
       p2 =       96.34  (96.16, 96.52)
Model 3. Sigmoidal model
     General model:
     y(x) = 100-a*exp(-b/x)
     Coefficients (with 95% confidence bounds):
       a =       52.75  (52.24, 53.26)
       b =       4.267  (4.206, 4.328)
Model 4. VDOT model
     General model:
     y(x) = 100-exp(a+b*c^(log(1/x)))
     Coefficients (with 95% confidence bounds):
       a =      -10.71  (-18.59, -2.832)
       b =       12.51  (4.662, 20.36)
       c =      0.9437  (0.9124, 0.975)
Model 5. Negative Binomial model
General model:
y(x)=100-(x^b2)*exp(b0+b1*x)
Coefficients
 b0 = 1.8743
 b1 = 0.060172
 b2 = 0.49655
 alpha = 1.3218
```

Figure D- 1. Regression parameter results for the Primary System

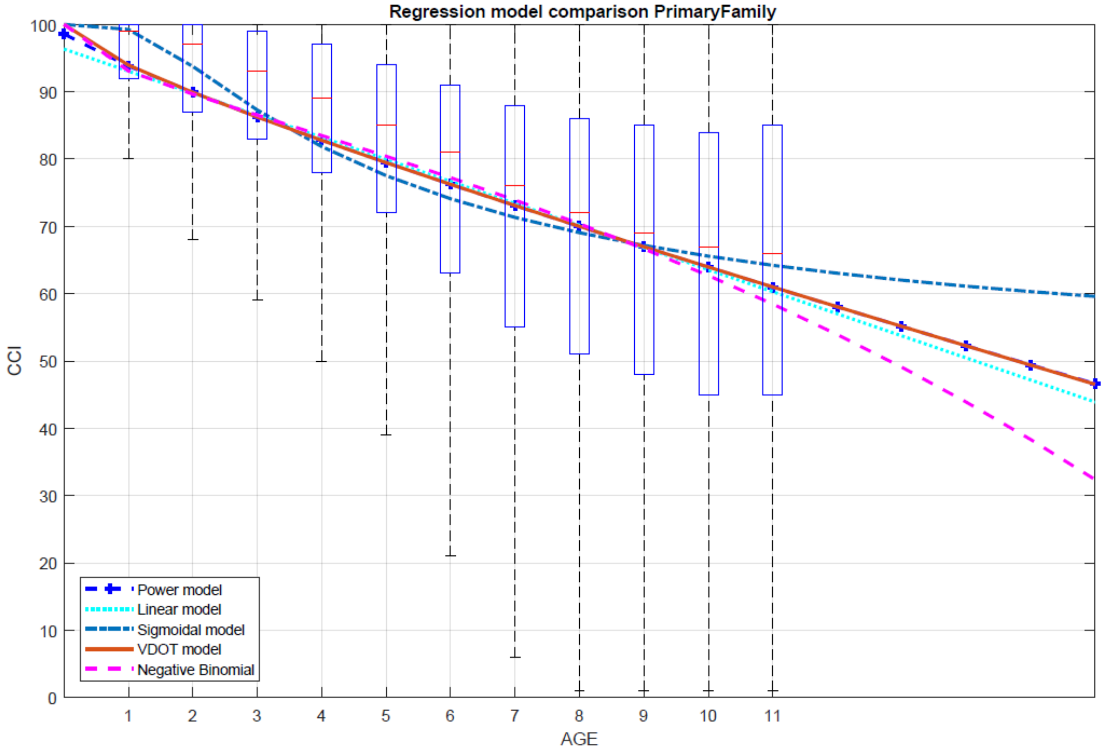


Figure D- 2. Regression model comparison for the Primary System

APPENDIX E. EMPIRICAL BAYES APPLICATION

This appendix describes the application of the Linear Empirical Bayes (LEB) to obtain an estimate of next year pavement condition, CCI_{EB} compared with the prediction obtained from the deterioration model ($CCI_{NBmodel}$).

Input Data from VDOT Primary System model:
CCI=67 ; age = 4 ; Parameters Models: β_0=1.353; β_1=0.228; β_2=-0.250; ϕ=1.322

Calculation of NB model estimation:

1. Calculate DI = 100-CCI=100-67 = 33
2. Calculate $DI_{NBmodel} = age^{\beta 1} * \exp(\beta_0 + \beta_2 * age) = 5^{0.228} * \exp(1.353 - 0.250 * 5) = 19.58$
3. Calculate $CCI_{NBmodel} = 100 - DI_{NBmodel} = 100 - 19.58 = 80.42$

Application of Empirical Bayesian Approach:

1. Calculate DI_{EB}:

$$DI_{EB} = \frac{1}{\phi * DI_{NB_{model}} + 1} * DI_{NB_{model}} + \left(1 - \frac{1}{\phi * DI_{NB_{model}} + 1}\right) * DI$$

$$DI_{EB} = \frac{1}{1.322 * 19.58 + 1} * 19.58 + \left(1 - \frac{1}{1.322 * 19.58 + 1}\right) * 33 = 32.50$$

2. Calculate $CCI_{EB} = 100 - DI_{EB} = 100 - 32.50 = 67.50$